Georgina Nassif & Tony Nassif

# Geotechnical Investigation Report

Proposed Development at:
26 Ralston Road
Palm Beach NSW 2108

G21118-1-Rev A

18th September 2024

Geotechnical Consultants Australia Pty Ltd
(02) 9788 2829
info@geoconsultants.com.au
www.geoconsultants.com.au

**Report Distribution**

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**Geotechnical Consultants Australia Pty Ltd**
2 Harold Street
Parramatta NSW 2150
(02) 9788 2829
www.geoconsultants.com.au
info@geoconsultants.com.au

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# 1. INTRODUCTION

## 1.1 Background

This geotechnical engineering report presents the results of a geotechnical investigation undertaken by Geotechnical Consultants Australia Pty Ltd (GCA) for a proposed development at No. 26 Ralston Road Palm Beach NSW 2108 (the site). The investigations were commissioned by Georgina Nassif & Tony Nassif (the client) and was carried out on the 12th March 2021 and 12th September 2024.

The purpose of the investigation was to assess the subsurface conditions over the site at the selected boreholes and testing locations (where accessible and feasible) and provide necessary recommendations from a geotechnical perspective for the proposed development.

The findings presented in this report are based on our subsurface investigation and our experience with subsurface conditions in the area and local region. This report presents our assessment of the geotechnical conditions and has been prepared to provide preliminary geotechnical advice and recommendations to assist in the preparation of preliminary designs and construction of the ground structures for the proposed development.

For your review, **Appendix A** contains a document prepared by GCA entitled "Important Information About Your Geotechnical Report", which summarises the general limitations, responsibilities and use of geotechnical engineering reports.

## 1.2 Proposed Development

Information provided by the client indicates the proposed development comprises demolition of the existing infrastructure onsite, followed by construction of a new two (2) storey dual occupancy, overlying a partial basement (garage) level. In-ground swimming pools are also proposed to be constructed as part of the planned development.

The Finished Floor Levels (FFL)s of the proposed developments basement and ground floor levels are set to be at Reduced Levels (RL)s of:

- Basement floor level: RL98.500m Australian Height Datum (AHD).
- Ground floor level: RL100.200m to RL101.350m AHD.

Based on this information and the existing site levels and topography, maximum excavation depths varying from approximately 1.0m to 3.8m are expected to be required for construction of the proposed basement level and in-ground swimming pools, with cut and fill elsewhere. Locally deeper excavations for the proposed building footings and service trenches are also anticipated to be required.

It should be noted that excavation depths are expected to vary across the site and are inferred off the FFLs shown on the architectural drawings and existing levels on the site survey plan, referenced in Section 1.3 below.

## 1.3 Provided Information

The following relevant information was provided to GCA prior to the geotechnical investigation and during preparation of this report:

- Architectural drawings prepared by Crawford Architects, titled "Ralston Road Palm Beach", referenced project No. 19031, and dated March 2024.
- Site survey plan prepared by C & A Surveyors NSW Pty Ltd, titled "Detail Survey of Lot 4, 5 Sec. 10 in DP14048, Located at No. 26 Ralston Street, Palm Beach", referenced No. 11303-19, sheet 1 of 1 and dated 21st May 2019.

### 1.4 Geotechnical Assessment Objectives

The objective of the geotechnical investigation was to assess the site surface and subsurface conditions at the selected boreholes and testing locations within the site, and to provide professional geotechnical advice and recommendations on the following requirements provided to GCA by the client:

- General assessment of any potential geotechnical issues that may affect the proposed development any surrounding infrastructure, buildings, council assets, etc.
- Excavation conditions and recommendations on excavation methods in soil and rock to restrict any ground vibrations.
- Recommendations on suitable shoring (retention) systems for the site.
- Design parameters based on the ground conditions within the site for retaining walls, cantilever shoring walls and propped shoring.
- Recommendations on suitable foundation types and design for the site.
- End bearing capacities and shaft adhesion for shallow and deep foundations based on the ground conditions within the site.
- Groundwater levels which may be determined during the geotechnical investigation.
- Recommendations on groundwater maintenance and limiting.
- Preliminary site lot classification in accordance with Australian Standards (AS) 2870-2011.
- Preliminary slope risk assessment in accordance with guidelines published by the Australian Geomechanics Society (AGS) "Practice Note Guidelines for Landslide Risk Management – AGS 2007c".
- General geotechnical advice on site preparation, filling and subgrade preparation.

### 1.5 Scope of Works

Fieldwork for the geotechnical investigation was undertaken by an experienced geotechnical engineer, following in general the guidelines outlined in AS 1726-2017 "Geotechnical Site Investigations". The scope of works included:

- Service locating carried out using electromagnetic detection equipment to ensure the area is free of any underground services at the selected boreholes and testing locations.
- Review of site plans and drawings to determine appropriate testing locations (where accessible and feasible), and identify any relevant features of the site.
- Hand augering of three (3) boreholes at selected locations within the site (where accessible and feasible), identified as BH1, BH101 and BH102, and carried out using hand operated equipment to varying practical refusal depths of approximately 0.5m to 0.8m below the existing ground level within the site (bgl).
- Dynamic Cone Penetrometer (DCP) testing immediately adjacent to the boreholes and at selected locations within the site (where accessible and feasible), using hand operated equipment to varying practical refusal depths of approximately 0.35m to 2.55m bgl. The DCP tests are identified as DCP1 to DCP3 inclusive, and DCP101 to DCP103 inclusive.
    - The approximate locations of the boreholes and DCP tests are shown on **Figure 1, Appendix B** of this report.
- Collection of soil samples during fieldwork for any laboratory testing which may be required.
- Reinstatement of the boreholes with available soil displaced during augering.
- Preparation of this geotechnical engineering report.

### 1.6 Constraints

The discussions and recommendations provided in this report have been based on the results obtained at the selected boreholes and testing locations within the site. It is recommended that further geotechnical inspections should be carried out during construction to confirm the subsurface conditions across the site and foundation bearing capacities have been achieved.

Consideration should also be given to machine drilled boreholes and rock strength testing carried out to confirm the ground conditions and estimated rock strength underlying the site, and to help assist in final designs of the proposed development.

## 2. SITE DESCRIPTION

### 2.1 Overall Site Description

The overall site description and its surrounding are presented in Table 1 below.

**Table 1. Overall Site Description and Site Surroundings**

| Information | Details |
|---|---|
| **Overall Site Location** | The site is located within a residential area along Ralston Road carriageway, approximately 290m east of Barrenjoey Road thoroughfare. |
| **Site Address** | 26 Ralston Road Palm Beach NSW 2108 |
| **Approximate Site Area[1]** | 1,497.3m$^2$ |
| **Local Government Authority** | Northern Beaches Council |
| **Site Description** | At the time of the investigation, a residential dwelling was present within the site accompanied by associated concrete pavements and a number of retaining walls. The remaining site area was mainly covered in grass, vegetation and some mature trees scattered throughout. |
| **Approximate Distances to Nearest Watercourses (i.e. rivers, lakes, creeks, etc.)** | • Sandy Beach – 360m south-west of the site. |
| **Site Surroundings** | The site is located within an area of residential use and is bounded by:<br>• Ralston Road carriageway to the north.<br>• Residential property at No. 24 Ralston Road to the east.<br>• Residential property at No. 8 Ebor Road to the west.<br>• Residential property at No. 28 Ralston Road to the west. |

[1]Site area is approximate and based on the site survey plan referenced in Section 1.3.

## 2.2 Topography

The local and site topography generally falls towards the west to north-west. Levels within the site vary from approximately RL103.5m to RL98.4m AHD.

General onsite assessment and review of the site survey plan indicates the site has an overall moderate to strong slope of around 5° to 13° (varying throughout).

It should be noted that the site topography, levels and slopes are approximate and based on the site survey plan referenced in Section 1.3, observations made during the geotechnical investigation and reference to NSW Six Maps (https://maps.six.nsw.gov.au/).

The actual topography in areas inaccessible during the site investigation, including areas under the existing infrastructure, along with the site and local topography and levels are expected to vary from those outlined in this report.

## 2.3 Regional Geology

Information obtained on the local regional subsurface conditions, referenced from the Department of Mineral Resources, Sydney 1:100,000 Geological Series Sheet 9130 First Edition, dated 1983, by the Geological Survey of New South Wales, indicates that the site is located within a geological region generally underlain by the Triassic Aged Hawkesbury Sandstone (Rh). The Hawkesbury Sandstone (Rh) typically comprises "medium to coarse grained quartz Sandstone, very minor Shale and laminite lenses".

We note that the site is also situated approximately 60m east of a geological boundary/region typically underlain by Triassic Aged Newport Formation and Garie Formation of the Narrabeen Group (Rnn). The Newport Formation and Garie Formation (Rnn) usually consists of "interbedded laminite, Shale and quartz to lithic-quartz Sandstone: minor red claystone north of Hawkesbury River. Clay pellet sandstone (Garie Fm) south of Hawkesbury River".

Furthermore, reference made to MinView by the State of New South Wales through Regional NSW 2024 specifies the site is positioned within a geological region underlain by Sandstone (Tuth) and in close proximity to Siliciclastic Sedimentary Rock (Tngn).

A review of the regional maps by the NSW Government Environment and Heritage shows the site is set within the Gymea (gy) landscape group. The Gymea (gy) landscape group is typically recognised by undulating to rolling rises and low hills on Hawkesbury Sandstone. Local reliefs are generally 20m to 80m, and slopes typically 10% to 25% in gradient, with noticeable rock outcrops of classically less than 25%. Soils of the Gymea group typically have localised steep slopes, high soil erosion hazard, rock outcrops, shallow highly permeable soils and very low soil fertility. Soils of the Gymea landscape group are generally slightly (pH 6.5) to strongly (pH 4.0) acidic.

We note that the site is also located near the Watagan (wn) landscape group. The Watagan (wn) landscape group which is typically recognised by rolling to very steep hills on fine-grained Narrabeen Group sediments. Local reliefs range from 60m to 120m and slopes typically greater than 25% in gradient. Soils of the Watagan group typically have mass movement hazard, steep slopes, severe soil erosion hazard and occasional rock outcrops. Soils of the Watagan landscape group range from slightly acidic (pH 6.0) to extremely acidic (pH 3.5).

The Gymea (gy) and Watagan (wn) landscape group reports are attached in **Appendix G**.

# 3. SUBSURFACE CONDITIONS AND ASSESSMENT RESULTS

## 3.1 Stratigraphy

A summary of the surface and subsurface conditions from across the site encountered during the geotechnical investigation is presented in Table 2a and Table 2b below and conditions are interpreted from the assessment results. It should be noted Table 2a and Table 2b presents a summary of the overall site conditions and reference should be made to the detailed engineering borehole logs presented in **Appendix D**, in conjunction with the geotechnical explanatory notes presented in **Appendix C**.

Fill, soil and rock descriptions are in accordance with AS 1726-2017 Geotechnical Site Investigations, and rock classification, where given, is in accordance with Pells P.J.N, Mostyn G. & Walker B.F. (1998) Foundations on Sandstone and Shale in the Sydney Region, Australian Geomechanics Journal, December 1998.

It should be noted that estimated soil consistency/strength assessed by DCP testing in the site during the geotechnical investigation are approximate and variances should be expected throughout. Due to the variable ground conditions throughout the site, it is recommended that confirmation of the subsurface materials be carried out during construction by inspection.

It should also be noted that ground conditions within the site are expected to differ from those encountered and inferred in this report, since no geotechnical or geological exploration program, no matter how comprehensive, can reveal and identify all subsurface conditions underlying the site.

From the boreholes (BH1, BH101 and BH102) carried out within the site, the subsurface conditions at the test locations (where accessible and feasible) generally comprised:

- Unit 1: Silty SAND and SAND fill, gravel inclusions, from the existing ground level within the site and extending to depths of approximately 0.3m to 0.5m bgl (or greater), underlain by:
- Unit 2: Natural SAND, fine to medium grained, trace of clay, gravel inclusions, estimated medium dense to dense and present to at least 0.6m to 0.8m bgl (BH101 and BH102 only). Inferred to extend to the DCP testing refusal depths and largely anticipated to overlie:
- Unit 3: *Inferred* SANDSTONE bedrock (subject to confirmation).

Based on the geotechnical investigation at the selected testing locations, along with our experience and observations made within the site and local region, it is inferred that bedrock of variable composition, strength and weathering is underlying the majority of the site area at varying depths of approximately 0.4m to 2.6m bgl (expected to vary throughout). It should be noted that DCP1 was carried out from the top of a retaining wall and bedrock is inferred to be present at relatively deeper depths in the vicinity of this test.

In addition, variable composition and consistency/strength natural soils are also likely to be present throughout the site, predominately at locations and depths not assessed during the geotechnical investigation.

A summary of the inferred subsurface conditions encountered and inferred during DCP testing are summarised in Table 2a and Table 2b below, with the DCP testing results attached in **Appendix E**. Ground conditions depicted in Table 2a and Table 2b below are inferred based on the DCP testing results and confirmation should be carried out by additional testing and during construction by inspection. This also assumes a similar subsurface profile observed during the geotechnical investigation to be present over the remainder of the site and throughout the testing depths indicated.

It should be noted that DCP testing and higher blow counts encountered may be affected by factors such as gravels, ironstone bands, well consolidated soils and highly cemented sands, and other deleterious materials which may be present within the underlying soils, along with tree rootlets extending throughout the soils from trees and vegetation within the vicinity. These results should be read in conjunction with the boreholes and geotechnical confirmation should be made as site conditions may vary.

Included in **Appendix I** are general Geotechnical Cross Sections (A-A' & B-B') showing inferred ground conditions beneath the site.

**Table 2a. Summary of Inferred Subsurface Conditions From DCP Testing from 2021**

<table>
<tr><th colspan="2">*DCP ID*</th><th>DCP1</th><th>DCP2</th><th>DCP3 (BH1)</th></tr>
<tr><th>Unit</th><th>Unit Type</th><th colspan="3">Depth/Thickness of Unit (m bgl)</th></tr>
<tr><td>1</td><td>*Inferred* Fill[1]</td><td rowspan="2">0.0 – 2.55</td><td rowspan="2">0.0 – 0.35</td><td rowspan="2">0.0 – 0.9</td></tr>
<tr><td>2</td><td>Natural Soils[2]</td></tr>
<tr><td>3</td><td>*Inferred* Bedrock[3]</td><td>2.6</td><td>0.4</td><td>0.9</td></tr>
</table>

**Table 2b. Summary of Inferred Subsurface Conditions From DCP Testing from 2024**

<table>
<tr><th colspan="2">*DCP ID*</th><th>DCP101 (BH101)</th><th>DCP102</th><th>DCP103 (BH102)</th></tr>
<tr><th>Unit</th><th>Unit Type</th><th colspan="3">Depth/Thickness of Unit (m bgl)</th></tr>
<tr><td>1</td><td>*Inferred* Fill[1]</td><td>0.0 – 0.5</td><td rowspan="2">0.0 – 1.17</td><td>0.0 – 0.3</td></tr>
<tr><td>2</td><td>Natural Soils[2]</td><td>0.5 – 1.17</td><td>0.3 – 0.78</td></tr>
<tr><td>3</td><td>*Inferred* Bedrock[3]</td><td>1.2</td><td>1.2</td><td>0.8</td></tr>
</table>

[1]Thickness of the fill layer is expected to vary from those indicated in Table 2a and Table 2b.
[2]Estimated soil consistency/strength is based on DCP testing to the maximum practical refusal depths at the selected testing locations within the site. The potential for weak or softer layers throughout the unit should be considered.
[3]Inferred bedrock composition, continuity, strength and depth should be confirmed by a geotechnical engineering prior to construction by additional borehole drilling and rock strength testing, and during construction by inspection. Bedrock inferred to be present at or shortly below the practical DCP testing refusal depths at the selected testing locations within the site.

Notes:

- Inferred bedrock estimated strength is expected to vary across the site, due to the limited investigation carried out.
- Clay seams, defects and fractured/extremely weathered zones are expected to be present throughout the underlying inferred bedrock, predominantly at depths and locations unobserved during the geotechnical investigation.
- Ground conditions are expected to vary across the site and should be confirmed by a geotechnical engineer, predominantly in areas unobserved during the geotechnical investigation.

### 3.2 Groundwater

No groundwater was encountered or observed during the site investigations to maximum depths of approximately 0.8m bgl (BH101) and 2.6m bgl (DCP1).

It is noted that the boreholes were immediately backfilled following completion of augering which precluded longer term monitoring of groundwater levels. Although no groundwater was encountered or observed during the investigations, its presence should not be precluded within the site and during construction.

Thus, based on observations made at the selected boreholes and testing locations and geological position of the site, groundwater which may be present within the site is expected to be in the form of seepage through voids within the underlying fill material and pore spaces between particles of unconsolidated natural soils, or through networks of fractures and solution openings in consolidated inferred bedrock underlying the site. Seepage may occur at the soil/rock interface.

It should be noted that groundwater levels have the potential to elevate during daily or seasonal influences such as tidal fluctuations, heavy rainfall, damaged services, flooding, etc., and moisture content within soils may be influenced by events within the site and adjoining properties. Groundwater monitoring should be carried out during construction to assess any groundwater inflow throughout the excavation areas.

We note that no provision was made for longer term groundwater monitoring within the site. Where groundwater conditions vary from those outlined in this report, GCA should be contacted for further advice.

# 4. PRELIMINARY LANDSLIDE RISK ASSESSMENT

## 4.1 General

The overall stability of the site including slope angles, depth of soils and overall strength, movements of groundwater and surface runoff, drainage and potential slides planes within the interfaces of rocks and soils was assessed by GCA as part of the geotechnical investigation. The overall assessment was carried out in accordance with guidelines published by the AGS "Practice Note Guidelines for Landslide Risk Management – AGS 2007c".

Due to the sloping nature of this site, a geotechnical investigation and assessment in accordance with guidelines published by the AGS was carried out in order to demonstrate that the proposed development is justified in terms of geotechnical stability. Therefore, the following sections are a preliminary assessment based on the AGS guidelines for the stability of the site prior and following construction.

It should be noted that this preliminary landslide risk assessment is limited to the proposed development area and areas accessible during the time of our site investigation, including information available at the time of reporting.

## 4.2 Site Assessment

The overall site area and topography generally slopes towards the west to north-west, as discussed in Section 2.2 of this report. Table 3 summarises results of the overall stability within the site.

**Table 3. Summary of Overall Site Stability**

| Observations | Identification | Comments |
|---|---|---|
| Site Topography | N/A | The overall site area varies throughout and slopes generally towards the west to north-west, as discussed in Section 2.2 and shown in **Figure 1, Appendix B** of this report.<br>Reference should be made to this section and site plan for a general description of the site area. |
| Overall Site Description | N/A | The site area was generally covered in healthy mature trees, vegetation and grass.<br>Associated concrete pavements, retaining walls and the existing dwelling covered majority of the remaining site area. |
| Groundwater | No | No groundwater was encountered or observed during augering of borehole BH101 and DCP1 testing to maximum depths of approximately 0.8m and 2.55m, respectively, as discussed in Section 3.2.<br>It is expected that groundwater which may be encountered within the site will be in the form of seepage through the voids within the underlying soils and defects in the underlying inferred bedrock.<br>Based on the regional and site topography, we expect groundwater flows (including surface water) to flow towards the west (varying throughout). |
| Surface Water | No | No surface water or ponding or seepage were observed within the site. No seepage was also visible through any retaining walls onsite at the time of the investigations.<br>Soils were predominately moist, and are inferred to comprise mainly fill material and natural soils, underlain by inferred bedrock at varying depths throughout the site area (as discussed in Section 3). |

| | | | |
|---|---|---|---|
| Outcrops | | Yes | No outcrops were observed within the site or adjoining properties. |
| Loose Boulders or Rock Mass' | | No | No loose boulders were observed across the site, within the proposed development and adjoining properties. |
| Bedrock Deterioration | | N/A | N/A |
| Structural Distress | Existing Dwelling and Infrastructure | N/A | No sign of structural distress or movement were observed to the existing dwelling and infrastructure within the site.<br>Any cracks observed within the concrete pavements are inferred to be associated with concrete shrinkage and loading over time. |
| | Retaining Walls | | No sign of structural distress or movement were observed to any retaining walls throughout the site. |
| Adjoining Properties | | N/A | Infrastructure adjoining the site were observed to be in a generally good condition and trees within the vicinity were observed to have no sign of deformation. |
| Ground Movement | | No | No signs of cracks in the ground, slumping, or other signs of landslip observed within the site. No ground deformation was observed within the site (where accessible and feasible). |
| Tilting or Bending Trees | | No | No trees showed any signs of tilting or bending at the time of the investigations within the site and investigation area. Typically, tilting, bending or curved trees can indicate rotation due to soil creep or movement (where accessible and feasible). |
| Soil Creep or Shallow Failure | | No | No sign of soil creep or shallow failure was observed within the fill material or natural soils present within the site, and throughout the site and adjoining properties (where accessible and feasible). |

It should be noted that trees, vegetation and grass present within the site are considered to contribute to the stability of the site. Retaining walls in their current state are also considered to contribute to the stability and retention of the soils behind the retaining walls.

