Geotechnical Consultants Australia

Krishathi Pty Ltd

Geotechnical Investigation Report

Proposed Development at:
225 & 227 Bungarribee Road
Blacktown NSW 2148

G21674-1

28th October 2021

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Report Distribution

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Address: 225 & 227 Bungarribee Road Blacktown NSW 2148

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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 proposed development at nos. 225 & 227 Bungarribee Road Blacktown 2148 (the site). The investigation was commissioned by Krishathi Pty Ltd (the client) and was carried out on the 15th October 2021.

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

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

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 existing infrastructures onsite, followed by construction of a boarding house building, overlying a single basement level within each property.

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

- No. 225 Bungarribee Road: RL65.72m to RL65.78m Australian Height Datum (AHD).
- No. 227 Bungarribee Road: RL66.62m AHD.

Based on this information and the existing site levels and topography, maximum excavation depths varying from approximately 1.0m to 4.5m (varying throughout) are expected to be required for construction of the proposed developments, with cut and fill in certain areas. Locally deeper excavations for the lift shafts, and building footings and service trenches are also anticipated to be required as part of the planned development.

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

1.3 Provided Information

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

- Architectural drawings prepared by Gus Fares Architects Pty Ltd, titled "Proposed 12 Rooms Boarding House at 225 Bungarribee Rd Blacktown NSW", referenced project No. 2020-19 and included drawing nos. A000, A001, A003, A004, A101 to A103 inclusive, A201 and A202.
- Architectural drawings prepared by Gus Fares Architects Pty Ltd, titled "Proposed 12 Rooms Boarding House at 227 Bungarribee Rd Blacktown NSW", referenced project No. 2020-19 and included drawing nos. A000, A001, A003, A004, A101 to A103 inclusive, A201 and A202.
- Site survey plan prepared by AB Dimensions Pty Ltd, titled "Detail Survey of Land at 225 Bungarribee Road, Blacktown NSW 2148", referenced drawing No. A1-20204-1-A, sheet 1 of 1 and dated 2nd December 2020.
- Site survey plan prepared by AB Dimensions Pty Ltd, titled "Detail Survey of Land at 227 Bungarribee Road, Blacktown NSW 2148", referenced drawing No. A1-20204-1-A, sheet 1 of 1 and dated 2nd December 2020.

1.4 Geotechnical Assessment Objectives

The objective of the geotechnical investigation was to assess the site surface and subsurface conditions at the selected borehole testing locations within the site (where accessible and feasible), and to provide professional geotechnical advice and recommendations on the following based on 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 rocks to restrict any ground vibrations.
- Recommendations on suitable shoring (retention) systems for the site.
- Design parameters based on the ground conditions within the site, for retaining walls, cantilever shoring walls and propped shoring.
- Recommendations on suitable foundation types and design for the site.
- End bearing capacities and shaft adhesion for shallow and deep foundations based on the ground conditions within the site (for ultimate limit state and serviceability loads).
- Groundwater levels which may be determined during the geotechnical investigation.
- Recommendations on groundwater maintenance and limiting.
- Preliminary site lot classification in accordance with Australian Standards (AS) 2870-2011.
- Preliminary subsoil class for earthquake design for the site in accordance with AS 1170.4-2007.
- Preliminary aggressivity and salinity assessment within the site based on laboratory testing results at the selected borehole locations.
- General geotechnical advice on site preparation, filling and subgrade preparation.

1.5 Scope of Works

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

- Submit and review Dial Before You Dig (DBYD) plans and any other plans provided by the client on existing buried services within the site.
- Service locating carried out using electromagnetic detection equipment to ensure the area is free of any underground services at the selected borehole testing locations.
- Review of site plans and drawings to determine appropriate testing locations (where accessible and feasible), and identify any relevant features of the site.
- Machine drilling of four (4) boreholes at selected locations within the site (where accessible and feasible) by a specialised trailer mounted drilling rig, using solid flight augers equipped with a 'Tungsten Carbide' (TC) bit, and identified as boreholes BH1 to BH4 inclusive. The drilling rig is owned and operated by a specialist subcontractor.
    - The boreholes were drilled to varying practical TC bit refusal depths of approximately 3.2m to 6.0m below the existing ground level within the site (bgl).
        - The approximate locations of the boreholes are shown on **Figure 1, Appendix B** of this report.
- Collection of soil and rock samples during drilling for the following laboratory testing required:
    - Laboratory testing by a National Association of Testing Authorities, Australia (NATA) accredited laboratory (ALS Environmental) on four (4) selected samples collected during drilling of the boreholes to determine the pH, chloride and sulphate content, and electrical conductivity of the selected samples. Laboratory analysis was undertaken for the purpose of a preliminary aggressivity and salinity assessment within the site.
- Reinstatement of boreholes BH1 to BH4 inclusive with available soil displaced during drilling.
- Preparation of this geotechnical engineering report.

