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# GEOTECHNICAL INVESTIGATION REPORT

**for**

**PROPOSED NEW DEVELOPMENT**

**at**

**32 GOLF AVENUE, MONA VALE, NSW**

**Prepared For**

**LAXDTX 2 Pty Ltd**

**Project No.: 2024-004.1**
**July 2024**

## Document Revision Record

| Issue No | Date | Details of Revisions |
|---|---|---|
| 0 | 23rd February 2024 | Original Issue |
| 1 | 1st July 2024 | Updated 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** | 32 Golf Avenue, Mona Vale, NSW |

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

*I,* __***Troy Crozier***__ *on behalf of* ___***Crozier Geotechnical Consultants*** _on this the 1st July 2024_ 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 Development

**Report Date:** 01/07/2024 **Project No.:** 2024-004.1

**Author:** S. Demirbag and T. Crozier

**Author's Company/Organisation:** Crozier Geotechnical Consultants

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

| |
|---|
| Architectural drawings by Walsh Architects, Drawing No.: DA000, DA010, DA020, DA030, DA035, DA040, DA100 – DA104, DA201 – DA204, DA300, DA301, DA400, DA501 – DA503, DA600 – DA602, Revision B, Dated: 22/06/2024. |
| Survey Plan by Bee & Lethbridge, Reference No.: 23104, Drawing No.: 23104-01, Revision 01, Dated: 09/01/2024 |

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

Signature ..........................................................

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

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

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

Company... **Crozier Geotechnical Consultants**

AUSTRALIAN INSTITUTE OF GEOSCIENTISTS
TROY CROZIER
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## 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** __32 Golf Avenue, Mona Vale, NSW ______________

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

**Geotechnical Report Details:**

**Report Title:** Geotechnical Report for Proposed New Development
**Report Date:** 01/07/2024 **Project No.:** 2024-004.1
**Author:** S. Demirbag and T. Crozier
**Author's Company/Organisation:** Crozier Geotechnical Consultants

**Please mark appropriate box**

- ■ Comprehensive site mapping conducted ___15th January 2024________
- ■ 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 January 2024………..
- ■ 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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# TABLE OF CONTENTS

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**GEOTECHNICAL INVESTIGATION FOR PROPOSED NEW DEVELOPMENT**
**32 GOLF AVENUE, MONA VALE, NSW**

## 1. INTRODUCTION:

This report details the results of a geotechnical investigation carried out for proposed new development at 32 Golf Avenue, Mona Vale, NSW. The investigation was undertaken by Crozier Geotechnical Consultants (CGC) at the written request of the client LAXDTX 2 Pty Ltd.

It is understood that the proposed works involve demolition of existing structures and construction of a new two storey residential apartment building over a basement parking level and a service level. Bulk excavation to a maximum of approximately 4.00m depth will be required for the proposed basement level to achieve a finished floor level (FFL) of RL 16.38 and RL 17.125 towards the front south dipping to RL 15.41 towards the rear north. The service level will require excavation to a maximum of approximately 6.20m in an isolated location towards the middle portion of the site.

The site is not located within a geotechnical landslip hazard zone (H1 or H2) as identified within Northern Beaches Councils precinct (Geotechnical Risk Management Policy for Pittwater – 2009 – Map Sheet GTH_018). However, the works trigger the policy in regard to excavation and filling (excavation >1.50m depth or within 1.0m of the boundary/a structure; fill >1.0m deep).

To meet the Councils Policy requirements for works which trigger the landslip policy a detailed Geotechnical Report which meets the requirements of Paragraph 6.5 of that policy is required. This report must include a landslide risk assessment of the site and proposed works, plans, geological sections and provide recommendations for construction and to ensure stability is maintained for a design life of 100 years. It is recommended that the client make themselves aware of the Policy and its requirements.

The original report has been updated to reflect architectural design changes at the request of Walsh Architects.

The investigation and reporting were undertaken as per the Fee Proposal No.: P24-001, Dated: 9th January 2024. The investigation comprised:

a) A detailed geotechnical inspection and mapping of the site and adjacent properties by a Geotechnical Engineer.
b) Dial Before You Dig (DBYD) plan request/review and onsite test location clearance by an accredited service locating sub-contractor.
c) Drilling of three auger boreholes to identify sub-surface geology using a restricted access drill rig along with Dynamic Cone Penetrometer (DCP) tests adjacent to boreholes and at four additional locations.
d) Soil sample collection and logging as per "AS1726: 2017 Geotechnical Site Investigation" and test analysis at NATA accredited chemical laboratories (Envirolab) to determine aggressivity of the ground to concrete and steel.

The following plans and drawings were supplied and were relied on for the proposal, investigation, and preparation of this report:

- Architectural Drawings – Walsh Architects, Drawing No.: DA000, DA010, DA020, DA030, DA035, DA040, DA100 – DA104, DA201 – DA204, DA300, DA301, DA400, DA501 – DA503, DA600 – DA602, Revision B, Dated: 22/06/2024.
- Survey Drawing – Bee and Lethbridge Pty Ltd, Reference No.: 23104, Drawing No.: 23104-01, Sheet 1 of 1, Revision No.: 01, Dated: 09/01/2024.

## 2. PROPOSED WORKS:

It is understood that the proposed works involve demolition of all existing site structures and construction of a new two storey residential apartment building over a basement carparking level with a service level beneath the proposed basement level in an isolated location towards the middle portion of the site. A proposed driveway from the south-west boundary with Golf Avenue provides vehicle access to the basement level.

The proposed development will comprise of Level 1 (RL22.08) and the Ground Floor Level (RL18.68) across a northern and southern structure, with the Basement Floor (RL15.41 – RL17.475) extending across the majority of the block and below both overlying structures. The Service Floor (RL13.39) underlies the basement Floor only within a central portion.

The basement will require bulk excavation to varying depths between 2.30m and 4.00m depth. Increased bulk excavation will be required within central portions for the service Floor to approximately 6.20m depth.

The basement level excavation will be setback from the northern and southern boundaries by approximately a minimum of approximately 4.00m and 6.50m respectively and will be setback from the eastern and western boundaries by a minimum of 2.40m. The driveway gradually reduces to nil at the south-west boundary with Golf Avenue. The service level will be setback from the eastern and western boundaries by 2.60m and 7.60m respectively.

## 3. SITE FEATURES:

### 3.1. Description:

The site (32 Golf Avenue, Mona Vale) (SP57603) is a rectangular shaped block situated on the low north side of Golf Avenue within gentle north-west dipping topography near a ridge crest. The survey indicates ground surface levels within the site extends from a high of approximately RL 20.52 at the front south-eastern corner to a low of approximately RL 17.35 at the rear north-western corner. For the purposes of this report, the front Golf Avenue roadway is referenced as the southern boundary, with the other boundaries referenced accordingly.

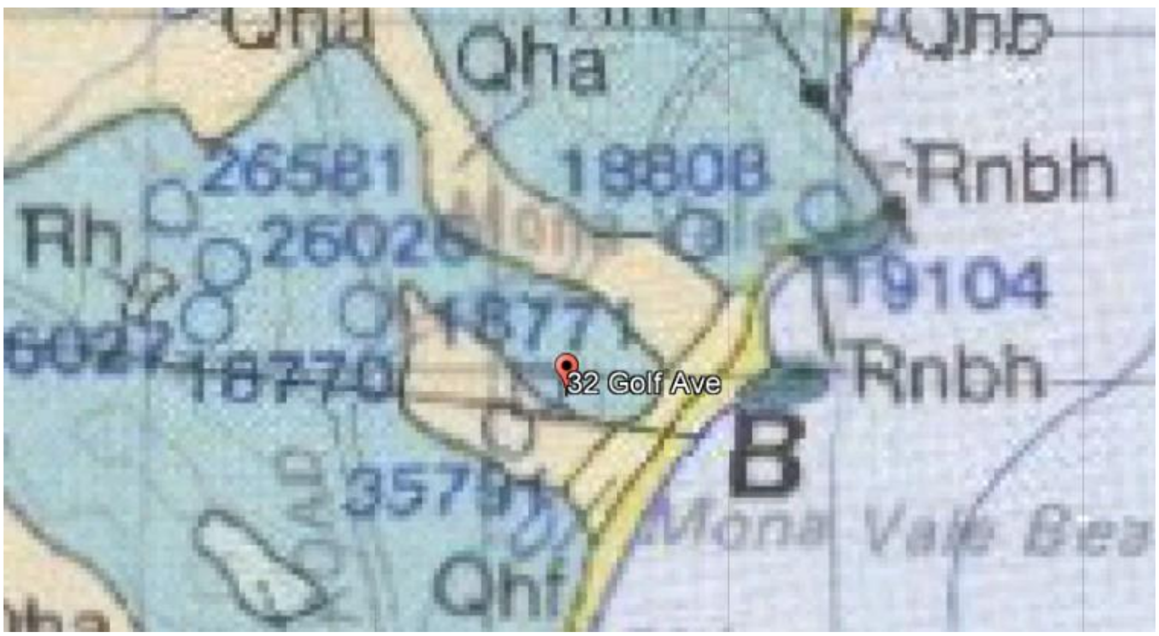
According to the provided survey plan, the site has front southern and rear northern boundaries of 19.81m respectively, whilst the side eastern and western boundaries are 70.41m respectively, covering a site area of approximately 1394m$^2$. An aerial photograph of the site and its surrounds with boundary designations is shown below in Photograph 1, as sourced from NSW Government Six Map Spatial Data.


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

**3.2. Geology:**

Reference to the Sydney 1:100,000 Geological Series sheet (9130) indicates that the site is situated near the boundary of Newport Formation (Upper Narrabeen Group) rocks which are of middle Triassic in age and Quaternary Sands (Qha) which have been deposited during the Holocene Period to the south and north of the site. The Newport Formation typically comprises interbedded laminite, shale and quartz to lithic quartz sandstones and pink clay pellet sandstones whilst the Quaternary Sands deposit typically comprises of silty to peaty quartz sand, silt and clay, ferruginous and humic cementation in places and common shell layers.

