Geotechnical Consultants Australia

Naycon Building Solutions Pty Ltd

Geotechnical Investigation Report

Proposed Development at:

2 Prince Edward Road

Seaforth NSW 2092

G2535-1

14th February 2025

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Report Distribution

Geotechnical Investigation Report

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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 the proposed development at No. 2 Prince Edward Road Seaforth NSW 2092 (the site). The investigation was commissioned by Naycon Building Solutions Pty Ltd (the client) and the fieldwork was carried out on the 11th February 2025.

The purpose of the investigation was to assess the subsurface conditions over the site at the selected boreholes and testing locations and to 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 is prepared to provide preliminary geotechnical advice and recommendations to assist in the preparation of 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 onsite infrastructure, followed by construction of a new two (2) storey dwelling, overlying a partial basement level. An in-ground swimming pool will also be included, as well as a secondary dwelling located west of the main property.

The Finished Floor Levels (FFL)s for the proposed development are set to be at Reduced Levels (RL)s of:

- Basement level: RL81.500m relative to the Australian Height Datum (AHD).
- Ground floor level: RL84.500m AHD.

Based on this information and the existing site levels and topography, maximum inferred excavation depths of up to 2.2m are expected to be required for construction of the proposed basement level and in-ground swimming pool, with cut and fill elsewhere. Locally deeper excavations for the building footings and service trenches are also anticipated.

Excavation depths are estimated to be 200mm below the FFLs shown on the architectural drawings.

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

1.3 Provided Information

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

- Architectural drawings prepared by New Paradigm Design Pty Ltd, titled "Proposed New Residence", and dated 13th November 2024.
- Site survey plan prepared by Usher & Company Surveying & Land Development Consultants, referenced No. 6533-DET, Issue: 1 and dated 24th February 2025.

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 any surrounding infrastructures, buildings, council assets, etc., along with the proposed development.
- Excavation conditions and recommendations on excavation methods in soils and rock to restrict any ground vibrations.
- Appropriate permanent and temporary batter slopes within the site based on ground conditions encountered during the site investigation.
- Vibration control and recommendations to restrict 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 ground conditions within the site.
- Groundwater levels which may be determined during the site investigation.
- Recommendations on groundwater maintenance and limiting inflow (if encountered).
- 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).

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:

- Review of Before You Dig Australia (BYDA) plans and 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 the site plans and drawings to determine appropriate borehole and Dynamic Cone Penetrometer (DCP) test locations
- A site walkover to identify any relevant features of the site, for the purpose of a preliminary slope risk assessment. The site features are shown on **Figure 1, Appendix B** and include:
  - Rock outcrops.
  - Slope breaks.
  - Approximate slope angles and directions.
- Hand augering of two (2) boreholes at selected locations within the site (accessible locations chosen), identified as boreholes BH1 and BH2, and carried out using hand operated equipment to refusal depths of approximately 0.4m and 0.45m below the existing ground level within the site (bgl).
- DCP testing immediately adjacent to the boreholes and at two (2) other selected locations within the site (accessible locations chosen), using hand operated equipment to varying practical refusal depths of approximately 0.28m to 0.45m bgl. The DCP tests are identified as DCP1 to DCP4 inclusive.
    - The approximate locations of the boreholes and DCP tests are shown on **Figure 1, Appendix B** of this report.
- Reinstatement of boreholes with available soil displaced during augering.
- Preparation of this geotechnical engineering report.

1.6 Constraints

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

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 on the north-western corner of Lister Avenue and Prince Edward Road in a residential area. The site is located approximately 40m east of Wakehurst Parkway thoroughfare.
Site Address	2 Prince Edward Road Seaforth NSW 2092
Approximate Site Area[1]	494m^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 detached garage. The remaining site area was mainly covered in grass, vegetation and some mature trees in the front portion of the site. Sandstone rock outcrops were present beyond the eastern fence line at the site. The approximate locations of the Sandstone rock outcrops are shown on **Figure 1, Appendix B.**
Approximate Distances to Nearest Watercourses (i.e. rivers, lakes, creeks, etc.)	• Burnt Bridge Creek – 230m north-east of the site. • Sugarloaf Bay – 440m to 460m west of the site.
Site Surroundings	The site is located within an area of residential use and is bounded by: • Residential property at No. 4 Prince Edward Road to the north. • Prince Edward Road carriageway to the east. • Lister Avenue thoroughfare to the south. • Residential property at No. 29 Lister Avenue to the west.

[1]Site area is approximate and obtained from the architectural drawings referenced in Section 1.3.

2.2 Topography

The local and site topography generally falls towards east. Levels within the site vary from approximately RL85.1m to RL81.9m AHD.

General onsite assessment and review of site levels shown on the site survey plan provided in Section 1.3 indicates the site has an overall very gently to gentle slopes of ~2° to ~4°, varying across the site. Eastwards of the eastern boundary fence at the site, the slope angle increases to approximately 15°, providing a strong slope down towards Prince Edward Road.

It should be noted that the site topography, levels and slopes are approximate and based on observations made during the geotechnical investigation, and reference to NSW Six Maps (https://maps.six.nsw.gov.au/) and Mecone Mosaic (https://meconemosaic.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 could vary from those outlined in this report.

2.3 Regional Geology

Reference to the Sydney 1:100,000 Geological Series Sheet 9130 First Edition, dated 1983, by the Geological Survey and Department of Mineral Resources of New South Wales, indicates that the site is underlain by the Triassic Age Hawkesbury Sandstone (Rh). The Hawkesbury Sandstone (Rh) generally comprises "medium to coarse-grained quartz Sandstone, very minor Shale and laminite lenses".

Furthermore, reference made to MinView by the State of New South Wales through Regional NSW 2024 also shows the site is underlain by Hawkesbury Sandstone (Tuth).

A review of the regional maps by the NSW Government Environment and Heritage shows that the site is located in the Lambert (la) landscape group.

The Lambert soil landscape group is characterised by undulating to rolling rises and low hills on Hawkesbury Sandstone, with gently to moderately inclined slopes dipping at 20% and typically containing >50% rock outcrop. Soils of the Lambert soil landscape group typically comprises sandy and/or clayey loams or clayey sands and these are generally strongly acidic (pH 4.0) to moderately acidic (pH 5.5). This soil landscape may contain organic or peaty soils, particularly in areas of poor drainage. This soil landscape generally has a high soil erosion hazard, seasonally perched water tables, highly permeable soil and has very low soil fertility.

The Lambert (la) soil landscape group report is attached in **Appendix G**.

3. SUBSURFACE CONDITIONS AND ASSESSMENT RESULTS

3.1 Stratigraphy

A summary of the surface and subsurface conditions within the investigation area of the proposed development is presented in Table 2 below and in the detailed engineering borehole logs presented in **Appendix D.** These should be read in conjunction with the geotechnical explanatory notes detailed in **Appendix C**, and with the DCP test results in **Appendix E**.

The fill and soil descriptions provided 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. Foundations on Sandstone and Shale in the Sydney Region, Australian Geomechanics Journal, December (1998).

The estimated soil consistency and strength assessed by DCP testing in the site during the geotechnical investigation is approximate and could vary within the site. It is recommended that an experienced geotechnical engineer confirm the subsurface materials exposed during construction by inspection.

Ground conditions within the site may 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 and BH2) carried out within the site, the subsurface conditions at the test locations generally comprised:

- (Unit 1): FILL: Silty Clay material, low plasticity, from surface and observed to an average depth of up to 0.3m bgl, underlain by:
- (Unit 2): Natural Sandy CLAY, typically medium plasticity, fine to coarse sand, observed to depths of 0.4m to 0.45m bgl.

The composition, depth and consistency/strength of the natural soils are likely to vary throughout the site, predominantly at locations and depths not assessed during this geotechnical investigation.

Based on the geotechnical investigation at the selected boreholes and DCP testing locations, observations of Sandstone rock outcrops near the site, along with our experience in the local region, it is inferred that Sandstone bedrock of variable strength and weathering underlies most of the site area at varying depths between approximately 0.3m to 0.5m bgl. The outcrops were generally assessed to be highly weathered and medium estimated strength.

Ground conditions shown in Table 2 below are in part inferred from the DCP testing results. A similar profile observed during the geotechnical investigation is likely 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 detailed engineering borehole logs presented in **Appendix D** and an experienced geotechnical engineer should confirm the subsurface materials exposed during construction by inspection, as site conditions may vary.