Based on the subsurface conditions encountered within the site during the geotechnical investigation, it is anticipated that fill material and natural soils will underlie majority of the proposed development area, overlying inferred bedrock at varying depths throughout, as discussed in Section 3 above.

### 4.3 Pre-Development (Assessed Risk to Property)

Based on the geotechnical investigation, site topography and existing ground conditions within the site, assessment on the potential effects which may be associated with the hazards on the adjoining properties, along with the buildings, lands and occupiers within. The adjoining properties, and existing dwelling/proposed development are considered as part of the risk levels to the property pre-development. presented in Table 4 below.

**Table 4. Pre-Development – Assessed Risk To Property**

| Potential Hazard | Qualitative Measures of Likelihood (AGS) | Qualitative Measures of Consequences to Property (AGS) | Qualitative Risk Analysis – Level of Risk to Property (AGS) |
|---|---|---|---|
| **Soil Creep[1]** | C – Possible ($10^{-3}$) | 3 – Medium (20%) | Moderate |
| **Shallow Failure[1]** | C – Possible ($10^{-3}$) | 3 – Medium (20%) | Moderate |

[1]Within the fill material and natural soils present within the site.

Based on the assessed conditions within the site, the overall slope instability assessed risk to the property under the existing conditions prior to construction of the currently proposed development is assessed to be "moderate".

According to AGS 2007c, the "moderate risk level" may be tolerated in certain circumstances, however, requires investigation, planning and implementation of treatment options which will be required to reduce the risk to "low level risk".

It should be noted that the AGS guidelines recommend tolerable loss of life for the person most at risk for exiting slope/existing development to be 1 x $10^{-4}$/annum.

### 4.4 Mitigation and Control Measures

To ensure the stability of the site and to reduce the risk (to "low" risk) of any instability for the proposed development within the site, the following recommendations should be considered along with recommendations presented in this report (not limited to):

- The design and construction of earthworks, foundations, retaining structures, excavation stabilisation and drainage measure for the proposed development should adhere to good engineering practice for hillside construction as set out in Appendix G of AGS 2007c Vol. 42 guidelines, attached as **Appendix H** in this report.
- Any cause of instability of the ground profile within the site and neighbouring properties should be addressed prior to any excavation or construction work, and proposed stabilisation actions should be implemented. In this case, GCA should be contacted on further geotechnical advice for any stabilisations actions which may be required.
- Excavation, pile installation and any rock ripping and hammering (or the like) are expected to cause vibrations within the underlying bedrock. Monitoring of any retaining walls, soils and bedrock underlying the site should be carried out and inspected by a geotechnical engineer during construction.
    - Any observable movement within the underlying soils and/or retaining walls should cease work immediately, and GCA be contacted for further advice.
- Any excavation should be monitored by a suitably qualified geotechnical engineer, which should monitor ground movement and vibrations, as well as any retaining walls or infrastructure within and adjoining the site. This includes any batter slopes which may be adopted during construction which will require ongoing inspections and approvals by a geotechnical engineer or engineering geologist. General advice on excavation stability is provided in Section 5.8 of this report.

- Any vertical cut or fill exceeding 0.5m in depth within soils should be retained by an appropriately designed retaining wall.
- All retaining walls should be designed using appropriate geotechnical design parameters for the subject site and ground conditions provided in Section 5.8.3.
- Any excavation should be commenced from higher levels and should be carried out in stages progressing towards the lower levels within the site. Excavations (including any batter slopes) are to be monitored and approved by a geotechnical engineer familiar with the site conditions.
- Backfilling should be placed and compacted to engineering standards in accordance with AS 3798-2007 and AS 1289, with reference to Section 5.10 of this report. This includes all batters, pavements, driveways, etc.
    - Reference should be made to these sections for preparation of pavements within the site. Further advice should also be sought from GCA prior and during construction.
- Backfilling behind any walls should also be carried out in accordance with AS 3798-2007 and AS 1289. This should include appropriate materials, compaction criteria and testing, site preparation and fill construction, methods of testing and inspections, and constant testing. Appropriate backfill drainage should also be provided.
- Appropriate drainage methods should be incorporated to ensure all surface and subsurface water flows are diverted away from the slopes, adjoining properties and proposed development, into a stormwater drainage system or appropriate discharge. This includes appropriate drainage behind any excavations and <u>all</u> retaining walls, and if required, beneath slabs. This should be carefully assessed, designed and detailed by the project stormwater engineer. Groundwater monitoring of seepage should also be implemented during any excavation stage to confirm the capacity of the drainage system and groundwater entering the excavation area.
    - All stormwater and drainage within the site should be in accordance with the approved stormwater engineering drawing.
- The foundation system for the proposed development should comprise combination of shallow foundations and piles sufficiently founded/embedded into consistent and competent bedrock underlying the site, as discussed in Section 5.9 of this report. Piles are necessary in order to increase resistance against sliding.
- Foundation systems for the proposed development, building structures, retaining walls and any water tanks, etc., should be sufficiently founded/embedded into the underlying bedrock, and where necessary designed for lateral earth pressures induced by soil movement along the interface between soils and the underlying bedrock.
- Foundations should be inspected and approved by a suitably qualified geotechnical engineer, with all structural elements also inspected and approved by the project structural engineer.
- All retaining walls and footings to be designed by a qualified structural engineer in accordance with recommendations in this report, and any future geotechnical investigation report which may be necessary for the site.
- Maintenance and inspection of permanent retaining walls should be carried out regularly.
- Inspection of surface and subsurface movement following any removal of trees or vegetation within the site.
- Plantation of trees and vegetation following construction of any proposed development in the future. Specific advice should be sought on plantation of trees near structures from AS 2870-2011.
- Construction activities should be carefully observed by a geotechnical engineer, where further assessment and necessary mitigation and control measures may be provided.
- Care should be taken for all construction activities within the site, with constant supervision by the project site manager, geotechnical engineer and structural engineer. Any observable movement within the underlying soils and/or retaining walls should cease work immediately, and GCA be contacted for further advice.

- Vibration levels during excavation and construction should be maintained to appropriate levels within the site, predominately where existing retaining walls or sensitive structures exist. Further general advice is provided in Section 5.6.

Implementation of the measures recommended in this report (not limited to these measures) should constitute as "Hold Points".

### 4.5 Quantitative Risk Assessment (Risk to Life)

The annual probability of loss of life (death) of an individual post-development is calculated using the following formula:

$$R_{(LOL)} = P_{(H)} \times P_{(S:H)} \times P_{(T:S)} \times v_{(D:T)}$$

Where:

| | |
|---|---|
| **R** (LOL) | is the risk (annual probability of loss of life (death) of an individual). |
| **P** (H) | is the annual probability of the landslide. |
| **P** (S:H) | is the probability of spatial impact of the landslide impacting a building (location) taking into account the travel distance and travel direction given the event. |
| **P** (T:S) | is the temporal spatial probability (e.g. of the building or location being occupied by the individual) given the spatial impact and allowing for the possibility of evacuation given there is warning of the landslide occurrence. |
| **V** (D:T) | is the vulnerability of the individual (probability of loss of life of the individual given the impact). |

It should be noted that the AGS guidelines recommend tolerable loss of life for the person most at risk for a new development to be 1 x $10^{-5}$/annum.

**Annual Probability of Landslide**

No evidence of movement was observed on the site during the time of the investigations.

**P** (H) = 0.0001/annum

**Probability of Spatial Impact**

Construction of the proposed development anticipated to be located towards the southern portion of the site. Review of the proposed developments drawings and from onsite investigations, we anticipate an area of approximately 870m$^2$ to be at risk of soil creep or shallow failure, which is roughly 58% of the total site area.

**P** (S:H) = 0.58

**Possibility of the Location Being Occupied During Failure**

The average household is taken to be occupied by 12 people (6 people per occupancy). It is estimated/assumed that 8 people are in the house for 18 hours a day, 7 days a week. It is estimated/assumed that 4 people are in the house 12 hours a day, 5 days a week.

$$\left(\frac{8}{12} \times \frac{18}{24} \times \frac{7}{7}\right) + \left(\frac{4}{12} \times \frac{12}{24} \times \frac{5}{7}\right) = 0.62$$

**P** (T:S) = 0.62

**Probability of Loss of Life on Impact of Failure**

Based on the volume of land sliding and its likely velocity when it impacts the house, it is estimated that the vulnerability of a person to being killed in the house when a landslide hit is 0.1.

**V** (D:T) = 0.1

**Risk Estimation**

**R** (LOL) = 0.0001 x 0.58 x 0.62 x 0.1

= 0.0000036

**R** (LOL) = 3.6 x $10^{-6}$/annum.

Therefore, in accordance with AGS (2007c) this level of risk is considered to be "ACCEPTABLE".

### 4.6 Post-Development (Assessed Risk to Property)

Based on the existing site levels and topography, maximum inferred excavation depths of up to 3.8m are expected to be required for construction of the proposed basement level and in-ground swimming pools, with cut and fill elsewhere. Locally deeper excavations for the building footings and service trenches are also anticipated to be required as part of the planned development.

Therefore, appropriate measures against the potential for any instability should be incorporated into the design and construction of the proposed development, predominately into the design and construction of the building foundations and any retaining walls, as discussed and outlined in this report.

On the condition that the recommendations and design parameters presented in this report are taken into consideration during the design and construction of the proposed development, as well as post construction, the following assessed risks relating to the stability of the property upon completion of any infrastructure, building foundations and retaining walls are presented in Table 5 below.

**Table 5. Post-Development – Assessed Risk To Property**

| Potential Hazard | Qualitative Measures of Likelihood (AGS) | Qualitative Measures of Consequences to Property (AGS) | Qualitative Risk Analysis – Level of Risk to Property (AGS) |
|---|---|---|---|
| **Soil Creep[1]** | D – Unlikely ($10^{-4}$) | 4 – Minor (5%) | Low |
| **Shallow Failure[1]** | D – Unlikely ($10^{-4}$) | 4 – Minor (5%) | Low |

[1]Within the fill material and natural soils present within the site.

Based on the assessed conditions within the site, the overall slope instability assessed risk to the property following construction of the currently proposed development is assessed to be "low".

Therefore, providing the recommendations outlined in Section 4.4 and Section 5 are implemented for the design and construction of the proposed development, the above risk is considered acceptable for the proposed development within the site.

Geotechnical inspections are to be undertaken during construction of the proposed development foundation system in order to confirm ground conditions and that the allowable bearing capacities are achieved. Appropriate certifications should also be provided during staged inspections by the project structural engineer and geotechnical engineer.

# 5. GEOTECHNICAL ASSESSMENT AND RECOMMENDATIONS

## 5.1 Dilapidation Survey

It is recommended that prior to demolition, excavation and construction, a detailed dilapidation survey be carried out on all adjacent buildings, structures, council assets, road reserves and infrastructure that fall within the "zone of influence" of the proposed excavation and vicinity of the proposed development. A dilapidation survey will record the condition of existing defects prior to any works being carried out within the site. Preparation of a dilapidation report should constitute as a "Hold Point".

## 5.2 General Geotechnical Issues

The following aspects have been considered main geotechnical issues for the proposed development:

- Preliminary site lot classification.
- Excavation conditions.
- Groundwater management.
- Stability of excavation and retention of adjoining properties and infrastructure.
- Foundations and founding materials.

Based on results of our assessment, a summary of the geotechnical aspects above and recommendations for construction and designs are presented below.

## 5.3 Preliminary Site Lot Classification

Based on the geotechnical investigation and observations made at the selected testing locations within the site, fill and natural soils are expected to be underlain by inferred bedrock at varying depths throughout the site.

The governing site lot classification in accordance with AS 2870-2011 has been identified as "Class P" (Problematic Site) for the overall site, due to:

- The presence of infrastructure and trees within and adjoining the site, causing abnormal and changing moisture conditions.
- The possible presence of deep fill material in certain areas of the site, considered as "uncontrolled fill".
- The sloping nature of the site.

Based on the boreholes and DCP tests carried out within the site, and proposed excavations which will result in the removal of majority of the fill material and natural soils (in the basement footprint), AS 2870-2011 indicates the site may be classified as a **"**Class S**"** site for design and construction of the proposed basement foundation system founded below any soft/loose soils, topsoil, slopewash, fill or other deleterious material, being <u>entirely</u> on bedrock underlying the proposed development area (subject to confirmation).

A higher classification of "Class M" should be adopted for structures built at ground surface level (i.e. portions of the ground floor level, footings, fences, etc.), and/or where fill/natural soils are present at depths of equal to or greater than 0.5m below the proposed development FFLs. This should be confirmed/monitored during construction.

The above classification is solely based on assessment of the subsurface conditions at the selected boreholes and testing locations/depths within the site (no clayey soils present within the site), and confirmation should be carried out as described in this report. It should be noted that the classification given above is appropriate for the undeveloped lot at the time of this report and as such, AS 2870-2011 recommends that the classification of a site should be reconsidered if the depth of subsequent cutting exceeds 0.5m or depth of subsequent filling exceeds 0.4m

Foundation design and construction should be carried out as outlined in Section 5.9 below, with reference made to AS 2870-2011. Geotechnical inspections and confirmation of the actual depth of underlying fill material, natural soils and inferred bedrock should be made prior to construction by additional borehole drilling and rock strength testing, and during construction by inspection.

GCA should be contacted where ground conditions vary from those outlined in this report at the boreholes and testing locations. Where the building foundations are not proposed to be constructed on the bedrock underlying the site, GCA should also be contacted immediately for further advice.

Footing designs should take into consideration the effect of recent removal and planting of trees, along with any future tree removal within the vicinity of the proposed development on soil moisture conditions. Sufficient time should be given for soil moisture to re-equilibrate following any removal or planting of trees within the proposed development area, or specific engineering assessment and design will be required on the foundation design.

Although trees and vegetation are considered to contribute to the stability of the site, we recommend that planting of trees around the development area (i.e. in close proximity to the proposed building foundations) be limited as they can also affect moisture changes within the soil and cause significant displacement/damage within the building foundations by extensive tree root system movement.

It is recommended that reference is made to the recommendations provided by CSIRO "Guide to Home Owners on Foundation Maintenance and Footing Performance", attached as **Appendix F**.

### 5.4 Inspection Pits and Underpinning

Consideration should be given to inspection pits carried out for the existing adjacent buildings and infrastructure, particularly where they fall within the "zone of influence" (obtained by drawing a line $45^0$ above horizontal from the base of the proposed excavations) of the proposed development. This should be carried out prior to any demolition, excavation or construction activities, and will provide an assessment of the existing foundations of the adjacent buildings.

The assessment of the adjacent building footings should include assessment of the underlying soils, which will determine the need for additional support, such as underpinning, prior to installation of shoring piles, or any demolition, excavation and construction activities.

### 5.5 Excavation

Maximum excavation depths of up to 1.0m to 3.8m are expected to be required for construction of the proposed basement level and in-ground swimming pools, with cut and fill elsewhere. Locally deeper excavations for the building footings and service trenches are also anticipated.

Based on this information and existing ground conditions as encountered during the geotechnical investigation, it is anticipated that excavations will extend through fill material, natural soils and into *inferred* bedrock throughout the majority of the proposed development area, as discussed in Section 3 above.

The possibility of encountering higher estimated strength (i.e. medium to high estimated strength, or better) bedrock should not be precluded during excavation, predominantly where deeper excavations are required across the site, and in areas and at depths not assessed during the geotechnical investigation, due to the limited investigation carried out within the site.

Bedrock strength variation and higher strength rock bands may be expected across the site area. Therefore, consultation should be made with subcontractors to discuss the feasibility and capability of machinery for the proposed development for the existing site conditions.

### 5.5.1 Excavation Assessment

Excavation through softer soils and extremely low to low estimated strength bedrock should be feasible using conventional earth moving excavators, typically medium to large hydraulic excavators. Smaller sized excavators may encounter difficulty in high strength bands of soils and rocks which may be encountered. Where high strengths bands are encountered, rock breaking or ripping should be allowed for. Removal of the existing pavements and associated infrastructure within the site are also expected to require larger excavators and rock breaking and ripping.

Excavation of medium to higher estimated strength bedrock which is anticipated to be encountered during construction would require higher capacity excavators, bulldozers or similar, for effective removal of the rock. This excavation will require the use of heavy ripping and rock breaking equipment or vibratory rock breaking equipment. Furthermore, excavation for the proposed building footings, and service trenches may require the use of heavy ripping and rock breaking equipment or vibratory rock breaking equipment, with the possibility of rock saw cutting.

Should rock hammering be used for the excavation in the underlying bedrock, excavation should be carried out away from adjoining structures, with vibrations transmitted through the ground being monitored to maintain vibrations within acceptable limits. Rock saw cutting should be carried out (where required) around the perimeter of excavations, prior to any rock breaking commencing, to minimise transmission of ground vibrations.

Demolition, excavation and construction activities (or the like) will generate both vibration and noise, predominantly whilst being carried out within the underlying bedrock. Therefore, vibration control measures should be considered as part of the construction process, mainly where excavations are expected to be conducted within the underlying bedrock of higher estimated strength and fall within the "zone of influence" of adjoining infrastructure.

All excavation works should be carried out in accordance with the NSW WorkCover code of practice for excavation work.

## 5.6 Vibration Monitoring and Controls

Particular care will be required to ensure that adjacent buildings and infrastructure (i.e. road reserves, buildings, etc.), are not damaged during demolition, excavation and construction activities (or the like) due to excessive vibrations. Therefore, appropriate excavation and construction methods should be adopted which will limit ground vibrations to limits not exceeding the following maximum Peak Particle Velocity (PPV) for adjacent structures, as outlined in AS 2187.2-2006:

- Sensitive and/or historical structures – 2mm/sec.
- Residential and/or low rise structures – 5mm/sec.
- Unreinforced and/or brick structures – 10mm/sec.
- Reinforced and/or steel structures – 25mm/sec.
- Commercial and/or industrial buildings – 25mm/sec.

In order to reduce resonant frequencies, rock hammers should be used in short bursts and oriented away from the site boundaries and adjoining structures, and into the proposed excavation area.

Vibrations transmitted by the use of rock hammers are unacceptable and not recommended. To minimise vibration transmission to any adjoining infrastructure, and to ensure vibration limits remain within acceptable limits, rock saw cutting using a conventional excavator with a mounted rock saw (or similar) should be carried out as part of excavation prior to any rock breaking commencing.

Although rock hammering is unacceptable and not recommended, if necessary during excavation, it is recommended that hammering be carried out horizontally along pre-cut rock boulders or blocks provided by rock saw cutting, and should remain within limits acceptable. This should be monitored at all times during excavation.

The effectiveness of all the above-mentioned approaches must be confirmed by the results of vibration monitoring. The limits of 5mm/sec and 10mm/sec are expected to be achievable if rock breaker equipment or other excavations are restricted to the values indicated in Table 6 below.

**Table 6. Rock Breaking Equipment Recommendations**

| Distance From Adjoining Structures (m) | Maximum PPV 5mm/sec | | Maximum PPV 10mm/sec[1] | |
|---|---|---|---|---|
| | Equipment | Operating Limit (Maximum Capacity %) | Equipment | Operating Limit (Maximum Capacity %) |
| 1.5 to 2.5 | Jack Hammer Only (hand operated) | 100 | 300kg Rock Hammer | 50 |
| 2.5 to 5.0 | 300kg Rock Hammer | 50 | 300kg Rock Hammer | 100 |
| | | | 600kg Rock Hammer | 50 |
| 5.0 to 10.0 | 300kg Rock Hammer | 100 | 600kg Rock Hammer | 100 |
| | 600kg Rock Hammer | 50 | 900kg Rock Hammer | 50 |

[1]Vibration monitoring is recommended for the use of a maximum PPV of 10mm/sec.

Consideration should be given to a vibration monitoring plan to monitor construction activities and their effects on adjoining infrastructure, mainly where excavations are expected to be conducted within the underlying bedrock of higher estimated strength and fall within the “zone of influence” of adjoining infrastructure.

A vibration monitoring plan may be carried out attended or unattended. An unattended vibration monitoring must be fitted with alarms in the form of strobe lights, sirens or live alerts sent to the vibration monitoring supervisor, which are activated when the vibration limit is exceeded. If adopted/considered, consultation should be made with appropriate subcontractors/consultants for the installation of vibration monitoring instruments.

A geotechnical engineer should be contacted immediately if vibrations during construction or in adjacent structures exceed the values outlined above and work should immediately cease. Rock excavation methodology should also consider acceptable noise limits as per the “Interim Construction Noise Guideline” (NSW EPA). It is recommended a dilapidation report be carried out prior to any excavation or construction, as discussed in Section 5.1. This should be considered a “Hold Point”.