1.6 Constraints

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

Consideration should be given to additional machine drilled boreholes and rock strength testing following demolition of existing onsite infrastructures within the site, in order to confirm the ground conditions and estimated rock strength underlying the site, and to help assist in final designs of the proposed development. This recommendation should be confirmed by the project geotechnical engineer and structural engineer during/following design stages of the proposed development.

2. SITE DESCRIPTION

2.1 Overall Site Description

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

Table 1. Overall Site Description and Site Surroundings

Information		Details
Overall Site Location		The site is located within a residential area along Bungarribee Road thoroughfare.
Site Address		225 & 227 Bungarribee Road Blacktown NSW 2148
Approximate Site Area[1]	**No. 225 Bungarribee Road**	834.7m^2
	No. 227 Bungarribee Road	834.7m^2
Local Government Authority		Blacktown City Council
Site Description		At the time of the investigation, a residential dwelling was present within each property, accompanied by associated concrete pavements and detached sheds. The remaining site area was predominately covered in grass, vegetation and a number of mature trees scattered throughout.
Approximate Distances to Nearest Watercourses (i.e. rivers, lakes, creeks, etc.)		• Bungarribee Creek – 1.2km south-east of the site.
Site Surroundings		The site is located within an area of residential use and is bounded by: • Residential properties at No. 51, No. 53 and No. 55 Paul Street to the north. • Residential property at No. 223 Bungarribee Road to the east. • Bungarribee Road thoroughfare to the south. • Residential property at No. 229 Bungarribee Road to the west.

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

2.2 Topography

The local and site topography generally falls towards the north to north-east. Levels within both sites vary from approximately RL65.8m to RL72m AHD.

It should be noted that the site topography, levels and slopes are approximate and based off the site survey plans referenced in Section 1.3, observations made during the geotechnical investigation and reference to NSW Six Maps (https://maps.six.nsw.gov.au). The actual topography in areas inaccessible during the site investigation, including areas under the existing infrastructures, along with the site and local topography and levels are expected to vary from those outlined in this report.

2.3 Regional Geology

Information obtained on the local regional subsurface conditions, referenced from the Department of Mineral Resources, Penrith 1:100,000 Geological Series Sheet 9030 Edition 1, dated 1991, by the Geological Survey of New South Wales, indicates the site is located within a geological region generally underlain by Bringelly Shale (Rwb) of the Wianamatta Group. The Bringelly Shale (Rwb) typically comprises "shale, carbonaceous claystone, claystone, laminite, fine to medium grained lithic sandstone, rare coal and tuff".

Furthermore, reference made to MinView by the State of New South Wales through Regional NSW 2021 indicates the site is positioned mainly within a geological region underlain by Shale (Twib).

A review of the regional maps by the NSW Government Environment and Heritage shows the site is generally located within the Blacktown (bt) landscape group. The Blacktown (bt) landscape group which is normally recognised by gently undulating rises on Wianamatta Group shales. Soils of the Blacktown group typically have localised seasonal waterlogging and water erosion hazards, moderately reactive highly plastic subsoils and localised surface movement potential. Local reliefs are generally less than 30m with slopes typically greater than 5% in gradient. Soils of the Blacktown group are generally neutral (pH 7.0) to strongly (pH 4.0) acidic.

The Blacktown (bt) 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 from across the site during this geotechnical investigation are summarised in Table 2 below and are interpreted from the assessment results. It should be noted that Table 2 presents a summary of the overall site conditions and reference should be made to the detailed engineering borehole logs presented in **Appendix D**, in conjunction with the geotechnical explanatory notes detailed in **Appendix C**. Rock description has been based on Pells P.J.N, Mostyn G. & Walker B.F. Foundations on Sandstone and Shale in the Sydney Region, Australian Geomechanics Journal, December 1998.

It should be noted that estimated rock strengths assessed by observation during auger drilling penetration resistance are approximate and variances should be expected throughout the site. It is worth noting that auger penetration within various bedrock formations vary from each drilling rig and estimated rock strength variances across the site are expected.