Table 2: Summary of Subsurface Conditions

<table>
<tr><th colspan="2">Borehole/DCP ID</th><th>BH1/DCP1</th><th>DCP2</th><th>BH2/DCP3</th><th>DCP4</th></tr>
<tr><th colspan="2">Approximate Surface Level (m AHD)</th><td>RL82.3</td><td>RL82.8</td><td>RL84.4</td><td>RL84.4</td></tr>
<tr><th>Unit</th><th>Unit Type</th><th colspan="4">Depth/Thickness of Unit (m bgl)</th></tr>
<tr><td>1</td><td>FILL[1]. Silty Clay</td><td>0.0 – 0.3</td><td rowspan="2">0.0 – 0.28</td><td>0.0 – 0.3</td><td rowspan="2">0.0 – 0.39</td></tr>
<tr><td>2</td><td>Natural Soil[2]. Sandy CLAY, estimated soft to firm</td><td>0.3 – 0.45</td><td>0.3 – 0.4</td></tr>
<tr><td>3</td><td>Inferred Sandstone Bedrock[3]</td><td>Below 0.5</td><td>Below 0.3</td><td>Below 0.4</td><td>Below 0.4</td></tr>
</table>

[1]Thickness of the layer is expected to vary from those thicknesses indicated in Table 2.
[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. Reference should be made to the detailed engineering borehole logs presented in **Appendix D**.
[3]Inferred bedrock composition, class, strength and depth should be confirmed by a geotechnical engineering prior to construction by additional cored borehole drilling and rock strength testing, or during construction by inspection. Sandstone bedrock is inferred to be present at or shortly below the practical DCP test refusal depths at the selected testing locations within the site.

Notes:

- Inferred bedrock 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 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 augering of the boreholes and DCP testing to a maximum depth of approximately 0.45m (BH1/DCP1) bgl. No seepage was observed in the soil near any of the outcrops seen near the site.

It is noted that the boreholes were immediately backfilled following completion of fieldwork which precluded longer term monitoring of groundwater levels.

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 will most likely also occur at the soil-rock contact.

It should be noted that groundwater levels have the potential to fluctuate during daily or seasonal events such as tidal changes, heavy rainfall, damaged services, flooding, etc., and moisture content within soils may be influenced by events within the site and within adjoining properties. Groundwater monitoring should be carried out during excavation and 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 approximate slope angles and direction, depth of soils and overall strength, movement of groundwater and surface runoff, drainage and potential slide planes within the interfaces of rock and soil were assessed by GCA as part of the geotechnical investigation. The overall assessment was carried out in accordance with guidelines published by the Australian Geomechanics Society (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 is 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 east, 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 topography of the overall site area varies throughout and slopes generally towards the east, as discussed in Section 2.2 and shown in **Figure 1, Appendix B** of this report. 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 mature trees, vegetation and grass. Sandstone outcrops were also present beyond the eastern fence line at the site, and locations are shown in **Figure 1, Appendix B**. Associated concrete pavements, detached garage and the existing dwelling covered the remaining site area.
Groundwater	No	No groundwater was encountered or observed during this investigation to a maximum depth of approximately 0.45m bgl, as discussed in Section 3.2. It is expected that groundwater which may be encountered within the site will be in the form of seepage through soils and bedrock, and at the soil-rock contact. Based on the regional and site topography, we expect groundwater flows (including surface water) to flow towards the east, following in general the topography.

Observations		Identification	Comments
Surface Water		No	No surface water, ponding or seepage was observed within the site. No seepage was visible through the Sandstone outcrops at the time of the investigation. Fill materials at the site were predominantly moist and the natural soils were moist to wet, and are underlain by *inferred* Sandstone bedrock at varying depths throughout the site area (as discussed in Section 3).
Outcrops		Yes	Sandstone rock outcrops were present and observed near the eastern portion of the site. The locations of the Sandstone rock outcrops are given in **Figure 1, Appendix B**. The *inferred* Sandstone bedrock underlying the site is expected to become moderately to slightly weathered, becoming fresh and increasing in estimated strength with depth.
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	None	No signs of structural distress or movement were observed to the existing dwelling and infrastructure within the site.
	Retaining Walls	N/A	No retaining walls were observed within the site.
Adjoining Properties		N/A	Infrastructure adjoining the site was observed to be in a generally good condition and trees within the vicinity were observed to have no signs of deformation.
Ground Movement		No	No cracks in the ground, slumping, or other signs of landslip were observed within the site. No ground deformation was observed within the site (in accessible areas).
Tilting or Bending Trees		No	No trees showed signs of bending or tilting within the site. Typically, tilting, bending or curved trees can indicate rotation due to soil creep or movement.
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 (in accessible areas).

Based on the subsurface conditions encountered within the site during the geotechnical investigation, it is anticipated that fill material, natural soils and *inferred* Sandstone bedrock will underlie majority of the proposed development area, 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 of the potential effects which may be associated with the hazards onsite and on the adjoining properties, along with the buildings, land and occupiers within. The adjoining properties, and existing dwellings are considered as part of the risk levels to the property pre-development. GCA's pre-development assessment risk to property is summarised 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]	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 under the existing conditions prior to construction of the currently proposed development is assessed to be "low".

According to AGS 2007c, the "low risk level" is usually tolerated by regulators. Ongoing maintenance and stabilisation measures are still required at the site and to maintain stability.

It is noted that the AGS guidelines recommend tolerable loss of life for the person most at risk for the existing slopes and existing development to be 1×10^{-4}/annum.

4.4 Mitigation and Control Measures

To ensure the stability of the site and the proposed development within the site, the following recommendations should be considered along with (but not limited to) the recommendations presented in this report:

- The design and construction of earthworks, foundations, retaining structures, excavation stabilisation and drainage measures 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.
- Adopting stabilisations actions such as retaining walls or batter slopes which should address the instability issues associated with the ground profile within the site and neighbouring properties before any excavation or construction work commences.
- Excavation, pile installation and any rock ripping and hammering (or the like) are expected to cause vibrations within the underlying bedrock. Monitoring of existing retaining walls, soils and bedrock underlying the site is required by a geotechnical engineer during construction.
    - Any observable movement within the underlying soil, rock and retaining walls should cease work immediately, and GCA should be contacted for further advice, if not present onsite.
- Monitoring of excavation stability, batter slopes, ground movement and movement and deflections of retaining walls or any infrastructure within and adjoining the site. A suitably qualified geotechnical engineer should undertake this monitoring. To ensure the stability of the excavation and adjoining infrastructure, this will require ongoing

inspections and approvals by a geotechnical engineer or engineering geologist. General advice on excavation stability is provided in Section 5.9 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 5.9.3.
- Any excavation is started from higher surface levels and undertaken in stages progressing towards the lower surface levels within the site. Excavations, including any batter slopes, require monitoring and approval by a geotechnical engineer familiar with the site conditions.
- Backfilling, if required, is placed and compacted to engineering standards in accordance with AS 3798-2007 "Guidelines on Earthworks for Commercial and Residential Developments" and AS 1289.5.3.1-2004 "Methods of Testing Soils for Engineering Purposes", with reference to Section 5.11.1 and Section 5.11.2 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 to and during construction.
- Backfilling behind any walls is 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 inspection, and constant testing. Appropriate backfill drainage should also be provided.
- Appropriate drainage methods are incorporated to ensure that 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 requires carefully assessment, design and detail by the project stormwater engineer and may require council approval. 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 drawings.
- The foundation system for the proposed development should be founded/embedded into the *inferred* Sandstone bedrock underlying the site (of at least low estimated strength), as discussed in Section 5.10 of this report. Piles may be required to increase resistance against sliding on hillsides.
- Foundation systems for the proposed development, building structures, retaining walls and any water tanks, etc., should be sufficiently founded/embedded into the underlying *inferred* Sandstone bedrock, and where necessary designed for lateral earth pressures induced by soil movement along the interface between soils and the underlying bedrock.
- Foundations require inspection and approval 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. If any movement within the underlying soils, bedrock and/or retaining walls is observed, then work should cease immediately, and GCA be contacted for further advice.
- Vibration levels during excavation and construction are maintained to appropriate levels within the site, predominantly where existing retaining walls or sensitive structures exist. Further general advice is provided in Section 5.7.

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 investigation.

P (H) = 0.0001/annum

Probability of Spatial Impact

Upon reviewing the architectural drawings referenced in Section 1.3 and observations made from our onsite investigations, GCA anticipates approximately 190m^2 to be at risk of soil creep or shallow failure.

P (S:H) = 0.39

Possibility of the Location Being Occupied During Failure

The average household is taken to be occupied by 6 people. It is estimated/assumed that 4 people are in the main house and 2 people in the secondary dwelling for 18 hours a day, 7 days a week.

It is estimated/assumed that 2 people are in the main house and 1 person in the secondary dwelling 12 hours a day, 5 days a week.

$$\left(\frac{6}{8}\times\frac{18}{24}\times\frac{7}{7}\right)+\left(\frac{3}{8}\times\frac{12}{24}\times\frac{5}{7}\right)=0.69$$

P (T:S) = 0.69

Probability of Loss of Life on Impact of Failure

Based on the volume of material subject to landslide movement 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 0.05, indicating a very high chance of survival. This vulnerability value is recommended from Appendix F in Australian Geomechanics Society (AGS) “Practice Note Guidelines for Landslide Risk Management – AGS 2007c”

V (D:T) = 0.05

Risk Estimation

R (LOL) = 0.0001 x 0.39 x 0.69 x 0.05

= 0.0000014

R (LOL) = 1.4 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 2.2m are expected to be required for construction of the proposed basement level and in-ground swimming pool, with cut and fill elsewhere. Locally deeper excavations for the building footings and service trenches are also anticipated.