### 5.7 Groundwater Management

Based on the geotechnical investigation at the selected boreholes and testing locations, *inferred* groundwater seepage which may be encountered during construction is expected to be at varying depths across the site and possibly above the proposed basement FFL (subject to confirmation).

It should be noted that no provision was made for longer term groundwater monitoring within the site, and the presence of groundwater should not be precluded during construction and in the long term design life of the proposed building. It should also be noted that groundwater levels have the potential to fluctuate during daily or seasonal influences such as tidal fluctuations, heavy rainfall, damaged services, flooding, etc.

Thus, we expect any groundwater inflow into the excavation to be in the form of seepage through voids within the underlying soils and defects (such as bedding planes, joints, etc.) in the underlying inferred weathered bedrock. Seepage may also occur within the excavation areas through the fill material, and at the fill/natural soils and/or natural soils/inferred bedrock interfaces, predominately following heavy rain.

The rate of flow which may enter the excavation may initially be rapid, but is expected to decrease over time as the voids in the soils and defects in the underlying inferred bedrock are drained, and local water ingress decreases. As noted, groundwater levels are subject to fluctuations on a daily and seasonal basis, and the potential for groundwater to enter the excavation as moderate to rapid seepage should be considered as part of the long term design life of the building. The amount of seepage into the excavation will also depend on the shoring system being adopted.

Therefore, based on our assumptions that groundwater within the site is in the form of seepage, consideration should be given to precautionary drainage measures including (not limited to):

- A conventional sump and pump system which may be used both during construction and for permanent groundwater control below the proposed basement level floor slab.
- Drainage installed around the perimeter of the proposed basement level behind all retaining walls, and below the slab. This drainage should be connected to a sump and pump out system and discharged into the stormwater system (which may require council approval).
- Collection trenches or pipes and stormwater pits may be installed in conjunction with the above method, and connected to the building stormwater system.

Where a suitable drainage system has not been implemented or provided for the proposed development to collect and remove any groundwater, consideration may also be given to waterproofing of the proposed development level walls and slabs, with allowance given for nominal hydrostatic uplift.

#### 5.7.1 Groundwater Recommendations

To better assess groundwater levels and the potential for groundwater inflow into the excavations, it is recommended that test pits are carried out by a suitable excavator within the site prior to construction in order to confirm and monitor groundwater levels and inflow rates which may be intercepted during construction within the excavation areas. This assessment should also be carried through to ensure that a suitable drainage and retention system is implemented for the proposed development, as discussed in Section 5.8 below, and to provide confirmation of the hydrogeological characteristics of the site prior to construction.

Where a suitable drainage system has not been implemented or provided for the proposed development to collect and remove any groundwater, consideration may also be given to waterproofing of the basement level walls and slabs, with allowance given for nominal hydrostatic uplift.

Groundwater monitoring of seepage should also be implemented during the excavation stage to confirm the capacity of the drainage system and groundwater entering the excavation area. This should be monitored by the project geotechnical engineer, in conjunction with the project stormwater engineer.

Should the proposed development change and excavation depths exceed those inferred in this report, GCA should be made aware

## 5.8 Excavation Stability

Maximum excavation depths of up to 1.0m to 3.8m are expected to be required for construction of the proposed basement level and in-ground swimming pools, with cut and fill elsewhere. Locally deeper excavations for the building footings and service trenches are also anticipated.

Based on the ground conditions within the site, the total depth of excavation and the extent of the basement level walls to the site boundaries and adjoining infrastructure, it is critical from geotechnical perspective to maintain the stability of the adjacent structures and infrastructure during demolition, excavation and construction.

### 5.8.1 Batter Slopes

Temporary or permanent batters may be considered for certain areas of the proposed development where sufficient space exists between the proposed excavations and adjoining infrastructure. It should be noted that due to the nature of fill material, natural sandy soils and weathered bedrock underlying the site, and the potential for elevated groundwater levels within the excavation area, unsupported vertical cuts of the soils carry the potential for slump failure.

Temporary or permanent batter slopes should only be considered where sufficient space exists between the proposed development and adjoining infrastructure, and where the adjacent infrastructure is located outside the "zone of influence" (obtained by drawing a line $45^{0}$ above horizontal from the base of the proposed excavations).

Table 7 provides maximum recommended slopes for permanent and temporary batters.

**Table 7. Recommended Maximum Batter Slopes**

| Unit | | Maximum Batter Slope (H:V)[1] | |
|---|---|---|---|
| | | **Permanent** | **Temporary** |
| **Fill (Unit 1)** | | 4:1 | 2:1 |
| **Natural Sandy Soils (Unit 2)** | | | |
| **Inferred Bedrock (Unit 3)** | **EL – VL** | 2:1 | 1:1 to 0.75:1 |
| | **L or better[2]** | 1:1 | 0.5:1 |

[1]Subject to inspection and confirmation by a geotechnical engineer or engineering geologist. Remedial options may be required (i.e. soil nailing, rock bolting, shotcreting, etc.).
[2]Preliminary only and assumes no defects of adverse dipping is present in the bedrock and Sandstone bedrock underlies the site. Subject to confirmation by a geotechnical engineer during construction by inspection, and by borehole drilling, rock coring and rock strength testing.
Notes:

- EL = Extremely low estimated strength, VL = Very low estimated strength, L = Low estimated strength.

All batter slopes within the site should remain stable providing that all surcharge and construction loads are kept out of the "zone of influence" (obtained by drawing a line $45^{0}$ above horizontal from the base of the proposed excavations) plus an additional 1.0m. A geotechnical engineer or engineering geologist should inspect the batter slopes and excavated faces within the site, which we recommend at staged inspections at a minimum of the initial 0.5m depth, followed by 1.0m to 1.5m subsequent excavation depths, then at excavation depth intervals of 1.5m.

It should be noted that steeper batter slopes may be considered for higher strength (i.e. low to medium estimated strength, or better) and intact bedrock underlying the site, subject to confirmation by additional borehole drilling and rock strength testing. Consideration should be given to shotcreting and soil nailing where steeper batter slopes are to be used.

Temporary surface protection against erosion should be provided by covering the batter slopes with plastic sheets extending at least 1.5m behind the crest of the cut face or up to the common site boundaries. The sheets should be positioned and fastened to prevent any water infiltration onto or into the batter slopes. Other applicable methods may be adopted for temporary surface protection, and all surface protection should be placed following inspection of the temporary batters by a geotechnical engineer.

An appropriately designed retaining wall by a suitably qualified structural engineer should be implemented and constructed around the proposed basement level perimeter walls following any temporary or permanent batter slopes within the site. All retaining walls should be properly constructed on appropriate bedrock material underlying the site and should take into consideration the lateral earth pressures induced by soil movement along the interface between soils and the underlying bedrock.

### 5.8.2 Excavation Retention Support Systems

Where there is insufficient space between the proposed development and adjoining infrastructure (i.e. eastern and western site boundaries), or where adjacent infrastructure are located within the "zone of influence" (as outlined in Section 5.8.1 above), consideration should be given to a suitable retention system such as a soldier pile wall solution, with piles sufficiently embedded into consistent and competent strength bedrock underlying the site, and concrete and reinforcement infill panels for the support of the excavation and soils.

Closer spaced piles are recommended and may be required to reduce lateral movements particularly where adjacent infrastructure, such as buildings or pavements and road reserves are located near the excavation, and to prevent the collapse of loose/soft fill in-situ materials and natural soils (i.e. sandy soils), and weathered bedrock. Pile spacing should be analysed and designed by the project structural engineer and should consider horizontal pressures due to surcharge loads from adjacent infrastructure (i.e. buildings, road reserves, etc.), and long term loadings.

A soldier pile wall solution should only be considered where fill material and natural soils overlying the inferred bedrock may be safely battered to prevent the collapse of soils (see Section 5.7.1). Therefore, battering back of the soils will be required in certain areas (where permissible) of the site to permit installation of soldier piles and prevent the collapse of soils into the excavation area. This should be monitored by a geotechnical engineer familiar with these site conditions. Where this is not achievable, consideration should be given to an alternative retention system such as a contiguous pile wall solution.

The use of a more rigid retention system such as a cast in-situ contiguous pile wall solution should also be considered to reduce the lateral movements and risk of potential damage to adjacent infrastructure (i.e. buildings, infrastructure, adjacent road reserves, etc.). This option may also be adopted where excessive surcharges are adjacent to the proposed excavation and to meet acceptable deflection criteria, where loose/soft soils (i.e. sandy soils) are required to be retained, or where there is a potential for undermining of any adjoining building/infrastructure (refer to Section 5.4).

As noted, all piles should be sufficiently embedded into consistent and competent strength bedrock underlying the site and should be inspected and approved by a suitably qualified geotechnical engineer. Piles should not be founded into any soft or weak bands/layers (i.e. clay seams and/or extremely weathered/fractured zones) underlying the site. Furthermore, the retention system should be carefully selected by the project structural engineer, with all structural elements also inspected and approved by a suitably qualified structural engineer.

It should be noted that groundwater inflow may pass through shoring pile gaps during excavation. This may be controlled by the installation of strip drains behind the retention system connected to the buildings stormwater system. Shotcreting or localised grouting may also be used in weak areas of the retention system, predominately where groundwater seepage and loose/soft soils are visible. Shoring design should take into consideration both short term (during construction) and permanent conditions, along with surcharge loading and footing loads from adjacent infrastructure.

The design of retaining walls will depend on the method of construction being adopted. Common methods include (not limited to):

- Top-down construction.
- Bottom-up construction.
- Staged excavation and installation of props and/or partial berms.

Retaining or shoring walls may require anchors. In cases where anchoring is impractical, other temporary support for the adopted shoring system should be considered. This may include the staged excavation and installation of temporary berms or props in front of the retaining walls.

If considered, the shoring wall (and other retaining walls) can be designed using the recommended design parameters provided in Section 5.8.3. Bulk excavation and foundations (including pile installations) should be supervised, monitored and inspected by a geotechnical engineer, with all structural elements of the development by a structural engineer. Inspections should be considered as “Hold Points” to the project.

### 5.8.3 Design Parameters (Earth Pressures)

Pressures acting on a bored pile retaining wall or other types of retaining wall will depend on a number of factors including;

- Lateral earth pressure;
- Hydrostatic and earthquake pressures (if applicable);
- The stiffness of the retaining wall;
- Whether the wall is anchored;
- Presence and levels of groundwater behind the wall;
- Slope of the surface behind the wall;
- The nature of the material being retained, and;
- The construction sequence of the proposed development.

Lateral earth pressure is affected by external forces from applied surcharge loads in the zone of influence of the wall, such as loads imposed by existing structures, vehicle traffic and construction activities.

Therefore, the following parameters may be used for the design of temporary and permanent retaining walls at the subject site:

- A triangular earth pressure distribution may be adopted for derivation of active pressures where a simple support system (i.e. cantilevered wall or propped/anchored wall with only one row of props/anchors are required) is adopted. Cantilevered walls are typically less than 2.5m in height, and should ensure deflections remain within tolerable limits.
    - Flexible retaining structures (i.e. cantilevered walls or walls with only one row of anchors), should be based on active lateral earth pressure. “At rest” earth pressure coefficient should be considered to limit the horizontal deformation of the retaining structure. Lateral active (or at rest) and passive earth pressures for cantilever walls or walls with only one row of anchors may be determined as follows:

**Lateral active or “at rest” earth pressure:**

$$P_a = K\,\gamma\,H - 2c\sqrt{K}$$

**Passive earth pressure:**

$$P_p = K_p \gamma H + 2c\sqrt{K_p}$$

- Where lateral deflection exceeds tolerable limits, or where two or more rows of anchors are required, the retention/shoring system should be designed as a braced structure. This more complex support system should utilise advanced numerical analysis tools such as WALLAP or PLAXIS which can ensure deflections in the walls remain within tolerable limits and to model the sequence of anchor installation and excavation. For braced retaining walls, a uniform lateral earth pressure should be adopted as follows:

**Active earth pressure:**

$$P_a = 0.65\, K \gamma H$$

Where:

| | |
|---|---|
| $P_a$ | = Active (or at rest) Earth Pressure (kN/m$^2$) |
| $P_p$ | = Passive Earth Pressure (kN/m$^2$) |
| $\gamma$ | = Bulk density (kN/m$^3$) |
| K | = Coefficient of Earth Pressure ($K_a$ or $K_o$) |
| $K_p$ | = Coefficient of Passive Earth Pressure |
| H | = Retained height (m) |
| c | = Effective Cohesion (kN/m$^2$) |

- Support systems and retaining structures 'should be designed to withstand hydrostatic pressures, lateral earth pressures and earthquake pressures (if applicable). The applied surcharge loads in their “zone of influence” should also be considered as part of the design, where the “zone of influence” may be obtained by drawing a line 45$^0$ above horizontal from the base of the proposed excavations.

Support system designed using the earth pressure approach may be based on the parameters given in Table 8 below for soil and rock horizons underlying the site. Table 8 also provides preliminary coefficients of lateral earth pressure for the soils and rock horizons encountered in the site. These are based on fully drained conditions and that the ground behind the retention walls is horizontal.

**Table 8. Preliminary Geotechnical Design Parameters**

| Material | Fill (Unit 1) | Natural Soils (Unit 2) | Inferred Bedrock[3, 5] (Unit 3) |
|---|---|---|---|
| | | | VL or better |
| **Unit Weight ($kN/m^3$)[4]** | 16 | 17 | 20 |
| **Effective Cohesion c' (kPa)** | 0 | 0 | 25 |
| **Angle of Friction $\phi'$ (°)** | 24 | 30 | 30 |
| **Modulus of Elasticity $E_{sh}$ (MPa)** | 3 | 15 (medium dense, or better) | 70 |
| **Earth Pressure Coefficient At Rest Ko[1]** | 0.59 | 0.5 | 0.5 |
| **Earth Pressure Coefficient Active Ka[2]** | 0.42 | 0.3 | 0.3 |
| **Earth Pressure Coefficient Passive Kp[2]** | 2.37 | 3.0 | 3.0 |
| **Poisson Ratio $v$** | 0.4 | 0.35 | 0.3 |

[1]Earth pressure coefficient at rest (Ko) can be calculated using Jacky's equation.
[2]Earth pressure coefficient of active (Ka) and passive (Kp) can be calculated using Rankine's or Coulomb's equation.
[3]The values for rock assume no defects of adverse dipping is present in the bedrock and Sandstone bedrock underlies the site. All excavation rock faces should be inspected on a regular basis by an experienced engineering geologist or geotechnical engineer.
[4]Above groundwater levels.
[5]Subject to confirmation by a geotechnical engineer by borehole drilling and rock strength testing, and during construction by inspection. Conforming to at least Class V Sandstone (or better).

Notes:

- VL = Very low estimated strength.
- VL bedrock should conform to at least Class V Sandstone in accordance with Pells P.J.N, Mostyn G. & Walker B.F (1998).

### 5.9 Foundations

Based on current information available at the time of reporting, it is anticipated that fill material, natural soils and bedrock of variable weathering and estimated strength will be exposed at bulk excavation level (depending on the actual amount of excavation required).

Variable composition and consistency/strength natural soils and fill material are likely to result in total and differential settlement under working load, and not adequately support shallow foundations for the proposed development within the site. Removal of the fill material within the proposed development area should be carried out prior to construction of the proposed building foundation system.

It is noted that ground conditions within the site are expected to differ from those encountered and inferred in this report, since no geotechnical or geological exploration program, no matter how comprehensive, can reveal and identify all subsurface conditions underlying the site. It is therefore recommended that confirmation of the underlying ground conditions be confirmed by a geotechnical engineer during construction by inspection.

### 5.9.1 Geotechnical Assessment

Based on the proposed development and assessment of the subsurface conditions, a suitable foundation system comprising combination of shallow foundations typically containing pad and/or strip footings, and a piled foundation system are likely to be adopted for the proposed development and should be constructed and sufficiently embedded into consistent and competent strength bedrock underlying the site.

All piles should be sufficiently embedded into consistent and competent strength bedrock in areas where bedrock is not exposed at bulk excavation level and should fully support the building/infrastructure. Consideration should be given to Continuous Flight Auger (CFA) piling or lining/casing the upper sections of piles to retain the fill material and natural sandy soils.

Shallow foundations should only be considered in areas where bedrock is expected to be exposed at or shortly below bulk excavation level and should include local slab thickening to support internal walls and columns for shallow foundations, with consideration given to settlement reducing piles. Foundations should not be founded on any soft/weak bands (i.e. clay seams and/or extremely weathered/fractured zones) underlying the site.

Installation of piles and foundation construction should be complemented by inspections carried out by a geotechnical engineer during construction, to confirm ground conditions are consistent throughout and allowable bearing capacities have been achieved. The actual depth and embedment of the piles should be assessed by the project structural engineer, with all structural elements of the proposed development also inspected and approved by a suitably qualified structural engineer.

Given the potential for variable ground conditions and soil reactivity across the site, it is recommended that all foundations are constructed on consistent and competent bedrock throughout, in order to provide uniform support and reduce the potential for differential settlements. This could be attained by strip or pad footings where the suitable bearing capacity is achieved or exposed at bulk excavation level, and pile foundations elsewhere. Reference should be made to the estimated levels of the subsurface conditions outlined in this report and compared to the final bulk excavation levels across the site.

Installation of piles may be required where the axial and working loads transmitted through the building walls and columns exceed the bearing pressure of the bedrock exposed at the proposed developments FFLs. These should be socketed into consistent and appropriate bedrock underlying the site. For cases where resistance against lateral loading induced by earthquakes or winds, and to achieve higher bearing capacities, piles may also be required.

Piles sufficiently socketed into higher strength bedrock may achieve greater allowable bearing capacities, subject to confirmation from a geotechnical engineer during construction by inspection. Where higher estimated strength bedrock is present within the site, or where ground conditions vary from those encountered during the geotechnical investigation, GCA should be contacted for further advice.

Table 9 provides preliminary recommended geotechnical design parameters.

**Table 9. Preliminary Recommended Geotechnical Design Parameters**

| Unit Type/Material | | Maximum Allowable (Serviceability) Values (kPa) | | |
|---|---|---|---|---|
| | | End Bearing Pressure[1] | Shaft Adhesion (Compression) | Shaft Adhesion (Tension) |
| Fill (Unit 1) | | N/A | N/A | N/A |
| Natural Soils (Unit 2) | | N/A | N/A | N/A |
| Inferred Bedrock (Unit 3)[2] | VL | 700 | 40 | 20 |
| | L or better | 1,500 | 150 | 75 |

[1]Minimum embedment of 0.4m for shallow foundations and 0.5m for deep foundations. Assumes the presence of Sandstone bedrock underlying the entire site area.
[2]The composition, class, depth and estimated strength of the underlying bedrock material should be confirmed either prior to construction by further borehole drilling and rock strength testing, and during construction by inspection. Bedrock should conform to at least Class V Sandstone (or better) in accordance with Pells P.J.N, Mostyn G. & Walker B.F (1998).
Notes:

- VL = Very low estimated strength, L = Low estimated strength.
- VL and L bedrock should conform to at least Class V and Class IV Sandstone, respectively, in accordance with Pells P.J.N, et al (1998).
- Higher allowable bearing capacities may be attained for higher estimated strength rock assessed and confirmed by a geotechnical engineer.
- All shaft adhesion parameters are based on adequately clean and rough sockets of category "R2", or better.
- N/A = Not Applicable. Not recommended for the proposed development.
- It is recommended that geotechnical inspections on the foundations are completed by a geotechnical engineer to determine the material and confirm the required bearing capacity is achieved.

Footings designed using ultimate values and limit state design will need to consider serviceability which usually governs designs in these cases. For pile designs, a basic geotechnical reduction factor ($\Phi_{gb}$) should be calculated by the structural engineer from AS 2159-2009, taking into consideration the design, installation method and associated risk rating. Furthermore, the design structural engineer should check both 'piston' pull-out and 'cone' pull-out mechanics in accordance with AS 4678-2002.

### 5.9.1 Geotechnical Comments

Bearing capacity and settlement behaviour varies according to foundation depth, shape and dimensions, including method of installation for piles. Consultation should be made with specialist subcontractors to discuss the feasibility of piles for the existing site conditions. It should be noted that higher bearing capacities may be justified for the proposed foundations subject to confirmation by borehole drilling with rock coring, rock strength testing and by assessment during excavation and construction.

Specific geotechnical advice should be obtained for footing designs and end bearing capacities, and design of the foundation system (shallow and pile foundations) should be carried out in accordance with AS 2870-2011 and AS 2159-2009.

Foundations located within the "zone of influence" of any services or sensitive structures should be supported by a piled foundation. The depths of the piles should extend below the "zone of influence" and should ignore any shaft adhesion. Appropriate measures should be taken to ensure that any services or sensitive structures located within the "zone of influence" of the proposed development are not damaged during and following construction.