Due to the variable ground conditions throughout the site, it is recommended that confirmation of the subsurface materials be carried out during construction, or by additional borehole drilling and rock strength testing. It should also be noted that ground conditions within the site are expected to differ from those encountered and inferred in this report, since no geotechnical or geological exploration program, no matter how comprehensive, can reveal and identify all subsurface conditions underlying the site.

Based on the geotechnical investigation at the selected borehole locations, along with our experience and observations made within the site and local region, it is inferred that bedrock of variable composition, strength and weathering is underlying majority of the site area at varying depths of approximately 1.3m to 2.0m bgl (expected to vary throughout).

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

Table 2. Summary of Subsurface Conditions

			Borehole ID	BH1	BH2	BH3	BH4
Unit	**Unit Type**	**Description**	**Estimated Consistency/ Strength**	**Depth/Thickness of Unit (m bgl)**			
1	Fill	Clayey SILT, medium to high plasticity, gravel inclusions.	N/A	0.0 – 0.4	0.0 – 0.3	0.0 – 0.5	0.0 – 0.4
2	Residual Soils	Silty CLAY, medium to high plasticity, gravel inclusions.	–	0.4 – 1.3	0.3 – 2.0	0.5 – 1.8	0.4 – 1.0
		Shaly CLAY, medium plasticity, interbedded shale.		–	–	–	1.0 – 1.9
3	Bedrock[1]	SHALE, clay seams, with silt, extremely to highly weathered, becoming	EL	1.3 – 2.0	2.0 – 3.5	1.8 – 4.0	1.9 – 2.8
			VL	2.0 – 3.8	3.5 – 6.0	4.0 – 6.0	2.8 – 3.2
			Inferred L (or better)[2]	3.8	6.0	6.0	3.2

[1]The composition, class, depth and estimated strength of the underlying bedrock material should be confirmed either prior to construction by further borehole drilling and rock strength testing, or during construction by inspection.
[2]Higher estimated strength and/or class bedrock (i.e. low estimated strength, or better) is anticipated to be present at the approximate depths indicated in Table 2. This is based on observations made during auger penetration resistance at the time of drilling.

Notes:

- N/A = Not Applicable, EL = Extremely Low estimated strength, VL = Very Low estimated strength, L = Low estimated strength.
- Clay seams, defects, and fractured and extremely weathered zones are expected to be present throughout the underlying bedrock, predominately at depths and locations unobserved during the geotechnical investigation.
- Estimated rock strengths are based on observations made during auger penetration resistance at the time of drilling.
- Bedrock estimated strength is expected to vary across the site, due to the limited investigation carried out.
- Ground conditions are expected to vary across the site and should be confirmed by a geotechnical engineer, predominately in areas unobserved during the geotechnical investigation.

3.2 Groundwater

No groundwater was encountered or observed during and shortly after drilling (<30 minutes) of the boreholes to a maximum depth of about 6.0m bgl in boreholes BH2 and BH3.

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

Thus, based on the above observations and data available at the time of reporting, 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 bedrock underlying the site.

It should also be noted that groundwater levels have the potential to elevate during daily or seasonal influences such as tidal fluctuations, heavy rainfall, damaged services, flooding, etc., and moisture content within soils may be influenced by events within the site and adjoining properties.

Groundwater monitoring should be carried out during construction to assess any groundwater inflows within the site as no provision was made for longer term groundwater monitoring. Where groundwater conditions vary from those outlined in this report, GCA should be contacted for further advice.

4. LABORATORY TEST RESULTS

4.1 Aggressivity and Salinity

Four (4) selected samples were sent to a NATA accredited testing laboratory, ALS Environmental, to determine the pH, chloride and sulphate content, and electrical conductivity of the samples. A summary of the laboratory tests results is provided in Table 3 below with laboratory certificates presented in **Appendix F** of this report.

Table 3. Summary of Laboratory Test Results (Aggressivity and Salinity)

Borehole ID		BH1	BH2	BH3	BH4
Approximate Depth (m bgl)		1.0	6.0	2.0	3.0
Strata Type		Residual Soils	Bedrock	Residual Soils	Bedrock
Aggressivity and Salinity	**pH**	5.2	8.0	4.9	5.3
	Moisture Content (%)	17.2	9.5	11.5	11.0
	Chloride (mg/kg)	460	490	830	380
	Sulphate SO_4 (mg/kg)	180	60	250	240
Electrical Conductivity (µS/cm)	**EC (µS/cm)**	384	378	620	405
	EC (dS/m)	0.384	0.378	0.62	0.405
	Multiplication Factor[1]	8	15	8	15
	Saturation Extract ECe (dS/m)	3.07	5.67	4.96	6.08

[1]Multipication factor obtained from NSW Government, Catchment Management Authority, "Calculating Electrical Conductivity and Salinity" and Department of Natural Resources (DNR) publication "Site Investigations for Urban Salinity" – 2002.