Therefore, appropriate measures against the potential for any instability are incorporated into the design and construction of the proposed development, predominantly 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 adopted during the design and construction of the proposed development, as well as post construction maintenance, the following assessed risks relate to the stability of the property, upon completion of any infrastructure, building foundations and retaining walls. 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 risk to the property following construction of the proposed development, and assuming suitable retention measures are adopted, is assessed to remain "low".

Therefore, providing that 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 required during construction of the proposed development foundation system, excavation works and adopted retention systems to confirm the ground conditions exposed. Appropriate certifications should also be provided during staged inspections by the project structural engineer and geotechnical engineer.

5. GEOTECHNICAL ASSESSMENT AND RECOMMENDATIONS

5.1 Site Suitability for the Proposed Development

The geotechnical investigation found that the surface and subsurface conditions of the site generally comprise Silty Clay fill material observed to an average depth of 0.3m, overlying natural soils up to around 0.45m bgl, underlain by *inferred* Sandstone bedrock. Sandstone rock outcrops were observed near the site.

Based on this investigation and our assessment, it is in our opinion that the site is suitable for the proposed development, and poses a low risk of instability to the site and surrounding areas, provided that the design and construction of the proposed development adheres to all the recommendations throughout this report and mitigation measures described in Section 4.4.

It is recommended that an experienced geotechnical engineer confirm the stability of the site and retention systems with staged inspections during excavation and construction.

5.2 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 excavations and vicinity of the proposed development. The “zone of influence” is defined as the zone created by drawing a 45° line above horizontal from the boundaries at the base of bulk excavation, into the excavation faces to the surface.

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”.

5.3 General Geotechnical Issues

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

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

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

5.4 Preliminary Site Lot Classification

Based on the geotechnical investigation and observations made at the selected testing locations within the site, surface and subsurface conditions of the site generally comprise Silty Clay fill material observed to an average depth of 0.3m, overlying natural soils up to around 0.45m bgl, underlain by inferred Sandstone bedrock. Sandstone rock outcrops were observed near the site.

The governing site lot classification in accordance with AS 2870-2011 is 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 sloping nature of the site and location within a potential landslide risk region.

Based on the boreholes and DCP tests carried out within the site, and proposed excavations which will result in removal of fill material and natural soils, AS 2870-2011 indicates that the site may be classified as a "Class A" site for design and construction of the proposed <u>basement</u> foundation system, assuming that foundations for the basement level are founded below any soft/loose soils, topsoil, slope wash, fill or other deleterious material, entirely on Sandstone bedrock. Sandstone bedrock underlying the site will require confirmation by an experienced geotechnical engineer. The "Class A" classification generally applies to most sand and rock sites.

A higher classification of "Class M" should be adopted for the remaining structures built at ground surface level (i.e. portions of the ground floor level, secondary dwelling, footings, fences, etc.), and/or where fill/natural soils are present at depths ≥0.4m below the proposed developments FFLs. This should be confirmed/monitored during construction.

The above classification is based on assessment of the subsurface conditions at the selected boreholes and testing locations/depths within the site, and from Sandstone rock outcrop mapping onsite (see **Figure 1, Appendix B**). An experienced geotechnical engineer should confirm the subsurface conditions exposed during construction. It should be noted that the classification given above is appropriate for the site 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 5.10 below, with reference made to AS 2870-2011. An experienced geotechnical engineer should perform geotechnical inspections at the site and confirm the actual depth of underlying fill material, natural soils and Sandstone bedrock during excavation and construction.

Contacting GCA is required when the ground conditions vary from those described in this report at the boreholes and testing locations.

Footing designs on soil (if any) 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 for the foundation design.

GCA recommends that planting of trees around the development area (i.e. in close proximity to the proposed building foundations) be limited as they can affect moisture changes within the soil and cause significant displacement and damage within the building foundations by extensive tree root system growth and movement.

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

5.5 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 at 45^0 above horizontal from the base of the proposed excavations) of the proposed development. These inspection pits need digging out prior to any 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 and bedrock, which will determine the need for additional support, such as underpinning, prior to installation of shoring piles, or any excavation and construction activities.

5.6 Excavation

Maximum inferred excavation depths of up to 2.2m are expected to be required for construction of the proposed basement level and in-ground swimming pool, 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 encountered during the geotechnical investigation, it is anticipated that excavations will extend through Unit 1(fill materials), Unit 2 (natural soils) and the *inferred* underlying Sandstone bedrock throughout majority of the proposed development area, as discussed in Section 3 above.

Excavation for the basement level is anticipated to exposed Sandstone bedrock at final excavation level of RL81.500m AHD.

Excavation for the ground floor level at RL84.500m AHD is likely to encounter fill and natural soil materials, and may require fill emplacement to bring the level up to the required RL. It is recommended that the existing fill layer and any natural soil materials which are not of at least estimated stiff consistency/strength are removed from the site, before placing fill materials in accordance with Section 5.11.

The possibility of encountering higher strength (i.e. medium to high estimated strength, or better) Sandstone 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 performed within the site.

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

5.6.1 Excavation Assessment

Excavation through the fill and natural soil materials and very low to low strength bedrock is likely feasible using conventional earth moving excavators, typically medium to large hydraulic excavators. Smaller sized excavators may face difficulty when encountering high strength bands of soil and rock. Where high strengths bands are encountered, allow for rock breaking or ripping. 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 the basement level and the swimming pool will most likely encounter medium or higher strength Sandstone bedrock, and this 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. The use of a rock saw is recommended.

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 Sandstone 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.

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

All excavation works is performed in accordance with the NSW WorkCover code of practice for excavation work.

5.7 Vibration Monitoring and Controls

Particular care is required to ensure that adjacent buildings and infrastructure (i.e. road reserves, buildings, etc.), are not damaged during 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.

Vibrations transmitted using 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 acceptable limits. 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.

Rock hammering and rock sawing activities should require monitoring at all times during excavation.

The effectiveness of all the above-mentioned approaches must be confirmed by assessing the results of the initial (live) vibration monitoring. Achieving the limits of 5mm/sec and 10mm/sec is expected if rock breaker equipment or other plant 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 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 system 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 or considered, consultation should be made with appropriate subcontractors or consultants for the installation of vibration monitoring equipment.

A geotechnical engineer should be contacted immediately if vibrations during construction or in adjacent structures exceed the values described 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 that a dilapidation report be carried out prior to any excavation or construction, as discussed in Section 5.2. This should be considered a “Hold Point”.

5.8 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 above bulk excavation level, as excavations are anticipated to be cut mostly into the *inferred* Sandstone bedrock.

It is 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 development. It is 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 natural soils/inferred bedrock interfaces, predominantly 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 defects in the underlying *inferred* Sandstone 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 requires consideration as part of the long-term design life of the proposed development. 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, the precautionary drainage measures adopted can include, but 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 basement level walls and slab, with allowance given for nominal hydrostatic uplift.

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 (see Section 6). This assessment should also be carried through to ensure a suitable drainage and retention system is implemented for the proposed development, as discussed in Section 5.9 below, and to provide confirmation of the hydrogeological characteristics prior to construction.

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.

5.9 Excavation Stability

Maximum inferred excavation depths of up to 2.2m are expected to be required for construction of the proposed basement level and in-ground swimming pool, 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 excavation and construction.

5.9.1 Batter Slopes

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).

It is possible that due to the nature of fill material, natural soils and weathered bedrock underlying the site, and the potential for elevated groundwater levels and seepage within the excavation area, unsupported vertical cuts of material carry the potential for slump failure.

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[2]	VL	1:1	0.75:1
	L		0.5:1
	M – H or better[3]	Semi-Vertical to Vertical	

[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]Assumes the presence of Sandstone bedrock underlying the entire site area and no defects of adverse dipping are present in the bedrock.
[3]Preliminary only and is subject to further cored borehole drilling and rock strength testing and staged inspections during construction, undertaken by an experienced geotechnical engineer.
Notes:

- VL = Very low estimated strength, L = Low estimated strength, M = Medium estimated strength, H = High estimated strength.

All batter slopes within the site should remain stable providing 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 within the site, which we recommend at staged inspections at a minimum the initial 1.0m depth, followed by 1.0m to maximum 1.5m subsequent excavation depths.

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 which may underlie the site (not observed/assessed during the current geotechnical investigation), 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 requires placing following inspection of the temporary batters by a geotechnical engineer or engineering geologist.

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 sufficiently constructed on Sandstone 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.

5.9.2 Excavation Retention Support Systems

Where there is insufficient space between the proposed development and adjoining infrastructure (i.e. site northern and southern boundaries), or where there is adjacent infrastructure located within the "zone of influence" (as outlined in Section 5.9.1 above), then consideration should be given to a suitable retention system.

A suitable retention system could comprise an engineer designed reinforced retaining wall (where batter slopes are adopted and approved), or a soldier pile wall solution, with piles sufficiently embedded into the *inferred* Sandstone bedrock underlying the site, and concrete and reinforcement infill panels for the support of the excavation and soils.

For a piled wall, 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 fill and natural soil materials and extremely 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 Sandstone bedrock may be safely battered to prevent the collapse of soils (see Section 5.9.1). 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, which should be monitored by a geotechnical engineer familiar with these site conditions. Where this is not achievable, then an alternative retention system such as a contiguous pile wall solution, is required.

