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REPORT ON GEOTECHNICAL SITE INVESTIGATION

for

PROPOSED NEW RESIDENTIAL DEVELOPMENT

at

No.75-No.77 FOAMCREST AVENUE, NEWPORT, NSW

Prepared For

Provent Property Group

Project No.: 2020-202
November, 2020

Document Revision Record

Issue No	Date	Details of Revisions
0	12th November 2020	Original issue

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

Development Application for______________________________

Name of Applicant

Address of site No.75-No.77 Foamcrest Avenue, Newport

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 3rd November 2020_ certify that I am a ~~geotechnical engineer or~~ engineering geologist ~~or coastal engineer~~ as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above ~~organisation/~~company to issue this document and to certify that the ~~organisation/~~company has a current professional indemnity policy of at least $2million. I:

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

Geotechnical Report Details:

Report Title: Geotechnical Report for Proposed New Residential Development

Report Date: 12/11/2020 **Project No.:** 2020-202

Author: M. Lujan and T. Crozier

Author's Company/Organisation: Crozier Geotechnical Consultants

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

Site Survey Plan by Bee & Lethbridge Quality Surveying & Development Solutions, Ref. No.: 21796, Sheet No.: 1 of 1, Rev. No.: 00 and Date: 03/09/2020.
Architectural Drawings by Richard Cole Architecture Pty Ltd, Project Number: 1612, Date: September 2020 and Drawings: A01 to A20.

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

Signature ....

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

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

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

Company... **Crozier Geotechnical Consultants**

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

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

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

Development Application for____

Name of Applicant

Address of site __No.75-No.77 Foamcrest Avenue, Newport _____________

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

Geotechnical Report Details:

Report Title: Geotechnical Report for Proposed New Residential Development
Report Date: 12/11/2020 **Project No.:** 2020-202
Author: M. Lujan and T. Crozier
Author's Company/Organisation: Crozier Geotechnical Consultants

Please mark appropriate box

- ■ Comprehensive site mapping conducted ___15th November 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 15th November 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 REPORT FOR PROPOSED NEW RESIDENTIAL DEVELOPMENT
No.75- No.77 FOAMCREST AVENUE, NEWPORT, NSW

1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for a proposed residential development at No.75 - No.77 Foamcrest Avenue, Newport, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the request of Richard Cole Architecture on behalf of the client Provent Property Group (Martin Cork).

The site is not located within a landslip hazard zone as identified within Pittwater Councils Geotechnical Hazard Mapping (Geotechnical Risk Management Policy for Pittwater - 2009) (Map-Sheet GTH_017). However, the works trigger the policy in regard to excavation and filling - Section 3.2 (b) (iv). The site is designated as Acid Sulfate Soils hazard Class 4 (Map-Sheet ASS_017) – therefore an assessment of acid sulfate soils and groundwater conditions is also required for Development Application submissions.

This report includes a description of site and sub-surface conditions, a landslide risk assessment of the site and proposed works, plans, a geological section, an assessment of acid sulfate soils hazards and provides recommendations for construction and to ensure stability is maintained for a design life of 100 years.

The investigation and reporting were undertaken as per the Proposal No.: P20-454, Dated: 29th September 2020.

The investigation comprised:

a) A detailed geotechnical inspection and mapping of the site and adjacent properties by a Geotechnical Engineer.
b) DBYD plan request and onsite service clearance of borehole locations by accredited contractor.
c) Drilling of two boreholes using a restricted access drill rig and two boreholes using hand tools along with Dynamic Cone Penetrometer (DCP) testing to investigate the subsurface geology, depth to bedrock and identification of ground water conditions.

The following plans and drawings were supplied for the work:

- Architectural Drawings – by Richard Cole Architecture Pty Ltd, Project Number: 1612, Date: September 2020 and Drawings: A01 to A20.
- Survey Drawings – by Bee & Lethbridge Quality Surveying & Development Solutions, Ref. No.: 21796, Sheet No.: 1 of 1, Rev. No.: 00 and Date: 03/09/2020

2. PROPOSED DEVELOPMENT:

It is understood that the proposed works involve demolition of two existing dwellings and the construction of a new two storey unit development with a basement carpark. The proposed basement will require an excavation to 4.00m depth to achieve a proposed Basement Level (BL) of R.L.= 5.20m and will gradually reduce to ≤0.50m depth within the south-west corner of the site for a proposed new driveway ramp.

The excavation will extend to the south boundary and within approximately 4.50m of the north boundary, 6.50m of the west boundary and 6.50m of the east boundary.

Based on the available DBYD plans, a sewer main intersects the rear of the site, striking north-south approximately 4.50m from the rear east boundary. The proposed basement excavation is designed to be approximately 1.50m west from the sewer main.

3. SITE FEATURES:

3.1. Description:

The site is a rectangular shaped block located on the low east side of Foamcrest Avenue. The site contains two properties with two separate two storey clad houses, which are located within the centre of each block with front and rear lawns. It has west front and east rear boundaries of 30.48m and north and south side boundaries of 45.72m as referenced from the provided survey plan.

Ground surface levels within the site reduce from a high of approximately RL 8.96m adjacent to the north-west corner of the site to a low of approximately RL 8.10m in the south-east corner of the site.

An aerial photograph of the site and its surrounds is provided below, as sourced from NSW Government Six Map spatial data system, as Photograph-1. A general view of the site at the time of investigation are provided in Photograph-2 to Photograph-6 below.

Photograph-1: Aerial photo of site and surrounds.

Photograph-2: Front of No.77 Foamcrest Avenue. View looking east.

Photograph-3: Rear of No.77 Foamcrest Avenue. View looking west.

Photograph-4: Front of No.75 Foamcrest Avenue. View looking east.

Photograph-5: Rear of No.75 Foamcrest Avenue. View looking west.

Photograph-6: Rear of No.75 Foamcrest Avenue. View looking north.

3.2. Geology:

Reference to the Sydney 1: 100,000 Geological Series sheet (9130) indicates that the site is underlain by Quaternary sands (Qha) which comprise of silty to peaty quartz sand, silt, clay with common shell layers, ferruginous and humic cementation. The Quaternary sands are underlain by weathered bedrock at the boundary of the Newport Formation (Upper Narrabeen Group) rock (Rnn) of middle Triassic Age and the Bald Hill Claystone (Rnbh) of early Triassic Age. The Newport Formation typically comprises inter-bedded laminite, shale and quartz to lithic quartz sandstones and pink clay pellet sandstones and has a tendency to weather to significant depth, whilst the Bald Hill Claystone comprises dominantly red shale and fine to medium sandstone.

4. FIELD WORK:

4.1. Methods:

The field investigation comprised a walk over inspection and mapping of the site and adjacent properties on the 15th October 2020 by a Geotechnical Engineer. It included a photographic record of site conditions as well as geological/geomorphological mapping of the site and adjacent land with examination of existing structures. It also included the drilling of two auger boreholes (BH1 and BH2) using a restricted access drill rig employing solid stem, spiral flight augers and a tungsten carbide bit and two auger borehole (BH3 and BH4) using hand tools, due to limited access, to investigate sub-surface geology.

Dynamic Cone Penetrometer (DCP) testing was carried out from ground surface adjacent to the boreholes and through the base of the boreholes when they had progressed in accordance with AS1289.6.3.2 – 1997, "Determination of the penetration resistance of a soil – 9kg Dynamic Cone Penetrometer test" to estimate near surface soil conditions.

Strata identification was undertaken on material recovered from the boreholes with samples collected as per "AS1726: 2017 Geotechnical Site Investigation" for logging purposes.

Explanatory notes are included in Appendix: 1. Mapping information and test locations are shown on Figure: 1, along with detailed bore log and DCP sheets in Appendix: 2. A geological model/section is provided as Figure: 2 and Figure: 3, Appendix 2.

4.2. Field Observations:

The site is located on the low east side of Foamcrest Avenue within gently east dipping topography. Foamcrest Avenue is formed with a gently south dipping bitumen pavement where it passess the site, with low concrete gutter and kerbs, a concrete pathway, grass lawn and two trees (up to ≤ 10.0m height). The road pavement and road reserve appeared in good condition, there were no signs of significant undulations, deformations or underlying geotechnical issues.