It is recommended that suitable drainage and the use of impermeable surfaces be implemented as a precaution as part of the design and construction of the proposed development in order to divert surface water away from the building, and help eliminate or minimise surface water infiltration to minimise moisture within the soils.

The design and construction of the foundations should take into consideration the potential of flooding. All foundation excavations should be free of any loose debris and wet soils, and if groundwater seepage or runoff is encountered dewatering should be carried out prior to pouring concrete in the foundations. Due to the possibility of groundwater being encountered and possible groundwater seepage during installation of bored piles within the site, including the nature of collapsing fill material and natural sandy soils, it is recommended that consideration be given to using a liner in the section of fill and natural soils to avoid collapse of the soil. Other alternatives may be the use of CFA piles.

Shaft adhesion may be applied to socketed piles adopted for foundations provided that the socketed shaft lengths conform to appropriate classes of bedrock (subject to confirmation) in accordance with Pells et. al, (1998, 2019) and shaft sidewall cleanliness and roughness are to acceptable levels (minimum R2 category, refer to Pells (1999) and **Appendix C**). Shaft adhesion should be ignored or reduced within socket lengths that are clay smeared or fail to satisfy cleanliness requirements (i.e. at least 80%). It is recommended that where piles penetrate expansive soils present within the site (if encountered), which are susceptible to shrink and swell due to daily and seasonal moisture changes, shaft adhesion be ignored due to the potential for shrinkage cracking. Pile inspections should be complemented by the use of a downhole CCTV camera for inspection of sidewall cleanliness.

We recommend that geotechnical inspections of foundations be completed by an experienced geotechnical engineer to determine that the designed socket materials are reached and that the required bearing capacity is achieved. The geotechnical engineer should also determine any variations between the borehole carried out and inspected locations. Inspections should be carried out in dewatered foundations for a more accurate examination, and inspections should be carried out under satisfactory WHS requirements. Geotechnical inspections for verification capacities of the foundations should constitute as a "Hold Point".

### 5.10 Subgrade Preparation and Filling

Prior to emplacement of engineered Fill, the sub-grade must be suitably prepared.

#### 5.10.1 Subgrade Preparation

The following are general recommendations on subgrade preparation for earthworks and emplacement of engineered Fill, slab on ground construction and pavements:

- Remove existing fill and topsoil, including all materials which are unsuitable from the site.
- Excavate any natural soils and rock.
    - Excavated natural material is not considered suitable for engineered fill. Rock may be used for subgrade material underlying pavements, providing appropriate geotechnical inspections and laboratory testing of the material is undertaken to confirm its suitability.
- Any natural soils (predominantly clayey soils) exposed at the bulk excavation level should be treated and have a moisture condition of 2% OMC. This should be followed by proof rolling and compaction of the upper 150mm layer.
    - Any soft or loose areas should be removed and replaced with engineered or approved fill material.
- Any rock exposed at the bulk excavation level should be clear of any deleterious materials (and free of loose or softened materials). As a guideline, remove an additional 150mm from the bulk excavation level.
- Ensure the foundations and excavated areas are free of water prior to concrete pouring.
- Areas which show visible heaving under compaction or proof rolling should be excavated at least 300mm and replaced with engineered or approved fill and compacted to a minimum dry density ratio not less than 98% of the maximum dry density.

### 5.10.1 Filling Specifications

Where filling is required, the following recommended compaction targets should be considered:

- Place horizontal loose layers not more than 150mm thickness over the prepared subgrade.
- Compact to a minimum dry density ratio not less than 98% of the maximum dry density for the building platforms.
- The moisture content during compaction should be maintained at ±2% of the Optimal Moisture Content (OMC).
- The upper 150mm of the subgrade should be compacted to a dry density ratio not less than 100% of the maximum dry density.

Any soils which are imported onto the site for the purpose of filling and compaction of the excavated areas should be free of deleterious materials and contamination. The imported soils should also include appropriate validation documentation in accordance with current regulatory authority requirements. The design and construction of earthworks should be carried out in accordance with AS 3798-2007 and AS 1289. Inspections of the prepared subgrade should be carried out by a geotechnical engineer, and should include proof rolling as a minimum. These inspections should be established as “Hold Points.

## 6. ADDITIONAL GEOTECHNICAL RECOMMENDATIONS

Furthermore, following completion of the geotechnical investigation and report, GCA recommends the following additional work to be carried out:

- Implementation of <u>all</u> mitigation and control measures for the proposed development, as outlined in this report.
- The recommendations in the AGS document “Some Guidelines for Hillside Construction” (see **Appendix H**) should be followed.
- Dilapidation survey report on adjacent properties and infrastructure.
- Monitoring and supervision of excavations within the site, including appropriate inspections and approvals on all batters adopted throughout the site (where feasible). Geotechnical inspections of exposed materials in the excavated faces at an initial 0.5m depth, followed by maximum 1.5m excavation depth intervals, and at bulk excavation level.
- The composition, class, depth and estimated strength of the underlying inferred bedrock material <u>should be confirmed</u> prior to construction by further borehole drilling and rock strength testing, and during construction by inspection, predominantly in areas and at depths not assessed during the geotechnical investigation.
- Geotechnical inspections of exposed materials at bulk level excavation.
- Geotechnical inspections of shoring wall piles installations.
- Geotechnical inspections of foundations (shallow and pile foundations) to confirm the preliminary bearing capacities have been achieved.
- Monitoring of any groundwater inflows into the excavation areas within the site.
- Provision for longer term groundwater monitoring within the site. This could be undertaken by groundwater assessment by excavation of two (2) pits to bulk excavation level and monitoring groundwater levels, and groundwater or seepage inflow rates and volumes.
- Classification of all excavated material transported from the site.
- If required, a meeting to be carried out to discuss any geotechnical issues and inspection requirements.
- Final architectural and structural design drawings are provided to GCA for further assessment.

## 7. LIMITATIONS

Geotechnical Consultants Australia Pty Ltd (GCA) has based its geotechnical assessment on available information obtained prior and during the site inspection/investigation. The geotechnical assessment and recommendations provided in this report, along with the surface, subsurface and geotechnical conditions are limited to the inspection and test areas during the site inspection/investigation, and then only to the depths investigated at the time the work was carried out. Subsurface conditions can change abruptly, and may occur after GCA's field testing has been completed.

It is recommended that if for any reason, the site surface, subsurface and geotechnical conditions (including groundwater conditions) encountered during the site inspection/investigation vary substantially during construction, and from GCA's recommendations and conclusions, GCA should be contacted immediately for further testing and advice. This may be carried out as necessary, and a review of recommendations and conclusions may be provided at additional fees. GCA's advice and accuracy may be limited by undetected variations in ground conditions between sampling locations.

GCA does not accept any liability for any varying site conditions which have not been observed, and were out of the inspection or test areas, or accessible during the time of the investigations. This report and any associated information and documentations have been prepared solely for **Georgina Nassif & Tony Nassif,** and any misinterpretations or reliances by third parties of this report shall be at their own risk. Any legal or other liabilities resulting from the use of this report by other parties can not be religated to GCA.

This report should be read in full, including all conclusions and recommendations. Consultation should be made to GCA for any misundertandings or misinterpretations of this report.

For and behalf of

**Geotechnical Consultants Australia Pty Ltd (GCA)**

GeorgeA

George Abou-Antoun
BEng (Hons) (Civil - Geotechnical) & BPM
GradIEAust (6353384)
*Member of AGS and ISSMGE*
Geotechnical Engineer

**Reviewed by:**

Jnader

Joe Nader
B.E. (Civil – Construction), Dip.Eng.Prac.,
MIEAust., RPEng, NER (3943418), RPEQ (25570)
Cert. IV in Building and Construction
*Member of AGS and ISSMGE*
NSW Fair Trading DPR No.: DEP0000184
NSW Fair Trading PER No.: PRE0000174
Geotechnical Engineer
Director

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NSW Planning Portal.

NSW Six Maps.

Mecone Mosaic.

eSPADE NSW Environment & Heritage.

Nearmap.

# APPENDIX A

### Important Information About Your

# Geotechnical Report

This geotechnical report has been prepared based on the scopes outlined in the project proposal. The works carried out by Geotechnical Consultants Australia Pty Ltd (GCA), have limitations during the site investigation, and may be affected by a number of factors. Please read the geotechnical invesitgation report in conjunction with this "Important Information About Your Geotechnical Report".

**Geotechnical Services Are Performed for Specicif Projects, Clients and Purposes.**
Due to the fact that each geotechnical investigation is unique and varies from sites, each geotechnical report is unique, and is prepared soley for the client. A geotechnical report may satisfy the needs of structural engineer, where is will not for a civil engineer or construction contractor. No one except the client should rely on the geotechnical report without first conferring with the specific geotechnical consultant who prepared the report. The report is prepared for the contemplated project or original purpose of the investigation. No one should apply this report to any other or similar project.

**Reading The Full Report.**
Do not read selected elements of the report or tables/figures only. Serious problems have occurred because those relying on the specially prepared geotechnical invesitgation report did not read it all in full context.

**The Geotechnical Report is Based on a Unique Set of Project And Specific Factors.**
When preparing a geotechnical report, the geotechnical engineering consultant considers a number of unique factors for the specific project. These typially include:

- Clients objectives, goals and risk management preferences;
- The general proposed development or nature of the structure involved (size, location, etc.); and
- Future planned or existing site improvements (parking lots, roads, underground services, etc.);

Care should be taken into identifying the reason of the geotechnical report, where you should not rely on a geotechnical engineering report that was:

- Not prepared for your project;
- Not prepared for the specific site;
- Not prepared for you;
- Does not take into consideration any important changes made to the project; or
- Was carried out prior to any new infrastructure on your subject site.

Typical changes that can affect the reliabiliy if an existing geotechical investigation report include those that affect:

- The function of the proposed structure, where it may change from one basement level to two basement levels, or from a light structure to a heavy loaded structure;
- Location, size, elevation or configuration of the proposed development;
- Changes in the structural design occur; or
- The owner of the proposed development/project has changed.

The geotecnical engineer of the project should always be notified of any changes – even minor – and be asked to evaluate if this has any impact. GCA does not accept responsibility or liability for problems that occur because its report did not consider developments which it was not informed of.

**Subsurface Conditions Can Change**
This report is based on conditions that existed at the time of the investigation, at the locations of the subsurface tests (i.e. boreholes) carried out during the site investigation. Subfurface conditions can be affected and modified by a number of factores including, but not limited to, the passage of time, man-made influences such as construction on or adjacent to the site, by natural forces such as floods, groundwater fluctuations or earthquakes. GCA should be contacted prior to submitting its report to determine if any further testing may be required. A minor amount of additional testing may prevent any major problems.

**Geotechnical Findings Are Professional Opinions**
Results of subsurface conditions are limited only to the points where the subsurface tests were carried out, or where samples were collected. The field and laboratory data is analysed and reviewed by a geotechnical engineer, who then applys their professional experience and recommendations about the site's subsurface conditions. Despite investigation, the actual subsurface conditions may differ – in some cases significantly – from the results presented in the geotechnical investigation report, since no subsurface exploration program, no matter how comprehensive, can reveal all subsurface anomalies and details.

Therefore, the recommendations in this report can only be used as preliminary. Retaining GCA as your geotechnical consultants on your project to provide construction observations is the most effective method of managing the risks associated with unanticipated subsurface conditions.

**Geotechnical Report's Recommendations Are Not Final**
Because geotechnical engineers provide recommendations based on experience and judgement, you should not overrely on the recommendations provided – they are not final. Only by observing the actual subsurface conditions revealed during construction may a geotechnical engineer finalise their recommendations. GCA does not assume responsibility or liability for the report's recommendations if no additional observations or testing is carried out.

**Geotechnical Report's Are Subject to Misinterpretations**
The project geotechnical engineer should consult with appropriate members of the design team following submission of the report. You should review your design teams plans and drawings, in conjunction with the geotechnical report to ensure they have all be incorporated. Due to many issues arising from misinterpretation of geotechnical reports between design teams and building contractors, GCA should participate in pre-construction meetings, and provide adequate construction observations.

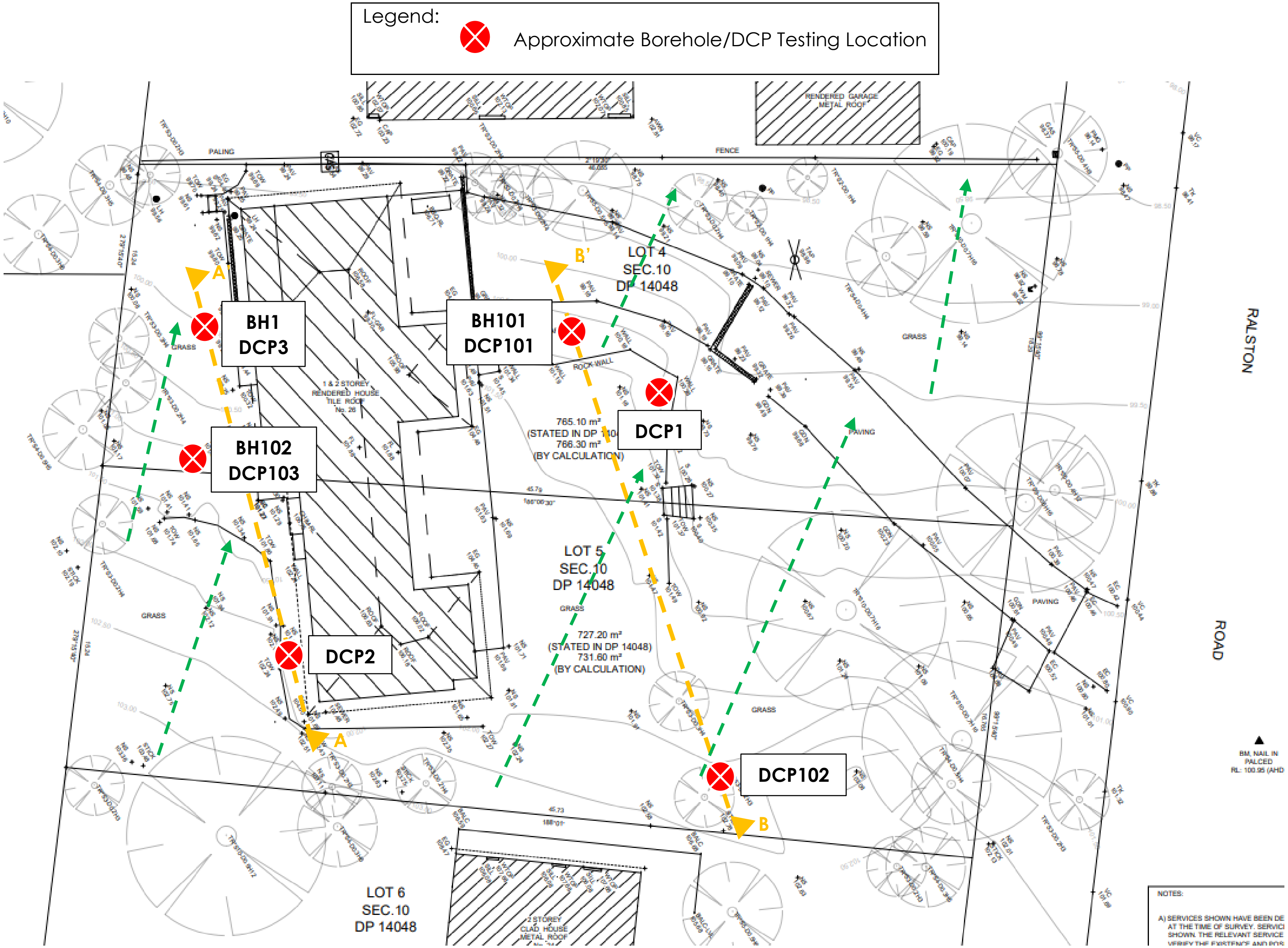
**Engineering Borehole Logs And Data Should Not be Redrawn**
Geotechnical engineers prepare final borehole and testing logs, figure, etc. based on results and interpretation of field logs and laboratory data following the site investigation. The logs, figure, etc. provided in the geotechnical report should never be redrawn or altered for inclusion in any other documents from this report, includined architectural or other design drawings.

**Providing The Full Geotechnical Report For Guidance**
The project design teams, subcontactors and building contractors should have a copy of the full geotechnical investigation report to help prevent any costly issues. This should be prefaced with a clearly written letter of transmittal. The letter should clearly advise the aforementioned that the report was prepared for proposed development/project requirements, and the report accuracy is limited. The letter should also encourage them to confer with GCA, and/or carry out further testing as may be required. Providing the report to your project team will help share the financial responsibilities stemming from any unanticipated issues or conditions in the site.

**Understanding Limitation Provisions**
As some clients, contractors and design professionals do not recognise geotechnical engineering is much broader and less exact than other engineering disciplines, this creates unrealistic expectations that lead to claims, disputs and other disappointments. As part of the geotechnical report, (in most cases) a 'limitations' explanatory provision is included, outlining the geotechnical engineers' limitations for your project – with the geotechnical engineers responsibilites to help other reduce their own. This should be read closely as part of your report.

**Other Limitations**
GCA will not be liable to revise or update the report to take into account any events or circumstances (seen or unforeseen), or any fact occurring or becoming apparent after the date of the report. This report is the subject of copyright and shall not be reproduced either totally or in part without the express permission of GCA. The report should not be used if there have been changes to the project, without first consulting with GCA to assess if the report's recommendations are still valid. GCA does not accept any responsibility for problems that occur due to project changes which have not been consulted.

# APPENDIX B




| GCA Geotechnical Consultants Australia | Figure 1<br>Site Plan | **Geotechnical Investigation** | Drawn: GN/NW | |
|---|---|---|---|---|
| | | Georgina Nassif & Tony Nassif | Date: 19/09/2024 | |
| | Job No.:<br>G21118-1-Rev A | 26 Ralston Road<br>Palm Beach NSW 2108 | Scale: NTS | |

Image source: Site survey plan prepared by C & A Surveyors NSW Pty Ltd, titled "Detail Survey of Lot 4, 5 Sec. 10 in DP14048, Located at No. 26 Ralston Street, Palm Beach", referenced No. 11303-19, sheet 1 of 1 and dated 21st May 2019.

# APPENDIX C

GCA
Geotechnical Consultants Australia

# Explanation of Notes, Abbreviations and Terms Used on Borehole and Test Pit Reports

## DRILLING/EXCAVATION METHOD

| Method | Description |
|---|---|
| AS | Auger Screwing |
| BH | Backhoe |
| CT | Cable Tool Rig |
| EE | Existing Excavation/Cutting |
| EX | Excavator |
| HA | Hand Auger |
| HQ | Diamond Core – 63mm |
| JET | Jetting |
| NMLC | Diamond Core – 52mm |
| NQ | Diamond Core – 47mm |
| PT | Push Tube |
| RAB | Rotary Air Blast |
| RB | Rotary Blade |
| RT | Rotary Tricone Bit |
| TC | Auger TC Bit |
| V | Auger V Bit |
| WB | Washbore |
| DT | Diatube |
| CC | Concrete Coring |

## PENETRATIION/EXCAVATION RESISTANCE

These assessments are subjective and dependant on many factors including the equipment weight, power, condition of the drilling tools or excavation, and the experience of the operator.

L **Low Resistance.** Rapid penetration possible with little effort from the equipment used.

M **Medium Resistance.** Excavation possible at an acceptable rate with moderate effort required from the equipment used.

H **High Resistance.** Further penetration is possible at a slow rate and required significant effort from the equipment.

R **Refusal or Practical Refusal.** No further progress possible within the risk of damage or excessive wear to the equipment used.

## WATER

Water level at date shown

Partial water loss

Water inflow

Complete water loss

***Groundwater not observed**:* The observation of groundwater, whether present or not, was not possible due to drilling water, surface seepage or cave in of the borehole/test pit.

***Groundwater not encountered**:* No free-flowing (springs or seepage) was intercepted, although the soil may be moist due to capillary water. Water may be observed in low permeable soils if the test pits/boreholes had been left open for at least 12-24 hours.

## MOISTURE CONDITION (AS 1726-2017)

Dry - Cohesive soils are friable or powdery
Cohesionless soil grains are free-running

Moist - Soil feels cool, darkened in colour
Cohesive soils can be moulded
Cohesionless soil grains tend to adhere

Wet - Cohesive soils usually weakened
Free water forms on hands when handling

For cohesive soils the following codes may also be used:

| | |
|---|---|
| MC>PL | Moisture Content greater than the Plastic Limit. |
| MC~PL | Moisture Content near the Plastic Limit. |
| MC<PL | Moisture Content less than the Plastic Limit. |

## SAMPLING AND TESTING

| Sample | Description |
|---|---|
| B | Bulk Disturbed Sample |
| DS | Disturbed Sample |
| Jar | Jar Sample |
| SPT* | Standard Penetration Test |
| U50 | Undisturbed Sample – 50mm |
| U75 | Undisturbed Sample – 75mm |

*SPT (4, 7, 11 N=18). 4, 7, 11 = Blows per 150mm. N= Blows per 300mm penetration following 150mm sealing.
SPT (30/80mm). Where practical refusal occurs, the blows and penetration for that interval is recorded.