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.) where required. This option may also be adopted where excessive surcharges are adjacent to the proposed excavation and to meet acceptable deflection criteria or where there is a potential for undermining of any adjoining building or infrastructure (refer to Section 5.5).

All piles require sufficient embedded into the *inferred* Sandstone bedrock underlying the site and need inspection and approval by a suitably qualified geotechnical engineer. It is recommended that piles are not founded on any soft or weak bands or layers (i.e. clay seams, extremely weathered zones, fractured zones, etc.) underlying the site. Furthermore, the project structural engineer should select the retention system, with all structural elements also inspected and approved by a suitably qualified structural engineer.

Groundwater inflows may pass through shoring pile gaps during excavation. Installing strip drains behind the retention system connected to the buildings stormwater system, can control any groundwater inflows encountered. In weak areas of the retention system, these may require shotcreting or localised grouting, predominantly where groundwater seepage and loose or soft materials are visible. Shoring design should consider 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 5.9.3 below. Bulk excavation and foundations (including pile installations) require supervision, monitoring and inspection 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.

5.9.3 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)
γ = 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)	Sandstone 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 ϕ' (°)	24	26	30
Modulus of Elasticity E_{sh} (MPa)	3	6 (soft) 10 (firm, or better)	70
Earth Pressure Coefficient At Rest Ko[1]	0.59	0.56	0.5
Earth Pressure Coefficient Active Ka[2]	0.42	0.39	0.3
Earth Pressure Coefficient Passive Kp[2]	2.37	2.56	3.0
Poisson Ratio v	0.4	0.35	0.3

[1]Earth pressure coefficient at rest (Ko) can be calculated using Jaky'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 additional cored borehole drilling and rock strength testing, or during construction by inspection.

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).

5.10 Foundations

Based on our geotechnical investigation carried out within the site, excavation for the basement level to approximately RL81.500m AHD is expected to expose Sandstone bedrock of variable strength and weathering. To achieve the ground floor RL of 84.500m AHD, this will likely require placing fill materials above the natural ground level at the site. Before placing these fill materials, it is recommended that the existing fill layer and any natural soil materials which are not of at least estimated stiff consistency/strength are removed from the site, before placing fill materials in accordance with Section 5.11.

Excavation for the swimming pool is likely to expose Sandstone bedrock at final excavation level, with excavation through fill and natural soils, overlying Sandstone bedrock of variable strength and weathering.

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. Therefore, it is recommended a geotechnical engineer confirm the underlying ground conditions encountered during construction by inspection.

5.10.1 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.

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 are 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. GCA should be present to witness the initial pile drilling stage.

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	30	15
	L or better[3]	1,500	100	50

[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 prior to construction by further borehole drilling and rock strength testing, or during construction by inspection from a geotechnical engineer.
[3]Conforming to at least Class IV Sandstone (or better).
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, Mostyn G. & Walker B.F. (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 (Φ_{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.

5.10.2 Geotechnical Comments

Bearing capacity varies according to foundation depth, shape and dimensions, including method of installation for piles. Although settlement behaviour can also vary, the proposed development's foundation system is expected to experience settlements of less than 1% of the minimum footing dimension (Reference Pells P.J.N, Mostyn, G. & Walker B.F., "Foundations on Sandstone and Shale in the Sydney Region", Australian Geomechanics Journal, 1998).

Consultation should be made with specialist subcontractors to discuss the feasibility of piles for the existing site conditions.

Specific geotechnical advice is recommended for footing designs and end bearing capacities, and the design of the foundation system (shallow and pile foundations) is performed 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 or a shallow foundation embedded into the underlying *inferred* Sandstone bedrock at the site. If piled foundations are adopted, 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 are implemented as a precaution as part of the design and construction of the proposed development. Implementing impermeable surfaces should divert surface water away from the proposed development's foundations and the excavation area, help eliminate or minimise surface water infiltration into the fill, natural soils and *inferred* Sandstone bedrock at the site.

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 collapsing nature of fill material, it is recommended that consideration be given to using a liner in the section of fill to avoid collapse of the soil. It is also recommended that concrete for the piles are tremie poured from the base of the pile excavation. Other alternatives may be the use of Continuous Flight Auger (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 boreholes 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.

5.11 Subgrade Preparation and Filling

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

5.11.1 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 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 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.

5.11.2 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, including the NSW Environment Protection Authority (EPA). 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 all mitigation and control measures for the proposed development, as outlined in this report, included in Section 4.4.
- Following the recommendations in the AGS document "Some Guidelines for Hillside Construction" (see **Appendix H**).
- Dilapidation survey report on adjacent properties and infrastructure.
- Monitoring and supervision of excavations within the site, including appropriate inspections and approvals on all batter slopes adopted throughout the site (where feasible). Geotechnical inspections of exposed materials in the excavated faces at an initial 1.0m depth, followed by 1.0m to maximum 1.5m subsequent excavation depths.
- Confirming the composition, class, depth and estimated strength of the underlying bedrock material before construction by further cored borehole drilling and rock strength testing, or during construction by inspection, undertaken by an experienced geotechnical engineer. Confirmation of the underlying bedrock should occur predominantly in areas and at depths not assessed during this geotechnical investigation.
- Geotechnical inspections of exposed materials at bulk level excavation.
- Geotechnical inspections of shoring wall piles installations (where installed).
- Geotechnical inspections of foundations (shallow and pile foundations) to confirm the exposed material in the foundation and the preliminary allowable 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, including a groundwater assessment by excavating two (2) test 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 to and during the site 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 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 is completed.

It is recommended that if for any reason, the site surface, subsurface and geotechnical conditions (including groundwater conditions) encountered during the site investigation vary substantially from conditions encountered during excavation and 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 borehole or test locations.

GCA does not accept any liability for varying site conditions which were not observed, and were out of the inspection or test areas, or were in inaccessible areas during the time of the investigation. This report and any associated information and documentation is prepared solely for **Naycon Building Solutions Pty Ltd,** and any misinterpretations or reliance on this report by third parties shall be at their own risk. Any legal or other liabilities resulting from the use of this report by other parties can not be transferred 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)

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8. REFERENCES

Pells P.J.N, Mostyn, G. & Walker B.F., "Foundations on Sandstone and Shale in the Sydney Region", Australian Geomechanics Journal, 1998.

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AS 1289.5.3.1-2004 Methods of Testing Soils for Engineering Purposes. Standards Australia.

AS 2870-2011 Residential Slabs and Footings. Standards Australia.

AS 2159-2009 Piling - Design and Installation. Standards Australia.

AS 4678-2002 Earth Retaining Structures. Standards Australia.

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NSW Department of Mineral Resources (1983) Sydney 1:100,000 Geological Series Sheet 9130 (Edition 1) Geological Survey of New South Wales. Department of Mineral Resources.

NSW Government Environment and Heritage, Soil and Land Information, Sydney 1:100,000 Soil Landscape Series Sheet 9130la.

MinView. State of New South Wales Through Regional NSW 2024.

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.

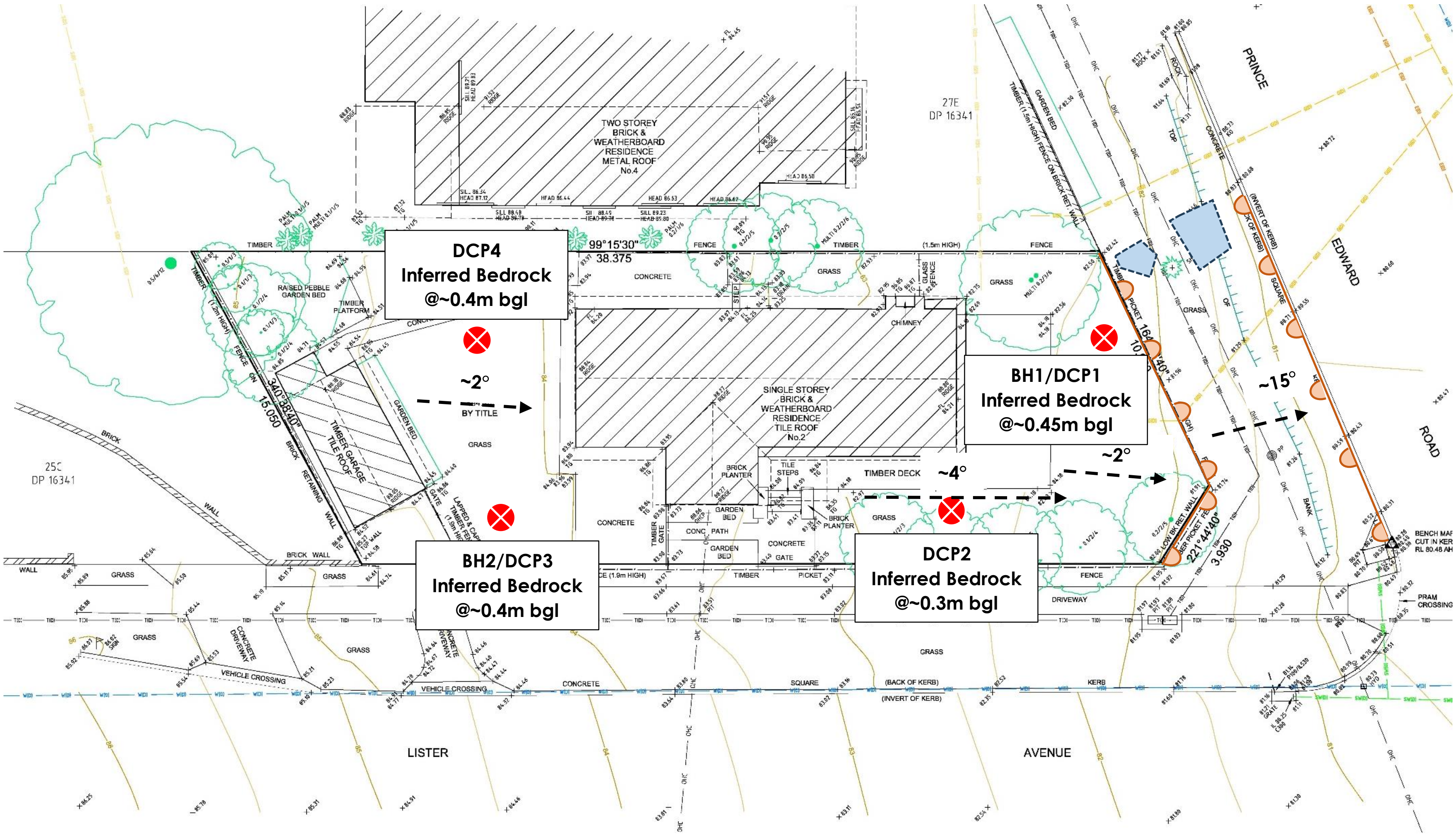
Understanding Limitation Provisions
As some clients, contractors and design professionals do not recognise geotechnical engineering is much broader and less exact than other engineering disciplines, this creates unrealistic expectations that lead to claims, disputs and other disappointments. As part 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