5. GEOTECHNICAL ASSESSMENT AND RECOMMENDATIONS

5.1 Dilapidation Survey

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

5.2 General Geotechnical Issues

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

- Preliminary aggressivity and salinity assessment.
- Preliminary site lot classification.
- Excavation conditions.
- Groundwater management.
- Stability of excavation and retention of adjoining properties and infrastructures.
- Preliminary site earthquake classification.
- Foundations.

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

5.3 Preliminary Aggressivity and Salinity Assessment

In accordance to AS 2159-2009 "Piling – Design and Installation" (as outlined in Table 4 below), the results of the laboratory tests and introduction of a multiplication factor for electrical conductivity indicates the following classification:

Table 4. Aggressivity and Salinity Reference Table

Reference	Element Type	High Perm. Soils	Low Perm. Soils	pH	Chloride (mg/kg)	Sulphate SO_4 (mg/kg)
AS 2159-2009	Concrete Elements	Mild	Non	>5.5	N/A	<5,000
		Moderately	Mild	4.5 – 5.5		5,000 – 10,000
		Severely	Moderately	4.0 – 4.5		10,000 – 20,000
		Very Severely	Severely	<4.0		>20,000
	Steel Elements	Non	Non	>5.0	<5,000	N/A
		Mild	Non	4.0 – 5.0	5,000 – 20,000	
		Moderately	Mild	3.0 – 4.0	20,000 – 50,000	
		Severely	Moderately	<3.0	>50,000	
Dry Salinity 1993	Electrical Conductivity Saturation Extract ECe (dS/m) value range, based on an introduction of a multiplication factor from DNR publication.			Non-Saline <2 Slightly Saline 2 – 4 Moderately Saline 4 – 8 Very Saline 8 – 16 Highly Saline >16		

- Underlying residual soils (from boreholes BH1 and BH3):
  - **Non** aggressive for buried steel structural elements in low permeability soils.
  - **Mildly** aggressive for buried steel structural elements in high permeability soils.
  - **Mildly** aggressive for buried concrete structural elements in low permeability soils.
  - **Moderately** aggressive for buried concrete structural elements in high permeability soils.
  - Electrical conductivity of saturated extract (ECe) ranging from approximately 3.07ds/m to 4.96ds/m, indicating generally "**moderately**" saline residual soils underlying the site.
- Underlying bedrock (from boreholes BH2 and BH4):
  - **Non** aggressive for buried steel structural elements in low and high permeability soils.
  - **Mildly** aggressive for buried concrete structural elements in low permeability soils.
  - **Moderately** aggressive for buried concrete structural elements in high permeability soils.
  - Electrical conductivity of saturated extract (ECe) ranging from approximately 5.67ds/m to 6.08ds/m, indicating generally "**moderately**" saline bedrock underlying the site.

It should be note that soil aggressivity and salinity may vary throughout the site and is based on testing at the selected borehole locations to the maximum depths indicated, in conjunction with multiplication factors for electrical conductivity, as described above. Ground conditions and soil aggressivity and salinity are expected to vary across the site as discussed in this report since no geotechnical or geological exploration program, no matter how comprehensive, can reveal and identify all subsurface conditions underlying the site.

Consideration should be given to additional borehole drilling and laboratory testing following demolition of existing infrastructures onsite, in order to confirm the findings presented above.

5.4 Preliminary Site Lot Classification

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

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

- The presence of existing infrastructures and trees within and adjoining the site, causing abnormal and changing moisture conditions.

Based on the boreholes carried out within the site, and proposed basement excavations which will result in the removal of majority of the fill material and natural clayey soils, AS 2870-2011 indicates the sites may be classified as **"Class M"** sites for design and construction of the proposed developments system, founded below any soft/loose soils, topsoil, slopewash, fill or other deleterious material, being <u>entirely</u> on bedrock underlying the proposed development area (subject to confirmation and providing bedrock is exposed entirely across the bulk excavation level).