## ROCK QUALITY

The fracture spacing is shown where applicable and the Rock Quality Designation (RQD) or Total Core Recovery (TCR) is given where:

$$TCR\ (\%) = \frac{\text{length of core recovered}}{\text{length of core run}}$$

$$RQD\ (\%) = \frac{\text{sum of axial lengths of core} > \text{100mm long}}{\text{length of core run}}$$

## ROCK STRENGTH TEST RESULTS

● Diametral Point Load Index test

■ Axial Point Load Index test

## SOIL ORIGINS

It is often difficult to accurately determine the origin of a soil. Soils can generally be classified as:

- **Residual soils**: derived from in-situ weathering of the underlying rock (see "rock material weathering" below).
- **Transported soils**: formed somewhere else and transported by nature to the site.
- **Filling**: moved/placed by man.

Transported soils may be further subdivided into:

- **Alluvium/alluvial**: river deposits.
- **Lacustrine**: lake deposits.
- **Aeolian**: wind deposits.
- **Littoral**: beach deposits.
- **Estuarine**: tidal river deposits.
- **Talus**: scree or coarse colluvium.
- **Slopewash or colluvium/colluvial**: transported downslope by gravity assisted by water. Often includes angular rock fragments and boulders.

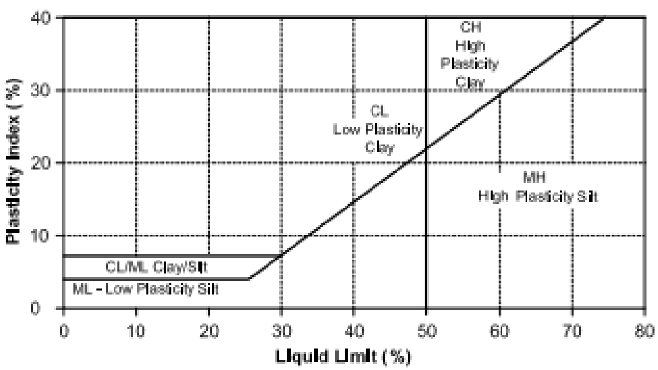
# Method and Terms for Soil and Rock Descriptions Used on Borehole and Test Pit Reports

Soil and Rock is classified and described in reports of boreholes and test pits using the preferred method given in AS 1726-2017, Appendix A. The material properties are assessed in the field by visual/tactile methods. The appropriate symbols in the Unified Soil Classification are selected on the result of visual examination, field tests and available laboratory tests, such as, sieve analysis, liquid limit and plasticity index.

## COHESIONLESS SOILS PARTICLE SIZE DESCRIPTIVE TERMS

| Name | Subdivision | Size |
|---|---|---|
| Boulders | | >200mm |
| Cobbles | | 63mm to 200mm |
| Gravel | coarse | 20mm to 63mm |
| | medium | 6mm to 20mm |
| | fine | 2.36mm to 6mm |
| Sand | coarse | 600µm to 2.36mm |
| | medium | 200µm to 600µm |
| | fine | 75µm to 200µm |

## PLASTICITY PROPERTIES




## COHESIVE SOILS – CONSISTENCY (AS 1726-2017)

| Strength | Symbol | Undrained Shear Strength, $C_u$ (kPa) |
|---|---|---|
| Very Soft | VS | < 12 |
| Soft | S | 12 to 25 |
| Firm | F | 25 to 50 |
| Stiff | St | 50 to 100 |
| Very Stiff | VSt | 100 to 200 |
| Hard | H | > 200 |
| Friable | Fr | Easily crumbled or broken into small pieces by hand |

## PLASTICITY

| Description of Plasticity | LL (%) |
|---|---|
| Low | <35 |
| Medium | 35 to 50 |
| High | >50 |

## COHESIONLESS SOILS - RELATIVE DENSITY

| Term | Symbol | Density Index | N Value (blows/0.3 m) |
|---|---|---|---|
| Very Loose | VL | 0 to 15 | 0 to 4 |
| Loose | L | 15 to 35 | 4 to 10 |
| Medium Dense | MD | 35 to 65 | 10 to 30 |
| Dense | D | 65 to 85 | 30 to 50 |
| Very Dense | VD | >85 | >50 |

## UNIFIED SOIL CLASSIFICATION

| USC Symbol | Description |
|---|---|
| GW | Well graded gravel |
| GP | Poorly graded gravel |
| GM | Silty gravel |
| GC | Clayey gravel |
| SW | Well graded sand |
| SP | Poorly graded sand |
| SM | Silty sand |
| SC | Clayey sand |
| ML | Silt of low plasticity |
| CL | Clay of low plasticity |
| OL | Organic soil of low plasticity |
| MH | Silt of high plasticity |
| CH | Clay of high plasticity |
| OH | Organic soil of high plasticity |
| Pt | Peaty Soil |

## ROCK MATERIAL WEATHERING

| Symbol | Term | Definition |
|---|---|---|
| RS | Residual Soil | Soil definition on extremely weathered rock; the mass structure and substance are no longer evident; there is a large change in volume but the soil has not been significantly transported |
| EW | Extremely Weathered | Rock is weathered to such an extent that it has 'soil' properties, i.e. It either disintegrates or can be remoulded in water |
| HW<br>*DW* | Highly Weathered<br>*Distinctly Weathered (as per AS 1726)* | The rock substance is affected by weathering to the extent that limonite staining or bleaching affects the whole rock substance and other signs of chemical or physical decomposition are evident. Porosity and strength is usually decreased compared to the fresh rock. The colour and strength of the fresh rock is no longer recognisable. |
| MW | Moderately Weathered | The whole of the rock substance is discoloured, usually by iron staining or bleaching, to the extent that the colour of the fresh rock is no longer recognisable |
| SW | Slightly Weathered | Rock is slightly discoloured but shows little or no change of strength from fresh rock |
| FR | Fresh | Rock shows no sign of decomposition or staining |

## ROCK STRENGTH (AS 1726-2017 and ISRM)

| Term | Symbol | Point Load Index $Is_{(50)}$ (MPa) |
|---|---|---|
| Extremely Low | EL | <0.03 |
| Very Low | VL | 0.03 to 0.1 |
| Low | L | 0.1 to 0.3 |
| Medium | M | 0.3 to 1 |
| High | H | 1 to 3 |
| Very High | VH | 3 to 10 |
| Extremely High | EH | >10 |

## ABREVIATIONS FOR DEFECT TYPES AND DECRIPTIONS

| Term | Defect Spacing | Bedding |
|---|---|---|
| Extremely closely spaced | <6mm | Thinly Laminated |
| | 6mm to 20mm | Laminated |
| Very closely spaced | 20mm to 60mm | Very Thin |
| Closely spaced | 0.06m to 0.2m | Thin |
| Moderately widely spaced | 0.2m to 0.6m | Medium |
| Widely spaced | 0.6m to 2m | Thick |
| Very widely spaced | >2m | Very Thick |

| Type | Definition |
|---|---|
| B | Bedding |
| J | Joint |
| HJ | Horizontal to Sub-Horizontal Joint |
| VJ | Vertical to Sub-Vertical Joint |
| F | Fault |
| Cle | Cleavage |
| SZ | Shear Zone |
| SM | Shear Seam |
| FZ | Fractured Zone |
| CZ | Crushed Zone |
| CS | Crushed Seam |
| MB | Mechanical Break |
| HB | Handling Break |

| Planarity | Roughness |
|---|---|
| P – Planar | C – Clean |
| Ir – Irregular | Cl – Clay |
| St – Stepped | VR – Very Rough |
| U – Undulating | R – Rough |
| | S – Smooth |
| | Sl – Slickensides |
| | Po – Polished |
| | Fe – Iron |

| Coating or Infill | Description |
|---|---|
| Clean (C) | No visible coating or infilling |
| Stain | No visible coating or infilling but surfaces are discoloured by mineral staining |
| Veneer | A visible coating or infilling of soil or mineral substance but usually unable to be measured (<1mm). If discontinuous over the plane, patchy veneer |
| Coating | A visible coating or infilling of soil or mineral substance, >1mm thick. Describe composition and thickness |
| Iron (Fe) | Iron Staining or Infill. |

## ROCK SOCKET ROUGHNESS CLASSIFICATION AND CATEGORIES

| Roughness Class | Description |
|---|---|
| R1 | Straight, smooth-sided socket, grooves or indentations less than 1mm deep |
| R2 | Grooves of depth 1mm to 4mm, width greater than 2mm, at spacing 50mm to 200mm |
| R3 | Grooves of depth 1mm to 4mm, width greater than 2mm, at spacing 50mm to 200mm |
| R4 | Grooves or undulations of depth >10mm, width >10mm, at spacing 50mm to 200mm |

Source: "State of Practice for the Design of Socketed Piles in Rock" by P.J.N. Pells, 1999." in 8th Australia - New Zealand Conference on Geomechanics (Hobart, 1999).

# APPENDIX D

Geotechnical Consultants Australia Pty Ltd
info@geoconsultants.com.au
www.geoconsultants.com.au
(02) 9788 2829

# BOREHOLE NUMBER BH1



**CLIENT** Georgina Nassif & Tony Nassif
**PROJECT NAME** Geotechnical Investigation
**PROJECT NUMBER** G21118-1-Rev A
**PROJECT LOCATION** 26 Ralston Road Palm Beach NSW 2108

**DATE STARTED** 12/3/21 **COMPLETED** 12/3/21 **R.L. SURFACE** 100 **DATUM** m AHD
**DRILLING CONTRACTOR** Geotechnical Consultants Australia Pty Ltd **SLOPE** 90° **BEARING** ---
**EQUIPMENT** Hand Operated Equipment **HOLE LOCATION** Refer To Site Plan (Figure 1) For Test Locations
**HOLE SIZE** 100mm Diameter **LOGGED BY** GN/GA **CHECKED BY** JN
**NOTES** RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

| Method | Water | RL (m) | Depth (m) | Graphic Log | Classification Symbol | Material Description | Samples Tests Remarks | Additional Observations |
|---|---|---|---|---|---|---|---|---|
| HA | Not Encountered During Augering | | | | | Silty SAND, fine grained, brown to dark brown, some fine grained gravel, grass rootlets, moist. | | FILL |
| | | | | | | Silty SAND, fine grained, brown, grey, some fine grained gravel, tree rootlets, moist. | | |
| | | 99.5 | 0.5 | | | SAND, fine to medium grained, brown to pale brown, brownish orange, with silt, trace of clay, some fine to medium grained gravel, organics, moist. | | |
| | | 99.0 | 1.0 | | | Borehole BH1 terminated at 0.5m | | Practical hand auger refusal at 0.5m bgl. |

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www.geoconsultants.com.au
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# BOREHOLE NUMBER BH101



**CLIENT** Georgina Nassif & Tony Nassif
**PROJECT NAME** Geotechnical Investigation
**PROJECT NUMBER** G21118-1-Rev A
**PROJECT LOCATION** 26 Ralston Road Palm Beach NSW 2108

**DATE STARTED** 12/9/24 **COMPLETED** 12/9/24 **R.L. SURFACE** 100.7 **DATUM** m AHD
**DRILLING CONTRACTOR** Geotechnical Consultants Australia Pty Ltd **SLOPE** 90° **BEARING** ---
**EQUIPMENT** Hand Operated Equipment **HOLE LOCATION** Refer To Site Plan (Figure 1) For Test Locations
**HOLE SIZE** 100mm Diameter **LOGGED BY** NW **CHECKED BY** JN
**NOTES** RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

| Method | Water | RL (m) | Depth (m) | Graphic Log | Classification Symbol | Material Description | Samples Tests Remarks | Additional Observations |
|---|---|---|---|---|---|---|---|---|
| HA | Not Encountered During Augering | 100.5 | | | | FILL: Silty Sand, fine to medium grained, brown to dark brown, with fine to coarse gravel, grass rootlets, moist. | | FILL |
| | | 100.0 | 0.5 | | SW | SAND, fine to medium grained, brown to yellow brown, trace of clay, trace of fine gravel, moist, estimated medium dense.<br>becoming dense from 0.7m bgl. | | NATURAL SOILS |
| | | | 1.0 | | | Borehole BH101 terminated at 0.8m | | Practical hand auger refusal at 0.8m bgl. |

# BOREHOLE NUMBER BH102



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www.geoconsultants.com.au
(02) 9788 2829

**CLIENT** Georgina Nassif & Tony Nassif
**PROJECT NAME** Geotechnical Investigation
**PROJECT NUMBER** G21118-1-Rev A
**PROJECT LOCATION** 26 Ralston Road Palm Beach NSW 2108

**DATE STARTED** 12/9/24 **COMPLETED** 12/9/24 **R.L. SURFACE** 101.2 **DATUM** m AHD
**DRILLING CONTRACTOR** Geotechnical Consultants Australia Pty Ltd **SLOPE** 90° **BEARING** ---
**EQUIPMENT** Hand Operated Equipment **HOLE LOCATION** Refer To Site Plan (Figure 1) For Test Locations
**HOLE SIZE** 100mm Diameter **LOGGED BY** NW **CHECKED BY** JN
**NOTES** RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

| Method | Water | RL (m) | Depth (m) | Graphic Log | Classification Symbol | Material Description | Samples Tests Remarks | Additional Observations |
|---|---|---|---|---|---|---|---|---|
| HA | Not Encountered During Augering | | | | | FILL: Silty Sand, fine to medium grained, brown to dark brown, with fine to coarse gravel, grass rootlets, moist. | | FILL |
| | | 101.0 | | | | | | |
| | | | | | SW | SAND, fine to medium grained, brown to pale brown, trace of clay, trace of fine to medium gravel, moist, estimated medium dense. | | NATURAL SOILS |
| | | | 0.5 | | | | | |
| | | | | | | Borehole BH102 terminated at 0.6m | | Practical hand auger refusal at 0.6m bgl. |
| | | 100.5 | | | | | | |
| | | | 1.0 | | | | | |

# APPENDIX E

# DYNAMIC CONE PENETOMETER RESULTS

| | | | |
|---|---|---|---|
| **Client:** | Georgina Nassif & Tony Nassif | **Test Date:** | 12/03/2021 |
| **Address:** | 26 Ralston Road Palm Beach NSW 2108 | **Job No.:** | G21118-1-Rev A |

| **Depths (mm bgl)** | **DCP No.** | | | | **Depths (mm bgl)** | **DCP No.** | | | |
|---|---|---|---|---|---|---|---|---|---|
| | **1** | **2** | **3** | | | | | | |
| 0-100 | 2 | 2 | 2 | | 0-100 | | | | |
| 100-200 | 3 | 2 | 1 | | 100-200 | | | | |
| 200-300 | 5 | 2 | 2 | | 200-300 | | | | |
| 300-400 | 7 | 2/50mm | 4 | | 300-400 | | | | |
| 400-500 | 9 | **Bouncing** | 12 | | 400-500 | | | | |
| 500-600 | 7 | | 5 | | 500-600 | | | | |
| 600-700 | 6 | | 5 | | 600-700 | | | | |
| 700-800 | 5 | | 5 | | 700-800 | | | | |
| 800-900 | 2 | | 22 | | 800-900 | | | | |
| 900-1000 | 3 | | **Bouncing** | | 900-1000 | | | | |
| 1000-1100 | 7 | | | | 1000-1100 | | | | |
| 1100-1200 | 10 | | | | 1100-1200 | | | | |
| 1200-1300 | 7 | | | | 1200-1300 | | | | |
| 1300-1400 | 6 | | | | 1300-1400 | | | | |
| 1400-1500 | 5 | | | | 1400-1500 | | | | |
| 1500-1600 | 4 | | | | 1500-1600 | | | | |
| 1600-1700 | 5 | | | | 1600-1700 | | | | |
| 1700-1800 | 5 | | | | 1700-1800 | | | | |
| 1800-1900 | 6 | | | | 1800-1900 | | | | |
| 1900-2000 | 3 | | | | 1900-2000 | | | | |
| 2000-2100 | 3 | | | | 2000-2100 | | | | |
| 2100-2200 | 5 | | | | 2100-2200 | | | | |
| 2200-2300 | 5 | | | | 2200-2300 | | | | |
| 2300-2400 | 9 | | | | 2300-2400 | | | | |
| 2400-2500 | 17 | | | | 2400-2500 | | | | |
| 2500-2600 | 17/50mm | | | | 2500-2600 | | | | |
| 2600-2700 | **Bouncing** | | | | 2600-2700 | | | | |
| 2700-2800 | | | | | 2700-2800 | | | | |
| 2800-2900 | | | | | 2800-2900 | | | | |
| 2900-3000 | | | | | 2900-3000 | | | | |
| 3000-3100 | | | | | 3000-3100 | | | | |
| 3100-3200 | | | | | 3100-3200 | | | | |
| 3200-3300 | | | | | 3200-3300 | | | | |
| 3300-3400 | | | | | 3300-3400 | | | | |
| 3400-3500 | | | | | 3400-3500 | | | | |
| 3500-3600 | | | | | 3500-3600 | | | | |
| 3600-3700 | | | | | 3600-3700 | | | | |
| 3700-3800 | | | | | 3700-3800 | | | | |
| 3800-3900 | | | | | 3800-3900 | | | | |
| 3900-4000 | | | | | 3900-4000 | | | | |

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|---|---|---|---|---|
| **Tested:** | **GN/GA** | **©Geotechnical Consultants Australia Pty Ltd** | **Sheet:** | **1 of 1** |

# DYNAMIC CONE PENETOMETER RESULTS

| | | | |
|---|---|---|---|
| **Client:** | Georgina Nassif & Tony Nassif | **Test Date:** | 12/09/2024 |
| **Address:** | 26 Ralston Road Palm Beach NSW 2108 | **Job No.:** | G21118-1-Rev A |

| **Depths (mm bgl)** | **DCP No.** | | | | **Depths (mm bgl)** | **DCP No.** | | | |
|---|---|---|---|---|---|---|---|---|---|
| | **101** | **102** | **103** | | | | | | |
| 0-100 | 1 | 1 | 1 | | 0-100 | | | | |
| 100-200 | 2 | 3 | 2 | | 100-200 | | | | |
| 200-300 | 3 | 5 | 5 | | 200-300 | | | | |
| 300-400 | 6 | 7 | 6 | | 300-400 | | | | |
| 400-500 | 10 | 8 | 6 | | 400-500 | | | | |
| 500-600 | 7 | 6 | 7 | | 500-600 | | | | |
| 600-700 | 8 | 8 | 8 | | 600-700 | | | | |
| 700-800 | 12 | 6 | 15/80mm | | 700-800 | | | | |
| 800-900 | 20 | 8 | **Bouncing** | | 800-900 | | | | |
| 900-1000 | 35 | 7 | | | 900-1000 | | | | |
| 1000-1100 | 25/70mm | 15/70mm | | | 1000-1100 | | | | |
| 1100-1200 | **Bouncing** | **Bouncing** | | | 1100-1200 | | | | |
| 1200-1300 | | | | | 1200-1300 | | | | |
| 1300-1400 | | | | | 1300-1400 | | | | |
| 1400-1500 | | | | | 1400-1500 | | | | |
| 1500-1600 | | | | | 1500-1600 | | | | |
| 1600-1700 | | | | | 1600-1700 | | | | |
| 1700-1800 | | | | | 1700-1800 | | | | |
| 1800-1900 | | | | | 1800-1900 | | | | |
| 1900-2000 | | | | | 1900-2000 | | | | |
| 2000-2100 | | | | | 2000-2100 | | | | |
| 2100-2200 | | | | | 2100-2200 | | | | |
| 2200-2300 | | | | | 2200-2300 | | | | |
| 2300-2400 | | | | | 2300-2400 | | | | |
| 2400-2500 | | | | | 2400-2500 | | | | |
| 2500-2600 | | | | | 2500-2600 | | | | |
| 2600-2700 | | | | | 2600-2700 | | | | |
| 2700-2800 | | | | | 2700-2800 | | | | |
| 2800-2900 | | | | | 2800-2900 | | | | |
| 2900-3000 | | | | | 2900-3000 | | | | |
| 3000-3100 | | | | | 3000-3100 | | | | |
| 3100-3200 | | | | | 3100-3200 | | | | |
| 3200-3300 | | | | | 3200-3300 | | | | |
| 3300-3400 | | | | | 3300-3400 | | | | |
| 3400-3500 | | | | | 3400-3500 | | | | |
| 3500-3600 | | | | | 3500-3600 | | | | |
| 3600-3700 | | | | | 3600-3700 | | | | |
| 3700-3800 | | | | | 3700-3800 | | | | |
| 3800-3900 | | | | | 3800-3900 | | | | |
| 3900-4000 | | | | | 3900-4000 | | | | |

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| | | | | |
|---|---|---|---|---|
| **Tested:** | **NW/EJ** |  | **Sheet:** | **1 of 1** |

# APPENDIX F

# Foundation Maintenance and Footing Performance: A Homeowner's Guide

CSIRO

BTF 18 replaces Information Sheet 10/91

Buildings can and often do move. This movement can be up, down, lateral or rotational. The fundamental cause of movement in buildings can usually be related to one or more problems in the foundation soil. It is important for the homeowner to identify the soil type in order to ascertain the measures that should be put in place in order to ensure that problems in the foundation soil can be prevented, thus protecting against building movement.