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




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

## 4. FIELD WORK:

### 4.1. Methods:

The field investigation comprised a geotechnical inspection, mapping of the site and limited inspection of adjacent properties on 15th January 2024 by a Geotechnical Engineer. It included a photographic record of site conditions as well as geological/geomorphological mapping of the site and adjacent land with examination of existing features and ground conditions.

The subsurface investigation comprised the drilling of three auger boreholes (BH1 – BH3) utilizing a restricted access drill rig operating solid stem, spiral flighted augers and a tungsten carbide blade bit to investigate sub-surface geology and depth to bedrock.

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

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

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

### 4.2. Field Observations:

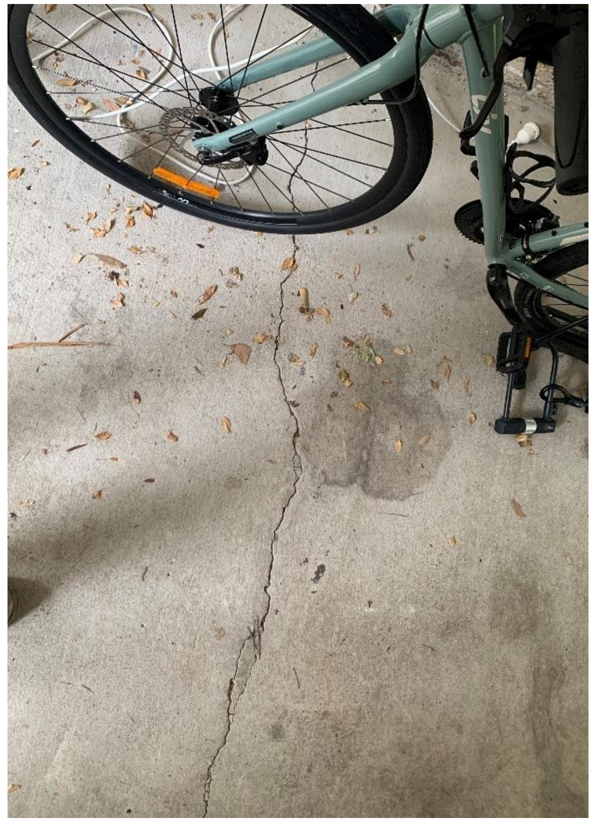
The site is situated on the northern side of Golf Avenue within gentle north-west dipping topography near a ridge crest. Golf Avenue is formed with a bituminous sealed pavement that is gently north-west dipping and is separated from the site by a concrete kerb, driveway, walkway and a grass lawn nature strip. There were no signs of settlement and cracking observed on the roadway to suggest any significant movement or underlying geotechnical issues.

The site contains a series of four connected, masonry townhouses situated broadly towards the middle of the site of anticipated construction age between 33 and 38 years, based on available NSW Government historical imagery. Each townhouse contains a masonry garage connected to the north. A concrete driveway runs along the eastern portion of the site from the front south-east corner extending to the site rear with a brick paved area and garden bed containing vegetation/shrubs situated between the townhouses and the driveway. Each townhouse's rear western portion is separated by a timber paling fence and comprises a grassed lawn area retained by a timber wall of approximately 0.30m height and concrete walkway. A garden area with vegetation and several mature trees is situated along the eastern boundary. Photograph 2 below provides a view of the site front south-east corner.

*Photograph 2: View of the site, facing broadly north from the front south-east corner*

The main structures appeared in reasonable condition with several large vertical cracks observed on all-townhouse exterior walls presumed to be a result of thermal expansion and deterioration over time, shown in Photograph 3. Cracking and settlement was observed on the concrete driveway and concrete floor slab of the garage, shown in Photograph 4. These cracks are presumed to be a result of differential settlement and deterioration over time.

*Photograph 3: Vertical crack within rear external wall*



*Photograph 4: Crack within concrete floor slab of garage*

The neighbouring property to the east (No. 34 - 36 Golf Avenue) contains five detached two-storey rendered apartment complex buildings situated throughout the property of approximate construction age of 18 years, based on available NSW Government historical imagery. A concrete driveway from the front south-west corner provides vehicle access to a below ground level basement garage adjacent to the sites (No. 32) boundary. Concrete footsteps and a rendered retaining wall supporting a garden area is situated adjacent to the driveway, approximately 0.50m from the shared site boundary. The neighbouring basement garage appears to be excavated up to a maximum 4.0m depth below pre-development ground levels. The closest apartment building to the site is situated towards the middle and is setback by approximately 4.0m from the shared timber fence defined boundary which is placed atop a rendered wall, whilst the basement garage level appears to be setback from the shared boundary by approximately 0.50m. The property appears to contain similar ground surface levels to the site along the common boundary with the exception of the basement garage and the neighbouring structures appeared in good condition with no signs of cracking or excessive settlement to indicate any underlying geotechnical concern.

The neighbouring property to the north (No. 33 Darley Street East) contains two modern detached two-storey rendered apartment buildings situated to the front north and rear south respectively of anticipated construction age of 5 years, based on google street view photography during construction. The rear apartment building is setback from the shared timber fence boundary by approximately 6.0m. The timber fence was observed to be leaning and in poor condition. A concrete driveway beginning from the front north-west corner of the property provides vehicle access to a below ground level basement garage which appears to be excavated to approximately 3.50m depth below pre-development ground levels. The remainder of the property is occupied by grass lawns with vegetation and trees. The property appears to contain similar ground surface levels to the site along the common boundary and the apartment buildings appeared in good condition with no visible signs of cracking, ground movement or underlying geotechnical issues.

The neighbouring property to the north-west (No. 35 Darley Street East) contains two detached multi-storey masonry apartment buildings situated to the front north and rear south respectively of anticipated construction age between 42 and 49 years, based on available NSW Government historical imagery. The rear apartment building is setback from the shared timber fence boundary by approximately 10.0m. A concrete driveway extends along the property's western boundary from the front north-west corner, extending to two carports towards the middle and the rear of the property and a swimming pool is situated towards the middle with the remainder occupied by grass lawns and vegetation. The property appears to contain similar ground surface levels to the site along the common boundary and the apartment buildings appeared in good condition with no visible signs of cracking, ground movement or underlying geotechnical issues.

The neighbouring property to the west (No. 28 – 30 Golf Avenue) contains three detached two storey rendered masonry apartment buildings situated towards the front, middle and rear portions of the property over a basement carpark level of anticipated construction age between 20 and 26 years, based on available NSW Government historical imagery. A concrete driveway from the front south-west corner provides vehicle access to the below ground level basement garage. A concrete block aboveground stormwater detention tank of 1.0m width is situated at the south-western boundary. The closest apartment building is setback from the shared timber boundary fence by approximately 4.0m. The neighbouring basement garage appears to be excavated up to a maximum 4.0m depth below pre-development ground levels. The property appears to contain similar ground surface levels to the site along the common boundary with the exception of the basement garage and the buildings appeared in good condition with no signs of cracking or excessive settlement to indicate any underlying geotechnical concern.

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

### 4.3. Ground Conditions:

Three boreholes (BH1 – BH3) were drilled across site and within the vicinity of the proposed works with BH1 drilled within the front garden bed, BH2 drilled towards the middle of the site with the brick paved area, BH3 drilled within the northern rear. All boreholes extended through a relatively shallow layer of topsoil/fill and subsequent intersection of natural clay interpreted as residual soils before encountering refusal atop shale/siltstone bedrock at depths ranging between 3.90m (BH1) and 4.70m (BH2 and BH3).

Dynamic Cone Penetrometer (DCP) testing were undertaken from ground surface level or beneath the brick pavement adjacent to the boreholes and at four additional locations. Solid or effective refusal was encountered within interpreted hard clays at varying depths of 0.95m (DCP4) and 2.20m (DCP3 and DCP7).

For a description of the ground conditions encountered at the borehole/DCP test locations, the Borehole Log and DCP results sheets should be consulted however a very broad summary of the subsurface conditions encountered is provided below.

- **TOPSOIL/FILL** – This layer was encountered in all boreholes and extended from ground surface or underlying a brick pavement to a maximum depth of 0.30m (BH1). The layer generally comprised a brown clayey/silty sand with organic matter, rootlets and gravels encountered within BH3.
- **RESIDUAL SOILS –** This layer was encountered underlying the topsoil/fill in all locations from a minimum of 0.17m depth (BH2) and extended to a maximum depth of 4.70m (BH2 and BH3). The material was initially encountered within BH1 and BH2 as pale brown to pale grey, medium to high plasticity, moist, near plastic limit clay with ironstone gravels. The deposit graded with depth to a mottled red to pale grey clay with trace silt and subsequently friable, dry of plastic limit clay with an increase in silt content. Clayey silt soils were encountered from 1.50m depth within BH3 and 4.0m depth within BH2. The strata was generally initially encountered as firm and quickly transitioned to hard clay from a minimum and maximum depth of 0.60m (DCP4) and 1.70m (DCP5) respectively.
- **SHALE/SILTSTONE BEDROCK** – Bedrock of at least low strength was interpreted via drill rig refusal within all boreholes at depths ranging between 3.90m (BH1) and 4.70m (BH2 and BH3). Interpreted extremely weathered material was interpreted via DCP refusal from a minimum and maximum depth of 0.95m (DCP4) and 2.20m (DCP3 and DCP7) respectively.

A freestanding ground water table or signs of significant water seepage was not identified within the boreholes, or on any of the returned DCP rods to the base excavation level at RL 13.30.

## 5. LABORATORY TESTING:

Chemical testing has been undertaken at a NATA accredited Chemical laboratory and the results are summarised and discussed in the following section. The laboratory test Report Sheet and Certificate of Analysis are included in Appendix: 3

Of the soil samples collected, representative disturbed soil samples were kept on ice and transported to NATA accredited laboratory (Envirolab) under standard chain of custody protocol for testing to determine aggressivity of the ground to concrete and steel.