Legend:

Approximate Borehole/DCP Testing Location
Approximate Location of Sandstone Rock Outcrops
7° Approximate Slope Direction and Angle
Slope Breaks (rounded)



GCA Geotechnical Consultants Australia	Figure 1 Site Plan	**Geotechnical Investigation**	Drawn: AN	
		Naycon Building Solutions Pty Ltd	Date: 11/02/2025	
	Job No.: G2535-1	2 Prince Edward Road Seaforth NSW 2092	Scale: NTS	

Image source: Site survey plan prepared by Usher & Company Surveying & Land Development Consultants, referenced No. 6533-DET, Issue: 1 and dated 24th February 2025.

APPENDIX C

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:

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

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

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.

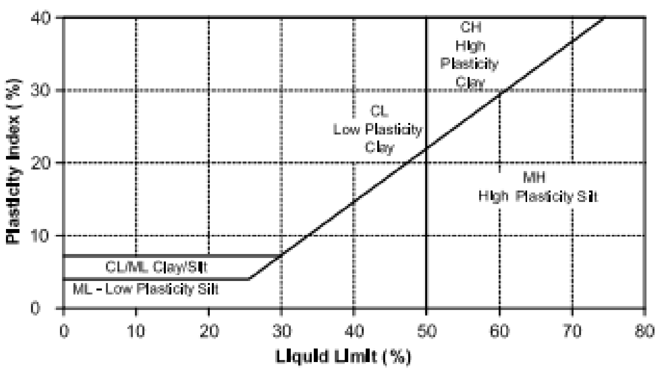
Method and Terms for Soil and Rock Descriptions Used on Borehole and Test Pit Reports

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	Highly Weathered	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.
DW	*Distinctly Weathered (as per AS 1726)*	
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 Naycon Building Solutions Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G2535-1
PROJECT LOCATION 2 Prince Edward Road Seaforth NSW 2092

DATE STARTED 11/2/25 **COMPLETED** 11/2/25 **R.L. SURFACE** 82.3 **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** AN **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 Clay, low plasticity, dark brown to brown, moist (w>PL), grass rootlets		FILL
		82.0			CLS-CIS	Sandy CLAY: low to medium plasticity, pale brown to brown, orange, fine to medium grained sand, wet (w~LL), estimated soft, slopewash (?).		NATURAL SOILS
			0.5			Borehole BH1 terminated at 0.45m		Practical hand auger refusal on interpreted bedrock at 0.45m bgl.
		81.5						
			1.0					
		81.0						
			1.5					
		80.5						
			2.0					

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BOREHOLE NUMBER BH2

CLIENT Naycon Building Solutions Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G2535-1
PROJECT LOCATION 2 Prince Edward Road Seaforth NSW 2092

DATE STARTED 11/2/25 **COMPLETED** 11/2/25
R.L. SURFACE 84.4 **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 AN **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 Clay, low plasticity, dark brown, moist (w>PL), rootlets, grass covering		FILL
		84.0			CIS	Sandy CLAY: medium plasticity, orange and pale brown, fine to coarse grained sand, moist (w>PL), estimated firm, slopewash (?).		NATURAL SOILS
			0.5			Borehole BH2 terminated at 0.4m		Practical hand auger refusal on interpreted bedrock at 0.4m bgl.
		83.5						
			1.0					
		83.0						
			1.5					
		82.5						
			2.0					

APPENDIX E

DYNAMIC CONE PENETROMETER RESULTS

Client:	Naycon Building Solutions Pty Ltd	**Test Date:**	11/02/2025
Address:	2 Prince Edward Road Seaforth NSW 2092	**Job No.:**	G2535-1

	DCP No.					**DCP No.**			
Surface RL	***82.3***	***82.8***	***84.4***	***84.4***	***Surface RL***				
Depths (mm bgl)	**1**	**2**	**3**	**4**	**Depths (mm bgl)**				
0-100	1	1	1	0	0-100				
100-200	1	2	2	2	100-200				
200-300	2	10/80mm	4	5	200-300				
300-400	1	**Bouncing**	10/90mm	7/90mm	300-400				
400-500	10/50mm		**Bouncing**	**Bouncing**	400-500				
500-600	**Bouncing**				500-600				
600-700					600-700				
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3600-3700					3600-3700				
3700-3800					3700-3800				
3800-3900					3800-3900				
3900-4000					3900-4000				

Notes:
- DCP1 at BH1 location.
- DCP3 at BH2 location.

Surface RL (m AHD) at top of the DCP test is approximate.

Tested: AN **Sheet:** 1 of 1

APPENDIX F

Foundation Maintenance and Footing Performance: A Homeowner's Guide

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).

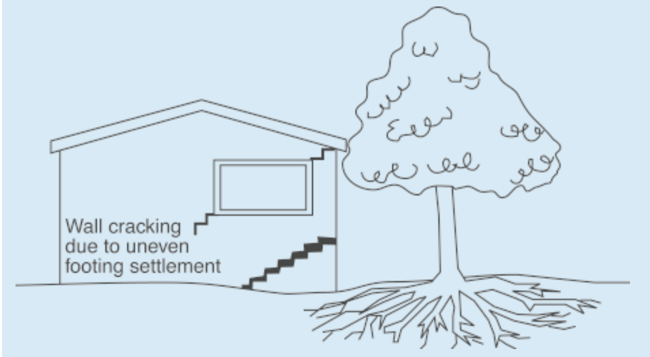
Isolated piers affected by erosion or saturation of foundations will eventually lose contact with the bearers they support and may tilt or fall over. The floors that have lost this support will become bouncy, sometimes rattling ornaments etc.

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.

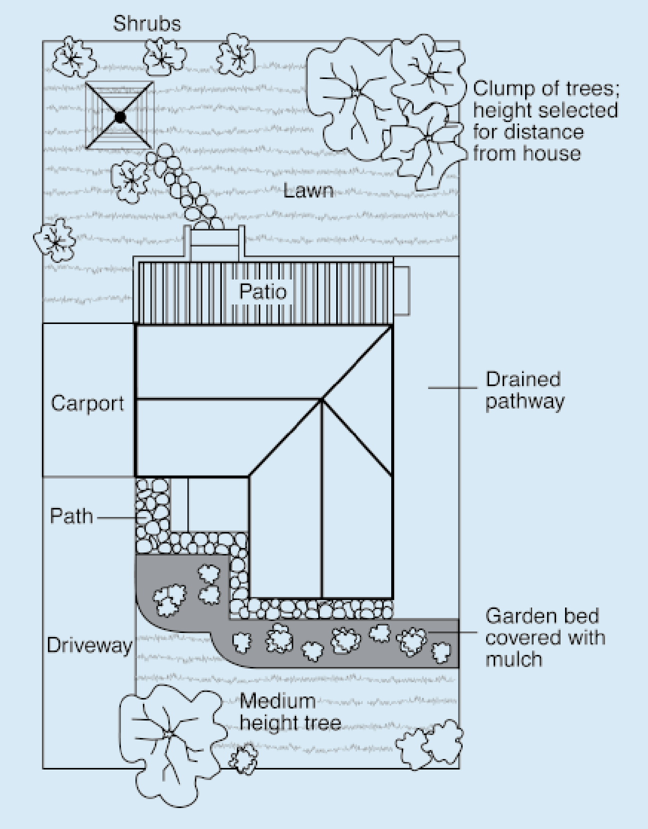
Protection of the building perimeter
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

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

la	LAMBERT	Erosional

Landscape—undulating to rolling rises and low hills on Hawkesbury Sandstone. Local relief 20–120 m, slopes 20%. Rock outcrop >50%. Broad ridges, gently to moderately inclined slopes, wide rock benches with low broken scarps, small hanging valleys and areas of poor drainage. Open and closed-heathland, scrub and occasional low eucalypt open-woodland.