Photograph-7: Foamcrest Avenue road reserve. View looking south.

No. 77 Foamcrest Avenue, contains a concrete driveway at the south-west corner of the block with a grass lawn to the north, subdivided by an east-west striking concrete entry pathway. The block contains a single storey clad cottage with a timber verandah at the north-east corner of the dwelling, founded on brick footings above Ground Surface Level (GSL) with an attached garage to the south. Access to the rear of the block is achieved through the dwelling and via a narrow pathway along the north boundary. The rear of the block contains a grass lawn that extends approximately 8.0m east from the dwelling and is bounded by a timber paling fence along the eastern, northern and southern sides. The dwelling, driveway, front and rear lawns appeared in good condition, signs of excessive cracking, deformation or underlying geotechnical issues were not observed within the structures.

No. 75 Foamcrest Avenue, contains a single clad house that broadly occupies the centre of the block and contains a timber deck within the north-east corner of the dwelling. Access to the rear of the block is achieved through the dwelling and via narrow pathways to the north and south of the dwelling. The rear of the block contains a grass lawn that extends approximately 14.0m east and extends north to the rear of No. 77 Foamcrest Avenue. The rear lawn is bounded by paling fences along the eastern, northern and southern sides and contains small to large trees within the eastern end of the block (up to 16.0m height). The dwelling, front driveway, front and rear lawns appeared in good condition, deformation or underlying geotechnical issues were not observed within the structures.

The neighbouring property to the north (No.79-83 Foamcrest Avenue) contains two separated two storey rendered unit buildings (within the west and east side of the property) that extend south to within approximately 4.50m to 7.00m from the common boundary at similar GSL to the site. The south-west corner of the property contains a concrete driveway ramp (Photograph-8), that leads down east to a basement carpark at a level approximately 3.0m below the site's GSL. The neighbouring dwellings, driveway and observed structures appeared in good condition, significant deformation or cracking or underlying geotechnical issues were not observed within the neighbouring property.

Photograph-8: No.79-No.83 Foamcrest's front driveway, directly to the north of the site. View looking east.

The neighbouring property to the south (No.73 Foamcrest Avenue) contains a single storey clad house with a detached fibro shed to the north east, front and rear grass lawns with a front strip driveway (Photograph-9) at the north-west corner of the property. The fibro shed is adjacent to the common boundary, whilst the dwelling extends north to approximately 2.50m from the common boundary. The front driveway and fibro shed, and rear grass lawn contain a similar GSL to the site. The property dwelling appeared in a relatively old condition, however significant deformation, cracking or underlying geotechnical issues were not observed within the neighbouring property.

Photograph-9: No.73 front driveway, directly to the south of the site. View looking east.

The neighbouring property to the east (No.405 Barrenjoey Rd.) contains a four storey brick residential unit building that broadly occupies the centre of the block and extends west to approximately 6.50m to 16.0m from the common boundary within the site. The neighbouring block contains a grass lawn at the front east with two concrete driveways (accessible from Barrenjoey Road) that continue west along the northern and southern boundaries of the property and then along the rear of the building. The rear of the building also contains a grass lawn with a clothes drying area directly adjacent to the site. The building and concrete driveways appeared in good condition, significant deformation, cracking or underlying geotechnical issues were not observed within this property.

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

4.3. Field Testing:

The boreholes (BH1 to BH4) were drilled approximately at the corners of the proposed basement. BH1 and BH2 were discontinued on a hard ironstone band/ extremely weathered sandstone/ siltstone bedrock (BH1) and sandy clay (BH2) at 3.40m depth and 5.00m depth, respectively. Hand auger refusal was encountered within BH4 and BH3 in silty/ sandy clay at varying depths between 0.90m and 1.20m, respectively.

Dynamic Cone Penetrometer (DCP) tests were carried out from ground surface and through the boreholes to refusal.

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

- **TOPSOIL/FILL –** this layer was encountered in all boreholes to a maximum depth of 0.80m below the existing ground surface. It comprised loose dark grey fine to medium grained moist silty sand with some roots.
- **SILTY/CLAYEY SAND** – this layer was encountered below the topsoil within BH1 and BH2. The silty/clayey sand was encountered to 1.40m depth within BH1 and down to 4.40m depth within BH2. This unit was intersected by a silty/ sandy clay layer within BH2 from 1.40m depth to 2.10m depth. The DCP testing results indicated it is very dense within BH1 and generally medium dense within BH2 from 0.30m to 0.75m depth and from 2.10m depth to 3.00m depth (below the silty sandy clay layer), becoming very dense to 4.40m depth. The silty/clayey sand was classified as dark brown/ grey, fine to medium grained, generally moist, and moist/wet between 2.10m depth to 2.60m depth (below the silty sandy clay layer).
- **SAND** – this layer was encountered below the topsoil within BH3 and BH4 to a maximum depth of 1.10m. It was classified as very dense, grey, fine to medium grained, moist, sand with trace of silt.
- **SILTY SANDY CLAY/ SANDY CLAY** – within the front of the site, this unit was encountered within BH1 and BH2 below the silty/ clayey sand unit through to 3.40m depth (refusal) within BH1 and to the maximum investigated depth of 5.0m within BH2. Within the rear of the site, this unit was encountered below the sand unit to a maximum of 1.20m depth (hand auger refusal) and is interpreted to ≥ 3.45m based on the DCP test results. This unit was classified as generally very stiff/ hard, orange/ brown, medium plasticity, moist, silty/ sandy clay.
- **SANDSTONE BEDROCK (EW/VLS) –** this unit was interpreted from dingo drill refusal encountered within BH1 only at 3.40m depth. Based on the DCP test results to 3.60m depth, it was classified as at least extremely weathered bedrock with low strength ironstone bands.

A freestanding ground water table or significant water seepages were not identified within the boreholes. However, within BH2, the DCP rod identified dampness from 3.0m depth through to 3.60m depth. Soil samples retrieved from BH2 identified a moist/ wet sandy unit (directly below the clayey unit) from 2.10m depth to 2.60m depth, becoming moist through to the maximum drilled depth. No free-standing ground water was identified within the bottom of BH2.

5. COMMENTS:

5.1. Geotechnical Assessment:

Within the western portion of the site, the site investigation identified the presence of silty sandy topsoil/ fill to a maximum depth of 0.80m, overlying a silty clayey sand unit to 1.40m depth (R.L.= 7.45m) and to 4.40m depth (R.L.= 3.95m) within BH1 and BH2, respectively. Increase in moisture was encountered within the clayey/ silty sand layer to 2.60m depth. Underlying the silty/ clayey sand unit, sandy clay (residual) was encountered to potentially interpreted bedrock within BH1 (3.40m depth, R.L.= 5.45m) and to the maximum drilled depth of 5.00m within BH2 (R.L.= 3.35m).

Within the eastern portion of the site, underlying the fill, very dense sand was encountered to a maximum depth of 1.10m, underlaid by a hard silty/ sandy clay unit to the maximum drilled depth of 1.20m (BH3, R.L.= 7.15m). Based on the DCP test results the consistency of the underlying unit was identified as hard through to a maximum tested depth of 3.45m (DCP4).

No groundwater or seepage were observed in the BHs and seepage was only observed from 3.0m depth upon retrieval of DCP2a. DCP effective refusal was encountered at 1.18m depth (DCP3) and 3.45m depth (DCP4a, R.L.= 5.15m) within the interpreted hard sandy clay/ clayey unit.

Based on the investigation test results, the excavation within the north-west portion of the proposed basement is expected to intersect fill, silty/ clayey sand, sandy clay (residual) and minor hard ironstone band/ extremely weathered bedrock to the excavation base. The excavation within the south-west portion of the proposed basement is expected to intersect fill, interbedded zones of silty/ sandy clay and silty/clayey sand through to the base of the excavation. Seepage is expected to be potentially intersected within the south-west corner of the basement excavation, within the silty clayey sand layer encountered within BH2 from 2.10m depth to 2.60m depth. This seepage will impact the stability of the overlying material.

Based on the limited test results within the eastern portion of the proposed basement carpark and information obtained from our database from a previous geotechnical investigation in a nearby site, the excavation is expected to intersect topsoil/ fill, sand and hard sandy clay through to the base of the basement excavation. The maximum borehole depth within the south-east corner was 1.20m, therefore the presence of seepage or interbedded zones of clayey sand or weak bedrock is not confirmed.