This Building Technology File is designed to identify causes of soil-related building movement, and to suggest methods of prevention of resultant cracking in buildings.

## Soil Types

The types of soils usually present under the topsoil in land zoned for residential buildings can be split into two approximate groups – granular and clay. Quite often, foundation soil is a mixture of both types. The general problems associated with soils having granular content are usually caused by erosion. Clay soils are subject to saturation and swell/shrink problems.

Classifications for a given area can generally be obtained by application to the local authority, but these are sometimes unreliable and if there is doubt, a geotechnical report should be commissioned. As most buildings suffering movement problems are founded on clay soils, there is an emphasis on classification of soils according to the amount of swell and shrinkage they experience with variations of water content. The table below is Table 2.1 from AS 2870, the Residential Slab and Footing Code.

## Causes of Movement

### Settlement due to construction

There are two types of settlement that occur as a result of construction:

- Immediate settlement occurs when a building is first placed on its foundation soil, as a result of compaction of the soil under the weight of the structure. The cohesive quality of clay soil mitigates against this, but granular (particularly sandy) soil is susceptible.
- Consolidation settlement is a feature of clay soil and may take place because of the expulsion of moisture from the soil or because of the soil's lack of resistance to local compressive or shear stresses. This will usually take place during the first few months after construction, but has been known to take many years in exceptional cases.

These problems are the province of the builder and should be taken into consideration as part of the preparation of the site for construction. Building Technology File 19 (BTF 19) deals with these problems.

### Erosion

All soils are prone to erosion, but sandy soil is particularly susceptible to being washed away. Even clay with a sand component of say 10% or more can suffer from erosion.

### Saturation

This is particularly a problem in clay soils. Saturation creates a bog-like suspension of the soil that causes it to lose virtually all of its bearing capacity. To a lesser degree, sand is affected by saturation because saturated sand may undergo a reduction in volume – particularly imported sand fill for bedding and blinding layers. However, this usually occurs as immediate settlement and should normally be the province of the builder.

### Seasonal swelling and shrinkage of soil

All clays react to the presence of water by slowly absorbing it, making the soil increase in volume (see table below). The degree of increase varies considerably between different clays, as does the degree of decrease during the subsequent drying out caused by fair weather periods. Because of the low absorption and expulsion rate, this phenomenon will not usually be noticeable unless there are prolonged rainy or dry periods, usually of weeks or months, depending on the land and soil characteristics.

The swelling of soil creates an upward force on the footings of the building, and shrinkage creates subsidence that takes away the support needed by the footing to retain equilibrium.

### Shear failure

This phenomenon occurs when the foundation soil does not have sufficient strength to support the weight of the footing. There are two major post-construction causes:

- Significant load increase.
- Reduction of lateral support of the soil under the footing due to erosion or excavation.
- In clay soil, shear failure can be caused by saturation of the soil adjacent to or under the footing.

**GENERAL DEFINITIONS OF SITE CLASSES**

| Class | Foundation |
|---|---|
| A | Most sand and rock sites with little or no ground movement from moisture changes |
| S | Slightly reactive clay sites with only slight ground movement from moisture changes |
| M | Moderately reactive clay or silt sites, which can experience moderate ground movement from moisture changes |
| H | Highly reactive clay sites, which can experience high ground movement from moisture changes |
| E | Extremely reactive sites, which can experience extreme ground movement from moisture changes |
| A to P | Filled sites |
| P | Sites which include soft soils, such as soft clay or silt or loose sands; landslip; mine subsidence; collapsing soils; soils subject to erosion; reactive sites subject to abnormal moisture conditions or sites which cannot be classified otherwise |

Tree root growth
Trees and shrubs that are allowed to grow in the vicinity of footings can cause foundation soil movement in two ways:

- Roots that grow under footings may increase in cross-sectional size, exerting upward pressure on footings.
- Roots in the vicinity of footings will absorb much of the moisture in the foundation soil, causing shrinkage or subsidence.

## Unevenness of Movement

The types of ground movement described above usually occur unevenly throughout the building's foundation soil. Settlement due to construction tends to be uneven because of:

- Differing compaction of foundation soil prior to construction.
- Differing moisture content of foundation soil prior to construction.

Movement due to non-construction causes is usually more uneven still. Erosion can undermine a footing that traverses the flow or can create the conditions for shear failure by eroding soil adjacent to a footing that runs in the same direction as the flow.

Saturation of clay foundation soil may occur where subfloor walls create a dam that makes water pond. It can also occur wherever there is a source of water near footings in clay soil. This leads to a severe reduction in the strength of the soil which may create local shear failure.

Seasonal swelling and shrinkage of clay soil affects the perimeter of the building first, then gradually spreads to the interior. The swelling process will usually begin at the uphill extreme of the building, or on the weather side where the land is flat. Swelling gradually reaches the interior soil as absorption continues. Shrinkage usually begins where the sun's heat is greatest.

## Effects of Uneven Soil Movement on Structures

Erosion and saturation
Erosion removes the support from under footings, tending to create subsidence of the part of the structure under which it occurs. Brickwork walls will resist the stress created by this removal of support by bridging the gap or cantilevering until the bricks or the mortar bedding fail. Older masonry has little resistance. Evidence of failure varies according to circumstances and symptoms may include:

- Step cracking in the mortar beds in the body of the wall or above/below openings such as doors or windows.
- Vertical cracking in the bricks (usually but not necessarily in line with the vertical beds or perpends).

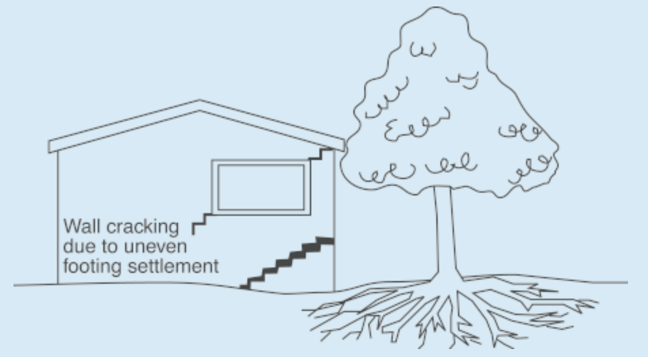
Isolated piers affected by erosion or saturation of foundations will eventually lose contact with the bearers they support and may tilt or fall over. The floors that have lost this support will become bouncy, sometimes rattling ornaments etc.

Seasonal swelling/shrinkage in clay
Swelling foundation soil due to rainy periods first lifts the most exposed extremities of the footing system, then the remainder of the perimeter footings while gradually permeating inside the building footprint to lift internal footings. This swelling first tends to create a dish effect, because the external footings are pushed higher than the internal ones.

The first noticeable symptom may be that the floor appears slightly dished. This is often accompanied by some doors binding on the floor or the door head, together with some cracking of cornice mitres. In buildings with timber flooring supported by bearers and joists, the floor can be bouncy. Externally there may be visible dishing of the hip or ridge lines.

As the moisture absorption process completes its journey to the innermost areas of the building, the internal footings will rise. If the spread of moisture is roughly even, it may be that the symptoms will temporarily disappear, but it is more likely that swelling will be uneven, creating a difference rather than a disappearance in symptoms. In buildings with timber flooring supported by bearers and joists, the isolated piers will rise more easily than the strip footings or piers under walls, creating noticeable doming of flooring.

Trees can cause shrinkage and damage




As the weather pattern changes and the soil begins to dry out, the external footings will be first affected, beginning with the locations where the sun's effect is strongest. This has the effect of lowering the external footings. The doming is accentuated and cracking reduces or disappears where it occurred because of dishing, but other cracks open up. The roof lines may become convex.

Doming and dishing are also affected by weather in other ways. In areas where warm, wet summers and cooler dry winters prevail, water migration tends to be toward the interior and doming will be accentuated, whereas where summers are dry and winters are cold and wet, migration tends to be toward the exterior and the underlying propensity is toward dishing.

Movement caused by tree roots
In general, growing roots will exert an upward pressure on footings, whereas soil subject to drying because of tree or shrub roots will tend to remove support from under footings by inducing shrinkage.

Complications caused by the structure itself
Most forces that the soil causes to be exerted on structures are vertical – i.e. either up or down. However, because these forces are seldom spread evenly around the footings, and because the building resists uneven movement because of its rigidity, forces are exerted from one part of the building to another. The net result of all these forces is usually rotational. This resultant force often complicates the diagnosis because the visible symptoms do not simply reflect the original cause. A common symptom is binding of doors on the vertical member of the frame.

Effects on full masonry structures
Brickwork will resist cracking where it can. It will attempt to span areas that lose support because of subsided foundations or raised points. It is therefore usual to see cracking at weak points, such as openings for windows or doors.

In the event of construction settlement, cracking will usually remain unchanged after the process of settlement has ceased.

With local shear or erosion, cracking will usually continue to develop until the original cause has been remedied, or until the subsidence has completely neutralised the affected portion of footing and the structure has stabilised on other footings that remain effective.

In the case of swell/shrink effects, the brickwork will in some cases return to its original position after completion of a cycle, however it is more likely that the rotational effect will not be exactly reversed, and it is also usual that brickwork will settle in its new position and will resist the forces trying to return it to its original position. This means that in a case where swelling takes place after construction and cracking occurs, the cracking is likely to at least partly remain after the shrink segment of the cycle is complete. Thus, each time the cycle is repeated, the likelihood is that the cracking will become wider until the sections of brickwork become virtually independent.

With repeated cycles, once the cracking is established, if there is no other complication, it is normal for the incidence of cracking to stabilise, as the building has the articulation it needs to cope with the problem. This is by no means always the case, however, and monitoring of cracks in walls and floors should always be treated seriously.

Upheaval caused by growth of tree roots under footings is not a simple vertical shear stress. There is a tendency for the root to also exert lateral forces that attempt to separate sections of brickwork after initial cracking has occurred.

The normal structural arrangement is that the inner leaf of brickwork in the external walls and at least some of the internal walls (depending on the roof type) comprise the load-bearing structure on which any upper floors, ceilings and the roof are supported. In these cases, it is internally visible cracking that should be the main focus of attention, however there are a few examples of dwellings whose external leaf of masonry plays some supporting role, so this should be checked if there is any doubt. In any case, externally visible cracking is important as a guide to stresses on the structure generally, and it should also be remembered that the external walls must be capable of supporting themselves.

Effects on framed structures
Timber or steel framed buildings are less likely to exhibit cracking due to swell/shrink than masonry buildings because of their flexibility. Also, the doming/dishing effects tend to be lower because of the lighter weight of walls. The main risks to framed buildings are encountered because of the isolated pier footings used under walls. Where erosion or saturation cause a footing to fall away, this can double the span which a wall must bridge. This additional stress can create cracking in wall linings, particularly where there is a weak point in the structure caused by a door or window opening. It is, however, unlikely that framed structures will be so stressed as to suffer serious damage without first exhibiting some or all of the above symptoms for a considerable period. The same warning period should apply in the case of upheaval. It should be noted, however, that where framed buildings are supported by strip footings there is only one leaf of brickwork and therefore the externally visible walls are the supporting structure for the building. In this case, the subfloor masonry walls can be expected to behave as full brickwork walls.

Effects on brick veneer structures
Because the load-bearing structure of a brick veneer building is the frame that makes up the interior leaf of the external walls plus perhaps the internal walls, depending on the type of roof, the building can be expected to behave as a framed structure, except that the external masonry will behave in a similar way to the external leaf of a full masonry structure.

## Water Service and Drainage

Where a water service pipe, a sewer or stormwater drainage pipe is in the vicinity of a building, a water leak can cause erosion, swelling or saturation of susceptible soil. Even a minuscule leak can be enough to saturate a clay foundation. A leaking tap near a building can have the same effect. In addition, trenches containing pipes can become watercourses even though backfilled, particularly where broken rubble is used as fill. Water that runs along these trenches can be responsible for serious erosion, interstrata seepage into subfloor areas and saturation.

Pipe leakage and trench water flows also encourage tree and shrub roots to the source of water, complicating and exacerbating the problem.
Poor roof plumbing can result in large volumes of rainwater being concentrated in a small area of soil:

- Incorrect falls in roof guttering may result in overflows, as may gutters blocked with leaves etc.
- Corroded guttering or downpipes can spill water to ground.
- Downpipes not positively connected to a proper stormwater collection system will direct a concentration of water to soil that is directly adjacent to footings, sometimes causing large-scale problems such as erosion, saturation and migration of water under the building.

## Seriousness of Cracking

In general, most cracking found in masonry walls is a cosmetic nuisance only and can be kept in repair or even ignored. The table below is a reproduction of Table C1 of AS 2870.

AS 2870 also publishes figures relating to cracking in concrete floors, however because wall cracking will usually reach the critical point significantly earlier than cracking in slabs, this table is not reproduced here.

## Prevention/Cure

Plumbing
Where building movement is caused by water service, roof plumbing, sewer or stormwater failure, the remedy is to repair the problem. It is prudent, however, to consider also rerouting pipes away from the building where possible, and relocating taps to positions where any leakage will not direct water to the building vicinity. Even where gully traps are present, there is sometimes sufficient spill to create erosion or saturation, particularly in modern installations using smaller diameter PVC fixtures. Indeed, some gully traps are not situated directly under the taps that are installed to charge them, with the result that water from the tap may enter the backfilled trench that houses the sewer piping. If the trench has been poorly backfilled, the water will either pond or flow along the bottom of the trench. As these trenches usually run alongside the footings and can be at a similar depth, it is not hard to see how any water that is thus directed into a trench can easily affect the foundation's ability to support footings or even gain entry to the subfloor area.

Ground drainage
In all soils there is the capacity for water to travel on the surface and below it. Surface water flows can be established by inspection during and after heavy or prolonged rain. If necessary, a grated drain system connected to the stormwater collection system is usually an easy solution.

It is, however, sometimes necessary when attempting to prevent water migration that testing be carried out to establish watertable height and subsoil water flows. This subject is referred to in BTF 19 and may properly be regarded as an area for an expert consultant.

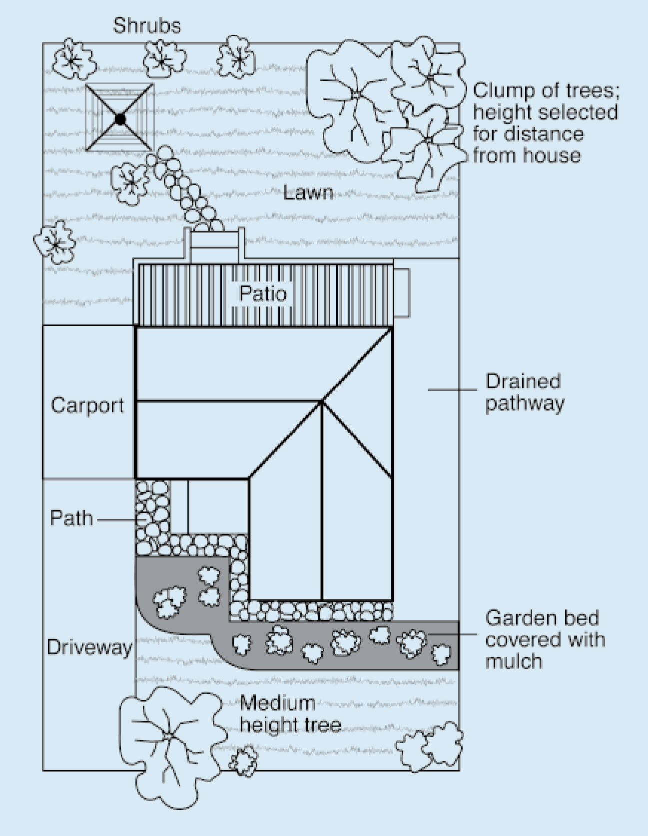
Protection of the building perimeter
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

For this reason, particularly where problems exist or are likely to occur, it is recommended that an apron of paving be installed around as much of the building perimeter as necessary. This paving

**CLASSIFICATION OF DAMAGE WITH REFERENCE TO WALLS**

| Description of typical damage and required repair | Approximate crack width limit (see Note 3) | Damage category |
|---|---|---|
| Hairline cracks | <0.1 mm | 0 |
| Fine cracks which do not need repair | <1 mm | 1 |
| Cracks noticeable but easily filled. Doors and windows stick slightly | <5 mm | 2 |
| Cracks can be repaired and possibly a small amount of wall will need to be replaced. Doors and windows stick. Service pipes can fracture. Weathertightness often impaired | 5–15 mm (or a number of cracks 3 mm or more in one group) | 3 |
| Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Window and door frames distort. Walls lean or bulge noticeably, some loss of bearing in beams. Service pipes disrupted | 15–25 mm but also depend on number of cracks | 4 |

**Gardens for a reactive site**




should extend outwards a minimum of 900 mm (more in highly reactive soil) and should have a minimum fall away from the building of 1:60. The finished paving should be no less than 100 mm below brick vent bases.

It is prudent to relocate drainage pipes away from this paving, if possible, to avoid complications from future leakage. If this is not practical, earthenware pipes should be replaced by PVC and backfilling should be of the same soil type as the surrounding soil and compacted to the same density.

Except in areas where freezing of water is an issue, it is wise to remove taps in the building area and relocate them well away from the building – preferably not uphill from it (see BTF 19).

It may be desirable to install a grated drain at the outside edge of the paving on the uphill side of the building. If subsoil drainage is needed this can be installed under the surface drain.

### Condensation

In buildings with a subfloor void such as where bearers and joists support flooring, insufficient ventilation creates ideal conditions for condensation, particularly where there is little clearance between the floor and the ground. Condensation adds to the moisture already present in the subfloor and significantly slows the process of drying out. Installation of an adequate subfloor ventilation system, either natural or mechanical, is desirable.

*Warning:* Although this Building Technology File deals with cracking in buildings, it should be said that subfloor moisture can result in the development of other problems, notably:

- Water that is transmitted into masonry, metal or timber building elements causes damage and/or decay to those elements.
- High subfloor humidity and moisture content create an ideal environment for various pests, including termites and spiders.
- Where high moisture levels are transmitted to the flooring and walls, an increase in the dust mite count can ensue within the living areas. Dust mites, as well as dampness in general, can be a health hazard to inhabitants, particularly those who are abnormally susceptible to respiratory ailments.

### The garden

The ideal vegetation layout is to have lawn or plants that require only light watering immediately adjacent to the drainage or paving edge, then more demanding plants, shrubs and trees spread out in that order.

Overwatering due to misuse of automatic watering systems is a common cause of saturation and water migration under footings. If it is necessary to use these systems, it is important to remove garden beds to a completely safe distance from buildings.

### Existing trees

Where a tree is causing a problem of soil drying or there is the existence or threat of upheaval of footings, if the offending roots are subsidiary and their removal will not significantly damage the tree, they should be severed and a concrete or metal barrier placed vertically in the soil to prevent future root growth in the direction of the building. If it is not possible to remove the relevant roots without damage to the tree, an application to remove the tree should be made to the local authority. A prudent plan is to transplant likely offenders before they become a problem.

### Information on trees, plants and shrubs

State departments overseeing agriculture can give information regarding root patterns, volume of water needed and safe distance from buildings of most species. Botanic gardens are also sources of information. For information on plant roots and drains, see Building Technology File 17.

### Excavation

Excavation around footings must be properly engineered. Soil supporting footings can only be safely excavated at an angle that allows the soil under the footing to remain stable. This angle is called the angle of repose (or friction) and varies significantly between soil types and conditions. Removal of soil within the angle of repose will cause subsidence.

## Remediation

Where erosion has occurred that has washed away soil adjacent to footings, soil of the same classification should be introduced and compacted to the same density. Where footings have been undermined, augmentation or other specialist work may be required. Remediation of footings and foundations is generally the realm of a specialist consultant.

Where isolated footings rise and fall because of swell/shrink effect, the homeowner may be tempted to alleviate floor bounce by filling the gap that has appeared between the bearer and the pier with blocking. The danger here is that when the next swell segment of the cycle occurs, the extra blocking will push the floor up into an accentuated dome and may also cause local shear failure in the soil. If it is necessary to use blocking, it should be by a pair of fine wedges and monitoring should be carried out fortnightly.