It should be noted that the sample descriptions provided on the summary tables are individual laboratory sample descriptions. No allowance has been made in the sample descriptions for sampling, sub sampling or test methodology. The mass material properties are provided on the Borehole Logs, as such the laboratory test results should be read in conjunction with the relevant borehole log.

### 5.1 Corrosion Potential:

Three soil samples recovered from the boreholes were tested to determine the corrosion potential of the site soils to provide durability classification for new steel and concrete structures as per AS2159. The reported results are summarised below in Table 1 and Certificate of Analysis is attached.

*Table 1: Summary of Envirolab Aggressivity Laboratory Test Results*

| Location | Depth (m) | pH | Electrical Conductivity (µS/cm) | Resistivity* (ohm.m) | Chloride Cl (mg/kg) | Sulphate SO4 (mg/kg) |
|---|---|---|---|---|---|---|
| BH1 | 2.7 – 3.0 | 5.5 | 40 | 250 | <10 | 25 |
| BH2 | 3.8 – 4.0 | 5.6 | 56 | 178.57 | <10 | 42 |
| BH3 | 4.5 – 4.6 | 5.7 | 72 | 138.89 | 22 | 48 |

*By calculation

The results of the soil chemical testing undertaken on the soil samples were compared against the Australian Standard AS 2159 – 2009 Pile Design and Installation.

The results were compared against Table 6.4.2 (C) Exposure Classification for Concrete Piles – Piles in Soil. The results indicate that the soils are '**non-aggressive'** to concrete from pH, chlorides and sulphate.

The results were also compared against Table 6.5.2 (C) Exposure Classification for Steel Piles – Piles in Soil. The results indicate that the soil is '**non-aggressive**' to steel with regard to pH, chlorides, and sulphate.

## 6. COMMENTS:

### 6.1. Geotechnical Assessment:

The site investigation identified a shallow layer of topsoil/fill overlying residual clayey soil deposits grading to extremely weathered material and subsequently shale/siltstone bedrock of at least low strength at depths ranging from 3.90m (BH1) and 4.70m (BH2 and BH3). The shale/siltstone bedrock has the potential to vary between extremely weathered and medium strength based on banding, which can only be confirmed through core drilling. The residual clays were initially encountered as firm within BH1 and BH3 and very stiff within BH2 and graded with depth to hard from a maximum of 1.70m (DCP5). A free-standing ground water table or significant seepage was not identified within the borehole, observed on the DCP rods on retrieval or in adjacent excavations to a base level of RL 13.30.

It is understood that the proposed works involve the demolition of all existing site structures for the construction of a new two storey residential apartment building over a basement carparking level and a service level. The proposed basement level will require bulk excavation below existing ground surface levels to a maximum of approximately 4.00m depth (RL 16.38) towards the front south, gradually reducing to 2.30m depth (RL 15.41) towards the rear north. The proposed service level will require maximum bulk excavation to approximately 6.20m depth (RL 13.39) in an isolated central location. The basement level excavation will be setback from the northern and southern boundaries by a minimum of approximately 4.00m and 6.50m respectively and will be setback from the eastern and western boundaries by a minimum of 2.40m respectively. The service level will be setback from the eastern and western boundaries by approximately 2.60m and 7.60m respectively.

Based on the results of the investigation, the excavation will extend through a relatively shallow layer of topsoil/fill before intersecting residual clay soils to potential extremely weathered material for the entirety of the basement level. The service level will extend through the residual clay soils and subsequently intersect shale/siltstone bedrock. The bedrock is anticipated to be low strength and highly variable initially however it is anticipated to grade to medium strength and potentially high strength with depth.

Based on the proposed excavation depths and setbacks as well as the depths of the surficial soils overlying the bedrock across site, safe batter slopes as detailed in Section 6.3 with regard to neighbouring property boundaries and the existing Sydney Water asset do not appear feasible along the northern, eastern and western excavation edges for the basement level and the eastern boundary only for the service level. However, continuous batter slopes >3.0m in height are not recommended on this site.

Pre-excavation support will be required where safe batter slopes are unachievable and may be considered where poor-quality bedrock is encountered. Pre-excavation support may consist of soldier pile (or similar) shoring walls. Pre-excavation support will need to be installed and taken through any soils and founded within competent bedrock or below the base levels of excavations. Careful control of pile drilling/support installation is required to avoid over excavation whilst all gaps in the wall must be sealed during excavation to prevent erosion between piles where supporting soil.

The actual strength and quality of the bedrock in the proposed excavation location is unconfirmed and requires core drilling, which will better define safe batter conditions and footing bearing capacities. Bedrock of at least low to medium strength can be excavated at steep to vertical batter slopes provided it is unfractured by the excavation works and does not contain unfavorable defects. Where these are encountered then support systems (i.e. rock bolts/shotcrete) can be implemented as excavation works progress.

There were limited stability hazards identified in the investigation. However, there is a potential for poorly oriented defects in both residual soils or weathered bedrock or localized zones of highly weathered bedrock (particularly near the upper surface) in the proposed excavation to result in localized rockslide/topple failure with potential impact to the site however there is very low potential for impact to properties adjacent. Installation of pre-excavation support systems will prevent major stability hazards.

The fill/soil and extremely weathered to very low strength bedrock can be excavated using conventional earthmoving equipment (e.g. buckets and rippers) which appears possible for the entire basement level and part of the service level. If excavation of low up to high strength rock is required, this will require the use of rock excavation equipment which can produce ground vibrations of a level which can potentially cause damage to neighbouring structures. Therefore, selection of suitable equipment and a sensible methodology are critical. The need for full time vibration monitoring will be determined based upon the type of rock excavation equipment proposed for use. Crozier Geotechnical Consultants should be consulted for assessment of the proposed equipment prior to its use. It is recommended that a rock saw and small (≤250kg) rock hammers be proposed for use at this site to avoid the need for full time monitoring. Larger rock hammers may be preferred and if utilised, further assessment and potentially full-time monitoring would be necessary.

Based on the expectation of excavation through bedrock for the lower part of the Service Level and the bedrock surface identified <0.50m depth below the proposed basement floor level towards the southern front, it is recommended that all new footings for the service and basement levels be founded onto competent shale/siltstone bedrock of at least low strength to minimise the risk of differential settlement. Preliminary allowable bearing pressures appropriate for the bedrock encountered underlying the site are provided in Section 6.3.1. The confirmation of consistent low strength bedrock below auger refusal depths will require core drilling.

A free-standing ground water table or significant seepage was not identified within the boreholes or observed on the DCP rods on retrieval to RL 13.30. As such, the water table will not be impacted by the proposed development which extends to a base excavation level of RL 13.39. Whilst seepage appears limited this could increase during/following rainfall. As the basement and service level are excavated into the site, they will accumulate seepage which will require control/removal unless a tanked structure is proposed.

The site is situated within ‘Class 5’ Acid Sulfate Soils hazard zone, however it is not anticipated based on excavation depths and investigation results to intersect acid sulfate soils or result in any lowering or impacting of the water table in adjacent Class 1, 2, 3 or 4 land (located >150m away). The preliminary assessment of the proposed works prepared in accordance with the Acid Sulfate Soils Manual identifies that an acid sulfate soils management plan is not required for the proposed works.

It is understood that a Sydney Water (SW) asset underlies the site across the rear northern side of the property. CGC has not undertaken any investigation into the construction/type/depth etc. of the asset however the BYDA plans indicate it as comprising a connected 225mm diameter Vitrified Clay pipe. The manholes within adjacent properties indicates the pipe invert is located at approximately 1.10m depth  Based on both the asset depth and scope of proposed works which will intersect the 45° influence zone of the sewer invert, as well as Sydney Water’s “Technical Guidelines for Building Over and Adjacent to Pipe Assets, 2015”, it is anticipated that a Specialist Engineering Assessment (SEA) will be required for the Construction Certificate (CC) phase of the proposed works.

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

The recommendations and conclusions in this report are based on an investigation utilising only surface observations and hand tools. This test equipment provides limited data from small isolated test points across the entire site with limited penetration into rock. Therefore, some minor variation to the interpreted sub-surface conditions is possible, especially between test locations. However, the results of the investigation provide a reasonable basis for the preliminary design of the proposed works.

### 6.2. Site Specific Risk Assessment:

Based on our site investigation and the proposed works, it is considered that the following geological/geotechnical landslip hazard which needs to be considered in relation to the existing site and the proposed works. The hazards are:

A. Landslip (earth slide <10m$^3$) from soils
B. Rockslide/topple (<3m$^3$) of bedrock around perimeter of excavation due to poorly oriented defects.

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

**Hazard A** was estimated to provide a **Risk to Life** of up to **2.11 x 10$^{-7}$** and Risk to Property of up to **'Low'** where unsuitable excavation methodologies are implemented as part of the excavation works.

**Hazard B** was estimated to provide a **Risk to Life** of up to **2.11 x 10$^{-8}$** and **Risk to Property** of up to **'Low'** where unsuitable excavation methodologies are implemented as part of the excavation works.

The risk to life and risk to property for both Hazards A and B was therefore generally considered to be 'Acceptable' when assessed via the criteria of the AGS 2007 against the Geotechnical Risk Management Policy for Pittwater - 2009. As such the project is considered suitable for the site provided the recommendations of this report are implemented.