We recommend that post the demolition works within the site (and prior to bulk excavation) or where access to the rear of the site for a drill rig can be created (>1.50m wide, >4.0m height), then an investigation should be completed in the eastern corners of the basement to confirm geology and geotechnical conditions to below basement level. The investigation results will allow for a more detailed geotechnical assessment, design parameters and identify any unforeseen geotechnical conditions that can cause delays during the construction process.

It appears possible that the north-west corner of the basement carpark will potentially expose and as such be founded onto bedrock. Therefore, we would generally recommend that the entire structure be similarly founded. This is expected to require pile footing through the silty clayey sand and sandy clay units however similar strength bedrock was not encountered in the rest of the site. As such additional investigation is required to confirm the existence/ condition of the bedrock at the north west corner and geological conditions at the rear of the site to allow confirmation of design requirement.

The proposed basement level will require an excavation to 4.00m depth to achieve a FFL of R.L. 5.20m and will extend to distances as per summarised in Section 5.3.2. It is anticipated that the majority of the bulk excavation at the site should be achievable using standard hydraulic plant and rock excavation machinery only potentially required within the north-west portion of the basement (BH1) if bedrock is confirmed.

The use of rock hammers can create ground vibrations which could damage the neighbouring and adjacent structures (including sewer main). Care will be required during the demolition, construction and excavation works to ensure the neighbouring properties, structures and services are not adversely impacted by ground vibrations. Small scale equipment (i.e. rock hammer <250kg) and a good excavation methodology can be used to maintain low vibration levels and avoid the need for full time vibration monitoring. Crozier Geotechnical Consultants (CGC) should be consulted regarding the size and type of demolition/excavation equipment proposed and demolition/excavation methodology prior to works.

Based on the proposed basement excavation and the safe temporary batter slopes as per Section 5.3.2, the excavation of safe batters appears achievable in most parts of the excavation with respect to property boundaries except along the southern side of the excavation. Therefore, the construction of support prior to excavation will be required along the southern side. However, geotechnical inspection of temporary batters is required and the potential need to install support systems including project planning and costing.

Where support prior to bulk excavation is required, driven piles, sheet piles or methodologies likely to generate significant vibrations to the adjacent structures are not recommended. The construction of a soldier to contiguous pile wall would be a viable option. All retaining structures must be constructed as per *Earth-retaining structures AS 4678-2002* and as per Section 5.3.3 of this report.

Subject to prevailing weather conditions, some minor seepage may be encountered within the sand layer identified within BH2 from 2.10m depth to 2.60m depth and at other locations in the site. If the side walls of the pier/ pile excavations do not remain stable during construction, an allowance should be made for temporary support e.g. liners/ casing.

The site is also classified as being within an Acid Sulphate Soils (ASS) Class 4 Zone, however, due to the ground conditions encountered in the site investigation and the proposed works there is a “very low” likelihood of intersecting these soils or impacting the local water table. As such, an ASS Management Plan (ASSMP) is not considered necessary.

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

The recommendations and conclusions in this report are based on an investigation utilising only surface observations and isolated boreholes from hand tools and a restricted access drill rig. This test equipment provides limited data from small isolated test points across the entire site, therefore some minor variation to the interpreted sub-surface conditions is possible, especially between test locations. However the results of the investigation provide a reasonable basis for the Development Application analysis and subsequent design of the proposed works.

Further investigation to confirm geological conditions in the rear of the site and the north-west corner is recommended following DA approval and/ or demolition.

5.2. Site Specific Risk Assessment:

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

A. Landslip of soils from basement excavation (<10m³).

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

The **Risk to Life** from **Hazard A** was estimated to be up to **1.25×10^{-6}** for a single person, whilst the **Risk to Property** was considered to be **'Moderate'** in all situations.

Although the 'Moderate' Risk to Property for Hazard A is considered to be 'Unacceptable', the assessments were based on excavations with no support or planning. Provided the recommendations of this report are implemented including installation of retaining wall prior to bulk excavation the likelihood of any failure becomes 'Rare' and as such the consequences reduce and risk becomes within 'Acceptable' levels when assessed against the criteria of the AGS. As such the project is considered suitable for the site provided the recommendations of this report are implemented.

5.3. Design & Construction Recommendations:

Design and the construction recommendations are tabulated below:

5.3.1. New Footings:	
Site Classification as per AS2870 – 2011 for new footing design	Class 'M' due to the clayey nature of the soils
Type of Footing	Strip/Pad or Slab at base of excavation, piers external to excavation or where high point loads are required
Sub-grade material and Maximum Allowable Bearing Capacity	- Stiff Silty CLAY: 100kPa - Very Stiff Silty CLAY: 200kPa - Hard Sandy CLAY: 400kPa - Weathered (EW-VLS) bedrock: 700kPa* - Weathered (LS) bedrock: 1000kPa*
Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia*	C_e – Shallow soil site

Remarks:

*Requires confirmation via core drilling investigation

All footings should be founded off material of similar strength unless the structure can accommodate potentially high differential settlements.

All new footings must be inspected by an experienced geotechnical professional before concrete or steel are placed to verify their bearing capacity and the in-situ nature of the founding strata. This is mandatory to allow them to be 'certified' at the end of the project.

5.3.2. Excavation:

Property Separation

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

Paved Patio Excavation

Table 1: Property Separation Distances

<table>
<tr><th rowspan="2">Boundary</th><th rowspan="2">Adjacent Property</th><th rowspan="2">Structure</th><th rowspan="2">Bulk Excavation Depth (m bgl)</th><th colspan="2">Separation Distances (m)</th></tr>
<tr><th>Boundary (m)</th><th>Structure</th></tr>
<tr><td>North</td><td>No.79 to No.83 Foamcrest Ave.</td><td>Driveway, terrace & pathway, and dwellings</td><td>3.60 to 4.0</td><td>4.50</td><td>-Adjacent to the boundary are the driveway, terrace, and patio.
-Building a further 4.50m.</td></tr>
<tr><td>South</td><td>No.73 Foamcrest Ave</td><td>Strip driveway, shed & grass lawn and dwelling</td><td>0.0 to 3.50</td><td>0</td><td>-Adjacent to the driveway, shed and rear grass lawn.
-House, a further 2.50m.</td></tr>
<tr><td rowspan="2">East</td><td>No.405 Barrenjoey Rd.</td><td>Grass lawn and clothes and drying area</td><td>3.30 to 3.60</td><td>6.50</td><td>-Adjacent to the building are the grass lawn and clothes and drying area.
-Building, a further 6.50m.</td></tr>
<tr><td colspan="5">Sewer Main approximately 1.50m east from the basement excavation</td></tr>
<tr><td>West</td><td>Foamcrest Ave. (Road Reserve)</td><td>Pathway and road pavement</td><td>3.60 to 4.00</td><td>6.50</td><td>- Pathway and road pavement a further >4.0m</td></tr>
</table>

<table>
<tr><td rowspan="2">Type of Material to be Excavated</td><td colspan="2">Fill ≤ 0.85m depth (BH1).</td></tr>
<tr><td colspan="2">Sand, silty/ clayey sand, sandy clay, silty/ sandy clay > 0.30m depth (BH2), through to the base of the basement excavation and to the ironstone band/ extremely weathered bedrock at 3.40m depth in BH1.</td></tr>
<tr><td colspan="3">Guidelines for batter slopes for this site are tabulated below:
<table>
<tr><th rowspan="2">Material</th><th colspan="2">Safe Batter Slope (H:V)</th></tr>
<tr><th>Short Term/ Temporary</th><th>Long Term/ Permanent</th></tr>
<tr><td>Fill/ Sand</td><td>1.5:1</td><td>2:1</td></tr>
<tr><td>Very stiff sandy clay/ clayey sand</td><td>1:1*</td><td>1.5:1*</td></tr>
</table>
*Dependent on seepage and assessment by engineering geologist</td></tr>
<tr><td colspan="3">Remarks:
Seepage along defects in the soil can also reduce the stability of batter slopes and invoke the need to implement additional support measures. Where safe batter slopes are not implemented the stability of the excavation cannot be guaranteed until the installation of permanent support measures. This should also be considered with respect to safe working conditions.</td></tr>
<tr><td rowspan="2">Equipment for Excavation</td><td>Topsoil/Sandy Soils</td><td>Excavator with bucket</td></tr>
<tr><td>Bedrock</td><td>Rock hammers and rock saw or rock grinders</td></tr>
<tr><td>Recommended Vibration Limits (Maximum Peak Particle Velocity (PPV))</td><td colspan="2">Yes, recommended for any rock hammer >250kg weight
Sewer 3 mm/s (subject to SW requirements)
Residential structures 5mm/s on nearby properties.</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 during construction</td><td colspan="2">Yes, recommended that these inspections be undertaken as per below mentioned sequence:
During construction of the retaining/support structures, prior to bulk excavation.
For assessment of batter slopes.
At every 2.00m depth interval of excavation.
At completion of excavation
Where unexpected ground conditions are encountered
Prior to the construction of footings for assessment of bearing.</td></tr>
<tr><td>Dilapidation Surveys Requiremen</td><td colspan="2">Survey of structures with 5.0m of proposed excavation will reduce the potential for spurious claims of damage.
<u>Note:</u> CGC have the experience in performing Dilapidation Surveys</td></tr>
</table>