**This BTF was prepared by John Lewer FAIB, MIAMA, Partner, Construction Diagnosis.**



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# APPENDIX G

| gy | GYMEA | Erosional |
|---|---|---|

*Source:* Soil and Land Resources of the Hawkesbury-Nepean Catchment *interactive DVD*

**Landscape**—undulating to rolling rises and low hills on Hawkesbury Sandstone. Local relief 20–80 m, slopes 10–25%. Rock outcrop <25%. Broad convex crests, moderately inclined sideslopes with wide benches, localised rock outcrop on low broken scarps. Extensively cleared open-forest (dry sclerophyll forest) and eucalypt woodland.

**Soils**—shallow to moderately deep (30-100 cm) Yellow Earths (Gn2.24) and Earthy Sands (Uc5.11, Uc5.23) on crests and inside of benches; shallow (<20 cm) Siliceous Sands (Uc1.21) on leading edges of benches; localised Gleyed Podzolic Soils (Dg4.21) and Yellow Podzolic Soils (Dy4.11, Dy5.11, Dy5.41) on shale lenses; shallow to moderately deep (<100 cm) Siliceous Sands (Uc1.21) and Leached Sands (Uc2.21) along drainage lines.

**Limitations**—localised steep slopes, high soil erosion hazard, rock outcrop, shallow highly permeable soil, very low soil fertility.

## LOCATION

Occurs extensively throughout the Hornsby Plateau and along the foreshores of Sydney Harbour and the Parramatta and Georges Rivers. Examples include areas of Northbridge, Forestville, Drummoyne, Balmain, Arcadia and Berrilee.

## LANDSCAPE

### Geology

Hawkesbury Sandstone, which is a medium to coarse-grained quartz sandstone with minor shale and laminite lenses.

**Topography**

Undulating to rolling low hills with local relief 20–80 m and slopes of 10–25%. Sideslopes with narrow to wide outcropping sandstone rock benches (10–100 m), often forming broken scarps of <5 m.

**Vegetation**

The original dry sclerophyll woodland and open-forest have been extensively cleared. Low, dry sclerophyll open-woodland dominates ridges and upper slopes. Common species include red bloodwood *Eucalyptus gummifera*, yellow bloodwood *E. eximia*, scribbly gum *E. haemastoma*, brown stringybark *E. capitellata* and old man banksia *Banksia serrata*. On the more sheltered slopes, black ash *E. sieberi*, Sydney peppermint *E. piperita* and smooth-barked apple *Angophora costata* are common tree species. The dry sclerophyll understorey consists of shrubs from the families Epacridaceae, Myrtaceae, Fabaceae and Proteaceae.

**Land use**

Land use is mostly urban residential. Developed suburbs include Forestville, Northbridge and Drummoyne. Steeper sections are used for recreational purposes and often remain covered with native vegetation. Grazing occurs at Berrilee and there are small hobby farms in the north-west.

**Existing Erosion**

Severe sheet erosion occurs following bushfires, which destroy or damage stabilising vegetative cover. Minor gully erosion occurs along unpaved or poorly maintained roads and fire trails especially those frequented by four-wheel-drive vehicles and trail bikes.

**Associated Soil Landscapes**

Small areas (<40 ha) of Hawkesbury (**ha**) and Lambert (**la**) soil landscapes have been included within the Gymea soil landscape. In many respects these landscapes have qualities in common with the Gymea soil landscape.

## SOILS

**Dominant Soil Materials**

**gy1—Loose, coarse sandy loam.** This is loamy sand to sandy loam with loose, apedal single-grained structure and porous sandy fabric. It generally occurs as topsoil (A1 horizon).

The colour often becomes lighter with depth and ranges from brownish-black (10YR 2/2), when organic matter is present, to bleached dull yellow-orange (10YR 7/2). It is often water repellent under native vegetation. The pH ranges from strongly acid (pH 4.0) to slightly acid (pH 6.0). Small sandstone and platy ironstone fragments, charcoal fragments and roots are common.

**gy2—Earthy, yellowish-brown clayey sand.** This is commonly yellowish-brown clayey sand with apedal massive structure and porous earthy fabric. It commonly occurs as subsoil over sandstone bedrock (B horizon). Where it is exposed at the surface it forms hardsetting topsoil.

Texture may increase gradually to a light sandy clay loam with depth. Colour is commonly yellowish-brown (10YR 6/8) and orange mottles are occasionally present with depth. The pH ranges from strongly acid (pH 4.0) to slightly acid (pH 6.5). Sandstone and ironstone fragments are common and are often concentrated in stone lines in the upper parts of this material. Charcoal fragments are common whilst roots are rare.

**gy3—Earthy to weakly pedal, yellowish-brown sandy clay loam.** This is commonly a yellowish-brown sandy clay loam to sandy clay with an apedal massive structure and an earthy porous fabric. It usually occurs as subsoil (B or C horizon) on coarse sandstone.

Texture is commonly sandy clay loam but may increase gradually with depth to sandy clay. Occasionally a weakly pedal structure of sub-angular blocky shaped peds are present. Peds are commonly rough-faced and porous and range in size from 5-20 mm. Colour is commonly yellowish brown (10YR 5/8, 6/6, 6/8; 2.5Y 5/6, 5/4). Orange mottles may occur with depth. The pH ranges from strongly acid (pH 4.5) to slightly acid (pH 6.0). Strongly weathered sandstone fragments are common. Roots and charcoal fragments are rare.

**gy4—Moderately to strongly pedal, yellowish-brown clay.** This is commonly a yellowish-brown sandy clay or light clay with a moderately to strongly pedal structure and either a smooth or rough-faced ped fabric. This material occurs as subsoil on shale bedrock (B and C horizons).

Peds ranging in size from 5–50 mm, are either smooth or rough-faced and are polyhedral to sub-angular blocky. Colour is commonly yellow-brown (10YR 6/6) but can vary from dark reddish-brown (2.5YR 3/6) to light grey (7.5YR 8/1). Red, orange and grey mottles are occasionally present at depth. The pH ranges from strongly acid (pH 4.0) to slightly acid (pH 6.0). Shale and ironstone fragments are often present, but charcoal fragments are absent, and roots are rare.

**Associated Soil Materials**

**Litter and decomposing organic debris.** In areas of natural bushland, litter and organic debris occur on the soil surface. The litter layer can be developed to depths of up to 10 cm. Charcoal fragments are common. This material is often found in debris dams in association with white, loose quartz sand.

**White, loose quartz sand.** A surface wash of quartz sand grains. It occurs in depositional areas such as small debris dams and fans on breaks of slope. It is often mixed with the litter layer and is usually water repellent.

**Occurrence and Relationships**

**Crests.** Generally up to 30 cm of loose, quartz sandy loam (**gy1**) overlies bedrock (Siliceous Sands and Lithosols (Uc 1.21)) or <30 cm of earthy, yellowish-brown clayey sand (**gy2**) (Earthy Sands (Uc5.11)). Occasionally (**gy2**) overlies up to 30 cm of yellow earthy/weakly pedal sandy clay loam (**gy3**) (Yellow Earths (Gn2.24)). Boundaries between soil materials are gradual. Total soil depth is <50 cm.

Where severe erosion has occurred, **gy2** or **gy3** is often exposed as a hardsetting layer at the surface. Bedrock is exposed in some areas, particularly where bushfires are frequent.

**Sideslopes.** The soils on the sideslopes are discontinuous and rock outcrop may cover up to 25% of the ground surface. On the outside of benches and areas close to rock outcrop, up to 20 cm of **gy1** overlies bedrock (Siliceous Sands/Lithosols (Uc1.21)). On the inside of benches, up to 30 cm of **gy1** overlies 10–30 cm of **gy2**. Occasionally **gy2** overlies up to 30 cm of **gy3**. The boundaries between soil materials are gradual. Total soil depth is 30–70 cm (Yellow Earths (Gn2.24), Earthy Sands (Uc5.11)).

**Shale lenses.** Where shale lenses occur on the inside of benches, up to 30 cm of **gy1** overlies up to 100 cm of strongly pedal yellowish-brown clay (**gy4**). The boundary between soil materials is sharp to clear. Total soil depth is <100 cm (Gleyed Podzolic Soils (Dg 4.21), Yellow Podzolic Soils (Dy 5.41)).

**Drainage lines.** Up to 100 cm of **gy1** overlies bedrock (Siliceous Sands (Uc1.2) and Leached Sands (Uc 2.21)).

## LIMITATIONS TO DEVELOPMENT

### Urban Capability

Generally, low to moderate capability for urban development.

### Rural Capability

Land not capable of being grazed or cultivated.

### Landscape Limitations

Erosion hazard
Rock outcrop
Rockfall hazard (localised)
Steep slopes (localised)
Shallow soil

### Soil Limitations

**gy1** High permeability
Low available water capacity
Stoniness
Low fertility

**gy2** Low available water capacity
Stoniness
Very low fertility
Very strongly acid
Very high aluminium toxicity

**gy3** Low available water capacity
Low wet strength (localised)
Low permeability (localised)
Stoniness (localised)
Very low fertility
Very strongly acid
High aluminium toxicity

**gy4** Low wet strength
High erodibility
Low permeability
Low available water capacity
Stoniness (localised)
Very low fertility
Very strongly acid
Very high aluminium toxicity

### Fertility

Very poor. The soils of this unit are generally shallow, stony, moderately acid and highly permeable with low available water capacities. They also have a low to very low nutrient status with very low phosphorus and nitrogen levels and very low CEC.

**Erodibility**

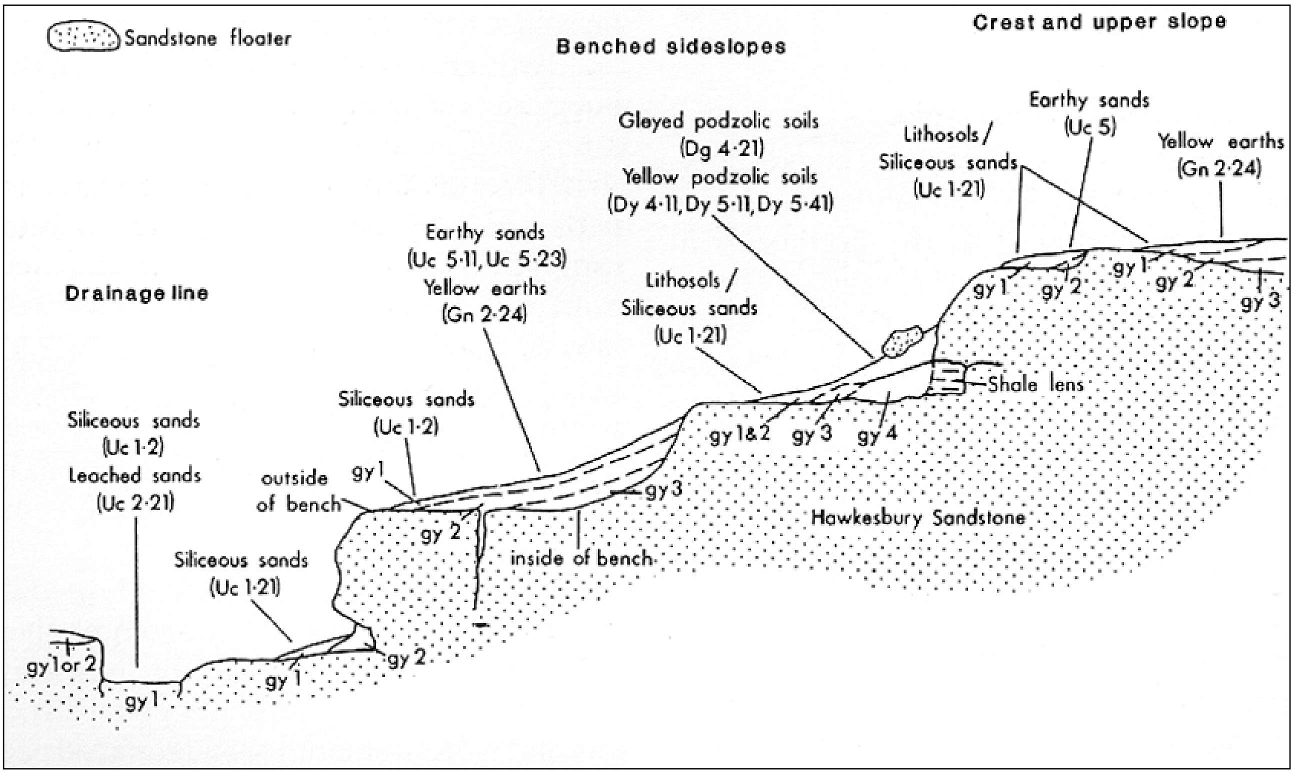
**gy1** and **gy2** are composed of coarse sand grains and have very low erodibilities as they are freely drained and are held together by high organic matter contents (**gy1**) and/or non-dispersive clays (**gy2**). However, (**gy3**) is moderately erodible as it has a weakly coherent earthy fabric with low organic matter content. **gy4** is highly erodible as it is very low in organic matter and consists dominantly of fine sands in a clay matrix.

**Erosion Hazard**

The erosion hazard for non-concentrated flows is generally high to very high but can range from moderate to extreme. Calculated soil loss for the first twelve months of development range up to 19 t/ha for topsoil and 464 t/ha for subsoil. Soil erosion hazard for concentrated flows is high to extreme.

**Surface Movement Potential**

The shallow sandy soils are stable to slightly reactive. In isolated instances where **gy4** is >100 cm thick soils may be moderately reactive.




*Schematic cross-section of Gymea soil landscape illustrating the occurrence and relationship of the dominant soil materials.*

| wn | WATAGAN | Colluvial |
|---|---|---|

*Source:* Soil and Land Resources of the Hawkesbury-Nepean Catchment *interactive DVD*

**Landscape**—rolling to very steep hills on fine-grained Narrabeen Group sediments. Local relief 60–120 m, slopes >25%. Narrow, convex crests and ridges, steep colluvial sideslopes, occasional sandstone boulders and benches. Tall eucalypt open-forest with closed-forest (rainforest) in sheltered positions.

**Soils**—shallow to deep (30–200 cm) *Lithosols/Siliceous Sands (Uc1.24)* and *Yellow Podzolic Soils (Dy3.21, Dy3.41, Dy4.11)* on sandstones; moderately deep (100–200 cm) *Brown Podzolic Soils (Db1.11)*, *Red {Podzolic Soils (Dr2.21)* and *Gleyed Podzolic Soils (Dg2.21)* on shales.

**Limitations**—mass movement hazard, steep slopes, severe soil erosion hazard, occasional rock outcrop.

## LOCATION

Occurs north of Collaroy, on steep sideslopes of the Erina Hills. It is associated with coastal headlands and bluffs along the Hawkesbury River and its lower tributaries. Also occurs along deep valleys in Bouddi National Park and along the escarpment west of Umina.

## LANDSCAPE

### Geology

Narrabeen Group of sediments. Mostly interbedded laminite and shale with quartz to lithic quartz sandstone. Minor red claystones occur north of the Hawkesbury River. Clay pellet sandstone occurs south of the Hawkesbury River (Herbert, 1983).

**Topography**

Rolling to very steep low hills. Local relief is 60–120 m. Slope gradients are steeper than 25%. Crests and ridges are convex and narrow. Hillslopes are steep with talus slopes containing sandstone boulders. Occasional sandstone benches and colluvial benches are present. Slopes with gradients >70% often have cliffs and scarps >10 m high

**Vegetation**

Mostly uncleared, tall eucalypt open-forest (wet sclerophyll) and closed-forest (rainforest). Much of the native vegetation on the Northern Beaches peninsula has been cleared.

Tall eucalypt open-forests occur on drier and more exposed slopes and crests. Tree species include spotted gum *Eucalyptus maculata*, grey ironbark *E. paniculata*, Sydney blue gum *E. saligna*, turpentine *Syncarpia glomulifera*, bangalay *E. botryoides*, rough-barked apple *Angophora floribunda* and forest oak *Allocasuarina torulosa*.

Rainforest occurs on sheltered slopes. Characteristic tree species include lilly pilly *Acmena smithii*, cheese tree *Glochidion ferdinandi*, coachwood *Ceratopetalum apetalum* and cabbage tree *Livistona australis*.

**Land use**

Most land has been gazetted as national park or nature reserve. Examples include Brisbane Water, Ku-ring-gai Chase and Bouddi National Parks and Muogamarra Nature Reserve. Some locations such as Taylors Point and Bayview are urban residential.

**Existing Erosion**

Minor gully erosion occurs along unpaved roads. Moderate sheet erosion occurs on the steep hillslopes. Landslip and rockfall have occurred on steep slopes with wet, unstable and disturbed soils. This has resulted in serious damage to roads and buildings at Newport.

## SOILS

**Dominant Soil Materials**

**wn1—Loose, stony, brownish-black sandy loam.** This is stony, brownish-black, loamy sand to loam-fine-sandy with loose apedal single-grained structure and sandy porous fabric. It usually occurs as topsoil (A1 horizon). Texture is commonly a fine sandy loam. Surface condition is generally loose but may also be friable when sufficient organic matter is present. Colour is usually brownish-black (10YR 2/2) or black (10YR 1.7/1). The pH ranges from moderately acid (pH 5.0) to slightly acid (pH 6.0). Strongly weathered, subrounded, small sandstone fragments, charcoal fragments and roots are common.

**wn2—Hardsetting, brown sandy clay loam.** This is a brown loam to fine sandy clay loam which is hardsetting when dry. It has apedal massive structure and slowly porous, earthy fabric. It usually occurs as an A2 horizon. Colour varies considerably and ranges from brownish-black (10YR 2/2) to yellowish-brown (10YR 5/6). The pH ranges from moderately acid (pH 5.5) to slightly acid (pH 6.0). Sandstone fragments may be common, and often occur as a stone line in the base of this material. Charcoal fragments and roots are often present.

**wn3—Strongly pedal, yellowish-brown fine sandy clay.** This is yellowish-brown sandy clay to medium clay, with moderately or strongly pedal structure and porous, rough-faced, ped fabric. This material occurs as subsoil on sandstone bedrock (B horizon). Fine sand is commonly present throughout this material. Peds are sub-angular blocky and range in size between 20–50 mm. Colour is commonly yellowish-brown (10YR 5/8) or orange yellowish-brown (7.5YR 6/8). Brown, red, yellow or grey mottles are common. The pH ranges from extremely acid (pH 3.5) to moderately acid (pH 5.0). Some sandstone rock fragments and roots are present, but charcoal fragments are absent.

**wn4—Strongly pedal clay.** This is light to medium clay with strongly pedal structure and dense, smooth-faced ped fabric. It generally occurs as subsoil on shale or siltstone bedrock (B horizon). Peds are crumb, polyhedral or sub-angular blocky in shape. They range in size from 2–10 mm and are smooth-faced and dense. Colour varies considerably depending on site drainage characteristics. Colours range from dark brown (10YR 3/4) and dark reddish-brown (2.5 YR 3/4) in freely drained areas to light grey (10YR 7/2) in poorly drained areas. Red, yellow or grey mottles also are present in poorly drained areas and/or at depth. The pH is usually strongly acid (pH 5.5). Siltstone and shale fragments are rare, as are roots. Charcoal fragments are absent.

**Associated Soil Materials**

Litter and decomposing organic debris. This can be identified readily as decomposing plant remains. The litter layer is particularly well developed to depths of up to 30 cm in forested areas.

**Occurrence and Relationships**

**Sandstone crests.** Up to 15 cm of hardsetting, brown sandy clay loam (**wn2**) overlies up to 50 cm of strongly pedal, yellowish-brown sandy clay (**wn3**). Total soil depth is 50–100 cm. Boundaries between soil materials are clear [Yellow Podzolic Soils (Dy 3.21)].

**Siltstone and shale crests.** Up to 10 cm of **wn2** overlies up to 100 cm of highly pedal, brown clay (**wn4**). Total soil depth is 100–150 cm and the boundary between the soil materials is sharp [Brown Podzolic Soil (Db 1.11)].

**Very steep sideslopes.** Where slope gradients generally exceed 40%, up to 20 cm of **wn1** overlies either bedrock (Siliceous Sands and Lithosols (Uc1.24)) or up to 50 cm of **wn3** [Yellow Podzolic Soils (Dy 4.11)].

**Moderately steep sideslopes with sandstone bedrock.** Up to 30 cm of **wn1** overlies 10–30 cm of **wn2** and up to 100 cm of **wn3**. Boundaries between soil materials are sharp. Total soil depth is 30–120 cm (Yellow Podzolic Soils (Dy 3.41)). Occasionally **wn1** is absent and **wn2** overlies **wn3** (Yellow Podzolic Soils (Dy3.21)).