### 6.3. Design & Construction Recommendations:

Design and the construction recommendations are tabulated below:

| **6.3.1. New Footings:** | |
|---|---|
| Site Classification as per AS2870 – 2011 for new footing design | Class ‘S’ for footings founded within clay<br>Class ‘A’ for footings on bedrock at excavation base |
| Type of Footing | Strip/Pad or Piles |
| Sub-grade material and Maximum Allowable Bearing Capacity | - Stiff Clay: 100kPa<br>- Very Stiff Clay: 200kPa<br>- Hard Clay: 400kPa<br>- Very Low Strength Shale/Siltstone: 800kPa<br>- Low Strength Shale/Siltstone: 1000kPa* |
| Site sub-soil classification as per *Structural design actions AS1170.4 – 2007, Part 4: Earthquake actions in Australia* | $C_e$ – Shallow soil site<br>The hazard factor (z) for Sydney is 0.08. |
| **Remarks:**<br>*Higher bearing pressures require further geotechnical testing including cored boreholes<br>All footings for each structure should be founded off material of similar strength to prevent differential settlement. All new footings must be inspected by an experienced geotechnical professional before concrete or steel are placed to verify their bearing capacity and the in-situ nature of the founding strata. This is mandatory to allow them to be ‘certified’ at the end of the project. | |

| **6.3.2. Excavation:** | |
|---|---|
| Depth of Excavation | Up to 4.00m depth for the Basement Level, increased to 6.20m for Service Level |
| **Property Separation:**<br>The table below shows the properties potentially affected by the proposed excavation, excavation depth and the separation distances to the shared property boundary/structure.<br>*Table 1: Property separation distances* | |

| Boundary | Property | Structure | Bulk Excavation Depth (m bgl) | Separation Distances (m)* | |
|---|---|---|---|---|---|
| | | | | Boundary | Structure |
| East | No. 34-36 Golf Avenue | Apartment buildings, Driveway/ Footsteps, Retaining wall, Basement level garage | 4.00m for Basement Level,<br>6.20m for Service Level | 2.40m to 2.60m | 0.50m for the garage, driveway/footsteps and retaining wall<br>4.0m for the apartment building |

| | | | | | |
|---|---|---|---|---|---|
| North | No. 33 Darley Street East | Apartment building | 3.00m for Basement Level | 4.00m to 5.40m | Apartment building a further 6.0m |
| North-West | No. 35 Darley Street East | Apartment building | 2.30m for Basement Level | 4.20m | Apartment building a further 10.0m |
| West | No. 28-30 Golf Avenue | Apartment building, Concrete stormwater detention tank | 3.0m for Basement Level,<br>5.80m for Service Level | 2.40m to 2.60m for Basement Level,<br>7.60m for Service Level | 4.0m for the apartment building, Stormwater tank directly adjacent to boundary |
| South | Golf Avenue | Concrete footpath and driveway, Roadway | 4.00m for Basement Level | 6.50m | Footpath a further 2.0m, Roadway a further 3.0m |

| | | |
|---|---|---|
| Type of Material to be Excavated | Up to 4.70m for the Service Level | Topsoil/fill and residual clay soils |
| | From minimum of 3.90m (BH1) | Sandstone bedrock – LS – MS, potentially HS |

Guidelines for <u>un-surcharged</u> batter slopes for this site are tabulated below:

| Material | Safe Batter Slope (H:V) | |
|---|---|---|
| | Short Term/Temporary | Long Term/Permanent |
| Topsoil/Fill | 1.5:1 | 2.0:1 |
| Clay to extremely weathered bedrock | 1.0:1 | 2.0:1 |
| Very Low (VLS) strength or fractured bedrock | 0.75:1 | 0.5:1* |
| Low to medium strength (LS - MS), defect free bedrock | Vertical* | Vertical* |

*Dependent on defects and assessment by engineering geologist.

**Remarks**: Seepage at the bedrock surface or along defects in the rock can also reduce the stability of batter slopes or rock cuts 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 permanent support measures are installed. This should also be considered with respect to safe working conditions. Batter slopes should not be left unsupported without geotechnical inspection and approval.

| | | |
|---|---|---|
| Equipment for Excavation | Fill/ Clayey soils | Excavator with bucket or hand tools |
| | VLS bedrock | Excavator with bucket and ripper |
| | LS – MS/HS bedrock | Rock hammer and rock saw |

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

**Remarks**:

Rock sawing of the hard rock excavation perimeters is recommended as it has several advantages. It often reduces the need for rock bolting as the cut faces generally remain more stable and require a lower level of rock support than hammer cut excavations, ground vibrations from rock saws are minimal and the saw cuts will provide a slight increase in buffer distance for use of rock hammers. It also reduces deflection across boundary of detached sections of bedrock near surface.

Based on previous testing of ground vibrations created by various rock excavation equipment within medium strength Hawkesbury Sandstone bedrock, to achieve a low level of vibration (3mm/s PPV) the below hammer weights and buffer distances are generally required:

| Maximum Hammer Weight | Required Buffer Distance from Structure |
|---|---|
| 300kg | 2.00m |
| 400kg | 3.00m |
| 600kg | 6.00m |
| ≥1 tonne | Up to 20.00m |

Onsite calibration and full-time vibration monitoring will provide accurate vibration levels to the site specific conditions and will generally allow for larger excavation machinery or smaller buffers to be used. Inspection of equipment and review of dilapidation surveys and excavation location is necessary to determine need for full time monitoring. Where monitoring is determined as necessary then it should be maintained directly between the excavation activity and the structure being monitored, as such the monitor may require relocation during excavation.

| | |
|---|---|
| Recommended Vibration Limits<br>(Maximum Peak Particle Velocity (PPV)) | Neighbouring residential dwellings = 5mm/s within 10m of excavation<br>Services = 3mm/s |
| Vibration Calibration Tests Required | If larger scale (i.e. rock hammer >250kg) excavation equipment is proposed |
| Full time vibration Monitoring Required | Pending proposed excavation equipment and vibration calibration testing results, if required |
| Geotechnical Inspection Requirement | Yes, recommended that these inspections be undertaken as per below mentioned sequence:<br>• During installation of excavation support<br>• At completion of basement level excavation<br>• At completion of the excavation<br>• Where ground conditions are exposed that differ to those expected |
| Dilapidation Surveys Requirement | Recommended on neighbouring structures or parts thereof within 10m of the excavation perimeter prior to site work |

| | |
|---|---|
| | to allow assessment of the recommended vibration limit and protect the client against spurious claims of damage. |
| **Remarks**:<br>Water ingress into exposed excavations can result in erosion and stability concerns in both soil and rock portions. Drainage measures will need to be in place during excavation works to divert any surface flow away from the excavation crest and any batter slope. | |

| **6.3.3. Retaining Structures:** | |
|---|---|
| Required | New retaining structures/excavation support wall will be required as part of the proposed development |
| Types | Pre-excavation bored pile wall where temporary batters are unachievable. Steel reinforced concrete/concrete block post excavation where batters possible. Designed in accordance with Australian Standards AS4678-2002 Earth Retaining Structures. Based on the separation distances to boundaries and adjacent structures, minor support wall deflection (i.e. ≤10-15mm) should have very limited potential to impact adjacent residential structures. |

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

| Material | Unit Weight (kN/m3) | Long Term (Drained) | Earth Pressure Coefficients | | Passive Earth Pressure Coefficient * | Modulus E (MPa) |
|---|---|---|---|---|---|---|
| | | | Active (Ka) | At Rest ($K_0$) | | |
| Topsoil/Fill | 18 | $\phi' = 30°$ | N/A | 0.5 | N/A | |
| Clay (firm to stiff) | 20 | $\phi' = 30°$ | 0.33 | 0.50 | N/A | 25 |
| Clay (very stiff to hard) | 22 | $\phi' = 35°$ | 0.27 | 0.40 | 3.75 | 40 |
| VLS - LS bedrock | 23 | $\phi' = 38°$ | 0.10 | 0.15 | 300kPa | 200 |

**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 utilise active earth pressure coefficients (Ka). |

| **6.3.4. Drainage and Hydrogeology** | | |
|---|---|---|
| Groundwater Table or Seepage identified in Investigation | | No |
| Excavation likely to intersect | Water Table | No |
| | Seepage | Minor (<1L/min per metre perimeter), at soil interfaces and defects |
| Site Location and Topography | | The site is located on the low north-eastern side of the road within gentle north-west dipping topography near a ridge crest |
| Impact of development on local hydrogeology | | Negligible |
| Onsite Stormwater Disposal | | Not possible |
| **Remarks**:<br>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. Ongoing seepage collection and discharge will be required unless a tanked basement and service level is implemented.<br>Trenches, as well as all new building gutters, downpipes and stormwater intercept trenches should be connected to a stormwater system designed by a Hydraulic Engineer which discharges to the Council's stormwater system off site. | | |

### 6.4. Conditions Relating to Design and Construction Monitoring:

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

1. Review and approve the structural design drawings for compliance with the recommendations of this report prior to construction,
2. Inspection of site and works as per Section 6.3 of this report
3. Inspect all new footings and earthworks to confirm compliance to design assumptions with respect to allowable bearing pressure, basal cleanness and the stability prior to the placement of steel or concrete,

4. Inspect completed works to ensure construction activity has not created any new hazards and that all retention and stormwater control systems are completed.

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

**7. CONCLUSION:**

The site investigation identified the presence of a shallow layer of topsoil/fill overlying residual clayey soils and subsequently shale/siltstone bedrock, encountered at minimum and maximum depths of 3.90m (BH1) and 4.70m (BH2 and BH3). No freestanding groundwater table was encountered within the investigation and is not expected within the envelope of proposed works, however, minor and seasonally variable seepage at the geological interfaces may be encountered in open excavations.

The proposed works involve the demolition of all existing site structures for the construction of a new two storey residential apartment building over a basement carparking level and a service level. The proposed basement level will require bulk excavation below existing ground surface levels to approximately 4.00m depth towards the front south reducing to 2.30m to 3.00m depth towards the rear north. The proposed service level will require maximum bulk excavation to approximately 6.20m depth in an isolated location towards the middle.

The proposed excavation for the basement level is anticipated to extend through a shallow layer of topsoil/fill before intersecting residual clay for the remainder. The service level will extend through the residual clay soils and subsequently intersect shale/siltstone bedrock of at least low strength which is anticipated to grade to medium strength and potentially high strength with depth and hard rock excavation is likely to be required for the service level. Safe batter slopes will not be achievable within the confines of the site along multiple excavation edges, triggering the requirement of pre-excavation support in the form of a soldier pile wall or similar.