5.3.3. Retaining Structures:	
Required	New retaining structures will be required as part of the proposed development along the southern and eastern sides of the basement excavation.
Types	Soldier to contiguous piles prior to bulk excavation and/or steel reinforced concrete/concrete block wall where safe temporary batters can be formed. Designed in accordance with Australian Standard AS 4678-2002 Earth Retaining Structures

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

Material	**Unit Weight (kN/m^3)**	**Long Term (Drained)**	**Earth Pressure Coefficients**		**Passive Earth Pressure Coefficient ***
			Active (Ka)	**At Rest (K_0)**	
Fill	18	$\phi' = 28°$	0.35	0.52	N/A
Silty clay (firm to stiff)	20	$\phi' = 30°$	0.33	0.50	3.00
Silty clay (very stiff to hard)	22	$\phi' = 35°$	0.27	0.42	3.50

Remarks:

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

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

5.3.4. Drainage and Hydrogeology		
Groundwater Table or Seepage identified in Investigation		No
Excavation likely to intersect	Water Table	No
	Seepage	Minor (<2L/min), at soil interfaces
Site Location and Topography		Low east side of road, within gently east dipping topography
Impact of development on local hydrogeology		Negligible

Onsite Stormwater Disposal	Subject to Hydraulic Engineer's design, clay soils have very low permeability, therefore only possible via dispersion.
Depth to rock	Potentially at 3.40m depth within BH1 and not encountered within the rest of the test locations to a maximum depth of 5.0m within BH2 and to 3.45m within DCP4a.
Minimum distance of stormwater system from down slope boundaries	≥ 5.00m to allow maximum transpiration within property boundaries
Remarks: As the excavation faces are expected to encounter some seepage, an excavation trench should be installed at the base of excavation cuts to below floor slab levels to reduce the risk of resulting dampness issues. Trenches, as well as all new building gutters, down pipes and stormwater intercept trenches should be connected to a stormwater system designed by a Hydraulic Engineer which preferably discharges to the Council's stormwater system off site.	

5.4. Conditions Relating to Design and Construction Monitoring:

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

1. Conduct addition investigation to confirm geotechnical conditions in rear inaccessible portions of the site and also to confirm bedrock in the north-west corner.
2. Review and approve the structural design drawings for compliance with the recommendations of this report prior to construction,
3. Inspection of site and works as per Section 5.3.1 of this report
4. Inspect all new footings and earthworks to confirm compliance to design assumptions with respect to allowable bearing pressure, basal cleanness and the stability prior to the placement of steel or concrete,
5. Inspect completed works to ensure construction activity has not created any new hazards and that all retention and stormwater control systems are completed.

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

5.5. Design Life of Structure:

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

- stormwater and subsoil drainage systems,
- retaining walls and 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 – 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 don't change from those present at the time this report was prepared, except for the changes due to this development.
- There is no change to the property due to an extraordinary event external to this site
- The property is maintained in good order and in accordance with the guidelines set out in;
    a) CSIRO sheet BTF 18
    b) Australian Geomechanics "Landslide Risk Management" Volume 42, March 2007.
    c) AS 2870 – 2011, Australian Standard for Residential Slabs and Footings

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

6. CONCLUSION:

The site investigation identified the presence of sandy topsoil to a maximum depth of 0.80m, underlaid by silty clayey sand (intersected by zones of silty sandy clay) within the western side of the site and by sand within the eastern side of the site. Underlying the silty clayey sand and sand units, sandy clay (residual unit) was encountered to a maximum depth of 5.00m within BH2 and to 3.40m depth within BH1, overlying a hard ironstone band/ interpreted extremely weathered bedrock of at least low strength. A groundwater table or significant seepage were not encountered within the investigation; however, seepage was encountered within the clayey/ silty sand layer within BH2 to a maximum depth of 2.60m.

The proposed basement excavation is expected to primarily extend through silty/ sandy clay and clayey/ silty sand through to majority of the excavation base however there appears the potential for bedrock of at least low strength within the north-west corner of the basement at 3.40m depth (although this requires confirmation). As such, conventional earth moving excavation machinery will be suitable along with minor rock excavation equipment (e.g. rock hammers, rock saw, ripper).

We recommend that post demolition of the site structures, additional investigation using a drill rig be performed within the eastern rear of the site to confirm sub-surface ground conditions and within the north west corner to confirm bedrock existence/ condition as this will significantly impact design and construction.

We recommend that all the footings be founded onto/ within similar founding material and bearing characteristics to prevent differential settlement. Therefore, pile inspection by a professional geotechnical consultant during construction is recommended. Where CGC is required to provide sign off on completion, all piles/ footings will need supervision/ inspection.

The construction of support prior to excavation structures will be required along the southern and eastern sides of the basement carpark. This may be achieved via a soldier to contiguous pile wall founded onto similar founding material.

Due to the observed geology and ground water conditions, the likelihood of intersecting Acid Sulfate Soils or impacting the water table is very low therefore no further investigation or reporting into these soils is necessary. Further investigation within the rear of the site will confirm this.

Provided the recommendations of this report are implemented in the design and construction phases of the development, it is considered that the works can be carried out with negligible impact to the site and neighbouring properties and as such are considered suitable for the site.

The potential risks associated with the proposed development will be within 'Unacceptable' levels where insufficient/unsuitable support systems are implemented. However, where suitable engineer designed systems are implemented the risks will be reduced and can be maintained within 'Acceptable' risk criteria for the design life of the development, taken as 100 years.

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

7. REFERENCES:

1. Australian Geomechanics Society 2007, "Landslide Risk Assessment and Management", Australian Geomechanics Journal Vol. 42, No 1, March 2007.
2. Pittwater Council Local Environmental Plan 2014, Acid Sulphate Soils Map – Sheet ASS_017 and Geotechnical Hazard Map – Sheet GTH_017.
3. Geological Society Engineering Group Working Party 1972, "The preparation of maps and plans in terms of engineering geology" Quarterly Journal Engineering Geology, Volume 5, Pages 295 - 382.
4. E. Hoek & J.W. Bray 1981, "Rock Slope Engineering" By The Institution of Mining and Metallurgy, London.
5. C. W. Fetter 1995, "Applied Hydrology" by Prentice Hall.
6. V. Gardiner & R. Dackombe 1983, "Geomorphological Field Manual" by George Allen & Unwin

Appendix 1

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NOTES RELATING TO THIS REPORT

Introduction

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

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

Description and classification Methods

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Standard Penetration Tests

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

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

The test results are reported in the following form.

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

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

Cone Penetrometer Testing and Interpretation

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

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

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

The information provided on the plotted results comprises: -

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

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

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

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

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.