Soils—shallow (<50 cm) discontinuous Earthy Sands (Uc5.11, Uc5.22) and Yellow Earths (Gn2.2) on crests and insides of benches; shallow (<20 cm) Siliceous Sands/Lithosols (Uc1.2) on leading edges; shallow to moderately deep (<150 cm) Leached Sands (Uc2.21), Grey Earths (Gn2.81) and Gleyed Podzolic Soils (Dg4.21) in poorly drained areas; localised Yellow Podzolic Soils (Dy4.1, Dy5.2) associated with shale lenses.

Limitations—very high soil erosion hazard, rock outcrop, seasonally perched watertables, shallow, highly permeable soil, very low soil fertility.

LOCATION

Exposed plateau surfaces, convex ridges and coastal headlands of the Hornsby Plateau. Typical areas include much of Brisbane Water National Park and the Lambert Peninsula in Ku-ring-gai Chase National Park. Smaller occurrences are found at Terrey Hills and in the Manly Warringah area, Dover Heights and La Perouse.

LANDSCAPE

Geology

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

Topography

Undulating to rolling low hills. Local relief 20–120 m and slopes <20%. Broad convex crests and plateau surfaces. Gently to moderately inclined sideslopes, often associated with small hanging valleys. Characteristic sandstone bedrock that outcrops as wide benches (10–100 m), with broken scarps 1–4 m high. Small, poorly drained seepage areas are common.

Vegetation

Predominantly uncleared open-heathlands, closed-heathlands and scrublands, with patches of low eucalypt woodland. The heathlands and scrublands are often exposed to strong winds. Their shallow, poorly drained soils fluctuate between being saturated or dry. Bushfires are frequent. Isolated lines and patches of trees are occasionally associated with joint crevices.

Shrub she-oak *Allocasuarina distyla* and/or heath banksia *Banksia ericifolia* are usually dominant. Other shrubs such as spiky hakea *Hakea teretifolia* may be locally dominant in areas subject to seepage or prolonged saturation. Associated shrubs include various spider flowers *Grevillea* spp., billy buttons *Kunzea* spp., eggs and bacon *Pultenaea* spp., teatree *Leptospermum* spp. and native heath *Epacris* spp.

Isolated occurrences of low eucalypt open-woodland with dry sclerophyll shrub understorey are found at sites with deeper soils and unimpeded soil drainage. Trees often have a mallee habit. Red bloodwood *Eucalyptus gummifera*, yellow-top ash *E. luehmanniana*, yellow bloodwood *E. eximia*, scribbly gum *E. haemastoma* and narrow-leaved apple *Angophora bakeri* are common mallee species.

Growth of introduced species in urban areas is stunted. Native trees rarely attain a height of 10 m.

Land use

Most of this unit is bushland managed by the National Parks and Wildlife Service. This includes Brisbane Water National Park, Ku-ring-gai Chase National Park, and Muogamarra Nature Reserve. National Parks and isolated vacant and crown land are used for recreational activities such as bushwalking. Urban residential areas include Dover Heights, Balgowlah Heights and Cromer.

Existing Erosion.

Severe sheet erosion can occur when bushfires destroy or damage vegetative ground cover. This is particularly so if the fires are followed by heavy rains (Atkinson, 1984). Poorly planned and maintained roads, fire trails, walking tracks and bridle trails are subject to severe erosion. Many gullies and rills on tracks and roads are eroded, exposing bedrock. Erosion can be severe and widespread in areas frequented by four-wheel drive vehicles, horses and trail bikes.

Associated Soil Landscapes

Hawkesbury (**ha**) soil landscape occurs in areas of steeper slopes. Small areas of North Head (**nh**) soil landscape and Newport (**np**) soil landscape are also included.

SOILS

Dominant Soil Materials

la1—Loose, stony, yellowish-brown sandy loam. This is stony brown loamy sand to sandy loam with apedal single-grained structure and porous sandy fabric. It generally occurs as topsoil (A1 horizon).

Colour, which can vary from olive brown (2.5Y 4/4) to dark brown (10YR 3/4) is commonly a yellowish-brown (10YR 5/4, 10YR 5/6, 10YR 5/8). The pH ranges from strongly acid (pH 4.0) to moderately acid (pH 5.5). Subrounded sandstone fragments and quartz pebbles are common and are occasionally concentrated as a stone line at depth. Charcoal fragments and roots are common.

la2—Earthy, yellow-brown, light sandy clay loam. This is commonly a yellow-brown, light sandy clay loam with apedal massive to weakly pedal structure and porous earthy fabric. This material occurs as subsoil (B horizon) or occasionally as an A2 horizon.

Texture can range from clayey sand to sandy clay loam. Texture often increases gradually with depth. Peds when present, are usually rough-faced and sub-angular blocky. They range in size from 10 mm to 50 mm. Porosity often decreases with depth. Colour ranges from yellowish-brown (10YR 5/6, 6/6) to brownish-yellow (10YR 6/8). The pH ranges from strongly acid (pH 4.0) to moderately acid (pH 5.5). Sandstone and ironstone fragments are common, but charcoal fragments and roots are rare.

la3—Angular blocky puggy clay. This is a fine sandy clay loam to medium clay with strongly developed angular blocky to occasionally prismatic structure when dry and apedal massive structure when wet. This material occurs as deep subsoil (B horizon) on shale lenses.

Peds are predominantly rough-faced (10-50 mm) and porous with isolated clusters of smooth faces and dense peds. Secondary sub-angular and polyhedral peds are common. When moist, this material is moderately sticky, and is apedal massive and plastic. It is equivalent to Buchanan's (1980) puggy clay. Colour in well-drained positions is commonly a yellowish-brown (10YR 6/6–6/8). In areas subject to prolonged saturation or seepage, colour varies from light yellow orange (10YR 8/4) to pale grey (10YR 8/2). Red, orange and grey mottles are common.

The pH ranges from extremely acid (pH 3.5) to moderately acid (pH 5.5). Platy, iron coated ironstone fragments are common. Roots and charcoal fragments are usually absent.

la4—Blackish-brown, loose sandy loam. This is a dark loamy sand to sandy loam with apedal single-grained structure and porous sandy fabric. It usually occurs as topsoil (A1 horizon).

This material is often water repellent. Colour usually ranges from greyish yellow brown (10YR 4/2) to brownish-black (10YR 3/2). The pH ranges between strongly acid (pH 4.0) and slightly acid (pH 6.0).

Sandstone and ironstone fragments, charcoal fragments, roots and decaying plant remains are common.

la5—Earthy, mottled, pale clayey sands. This is pale coloured clayey sand with apedal massive structure and porous earthy fabric. It generally occurs as subsoil in wet areas (B or C horizon).

Texture can vary from loamy sand to sandy clay loam, with clayey sands and sandy loams being the most common. Surface condition is loose and fabric is sandy. This material is characterised by pallid/grey soil colours such as light yellow (2.5Y 7/4) and bright yellowish-brown (2.5Y 7/6). In wet situations there are often rusty piped mottles around root traces. The pH ranges from extremely acid (pH 3.5) to moderately acid (pH 5.5). Sandstone fragments, charcoal fragments and roots are usually absent.

la6—Friable sandstone. This is soft, friable, deeply weathered, sandstone with a coarse sugary appearance. It commonly occurs as deeply weathered parent material (C horizon) in joint lines and beneath perched watertables.

Texture is commonly clayey sand which often becomes sandier with depth. Structure is usually apedal and massive and the fabric is sandy or occasionally earthy. Colour can vary from light grey (10YR 8/1) to dull yellow-orange (10YR 7/2-7/4). Pale yellow and orange mottles may be present. Rusty mottles occasionally occur which follow root traces. This material can be crushed by hand

and the disrupted material has a feel and appearance similar to sugar crystals. The pH ranges from extremely acid (pH 3.5) to moderately acid (pH 5.0). Occasional bands of dark red (2.5YR 3/6) mottles associated with platy, angular, ironstone fragments occur. These ironstone fragments often occur in undisturbed and stratified bands. Strongly weathered fragments of sandstone are found at depth. Roots are rare and charcoal fragments are absent.

Associated Soil Materials

Litter and decomposing organic debris. This material consists of easily recognisable remnants of leaves, flowers, bark and twigs. Distribution is variable and depends on exposure, fire regime, location of nearby species and surface wetness. Fungal and root mats are common. There is a sharp even boundary with the mineral soil.

White loose sand. This material is composed almost entirely of quartz sand grains and is found in recently deposited surface washes such as small debris dams and fans located on breaks of slope.