It is recommended that all new footings extend through any topsoil/fill and residual clays encountered and be founded on competent bedrock of at least very low strength to avoid variable settlement within the new structure.

The Sydney Water asset within the rear northern side of the site running through the property may require a Specialist Engineering Assessment (SEA) prior to site works, it is recommended that Sydney Water be contacted as soon as possible to confirm requirements.

The soils intersected in the investigation did not exhibit any characteristics inherent to Acid Sulfate Soils. Additionally, the water table will not be intersected within the envelope of proposed works, therefore there will be no requirement for dewatering or tanking and hence no alterations of water table depths in adjacent properties and Acid Sulfate Soils hazard classes. Therefore, a detailed ASS management plan is not required for this proposed development.

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

| Prepared By: | Reviewed By: |
|---|---|
| SD | |
| Sores Demirbag | Troy Crozier |
| Geotechnical Engineer | Principal |
| B.E. (Hons.) Civil | MIEAust., CPEng |
| | MAIG, RPGeo |
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**8. REFERENCES:**


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2. C. W. Fetter 1995, "Applied Hydrology" by Prentice Hall. V. Gardiner & R. Dackombe 1983, "Geomorphological Field Manual" by George Allen & Unwin
3. Australian Standard AS 2870 – 2011, Residential Slabs and Footings
4. Australian Standard AS 1726 – 2017, Geotechnical Site Investigations
5. Australian Standard AS 1289 – 2000, Method of Testing Soils for Engineering Purposes
6. Australian Standards AS4678-2002, Earth Retaining Structures

CROZIER
GEOTECHNICAL CONSULTANTS

# 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: -

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

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

$Q_c$ = (12 to 18) $C_u$

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

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

## Dynamic Penetrometers

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

Two relatively similar tests are used.

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

## Laboratory Testing

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

## Borehole Logs

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

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

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

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

## Ground Water

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

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

## Engineering Reports

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

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

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

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

### Site Anomalies

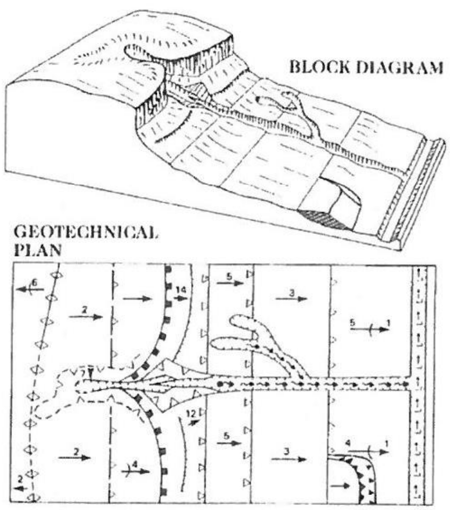
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed than at some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

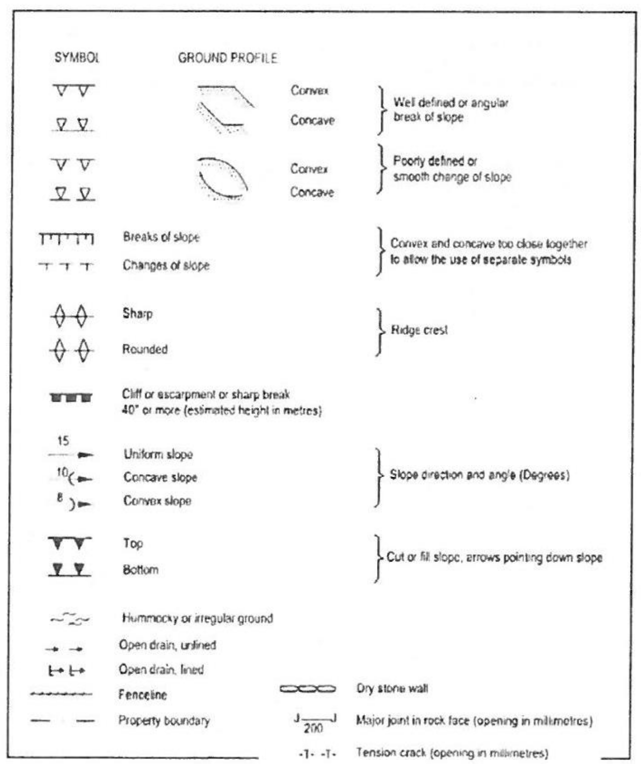
### Reproduction of Information for Contractual Purposes

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

### Site Inspection

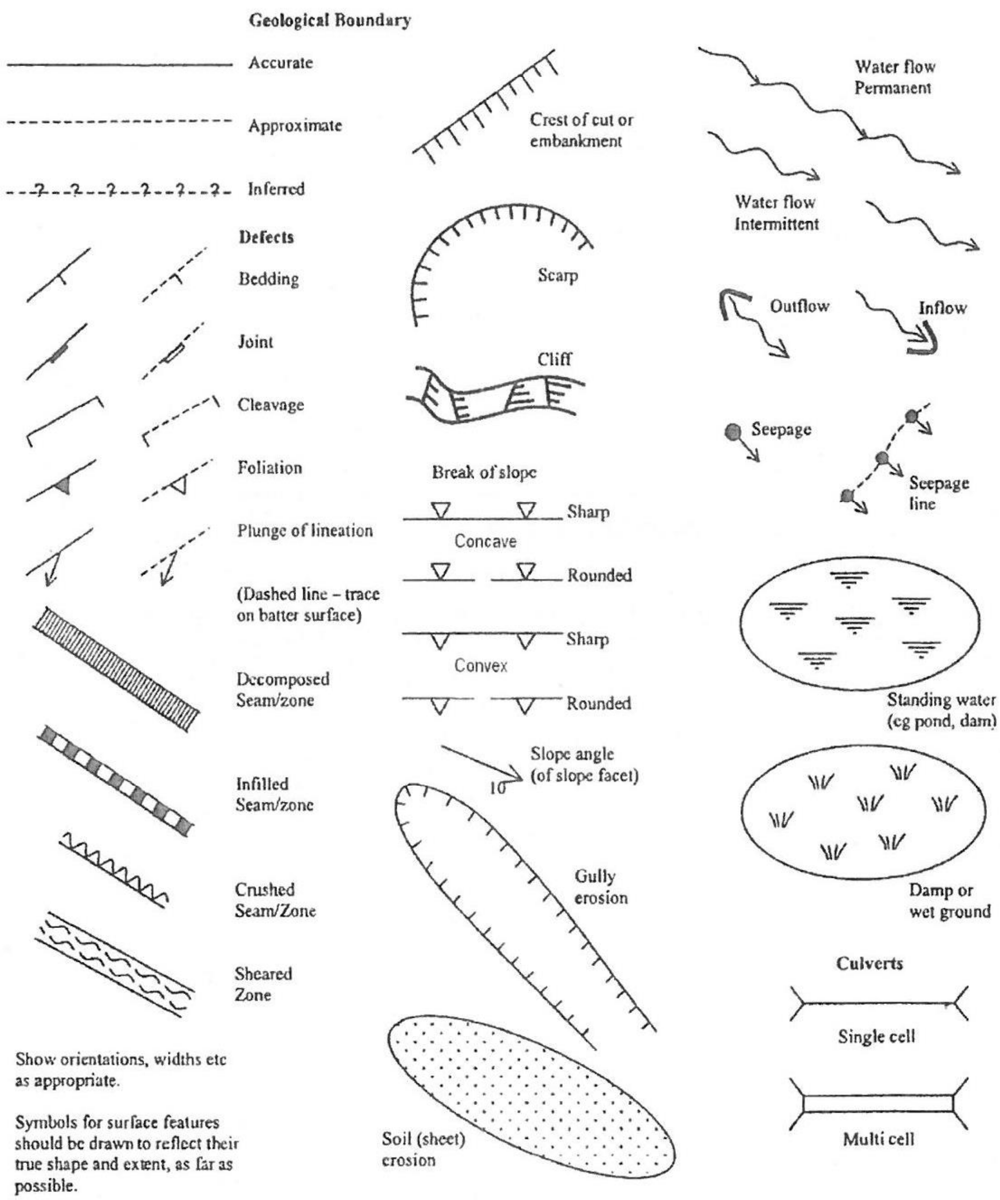
The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.