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

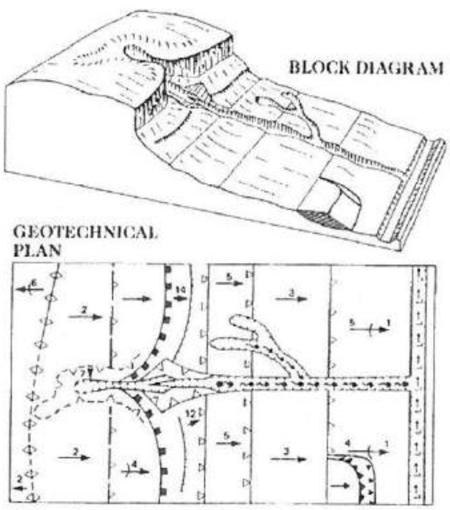
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

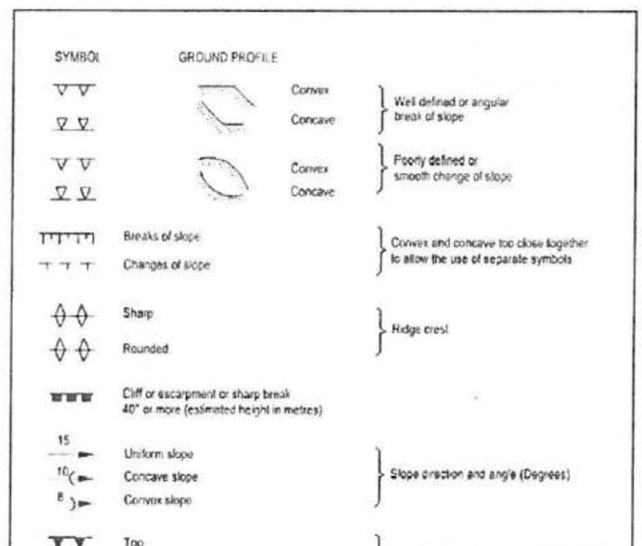
Reproduction of Information for Contractual Purposes

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

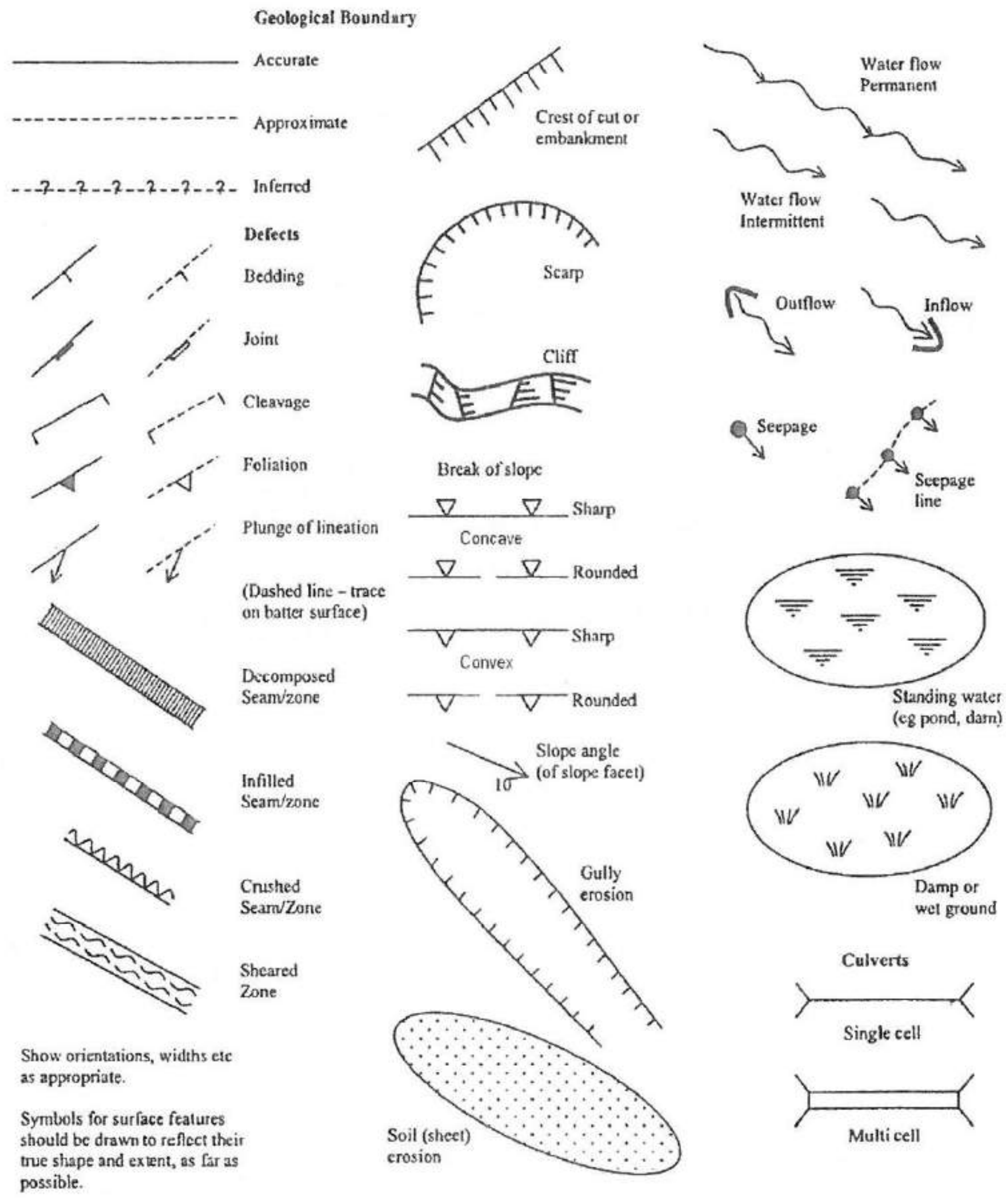
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.





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).

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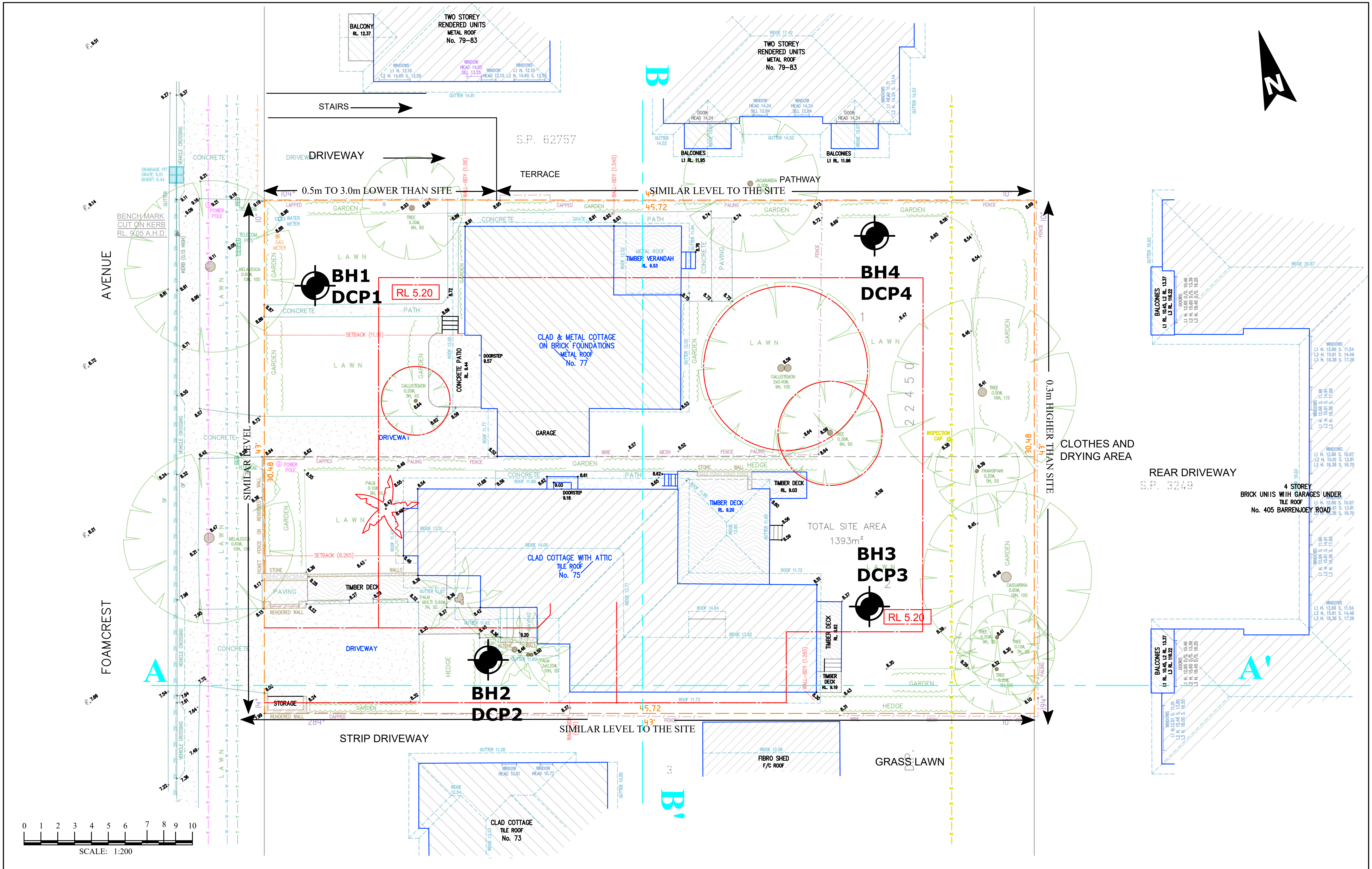
Appendix 2