**Moderately steep sideslopes with siltstone or mudstone (shale) bedrock.** Up to 15 cm of **wn1** overlies 20–30 cm of **wn2** and >60 cm of **wn4**. Total soil depth is >100 cm. Boundaries between soil materials are usually clear to sharp. Red and Brown Podzolic Soils (Dr2.11, Db1.11) occur on well-drained slopes with Gleyed Podzolic Soils (Dg2.21) on poorer drained footslopes.

**Drainage lines.** Generally, 100 cm of stony dark brown sandy loam (**wn1**) overlies bedrock [Siliceous sands (Uc1.22)].

## LIMITATIONS TO DEVELOPMENT

### Urban Capability

Generally not capable of urban development.

### Rural Capability

Generally not capable of regular cultivation or being grazed.

**Landscape Limitations**

Mass movement hazard
Rockfall hazard
Steep slopes
Extreme erosion hazard
Shallow depth (localised)
Surface movement potential (localised)

**Soil Limitations**

**wn1** Stoniness
Low available water capacity
Low fertility
Very strongly acid
High aluminium toxicity

**wn2** Stoniness (localised)
Hardsetting surface
Low fertility
High aluminium toxicity
Very strongly acid

**wn3** Low wet strength
Low permeability
Low fertility
Strongly acid
Very high aluminium toxicity

**wn4** Low wet strength
Low permeability
Low fertility
Strongly acid
Very high aluminium toxicity

**Fertility**

The general fertility is low to moderate. The soil materials are strongly acid and have low or moderate available water capacities, very low nutrient status, with low nitrogen and very low phosphorus levels and low to moderate CEC. The subsoils may have low permeability and pronounced aluminium toxicity.

**Erodibility**

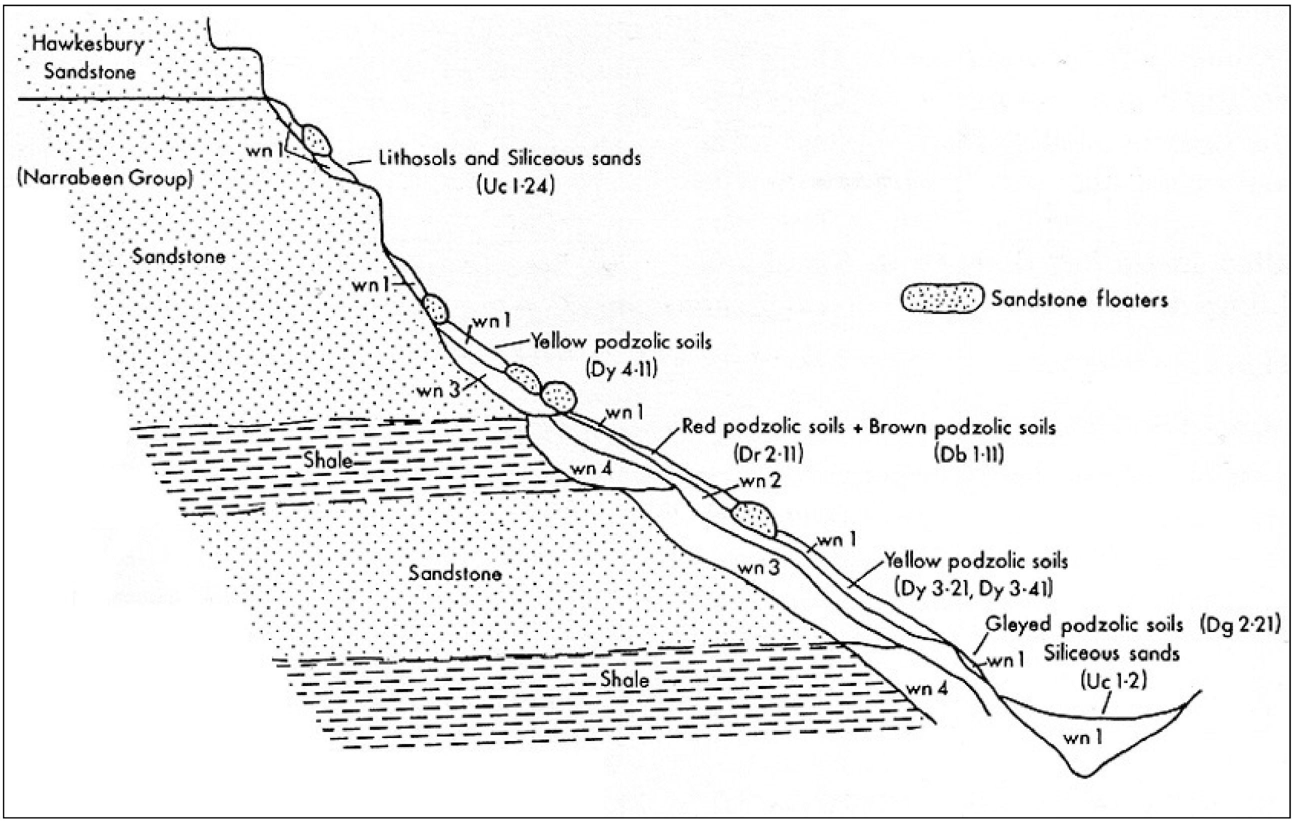
The wn1 soil material has low erodibility, consisting dominantly of highly permeable coarse sand grains. The other soil materials have moderate erodibility as they are well graded with porous and coherent fabric. Where **wn4** is dispersible it is highly erodible; otherwise it has low erodibility.

**Erosion Hazard**

Despite the low to moderate erodibility of the soil materials steep slopes produce an erosion hazard for non-concentrated flows which is extreme. Calculated soil losses for the first twelve months of urban development range up to 372 t/ha for topsoil and 547 t/ha for subsoil. Soil erosion hazard for concentrated flows is also extreme.

### Surface Movement Potential

Soils are generally shallow and therefore slightly reactive. Where profiles contain >50 cm of **wn3** or **wn4** they are moderately reactive. Large variations in soil properties occur over short distances.



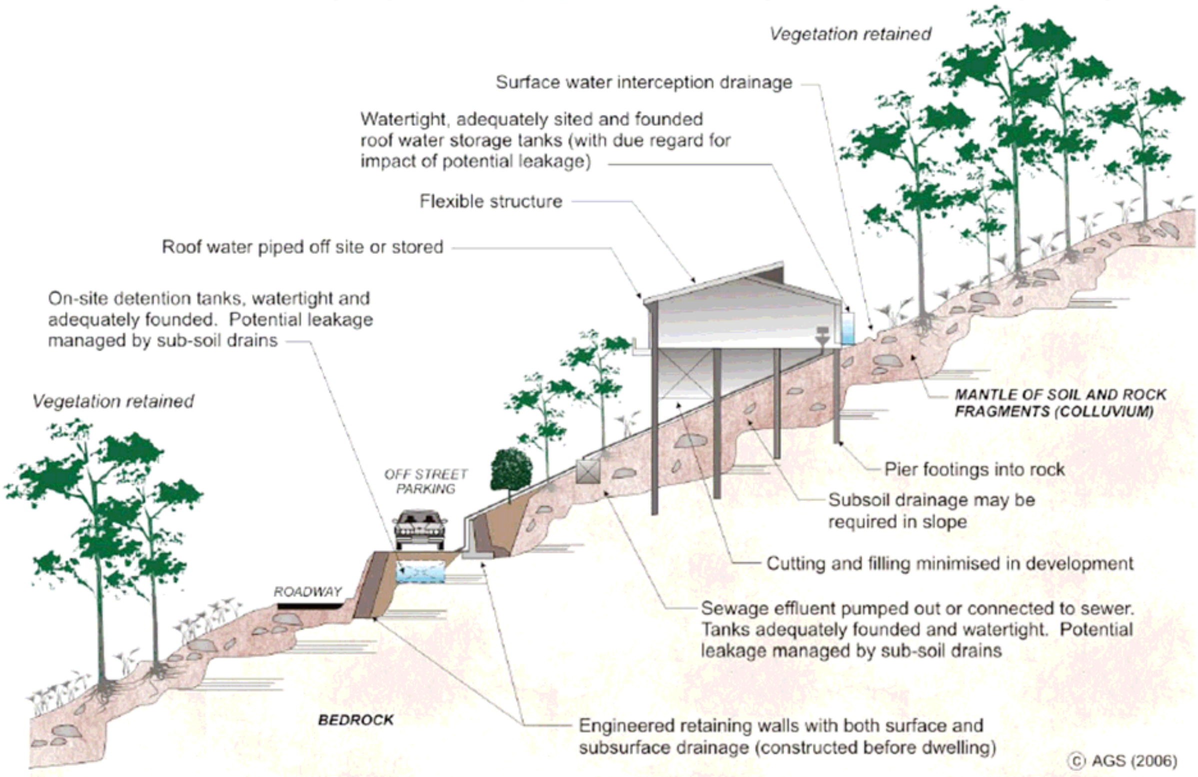

***Schematic cross-section of Watagan soil landscape illustrating the occurrence and relationship of the dominant soil materials.***

# APPENDIX H

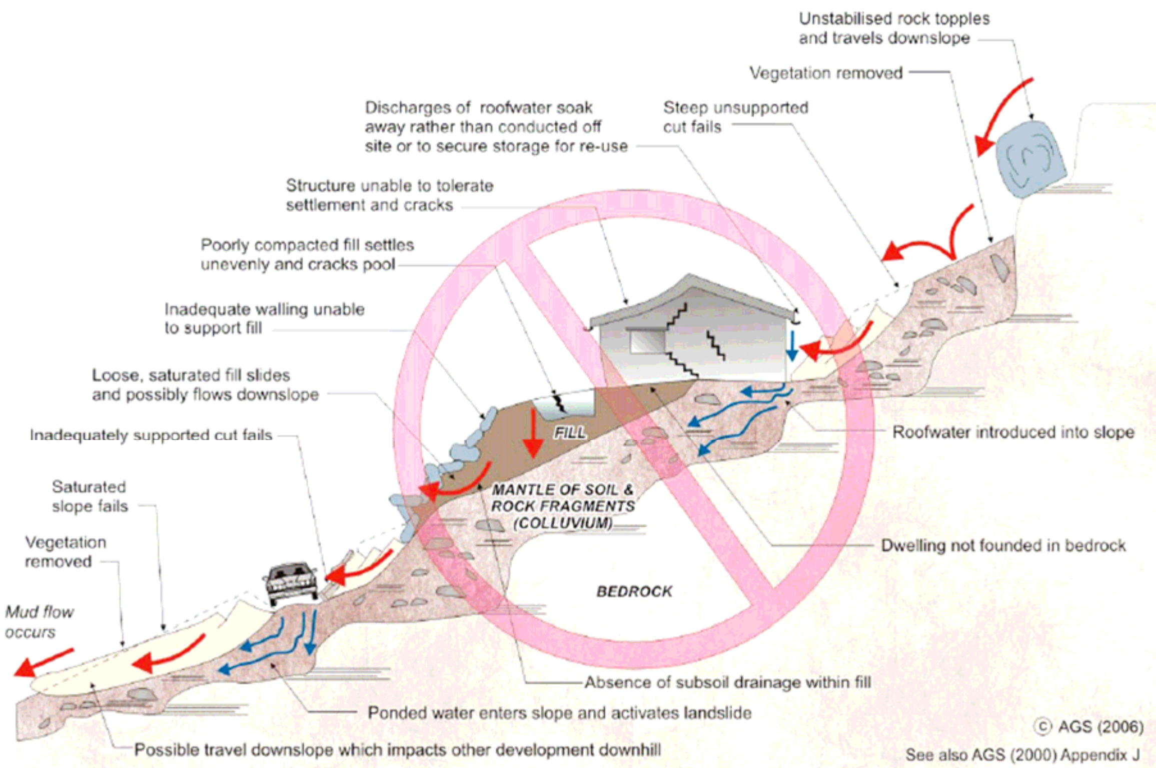
# APPENDIX G - SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

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

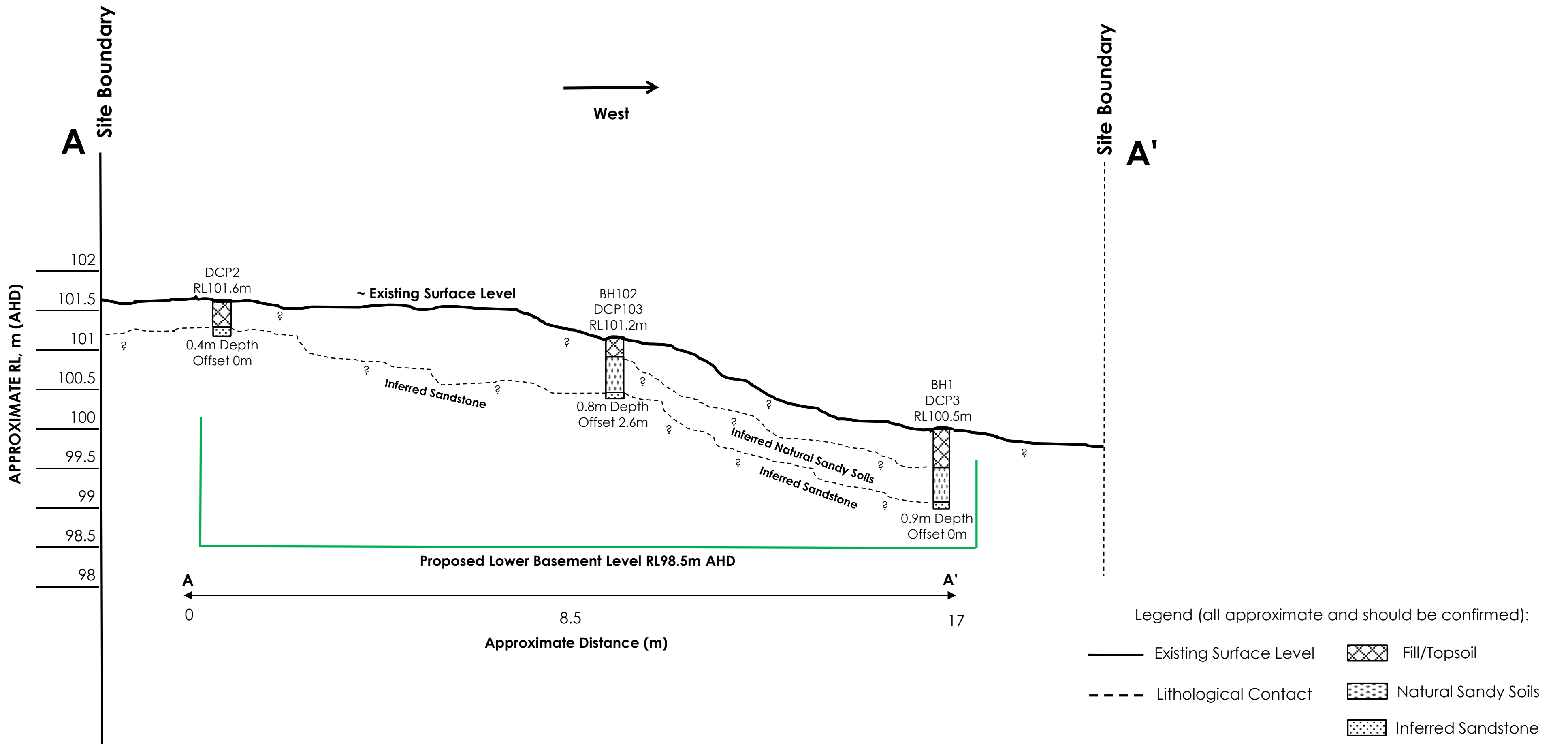
## EXAMPLES OF GOOD HILLSIDE PRACTICE




## EXAMPLES OF POOR HILLSIDE PRACTICE



# APPENDIX I




**Notes:**
1. Drawing Not To Scale (N.T.S).
2. Depths and RL's are approximate.
3. Drawing prepared based on provided survey RL's, subsurface conditions encountered by previous investigation and inferred subsurface conditions.
4. Groundwater levels may fluctuate with time.
5. Do not exaggerate vertical scale.

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|---|---|
| **Geotechnical Investigation** | |
| Georgina Nassif & Tony Nassif | |
| 426 Ralston Road<br>Palm Beach NSW 2108 | |

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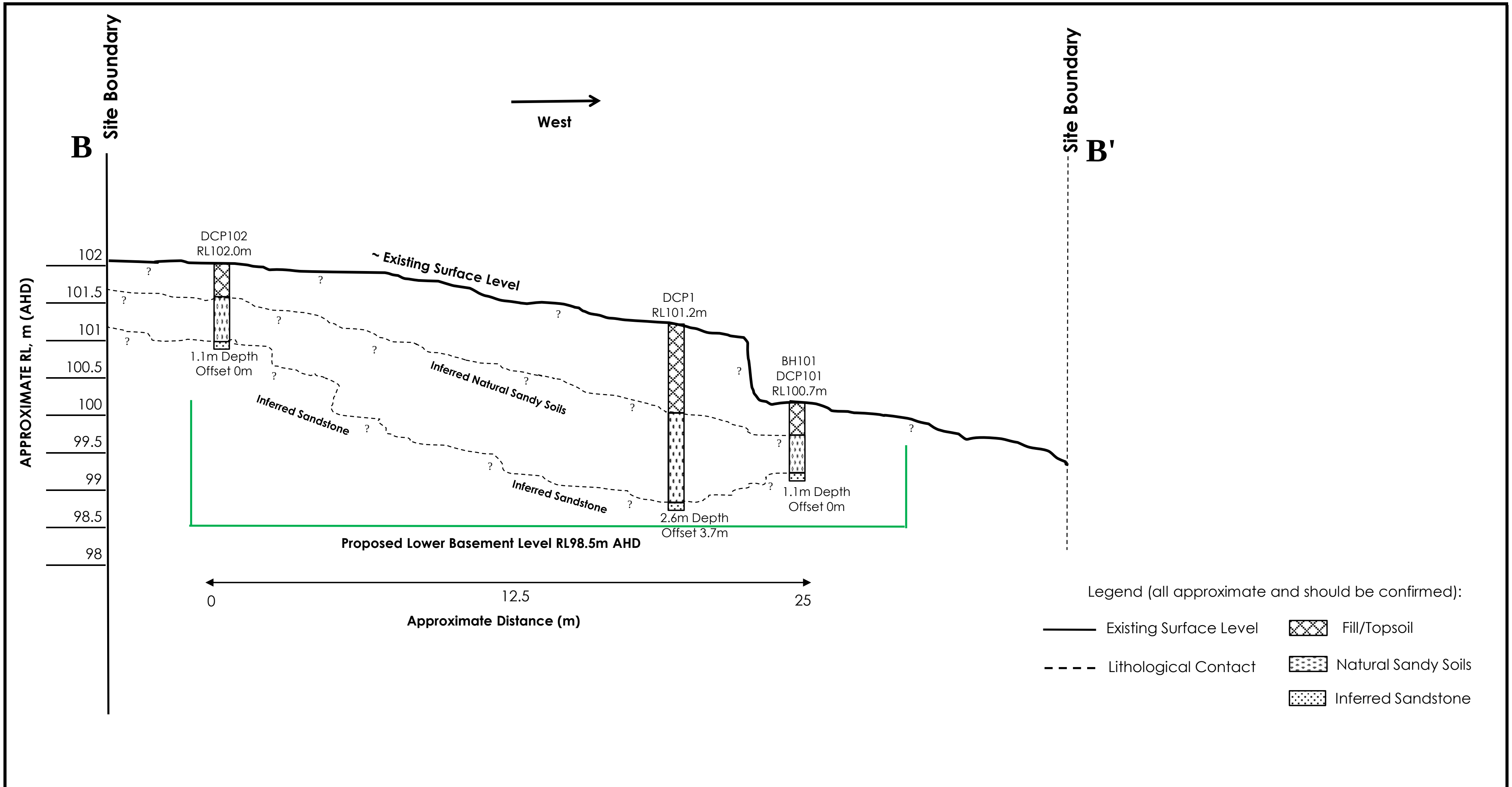
|---|---|
| Generalised Cross Section A-A' | |
| Drawn By: GN | Job No.: G21118-1-Rev A |
| Checked By: RF/JN | Figure 2A |
| Scale: N.T.S | Date: 18/09/2024 |




**Notes:**
1. Drawing Not To Scale (N.T.S).
2. Depths and RL's are approximate.
3. Drawing prepared based on provided survey RL's, subsurface conditions encountered by previous investigation and inferred subsurface conditions.
4. Groundwater levels may fluctuate with time.
5. Do not exaggerate vertical scale.

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|---|---|
| **Geotechnical Investigation** | |
| Georgina Nassif & Tony Nassif | |
| 426 Ralston Road<br>Palm Beach NSW 2108 | |

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| Generalised Cross Section B-B' | |
|---|---|
| Drawn By: GN | Job No.: G21118-1-Rev A |
| Checked By: RF/JN | Figure 2B |
| Scale: N.T.S | Date: 18/09/2024 |