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

# APPENDIX E - GEOLOGICAL AND GEOMORPHOLOGICAL MAPPING SYMBOLS AND TERMINOLOGY




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

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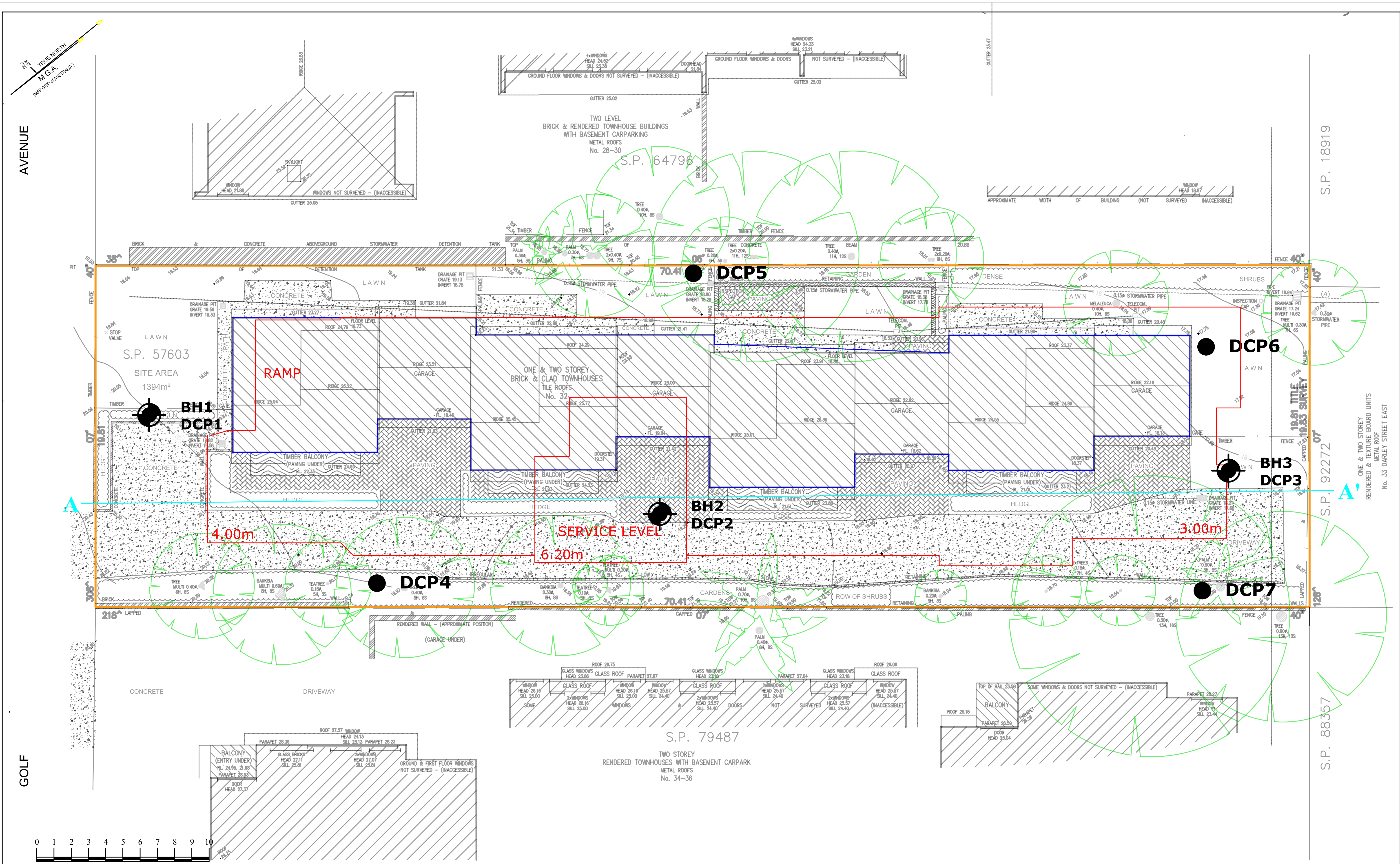
# Appendix 2




SITE PLAN & TEST LOCATIONS FIGURE 1.

(A) EASEMENT TO DRAIN WATER (0.9 WIDE) X131411

**LEGEND**

| | | | | |
|---|---|---|---|---|
| BH DCP — AUGER / DYNAMIC CONE PENETROMETER LOCATION | DCP — DYNAMIC CONE PENETROMETER | EXISTING STRUCTURES | PROPOSED EXCAVATION AND APPROXIMATE DEPTHS / PROPERTY BOUNDARY | A — A' SECTION LINE |

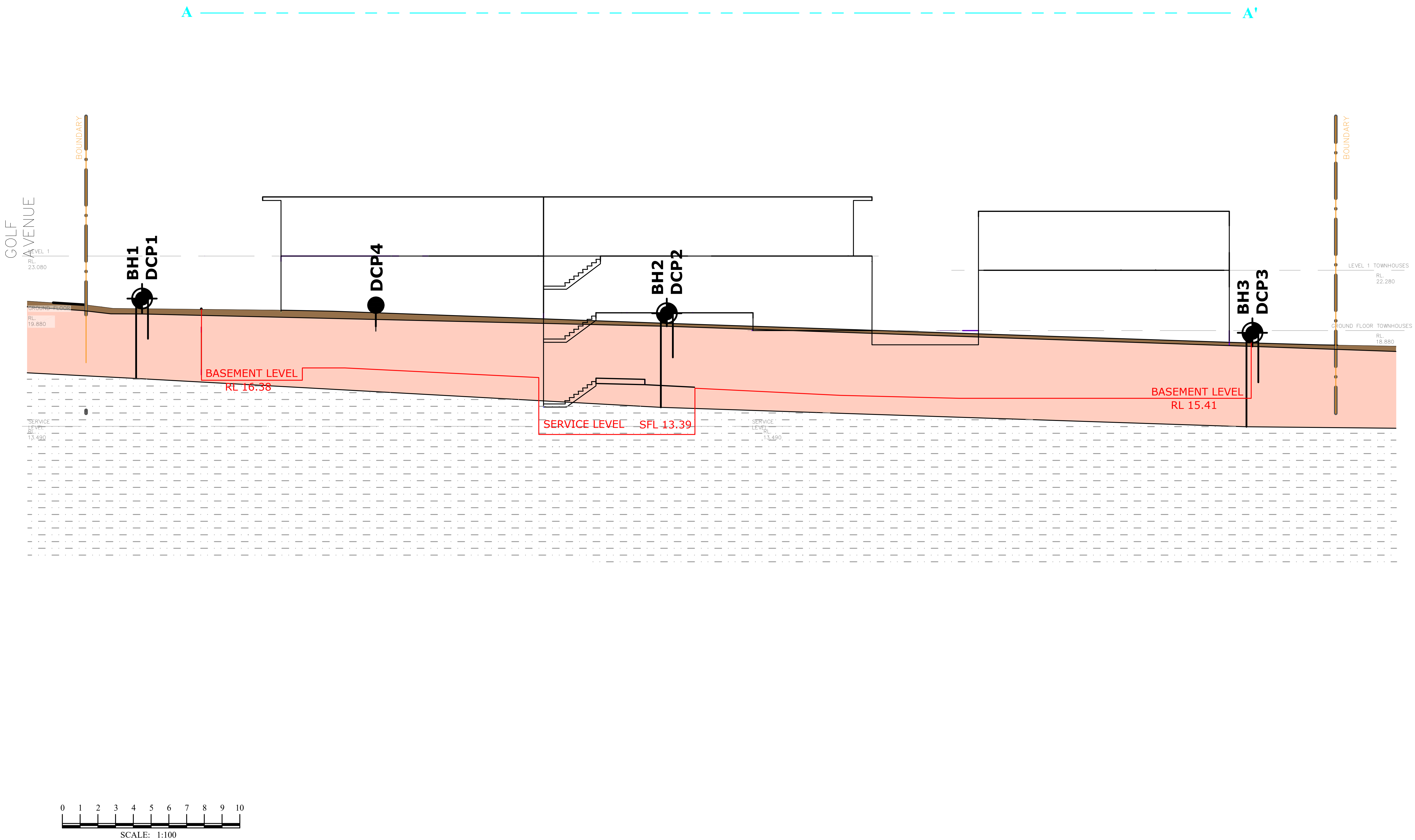
CROZIER GEOTECHNICAL CONSULTANTS
Crozier Geotechnical ABN: 96 113 453 624
Unit 12, 42-46 Wattle Road Phone: (02) 9939 1882
Brookvale NSW 2100 Fax: (02) 9939 1883
*Crozier Geotechnical is a division of PJC Geo-Engineering Pty Ltd*

SCALE: 1:100 @ A1
DRAWING: FIGURE 1
DATE: 07/2024

PREPARED FOR:
LAXDTX 2 Pty Ltd

APPROVED BY: TMC
DRAWN BY: SD
PROJECT: 2024-004

ADDRESS:
32 Golf Avenue, Mona Vale, NSW




SITE SECTION FIGURE 2.

CROZIER GEOTECHNICAL CONSULTANTS

Crozier Geotechnical
Unit 12, 42-46 Wattle Road
Brookvale NSW 2100
ABN: 96 113 453 624
Phone: (02) 9939 1882
Fax: (02) 9939 1883

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

**LEGEND**

| | |
|---|---|
| BH DCP | AUGER / DYNAMIC CONE PENETROMETER LOCATION |
| DCP | DYNAMIC CONE PENETROMETER |
| | PROPERTY BOUNDARY |
| | PROPOSED EXCAVATION |
| | TOPSOIL/FILL |
| | RESIDUAL SOIL |
| | SHALE/SILTSTONE BEDROCK |
| A —— A' | SECTION LINE |

| | |
|---|---|
| SCALE: | 1:100 @ A1 |
| DRAWING: | FIGURE 2 |
| DATE: | 07/2024 |
| APPROVED BY: | TMC |
| DRAWN BY: | SD |
| PROJECT: | 2024-004 |

PREPARED FOR:
LAXDTX 2 Pty Ltd

ADDRESS:
32 Golf Avenue, Mona Vale, NSW

# BOREHOLE LOG

**CLIENT:** LAXDTX 2 Pty Ltd **DATE:** 15/01/2024 **BORE No.:** 1

**PROJECT:** New Residential Apartment Building **PROJECT No.:** 2024-004.1 **SHEET:** 1 of 1

**LOCATION:** 32 Golf Avenue, Mona Vale **SURFACE LEVEL:** RL 20.0

| Depth (m) | Classification | Description of Strata<br>**PRIMARY SOIL** - consistency / density, colour, grainsize or plasticity, moisture condition, soil type and secondary constituents, other remarks | Sampling Type | Sampling Tests | In Situ Testing Type | In Situ Testing Results |
|---|---|---|---|---|---|---|
| 0.00 | | FILL: Soft, brown with orange mottle, moist clay | | | | |
| 0.30 | CI/CH | CLAY: Firm, pale brown, medium to high plasticity, moist, near plastic limit clay | | | | |
| 0.50 | | … stiff | | | | |
| 0.70 | | … becoming very stiff, red to pale grey mottled clay with iron stone gravels | | | | |
| 1.00 | | … hard | | | | |
| 1.30 | | … becoming moist, dry of plastic limit, friable with iron stone gravels | | | | |
| 2.00 | | | | | | |
| 2.70 | | | D | 2.70 | | |
| 3.00 | CL/CI | … becoming pale grey to red mottle silty clay (extremely weathered sandstone) | | 3.00 | | |
| 3.90 | | AUGER REFUSAL at 3.90m depth on interpreted very low strength siltstone bedrock | | | | |
| 4.00 | | | | | | |