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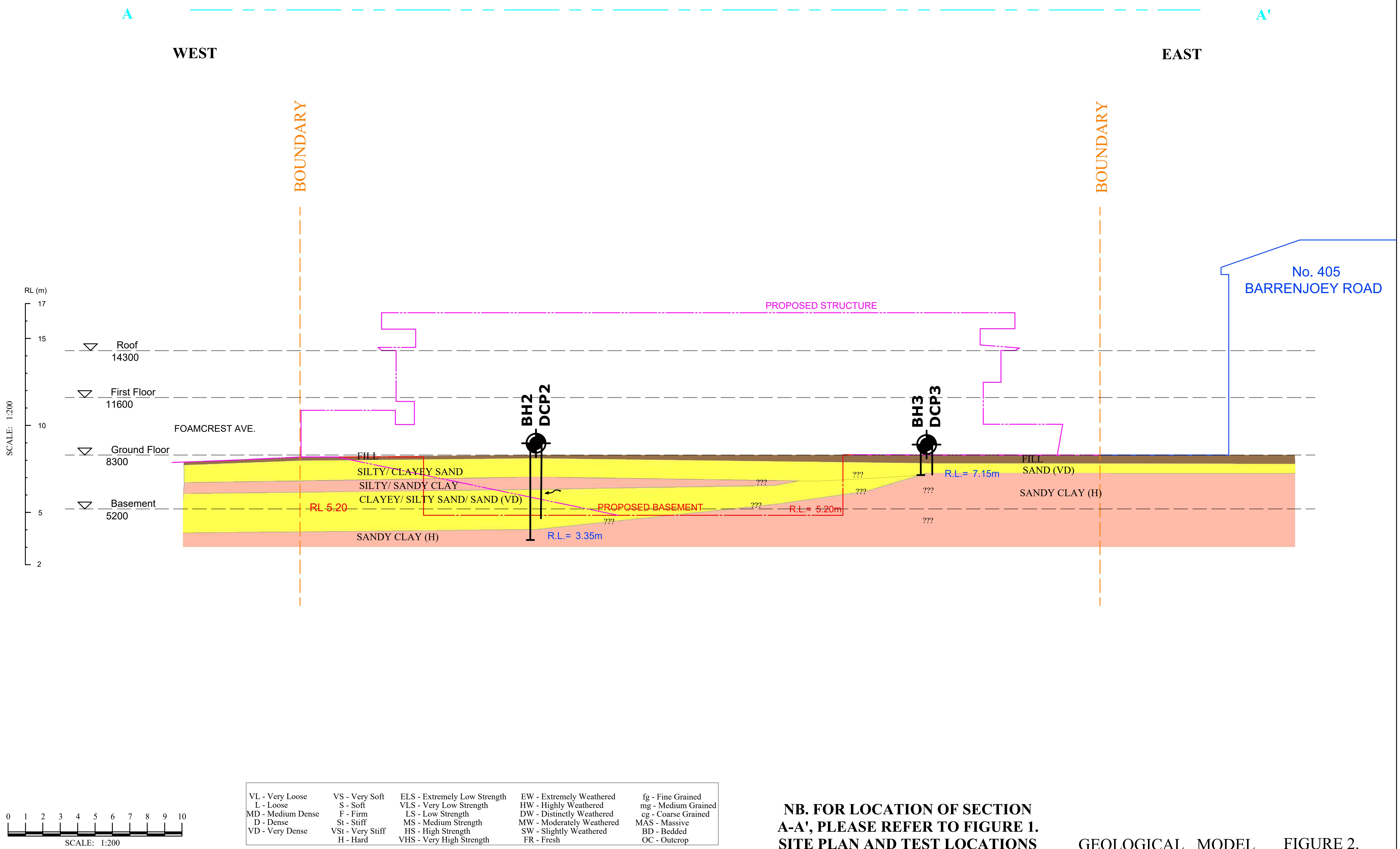
LEGEND

BH DCP	AUGER / DYNAMIC CONE PENETROMETER LOCATION
A — A'	CROSS-SECTION REFERENCE LINE
Blue line	EXISTING STRUCTURE
Red line	PROPOSED EXCAVATION
Orange dashed line	PROPERTY BOUNDARY
Magenta line	PROPOSED STRUCTURE

SCALE:	1:200 @ A3
DRAWING:	FIGURE 1
DATE:	30/10/2020
APPROVED BY:	TMC
DRAWN BY:	ML
PROJECT:	2020-202

PREPARED FOR:
MARTIN CORK

ADDRESS:
75-77 FOAMCREST AVENUE,
NEWPORT



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

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

GEOLOGICAL MODEL FIGURE 2.

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LEGEND

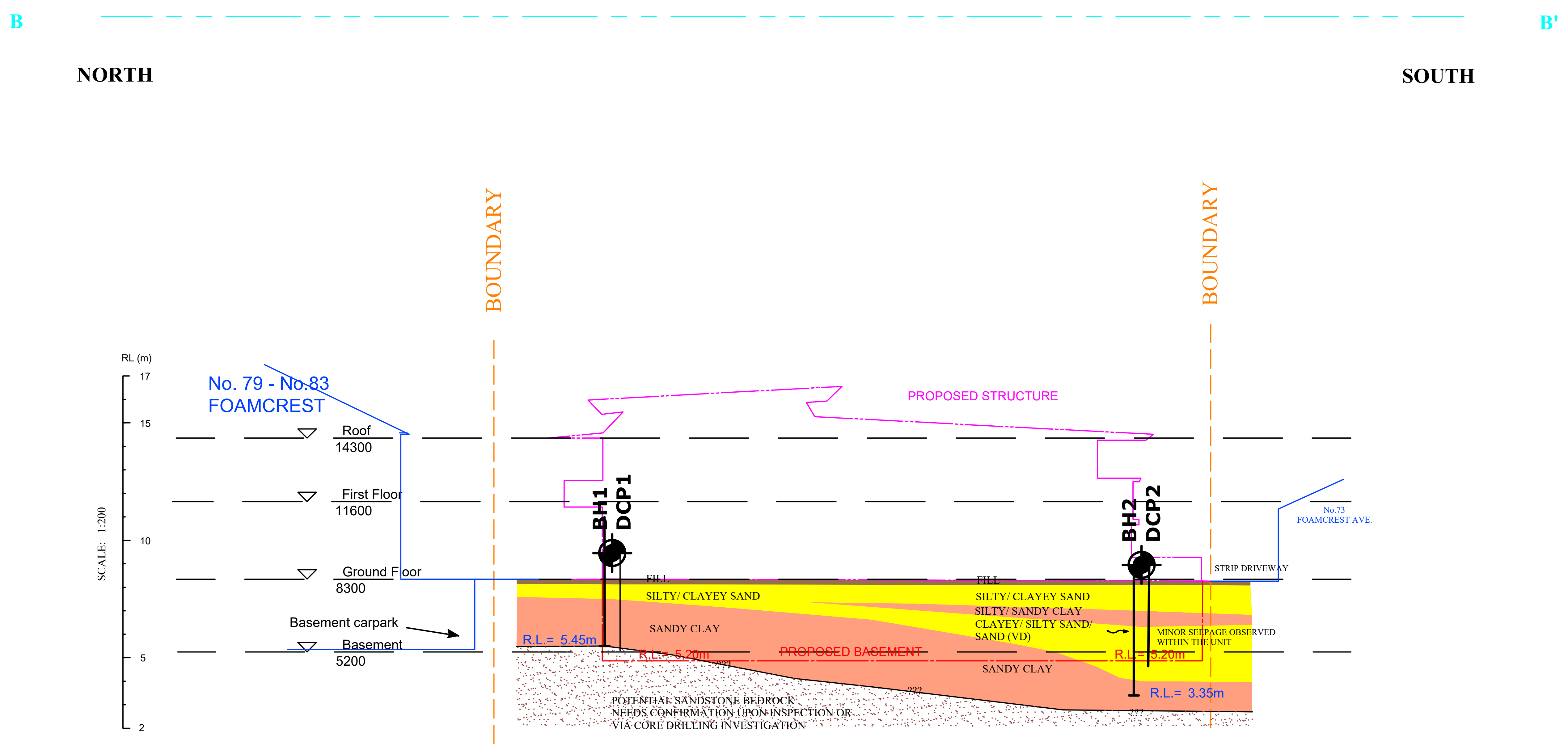
A — A'	CROSS-SECTION REFERENCE LINE
	PROPOSED EXCAVATION
	PROPOSED ADDITION
	SAND/ SILTY CLAYEY SAND
BH DCP	AUGER / DYNAMIC CONE PENETROMETER LOCATION
	PROPERTY BOUNDARY
	CLAY
	FILL