Dark peaty sand. In poorly drained areas heavy accumulations of organic matter are associated with shallow, dark, peaty sands.

Occurrence and Relationships

Crests and plateaux. Generally 20–100 cm of earthy, yellow-brown, light sandy clay loam (**la2**) occurs as both topsoil and subsoil, with texture characteristically increasing gradually with depth (Earthy Sands (Uc5.11, Uc5.22), Yellow Earths (Gn2.21)). This material may merge with friable sandstone (**la6**), or with sandstone bedrock. Total soil depth is <100 cm.

Occasionally up to 30 cm of loose, stony, yellow-brown sandy loam (**la1**) overlies 10–40 cm of **la2**. Total soil depth is <100 cm. The boundary between the soil materials may be gradual (Yellow Earths (Gn2.2)) or clear (Yellow Podzolic Soils (Dy2.61, Dy4.51)). A stone line is often present.

Plateau surfaces and larger benches are often characterised by areas of exposed bedrock with shallow (<30 cm), discontinuous pockets or islands of up to 10 cm of brownish-black sandy loam (**la4**) which overlies up to 10 cm of **la1**. Total soil depth is usually <60 cm. The boundary between the soil materials is gradational (Siliceous Sands/Earthy Sands/Lithosols (Uc1.21, Uc5.11)).

Sideslopes. The soils on sideslopes are discontinuous, with up to 50% of the surface covered by sandstone rock outcrop. On the benches, a variety of shallow soils occur (<50 cm). Soils in crevices such as joint lines may be >100 cm deep.

Outside of benches. The leading edges of most benches, adjacent to rock outcrops, have up to 20 cm of **la1** and/or **la4** overlying bedrock (Siliceous Sands/Lithosols (Uc1.2)). In other locations, up to 20 cm of **la4** overlies up to 20 cm of **la1** and up to 50 cm of **la2**. Total soil depth is <60 cm. Boundaries between soil materials are gradational (Yellow Earths, Earthy Sands (Gn2.24)).

Inside of benches. Up to 20 cm of **la1** or **la4** overlies up to 50 cm of **la2**. Total soil depth is usually <100 cm and the boundary between the soil materials is gradual (Earthy Sands (Uc5.2), Yellow Earths (Gn2.2)). Where occasional shale lenses have influenced soil formation, up to 20 cm of **la4** and/or **la1** overlie up to 50 cm of white puggy clay (**la3**) (Yellow Podzolic Soils (Dy4.11, Dy5.21, Dy5.51)). Total soil depth is <60 cm. Boundaries between the soil materials are clear to sharp.

Wet areas. Up to 20 cm of la4 overlies up to 50 cm of earthy, mottled, pale clayey sands (**la5**). **la3** may substitute for **la5** or occur below **la5**. Total soil depth rarely exceeds 100 cm. The boundary between the soil materials is gradual (Leached Sands (Uc2.21), Grey Earths (Gn2.81)) to sharp (Gleyed Podzolic Soils (Dg4.21)).

Drainage depressions and hanging valleys. Close to drainage depressions up to 20 cm of **la4** overlies up to 60 cm of **la5** and occasionally up to 30 cm of **la6**. Total soil depth is <100 cm. Boundaries between soil materials are gradual (Leached Sands (Uc2.21), Grey Earths (Gn2.81)). In other areas litter, decomposing organic debris and white loose sand commonly overlie up to 60 cm of **la1** (Siliceous Sands (Uc1.2)). Secondary depositional yellow earth material (**la2**) is often found adjacent to drainage lines (Paton, 1978).

Hanging valleys. The deep subsoil of the hanging valleys usually consists of **la6**, especially in waterlogged and swampy areas.

LIMITATIONS TO DEVELOPMENT

Urban Capability

Low to moderate capability for urban development.

Rural Capability

Land not capable of being cultivated or grazed.

Landscape Limitations

Seasonal waterlogging
Rock outcrop
Shallow depth
Erosion hazard
Perched watertables (localised)

Soil Limitations

la1 High permeability
Low available water capacity
Stoniness
Low fertility

la2 High permeability
Low available water capacity
Stoniness
Low fertility
Strongly acid
Very high aluminium toxicity

la3 Low wet strength
Low permeability
Stoniness (localised)
Very low fertility
Very strongly acid
High aluminium toxicity

la4 Stoniness (localised)
High organic matter (localised)
Low fertility
Very strongly acid
High aluminium toxicity

la5 Low available water capacity
Very low fertility

Strongly acid
High aluminium toxicity

la6 Low available water capacity
Low permeability (localised)
Stoniness (localised)
Very low fertility
Strongly acid
Very high aluminium toxicity

Fertility

The soils of this unit are shallow, stony, moderately acid, have low available water capacity, very low to low CEC and often are severely deficient in nitrogen and phosphorus. In many areas these soils are poorly drained. The subsoil has very high aluminium toxicity.

Erodibility

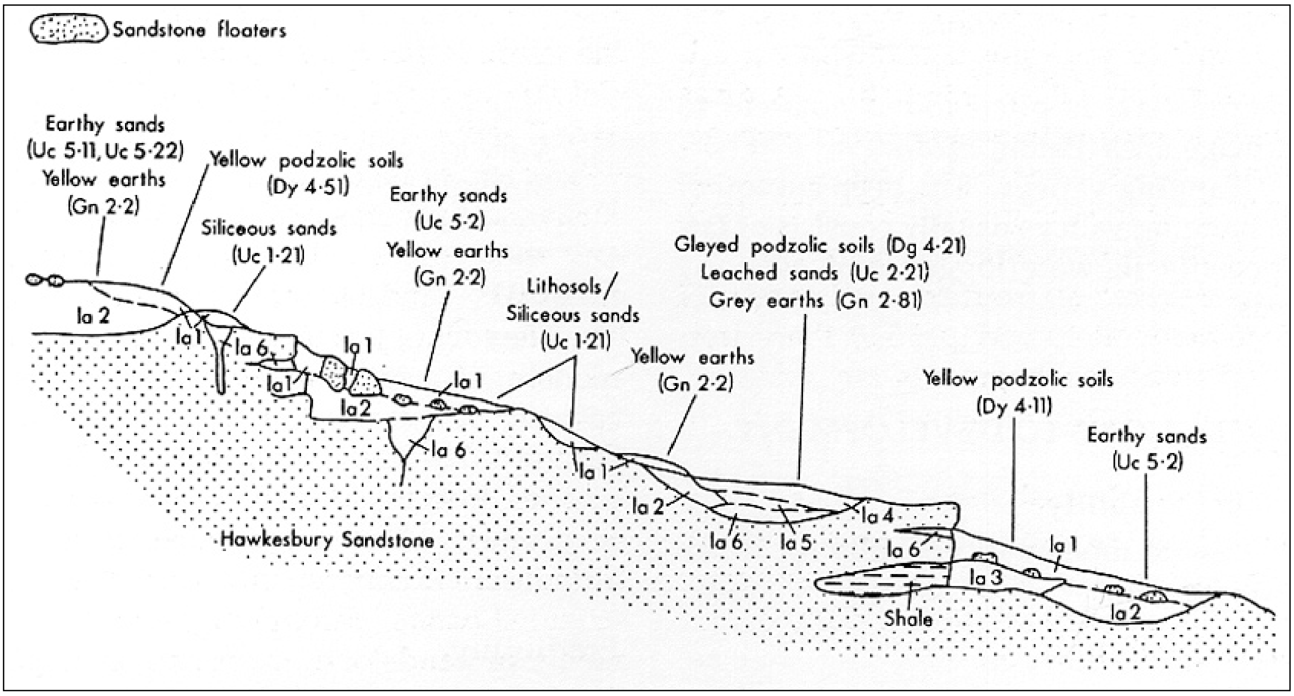
Soil materials **la1–la4** are moderately erodible. They consist of either well-drained coarse sand with moderate (**la2**) to high (**la1**, **la4**) amounts of organic matter or weakly cemented earths and clays (**la3**). Most aggregates are stable or prone only to slaking. The clays in **la3** are occasionally dispersible and this material is then considered to be highly erodible. However, **la5** and **la6** have low erodibility as they are firmly cemented by clays and/or iron oxides.

Erosion Hazard

The soil erosion hazard for non-concentrated flows is usually very high, but ranges from low to extreme. Calculated soil losses for the first twelve months of urban development range up to 17 t/ha for topsoils and 197 t/ha for exposed subsoils. The soil erosion hazard from channelled flow is extreme.

Surface Movement Potential

The sandy shallow soils are stable to slightly reactive. Only in isolated instances where **la3** is >100 cm thick would the reactivity be moderate.