RIG: K9-4 Dingo mini-digger with Ezi-probe drill mast DRILLER: A.C

METHOD: Solid stem spiral flighted auger with tungsten carbide bit LOGGED: S.D

GROUND WATER OBSERVATIONS: Not Encountered

REMARKS: N/A CHECKED: T.M.C

# BOREHOLE LOG

**CLIENT:** LAXDTX 2 Pty Ltd **DATE:** 15/01/2024 **BORE No.:** 2

**PROJECT:** New Residential Apartment Building **PROJECT No.:** 2024-004.1 **SHEET:** 1 of 1

**LOCATION:** 32 Golf Avenue, Mona Vale **SURFACE LEVEL:** RL 19.25

| Depth (m) | Classification | Description of Strata<br>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 | | BRICK PAVEMENT (70mm) | | | | |
| 0.17 | | FILL: Builders/Pavement sand | | | | |
| | CI/CH | CLAY: Very stiff, pale brown to pale grey, medium to high plasticity, moist, near plastic limit with small ironstone gravels trace silt | | | | |
| 0.80 | | … with red mottle | | | | |
| 0.90 | | … hard | | | | |
| 1.00 | | … becoming reddish brown to pale grey | | | | |
| 1.20 | | … becoming pale grey, trace red with increase in silt content | | | | |
| 1.50 | CI | … hard, pale grey to reddish brown, medium plasticity, mosit, near plastic limit silty clay | | | | |
| 2.00 | | | | | | |
| 3.00 | | … becoming pale grey to pale brown | | | | |
| | | | | 3.80 | | |
| 4.00 | | … becoming brown clayey silt, trace siltstone | D | 4.00 | | |
| 4.60 | | … becoming reddish brown to pale grey with trace siltstone | | | | |
| 4.70 | | AUGER REFUAL at 4.70m depth on interpreted very low strength siltstone | | | | |
| 5.00 | | | | | | |

RIG: K9-4 Dingo mini-digger with Ezi-probe drill mast DRILLER: A.C

METHOD: Solid stem spiral flighted auger with tungsten carbide bit LOGGED: S.D

GROUND WATER OBSERVATIONS: Not Encountered

REMARKS: N/A CHECKED: T.M.C

# BOREHOLE LOG

**CLIENT:** LAXDTX 2 Pty Ltd **DATE:** 15/01/2024 **BORE No.:** 3

**PROJECT:** New Residential Apartment Building **PROJECT No.:** 2024-004.1 **SHEET:** 1 of 1

**LOCATION:** 32 Golf Avenue, Mona Vale **SURFACE LEVEL:** RL 18.0

| Depth (m) | Classification | Description of Strata<br>**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: Brown clayey/silty sand with rootlets and gravels | | | | |
| 0.20 | | Silty SAND: Loose, pale brown to grey, fine grained, moist silty sand with ironstone gravels, trace clay | | | | |
| 0.40 | | CLAY: Firm, orange to pale brown, localised red, medium to high plasticty, moist, near plastic limit with small ironstone gravels, trace organics | | | | |
| 0.80 | | … stiff with red mottle and iron staining | | | | |
| 1.00 | | … pale grey to red mottle, moist dry of plastic limit, friable clay, trace silt with ironstone gravels | | | | |
| 1.10 | | … very stiff | | | | |
| 1.50 | | … becoming hard, yellow/pale grey/red mottle clayey silt | | | | |
| 3.00 | | | | | | |
| 4.00 | | | | | | |
| | | | | 4.50 | | |
| 4.60 | | … with reddish brown ironstone gravels and silt (possible extremely weathered) | D | 4.60 | | |
| 4.70 | | AUGER REFUSAL at 4.70m depth on interpreted very low strength siltstone | | | | |

RIG: K9-4 Dingo mini-digger with Ezi-probe drill mast DRILLER: A.C

METHOD: Solid stem spiral flighted auger with tungsten carbide bit LOGGED: S.D

GROUND WATER OBSERVATIONS: Not Encountered

REMARKS: N/A CHECKED: T.M.C

## DYNAMIC PENETROMETER TEST SHEET

**CLIENT:** LAXDTX 2 Pty Ltd **DATE:** 15/01/2024

**PROJECT:** New Residential Apartment Building **PROJECT No.:** 2024-004.1

**LOCATION:** 32 Golf Avenue, Mona Vale **SHEET:** 1 of 1

| | Test Location | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Depth (m) | 1 | 2 | 3 | 4 | 5 | 6 | 7 | | | |
| 0.00 - 0.10 | -- | 3 | -- | 1 | 2 | 1 | 2 | | | |
| 0.10 - 0.20 | 1 | 13 | 2 | 2 | 1 | 2 | 3 | | | |
| 0.20 - 0.30 | 1 | 25 | 2 | 3 | 1 | 1 | 4 | | | |
| 0.30 - 0.40 | 1 | 6 | 2 | 5 | 1 | 1 | 5 | | | |
| 0.40 - 0.50 | 2 | 5 | 2 | 4 | 2 | 4 | 4 | | | |
| 0.50 - 0.60 | 3 | 4 | 2 | 3 | 3 | 5 | 5 | | | |
| 0.60 - 0.70 | 4 | 5 | 2 | 5 | 3 | 7 | 4 | | | |
| 0.70 - 0.80 | 6 | 5 | 3 | 9 | 2 | 6 | 5 | | | |
| 0.80 - 0.90 | 8 | 7 | 4 | 11 | 4 | 8 | 8 | | | |
| 0.90 - 1.00 | 8 | 11 | 4 | 13 (B) at 0.95m | 4 | 8 | 7 | | | |
| 1.00 - 1.10 | 9 | 18 | 4 | | 4 | 9 | 14 | | | |
| 1.10 - 1.20 | 10 | 11 | 5 | | 4 | 10 | 8 | | | |
| 1.20 - 1.30 | 12 | 14 | 5 | | 3 | 8 | 8 | | | |
| 1.30 - 1.40 | 12 | 16 | 6 | | 7 | 10 | 20 | | | |
| 1.40 - 1.50 | 17 | 17 | 6 | | 7 | 11 | 17 | | | |
| 1.50 - 1.60 | 23 | 8 | 13 | | 7 | 12 | 15 | | | |
| 1.60 - 1.70 | 25 (B) at 1.68m | 10 | 15 | | 8 | 12 | 12 | | | |
| 1.70 - 1.80 | | 14 | 12 | | 20 stop at 1.75m | 18 | 14 | | | |
| 1.80 - 1.90 | | 23 stop at 1.90m | 12 | | | 3 (B) at 1.82m | 13 | | | |
| 1.90 - 2.00 | | | 11 | | | | 17 | | | |
| 2.00 - 2.10 | | | 14 | | | | 17 | | | |
| 2.10 - 2.20 | | | 13 (B) at 2.20m | | | | 20 (B) at 2.20m | | | |
| 2.20 - 2.30 | | | | | | | | | | |
| 2.30 - 2.40 | | | | | | | | | | |
| 2.40 - 2.50 | | | | | | | | | | |
| 2.50 - 2.60 | | | | | | | | | | |
| 2.60 - 2.70 | | | | | | | | | | |
| 2.70 - 2.80 | | | | | | | | | | |
| 2.80 - 2.90 | | | | | | | | | | |
| 2.90 - 3.00 | | | | | | | | | | |
| 3.00 - 3.10 | | | | | | | | | | |
| 3.10 - 3.20 | | | | | | | | | | |
| 3.20 - 3.30 | | | | | | | | | | |
| 3.30 - 3.40 | | | | | | | | | | |
| 3.40 - 3.50 | | | | | | | | | | |
| 3.50 - 3.60 | | | | | | | | | | |
| 3.60 - 3.70 | | | | | | | | | | |
| 3.70 - 3.80 | | | | | | | | | | |
| 3.80 - 3.90 | | | | | | | | | | |
| 3.90 - 4.00 | | | | | | | | | | |

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

**REMARKS:**
**(B)** Test hammer bouncing upon refusal on solid object
**--** No test undertaken at this level due to prior excavation of soils