SCALE: 1:200 @ A3
DRAWING: FIGURE 2
DATE: 30/10/2020

APPROVED BY: TMC
DRAWN BY: ML
PROJECT: 2020-202

PREPARED FOR:
MARTIN CORK

ADDRESS:
75-77 FOAMCREST AVENUE,
NEWPORT



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 3.

CROZIER
GEOTECHNICAL CONSULTANTS

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

LEGEND

- A — A' CROSS-SECTION REFERENCE LINE
- PROPOSED EXCAVATION
- PROPOSED ADDITION
- POTENTIAL BEDROCK
- CLAY
- SAND/ SILTY CLAYEY SAND
- BH DCP AUGER / DYNAMIC CONE PENETROMETER LOCATION
- PROPERTY BOUNDARY
- EXISTING STRUCTURE
- FILL
- SEEPAGE

SCALE:	1:200 @ A3
DRAWING:	FIGURE 3
DATE:	30/10/2020
APPROVED BY:	TMC
DRAWN BY:	ML
PROJECT:	2020-202

PREPARED FOR:
MARTIN CORK

ADDRESS:
75-77 FOAMCREST AVENUE,
NEWPORT

BOREHOLE LOG

CLIENT: Martin Cork **DATE:** 15/10/2020 **BORE No.:** 1

PROJECT: Demolition of 2 dwellings and construction of 2 storey unit block **PROJECT No.:** 2020-202 **SHEET:** 1 of 1

LOCATION: 75-77 Foamcrest Avenue, Newport 2106 **SURFACE LEVEL:** R.L.= 8.85m

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: Loose, dark grey, fine to medium grained, moist, silty sand with some plant roots				
0.70						
0.80		é becoming silty/clayey sand		0.80		
	SC	SILTY/ CLAYEY SAND: Very dense,dark brown/ grey, fine to medium grained, moist, silty clayey sand with trace of fine gravel	D	0.85		
			D	0.90		
1.40				1.40		
	CI	SANDY CLAY: Very stiff, yellow orange, medium plasticity, moist, sandy clay	D	1.50		
1.50		é hard				
2.00						
2.50		é pale grey mottled yellow/ orange				
2.80		é orange mottled pale grey				
				3.30		
3.40			D	3.40		
		DINGO REFUSAL at 3.40m depth on hard ironstone band/ potential extremely weathered sandstone bedrock				
4.00						

RIG: Dingo restricted access rig DRILLER: AC LOGGED: ML

METHOD: Solid stem spiral flight auger, tungsten carbide bit CHECKED: TMC

GROUND WATER OBSERVATIONS: No freestanding groundwater found

REMARKS:

BOREHOLE LOG

CLIENT: Martin Cork **DATE:** 15/10/2020 **BORE No.:** 2

PROJECT: Demolition of 2 dwellings and construction of 2 storey unit block **PROJECT No.:** 2020-202 **SHEET:** 1 of 1

LOCATION: 75-77 Foamcrest Avenue, Newport 2106 **SURFACE LEVEL:** R.L.= 8.35m

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, silty sand with plant roots				
0.30	SC	SILTY/ CLAYEY SAND: Medium dense, grey, fine to medium grained, moist, silty clayey sand				
0.75		é dense				
0.90		é very dense	D	0.90 1.00		
1.40		é moist/ wet				
	CI	SILTY/ SANDY CLAY: Hard, grey mottled yellow orange, low to medium plasticity, moist, silty sandy clay	D	1.40 1.50		
2.00						
2.10	SC	SILTY/ CLAYEY SAND: Medium dense, grey/ yellow orange, fine to medium grained, moist/ wet, clayey silty sand	D	2.30 2.40		
2.60		... moist, bands of sandy clay				
3.00		é very dense				
3.20		é grey/ yellow brown	D	3.20 3.30		
4.00		é orange brown/ grey	D	4.00 4.10		
4.40	CI	SANDY CLAY: Hard, orange brown mottled grey, medium plasticity, moist, sandy clay	D	4.40 4.50		
5.00		END OF BOREHOLE at 5.00m depth in sandy clay				

RIG: Dingo restricted access rig DRILLER: AC LOGGED: ML

METHOD: Solid stem spiral flight auger, tungsten carbide bit CHECKED: TMC

GROUND WATER OBSERVATIONS: No free-standing groundwater was detected at the end of the borehole at 5.0m depth

REMARKS: DCP rod was wet from 3.0m depth

BOREHOLE LOG

CLIENT: Martin Cork **DATE:** 15/10/2020 **BORE No.:** 3

PROJECT: Demolition of 2 dwellings and construction of 2 storey unit block **PROJECT No.:** 2020-202 **SHEET:** 1 of 1

LOCATION: 75-77 Foamcrest Avenue, Newport 2106 **SURFACE LEVEL:** R.L.= 8.35m

Depth (m)	Classification	Description of Strata **PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks	Sampling Type	Sampling Tests	In Situ Testing Type	In Situ Testing Results
0.00		TOPSOIL/FILL: Dark grey fine to medium grained moist silty sand with roots				
				0.40		
0.50			D	0.50		
	SM	SAND: Very dense, grey, fine to medium grained, moist, sand with trace of silt				
0.70		é becoming pale grey				
1.00		é becoming dark brown with bands of clayey sand		1.00		
1.05		é becoming brown	D	1.10		
1.10						
1.20	CL	SILTY/ SANDY CLAY: Hard, orange/ brown, low plasticity, silty sandy				
		HAND AUGER REFUSAL at 1.20m depth in hard silty/ sandy clay				
2.00						
4.00						

RIG: None DRILLER: AC LOGGED: ML

METHOD: Hand Auger CHECKED: TMC

GROUND WATER OBSERVATIONS: No freestanding groundwater found

REMARKS:

BOREHOLE LOG

CLIENT: Martin Cork | **DATE:** 15/10/2020 | **BORE No.:** 4

PROJECT: Demolition of 2 dwellings and construction of 2 storey unit block | **PROJECT No.:** 2020-202 | **SHEET:** 1 of 1

LOCATION: 75-77 Foamcrest Avenue, Newport 2106 | **SURFACE LEVEL:** R.L.= 8.60m

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: Dark grey fine to medium grained moist silty sand with roots				
0.40	SM	SAND: Very dense, grey, fine to medium grained, moist, sand with trace of silt clay				
0.70	CL	SILTY/ SANDY CLAY: Hard, orange/ brown, low plasticity, silty sandy				
0.90		HAND AUGER REFUSAL at 0.90m depth in hard silty/ sandy clay				
2.00						
4.00						