Where fill and natural soils are present at depths of equal to or greater than 1.8m below the proposed developments FFLs, GCA should be contacted immediately for further advice. This should be confirmed/monitored prior to and during construction as the site lot classification may vary (i.e. "Class H1" site).

The above classification is solely based on assessment of the subsurface conditions at the selected borehole testing locations/depths within the site (bedrock anticipated to be exposed at bulk excavation level) and current architectural drawings, and confirmation should be carried out as outlined in this report.

Foundation design and construction should be carried out as outlined in Section 5.11 below, with reference made to AS 2870-2011. Geotechnical inspections and confirmation of the actual depth of underlying fill material, natural soils and bedrock should be made prior to construction by additional borehole drilling and rock strength testing, or during construction by inspection.

GCA should be contacted where ground conditions vary from those outlined in this report at the borehole testing locations. Where the building foundations are not proposed to be constructed on the bedrock underlying the site, GCA should also be contacted and the building foundations be designed and constructed as a "Class P" site.

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

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

Based on the preliminary site lot classification outlined above, it is recommended that reference is made to the recommendations provided by CSIRO "Guide to Home Owners on Foundation Maintenance and Footing Performance", attached as **Appendix E.**

5.5 Inspection Pits and Underpinning

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

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

5.6 Excavation

Maximum excavation depths varying from approximately 1.0m to 4.5m (varying throughout) are expected to be required for construction of the proposed developments, with cut and fill in certain areas. Locally deeper excavations for the lift shafts, and building footings and service trenches are also anticipated to be required as part of the planned development.

Based on this information and existing ground conditions as encountered during the geotechnical investigation, it is anticipated that excavation will extend through Unit 1 (fill) to Unit 3 (bedrock) inclusive, throughout the majority of the proposed development area, as discussed in Section 3 above.

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

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

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

Should rock hammering be used for the excavation in the underlying bedrock, excavation should be carried out away from the adjoining structures, with vibrations transmitted 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.

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

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

5.7 Vibration Monitoring and Controls

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

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

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

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

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

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

Table 5. 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 infrastructures, mainly where excavations are expected to be conducted within the underlying bedrock of higher estimated strength and fall within the "zone of influence" of adjoining infrastructures.

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

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

5.8 Groundwater Management

Based on the geotechnical investigation at the selected borehole locations within the site (summarised in Section 3.2), *inferred* groundwater seepage which may be encountered during construction is expected to be at varying depths across the site and possibly above the proposed basement FFLs (subject to confirmation).

It should be noted that no provision was made for longer term groundwater monitoring within the site, and the presence of groundwater should not be precluded during construction and in the long term design life of the proposed buildings.

It should also be noted that these groundwater levels have the potential to elevate 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 weathered bedrock. Seepage may also occur within the excavation areas through the fill material, and at the fill/natural soils and natural soils/bedrock interfaces, predominately following heavy rain.

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

Therefore, consideration should be given to precautionary drainage measures including (not limited to):

- A conventional sump and pump system which may be used both during construction and for permanent groundwater control below the basement level floor slabs.
- Drainage installed around the perimeter of the basement level behind all retaining walls and below the slabs. 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 slabs, with allowance given for nominal hydrostatic uplift.

It is recommended that test pits are carried out by a suitable excavator within the site following demolition of the existing infrastructures and prior to construction, in order to confirm and monitor groundwater levels and inflow rates which may be intercepted during construction within the excavation areas.

This assessment should also be carried out to ensure a suitable drainage and retention system has been 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.

5.9 Excavation Stability

Maximum excavation depths are expected to vary within the sites from approximately 1.0m to 4.5m for construction of the proposed developments, with cut and fill in certain areas. Locally deeper excavations for the lift shafts, and building footings and service trenches are also anticipated to be required as part of the planned development.

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

5.9.1 Batter Slopes

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

Temporary or permanent batter slopes should only be considered where sufficient space exists between the proposed developments and adjoining infrastructures, and where the adjacent infrastructures are located outside the "zone of influence" (obtained by drawing a line 45º above horizontal from the base of the proposed basement level walls).

Table 6 provides maximum recommended slopes for permanent and temporary batters.