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# Appendix 3

**TABLE : A**

**Landslide risk assessment for Risk to life**

| HAZARD | Description | Impacting | Likelihood of Slide | Spatial Impact of Slide | | Occupancy | Evacuation | Vulnerability | Risk to Life |
|---|---|---|---|---|---|---|---|---|---|
| **A** | Landslip (earth slide <$10m^3$) from soils | | Up to 4.70m of soil | a) 0.50m from eastern site boundary, impact 20%<br>b) 4.0m from eastern site boundary, impact 1%<br>c) 1.0m from western site boundary, impact 20%<br>d) Abutting western site boundary, impact 30% | | a) Person in basement garage 4hrs/day avge.<br>b) Person in apartment 18hrs/day avge.<br>c) Person in rear courtyard 1hr/day avge.<br>d) Person by stormwater detention tank 0.5hrs/day avge. | a) Possible to not evacuate<br>b) Likely to not evacuate<br>c) Unlikely to not evacuate<br>d) Unlikely to not evacuate | a) Person in open space, partially buried<br>b) Person in apartment, minor damage only<br>c) Person in open space, partially buried<br>d) Person in open space, partially buried | |
| | | | **Possible** | **Prob. of Impact** | **Impacted** | | | | |
| | | a) Driveway/ Footsteps/ Retaining wall/ Basement garage of No. 34-36 Golf Avenue | 0.001 | 0.20 | 0.20 | 0.1667 | 0.5 | 0.05 | **1.67E-07** |
| | | b) Apartment buildings of No. 34 - 46 Golf Avenue | 0.001 | 0.05 | 0.01 | 0.750 | 0.75 | 0.75 | **2.11E-07** |
| | | c) Rear courtyard area of No. 28-30 Golf Avenue | 0.001 | 0.10 | 0.20 | 0.0417 | 0.25 | 0.05 | **1.04E-08** |
| | | d) Stormwater detention tank of No. 28-30 Golf Avenue | 0.001 | 0.10 | 0.30 | 0.0208 | 0.25 | 0.75 | **1.17E-07** |
| **B** | Landslip (rockslide/topple <$3m^3$) of bedrock around perimeter of excavation due to poorly oriented defects | | Potentially up to 1.30 of exposed bedrock | a) 0.50m from eastern site boundary, impact 5%<br>b) 4.0m from eastern site boundary, impact 1% | | a) Person in basement garage 4hrs/day avge.<br>b) Person in apartment 18hrs/day avge. | a) Possible to not evacuate<br>b) Likely to not evacuate | a) Person in open space, partially buried<br>b) Person in apartment, minor damage only | |
| | | | **Unlikely** | **Prob. of Impact** | **Impacted** | | | | |
| | | a) Retaining Wal/ Basement garage of No. 34-36 Golf Avenue | 0.0001 | 0.20 | 0.05 | 0.1667 | 0.5 | 0.05 | **4.17E-09** |
| | | b) Apartment building of No. 34-36 Golf Avenue | 0.0001 | 0.05 | 0.01 | 0.7500 | 0.75 | 0.75 | **2.11E-08** |

* 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 (earth slide <10m³) from soils | a) Driveway/ Footsteps/ Retaining wall/ Basement garage of No. 34-36 Golf Avenue | Unlikely | The event might occur under very adverse circumstances over the design life. | Minor | Limited Damage to part of structure or site or INSIGNIFICANT damage to neighbouring properties, requires some stabilisation . | Low |
| | | b) Apartment buildings of No. 34 - 46 Golf Avenue | Unlikely | The event might occur under very adverse circumstances over the design life. | Minor | Limited Damage to part of structure or site or INSIGNIFICANT damage to neighbouring properties, requires some stabilisation . | Low |
| | | c) Rear courtyard area of No. 28-30 Golf Avenue | Unlikely | The event might occur under very adverse circumstances over the design life. | Minor | Limited Damage to part of structure or site or INSIGNIFICANT damage to neighbouring properties, requires some stabilisation . | Low |
| | | d) Stormwater detention tank of No. 28-30 Golf Avenue | Unlikely | The event might occur under very adverse circumstances over the design life. | Minor | Limited Damage to part of structure or site or INSIGNIFICANT damage to neighbouring properties, requires some stabilisation . | Low |
| B | Landslip (rockslide/topple <3m³) of bedrock around perimeter of excavation due to poorly oriented defects | a) Retaining Wal/ Basement garage of No. 34-36 Golf Avenue | Rare | The event is conceivable but only under exceptional circumstances over the design life. | Medium | Moderate damage to some of structure or significant part of site or MINOR damage to neighbouring property, requires large stabilising works . | Low |
| | | b) Apartment building of No. 34-36 Golf Avenue | Rare | The event is conceivable but only under exceptional circumstances over the design life. | Medium | Moderate damage to some of structure or significant part of site or MINOR damage to neighbouring property, requires large stabilising works . | 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%.

* Cost of site development estimated at $1,000,000

# Appendix 4

**Envirolab Services Pty Ltd**
ABN 37 112 535 645
12 Ashley St Chatswood NSW 2067
ph 02 9910 6200 fax 02 9910 6201
customerservice@envirolab.com.au
www.envirolab.com.au

# CERTIFICATE OF ANALYSIS 341696

## Client Details

| | |
|---|---|
| **Client** | Crozier Geotechnical Consultants |
| **Attention** | Sores Demirbag |
| **Address** | Unit 12/42-46 Wattle Rd, Brookvale, NSW, 2100 |

## Sample Details

| | |
|---|---|
| **Your Reference** | **2024-004, 32 Golf Avenue, Mona Vale** |
| **Number of Samples** | 3 Soil |
| **Date samples received** | 17/01/2024 |
| **Date completed instructions received** | 17/01/2024 |

## Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

## Report Details

| | |
|---|---|
| **Date results requested by** | 24/01/2024 |
| **Date of Issue** | 24/01/2024 |

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Accredited for compliance with ISO/IEC 17025 - Testing. **Tests not covered by NATA are denoted with ***

**Results Approved By**
Diego Bigolin, Inorganics Supervisor

**Authorised By**
Nancy Zhang, Laboratory Manager

| Misc Inorg - Soil | | | | |
|---|---|---|---|---|
| Our Reference | | 341696-1 | 341696-2 | 341696-3 |
| Your Reference | UNITS | BH1 | BH2 | BH3 |
| Depth | | 2.7-3.0 | 3.8-4.0 | 4.5-4.6 |
| Date Sampled | | 15/01/2024 | 15/01/2024 | 15/01/2024 |
| Type of sample | | Soil | Soil | Soil |
| Date prepared | - | 17/01/2024 | 17/01/2024 | 17/01/2024 |
| Date analysed | - | 23/01/2024 | 23/01/2024 | 23/01/2024 |
| pH 1:5 soil:water | pH Units | 5.5 | 5.6 | 5.7 |
| Electrical Conductivity 1:5 soil:water | µS/cm | 40 | 56 | 72 |
| Chloride, Cl 1:5 soil:water | mg/kg | <10 | <10 | 22 |
| Sulphate, SO4 1:5 soil:water | mg/kg | 25 | 42 | 48 |

| Method ID | Methodology Summary |
|---|---|
| **Inorg-001** | pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times. |
| **Inorg-002** | Conductivity and Salinity - measured using a conductivity cell at 25°C in accordance with APHA latest edition 2510 and Rayment & Lyons. |
| **Inorg-081** | Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis.<br>Alternatively determined by colourimetry/turbidity using Discrete Analyser. |

| QUALITY CONTROL: Misc Inorg - Soil | | | | | Duplicate | | | | Spike Recovery % | |
|---|---|---|---|---|---|---|---|---|---|---|
| Test Description | Units | PQL | Method | Blank | # | Base | Dup. | RPD | LCS-1 | [NT] |
| Date prepared | - | | | 17/01/2024 | [NT] | [NT] | [NT] | [NT] | 17/01/2024 | [NT] |
| Date analysed | - | | | 23/01/2024 | [NT] | [NT] | [NT] | [NT] | 23/01/2024 | [NT] |
| pH 1:5 soil:water | pH Units | | Inorg-001 | [NT] | [NT] | [NT] | [NT] | [NT] | 101 | [NT] |
| Electrical Conductivity 1:5 soil:water | µS/cm | 1 | Inorg-002 | <1 | [NT] | [NT] | [NT] | [NT] | 102 | [NT] |
| Chloride, Cl 1:5 soil:water | mg/kg | 10 | Inorg-081 | <10 | [NT] | [NT] | [NT] | [NT] | 98 | [NT] |
| Sulphate, SO4 1:5 soil:water | mg/kg | 10 | Inorg-081 | <10 | [NT] | [NT] | [NT] | [NT] | 98 | [NT] |

## Result Definitions

| | |
|---|---|
| **NT** | Not tested |
| **NA** | Test not required |
| **INS** | Insufficient sample for this test |
| **PQL** | Practical Quantitation Limit |
| **<** | Less than |
| **>** | Greater than |
| **RPD** | Relative Percent Difference |
| **LCS** | Laboratory Control Sample |
| **NS** | Not specified |
| **NEPM** | National Environmental Protection Measure |
| **NR** | Not Reported |

## Quality Control Definitions

| | |
|---|---|
| **Blank** | This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples. |
| **Duplicate** | This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable. |
| **Matrix Spike** | A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist. |
| **LCS (Laboratory Control Sample)** | This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample. |
| **Surrogate Spike** | Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples. |

Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.

The recommended maximums for analytes in urine are taken from “2018 TLVs and BEIs”, as published by ACGIH (where available). Limit provided for Nickel is a precautionary guideline as per Position Paper prepared by AIOH Exposure Standards Committee, 2016.

Guideline limits for Rinse Water Quality reported as per analytical requirements and specifications of AS 4187, Amdt 2 2019, Table 7.2

## Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.

Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals (not SPOCAS); 60-140% for organics/SPOCAS (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Where matrix spike recoveries fall below the lower limit of the acceptance criteria (e.g. for non-labile or standard Organics <60%), positive result(s) in the parent sample will subsequently have a higher than typical estimated uncertainty (MU estimates supplied on request) and in these circumstances the sample result is likely biased significantly low.

Measurement Uncertainty estimates are available for most tests upon request.

Analysis of aqueous samples typically involves the extraction/digestion and/or analysis of the liquid phase only (i.e. NOT any settled sediment phase but inclusive of suspended particles if present), unless stipulated on the Envirolab COC and/or by correspondence. Notable exceptions include certain Physical Tests (pH/EC/BOD/COD/Apparent Colour etc.), Solids testing, total recoverable metals and PFAS where solids are included by default.

Samples for Microbiological analysis (not Amoeba forms) received outside of the 2-8°C temperature range do not meet the ideal cooling conditions as stated in AS2031-2012.

# Appendix 5

# APPENDIX A

# DEFINITION OF TERMS

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

## APPENDIX C: LANDSLIDE RISK ASSESSMENT

## QUALITATIVE TERMINOLOGY FOR USE IN ASSESSING RISK TO PROPERTY

### *QUALITATIVE MEASURES OF LIKELIHOOD*

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

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

### *QUALITATIVE MEASURES OF CONSEQUENCES TO PROPERTY*

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

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

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

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

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

### *QUALITATIVE RISK ANALYSIS MATRIX – LEVEL OF RISK TO PROPERTY*

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

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

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

### *RISK LEVEL IMPLICATIONS*

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

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