RIG: None | DRILLER: AC | LOGGED: ML

METHOD: Hand Auger | CHECKED: TMC

GROUND WATER OBSERVATIONS: No freestanding groundwater found

REMARKS:

DYNAMIC PENETROMETER TEST SHEET

CLIENT: Martin Cork **DATE:** 15/10/2020

PROJECT: Demolition of 2 dwellings and construction of 2 storey dwelling **PROJECT No.:** 2020-202

LOCATION: 75-77 Foamcrest Ave, Newport 2106 **SHEET:** 1 of 1

	Test Location							
Depth (m)	DCP1	DCP1a	DCP1b	DCP2	DCP2a	DCP3	DCP4	DCP4a
0.00 - 0.15	1	--	--	4	--	2	2	--
0.15 - 0.30	3	--	--	3	--	3	2	--
0.30 - 0.45	3	--	--	4	--	3	5	--
0.45 - 0.60	5	--	--	5	--	7	9	--
0.60 - 0.75	30	--	--	6	--	13	8	--
0.75 - 0.90	23	--	--	10	--	34	6	--
0.90 - 1.05	40	--	--	17	--	26	37	--
1.05 - 1.20	14 (D) @1.20m depth	--	--	15	--	27 (R) @1.18m depth	12	17
1.20 - 1.35		5	--	45	--		9	3
1.35 - 1.50		6	--	44 (R) @1.35m	--		10	5
1.50 - 1.65		9	--		--		10	4
1.65 - 1.80		13	--		--		12	9
1.80 - 1.95		18	--		--		16	11
1.95 - 2.10		21	--		--		14	12
2.10 - 2.25		22	--		--		11	11
2.25 - 2.40		24 (D) @2.40m depth	--		--		12	10
2.40 - 2.55			--		--		10	10
2.55 - 2.70			10		5		13	11
2.70 - 2.85			9		7		19	12
2.85 - 3.00			14		7		26 (D) @3.00m depth	23
3.00 - 3.15			22		19			31
3.15 - 3.30			28		25			29
3.30 - 3.45			25		26			28 (R) @3.45m depth
3.45 - 3.60			30 (R) @ 3.60m depth		20			
3.60 - 3.75					22 (R) @ 3.75m depth			
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
(D) Test discontinnued
-- No test undertaken at this level due to prior excavation of soils

Appendix 3

TABLE : A

Landslide risk assessment for Risk to life

HAZARD	Description	Impacting	Likelihood of Slide	Spatial Impact of Slide		Occupancy	Evacuation	Vulnerability	Risk to Life
A	Landslip of soils from basement excavation (<10m³)		landslide due to excavation up to 4.00m depth in sandy topsoil and generally silty sandy clay	a) Excavation within 4.50m of the common boundary, driveway adjacent to boundary, impact 1%; Terrace and pathway adjacent to boundary, impact 5%; building another 4.50m, impact 1% b) Excavation adjacent to the common boundary, driveway adjacent to boundary, impact 15%; single shed adjacent to boundary, impact 30%; grass lawn adjacent to boundary, impact 2%; dwelling another 2.50m, impact 20% c) Excavation within 6.50m of the common boundary; driveway adjacent to the boundary, impact 2%; grass, clothes and drying adea adjacent to boundary, impact 2%; building another 6.50m, impact 1% d) Excavation within 6.50m of the western boundary, pathway another 4.0m, impact 2%, road pavement another 6.0m, impact 1%		a) Person in driveway, 1hr/day avg. Person in the terrace and pathway, 5hr/day avg. Person in the building, 23hr/day avg. b) Person in driveway, 1hr/day avg. Person in the single shed, 5hr/day avg. Person in the grass lawn, 4hr/day avg. Person in the dwelling, 23hr/ day avg. c) Person in driveway, 5hr/day avg. Person in the grass area, 5 hr/day avg. Person in the building 24/24 avg. d) Peson in the pathway, 2hr/ day avg. Person in the road pavement 4hr/day avg.	a) Possible to not evacuate Likely to not evacuate Almost certain to not evacuate b) Possible to not evacuate Almost certain to not evacuate Likely to not evacuate Almost certain to not evacuate c) Possible to not evacuate Possible to not evacuate Almost certain to not evacuate d) Possible to not evacuate Possible to not evacuate	a) Person in open space, partly buried Person in open space, partly buried Person in building, minor injured b) Person in open space, partly buried Person in the shed, major injury where shed structure collapses Person in open space, partly buried Person in building, minor injured c) Person in open space, partly buried Person in open space, partly buried Person in building, minor injured, d) Person in open space, partly buried Person in open space, partly buried	
			Possible	**Prob. of Impact**	**Impacted**				
	a)	No. 79 - No.83 Foamcrest Av.(start of Driveway)	0.001	0.05	0.01	0.04	0.50	0.50	**5.21E-09**
		No. 79 - No.83 Foamcrest Av. (Terrace&pathway)	0.001	0.05	0.05	0.21	0.75	0.50	**1.95E-07**
		No. 79 - No.83 Foamcrest	0.001	0.01	0.01	0.96	1.00	0.01	**9.58E-10**
	b)	No. 73 Foamcrest Av. (Driveway)	0.001	0.80	0.15	0.04	0.50	0.50	**1.25E-06**
		No. 73 Foamcrest Av. (single shed)	0.001	0.80	0.30	0.21	0.10	0.70	**3.50E-06**
		No. 73 Foamcrest Av. (Grass lawn)	0.001	0.05	0.02	0.17	0.75	0.50	**6.25E-08**
		No. 73 Foamcrest Av. (Dwelling)	0.001	0.50	0.20	0.96	0.50	0.01	**4.79E-07**
	c)	No. 405 Barrenjoey Rd. (Driveway)	0.001	0.05	0.10	0.21	0.50	0.05	**2.60E-08**
		No. 405 Barrenjoey Rd. (Grass, clothes and drying area)	0.001	0.05	0.02	0.21	0.50	0.05	**5.21E-09**
		No. 405 Barrenjoey Rd. (Building)	0.001	0.00	0.01	1.00	1.00	0.01	**1.00E-10**
	d)	Foamcrest Ave. (Pathway)	0.001	0.05	0.02	0.08	0.50	0.10	**4.17E-09**
		Foamcrest Ave. (Road pavement)	0.001	0.05	0.01	0.17	0.50	0.05	**2.08E-09**

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

* likelihood of occurrence for design life of 100 years

* Spatial Impact - Probaility of Impact refers to slide impacting structure/area expressed as a % (i.e. 1.00 = 100% probability of slide impacting area if slide occurs).
Impacted refers to expected % of area/structure damaged if slide impacts (i.e. small, slow earth slide will damage small portion of house structure such as 1 bedroom (5%), where as large boulder roll may damage/destroy >50%)

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

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

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

* evacuation scale from Almost Certain to not evacuate (1.0), Likely (0.75), Possible (0.5), Unlikely (0.25), Rare to not evacuate (0.01). Based on likelihood of person knowing of landslide and completely evacuating area prior to landslide impact.

* vulnerability assessed using Appendix F - AGS Practice Note Guidelines for Landslide Risk Management 2007

TABLE : B

Landslide risk assessment for Risk to Property

HAZARD	Description	Impacting	Likelihood		Consequences		Risk to Property
A	Landslip of soils from basement excavation (<10m³)	a) No. 79 - No.83 Foamcrest Ave.	Unlikely	The event might occur under very adverse circumstances over the design life.	Minor	Limited Damage to part of structure or site requires some stabilisation or INSIGNIFICANT damage to neighbouring properties.	Low
		b) No. 73 Foamcrest Ave.	Possible	The event could occur under adverse conditions 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.	Moderate
		c) No. 405 Barrejoey Rd.	Unlikely	The event might occur under very adverse circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required or no impact to neighbouring properties.	Very Low
		d) Foamcrest Ave	Unlikely	The event might occur under very adverse circumstances over the design life.	Insignificant	Little Damage, no significant stabilising required or no impact to neighbouring properties.	Very Low

* hazards considered in current condition, without remedial/stabilisation measures and during construction works.

* qualitative expression of likelihood incorporates both frequency analysis estimate and spatial impact probability estimate as per AGS guidelines.

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

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

TABLE: 2

Recommended Maintenance and Inspection Program

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

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

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.