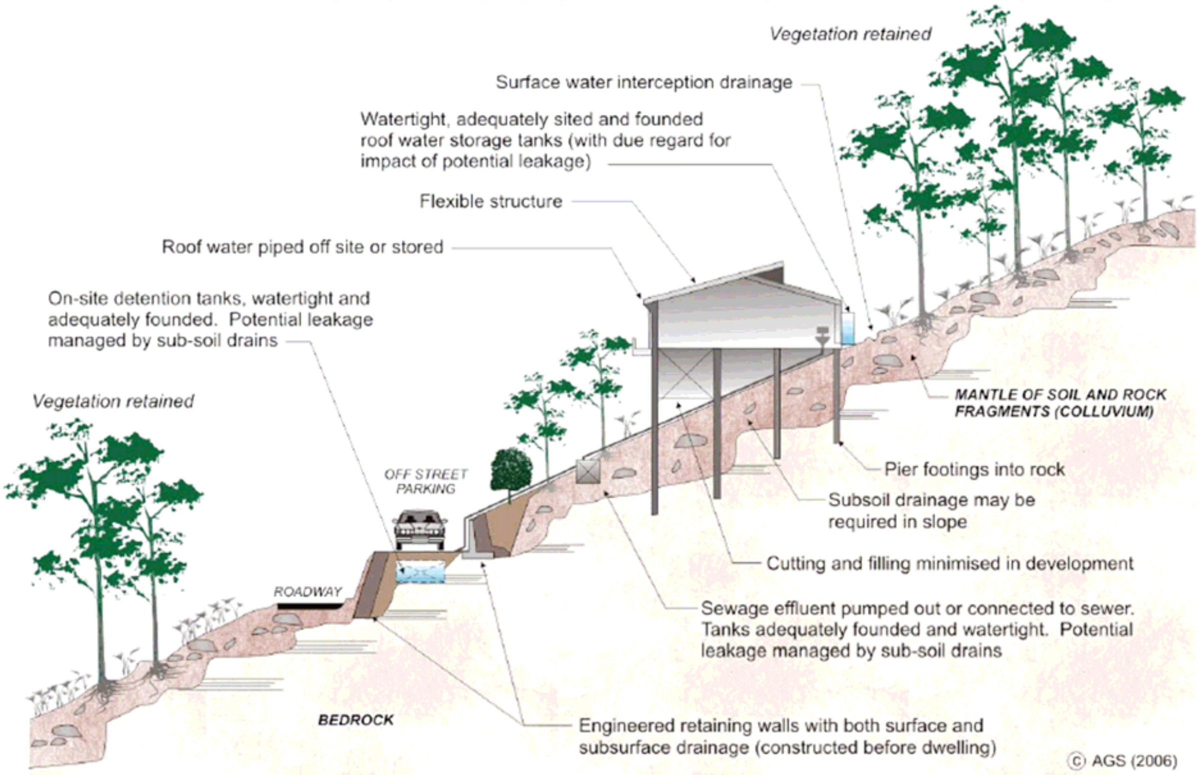
Schematic cross-section of Lambert soil landscape illustrating the occurrence and relationship of the dominant soil materials.

APPENDIX H

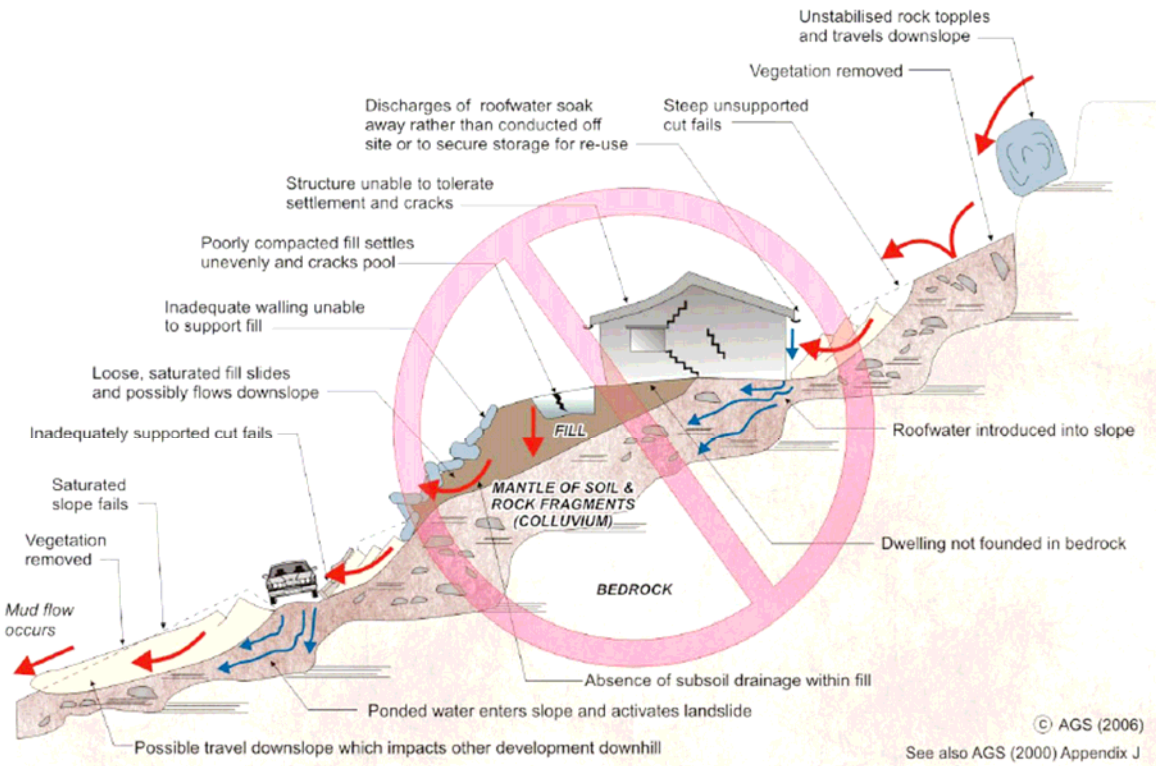
APPENDIX G - SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

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

EXAMPLES OF GOOD HILLSIDE PRACTICE



EXAMPLES OF POOR HILLSIDE PRACTICE



GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1 – To be submitted with Development Application

Development Application for ____________________

Name of Applicant

Address of site 2 Prince Edward Road Seaforth NSW 2092

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

I, Joe Nader *on behalf of* Geotechnical Consultants Australia Pty Ltd

(Insert Name) (Trading or Company Name)

on this the 14th February 2025 certify that I am a geotechnical engineer or engineering geologist or coastal engineer as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above organisation/company to issue this document and to certify that the organisation/company has a current professional indemnity policy of at least $5million.

I:

Please mark appropriate box

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

Geotechnical Report Details:

Report Title: Geotechnical Engineering Investigation Report, Proposed Development at: 2 Prince Edward Road Seaforth NSW 2092

Report Date: Report No. G2535-1, and dated 14th February 2025

Author: Joe Nader

Author's Company/Organisation: Geotechnical Consultants Australia Pty Ltd

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

Provided information outlined in Section 1.3 of the Geotechnical Investigation Report
All recommendations within the report are to be complied with for the design and construction of the proposed development

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

Signature ……JNader……

Name Joe Nader……

Chartered Professional Status MIEAust., CPEng, NER, RPEQ

Membership No. 3943418, 3224, 25570

Company Geotechnical Consultants Australia Pty Ltd

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER
FORM NO. 1(a) - Checklist of Requirements For Geotechnical Risk Management Report for Development Application

Development Application for ____________________

Name of Applicant

Address of site 2 Prince Edward Road Seaforth NSW 2092

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

Geotechnical Report Details:

Report Title: Geotechnical Engineering Investigation Report, Proposed Development at: 2 Prince Edward Road Seaforth NSW 2092

Report Date: Report No. G2535-1, and dated 14th February 2025

Author: Joe Nader

Author's Company/Organisation: Geotechnical Consultants Australia Pty Ltd

Please mark appropriate box

- ☒ Comprehensive site mapping conducted 11th February 2025 (date)
- ☐ Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:200 (as appropriate)
- ☐ Subsurface investigation required
  - ☐ No Justification ……………………………………………...
  - ☒ Yes Date conducted 11th February 2025
- ☐ Geotechnical model developed and reported as an inferred subsurface type-section
- ☒ Geotechnical hazards identified
  - ☒ Above the site
  - ☒ On the site
  - ☒ Below the site
  - ☒ Beside the site
- ☒ Geotechnical hazards described and reported
- ☒ Risk assessment conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
  - ☐ Consequence analysis
  - ☐ Frequency analysis
- ☒ Risk calculation
- ☒ Risk assessment for property conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ☒ Risk assessment for loss of life conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- ☒ Assessed risks have been compared to "Acceptable Risk Management" criteria as defined in the Geotechnical Risk Management Policy for Pittwater - 2009
- ☒ Opinion has been provided that the design can achieve the "Acceptable Risk Management" criteria provided that the specified conditions are achieved.
- ☐ Design Life Adopted:
  - ☐ 100 years
  - ☐ Other ……………………………………………… specify
- ☐ Geotechnical Conditions to be applied to all four phases as described in the Geotechnical Risk Management Policy for Pittwater - 2009 have been specified
- ☐ Additional action to remove risk where reasonable and practical have been identified and included in the report.
- ☐ Risk assessment within Bushfire Asset Protection Zone.

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

Signature JNader

Name Joe Nader

Chartered Professional Status MIEAust., CPEng, NER, RPEQ

Membership No. 3943418, 3224, 25570

Company Geotechnical Consultants Australia Pty Ltd