Table 6. Recommended Maximum Batter Slopes

Unit		Maximum Batter Slope (H:V)[1]	
		Permanent	**Temporary**
Fill (Unit 1)		4:1	2:1
Residual Soils (Unit 2)		3:1	1.5:1 to 1:1
Bedrock (Unit 3)	**EL – VL**	2:1	1:1 to 0.75:1
	L or better[2]	1:1	0.5:1

[1]Subject to inspection and confirmation by a geotechnical engineer and/or engineering geologist. Remedial options may be required (i.e. soil nailing, rock bolting, shotcreting, etc.).
[2]Preliminary only and inferred to be present within the site at depth. Assumes the presence of shale bedrock underlying the entire site area. Subject to confirmation by a geotechnical engineer and/or engineering geologist, as outlined in this report.
Notes:

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

All batter slopes within the site should remain stable providing all surcharge and construction loads are kept out of the "zone of influence" (obtained by drawing a line 45º above horizontal from the base of the proposed basement level walls) plus an additional 1.0m. A geotechnical engineer and/or engineering geologist should inspect the batter slopes within the site.

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, subject to confirmation by a geotechnical engineer during construction by inspection, or by additional borehole drilling and rock strength testing. Consideration should be given to shotcreting and soil nailing where steeper batter slopes are to be used.

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

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

5.9.2 Excavation Retention Support Systems

Where there is insufficient space between the proposed development and adjoining infrastructures, or where adjacent infrastructures are located within the "zone of influence" (as outlined in Section 5.9.1 above), consideration should be given to a suitable retention system such as a soldier pile wall solution, with piles sufficiently embedded into consistent and competent strength bedrock underlying the site, and concrete and reinforcement infill panels for the support of the excavation and soils.

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

Battering back of the soils may be required in certain areas of the site to permit installation of soldier piles and prevent the collapse of soils into the excavation area. This should be monitored by a geotechnical engineer familiar with these site conditions.

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 infrastructures (i.e. buildings, infrastructures, adjacent road reserves, etc.). This option may also be adopted where excessive surcharges are adjacent to the proposed excavation and to meet acceptable deflection criteria, or where loose/soft soils are required to be retained (i.e. sandy soils), or where there is a potential for undermining of any adjoining building/infrastructures (refer to Section 5.5).

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

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

Where groundwater is deemed to be relatively high and permeability rates are excessive, it is recommended that consideration be given to a contiguous pile wall with strip drains installed behind the piles and shotcreting in weak areas susceptible to groundwater inflow. This should be confirmed by measures discussed in Section 5.8 of this report.

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

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

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 can be designed using the recommended design parameters provided in Section 5.9.3. Bulk excavation and foundations (including pile installations) should be supervised, monitored and inspected by a geotechnical engineer, with all structural elements of the development by a structural engineer. Inspections should be considered as "Hold Points" to the project.

5.9.3 Design Parameters (Earth Pressures)

Excavation pressures acting on the support will depend on a number of factors including external forces from surcharge loading, the stiffness of the support, varying groundwater levels within the site, and the construction sequence of the proposed basement. 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 45º above horizontal from the base of the proposed basement wall.

Support system designed using the earth pressure approach may be based on the parameters given in Table 7 below for soils and rock horizons underlying the site. Table 7 also provides preliminary coefficients of lateral earth pressure for the soils and rock horizons encountered in the site, assuming shale bedrock underlies the site area. These are based on fully drained conditions and that the ground behind the retention walls is horizontal.

Where higher estimated strength bedrock is encountered, GCA should be contacted for further advice.

Table 7. Preliminary Geotechnical Design Parameters

Material	Fill (Unit 1)	Residual Soils (Unit 2)	Bedrock[3, 5] (Unit 3)	
			EL – VL	L or better[6]
Unit Weight (kN/m^3)[4]	16	18	21	22
Effective Cohesion c' (kPa)	0	5	20	40
Angle of Friction ϕ' (°)	24	24	26	28
Modulus of Elasticity E_{sh} (MPa)	3	15	50	160
Earth Pressure Coefficient At Rest Ko[1]	0.59	0.59	0.56	0.53
Earth Pressure Coefficient Active Ka[2]	0.42	0.42	0.39	0.36
Earth Pressure Coefficient Passive Kp[2]	2.37	2.37	2.56	2.77
Poisson Ratio ν	0.4	0.35	0.3	0.3

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

Notes:

- VL = Very Low estimated strength, L = Low estimated strength.
- VL and L bedrock should conform to at least Class V and Class IV Shale, respectively, in accordance with Pells P.J.N, Mostyn G. & Walker B.F.
- Inferred estimated bedrock strength is based on observations made during auger penetration resistance at the time of drilling and confirmation should be made by a geotechnical engineer.
- For undrained (temporary) clay soils, higher earth pressures (K=1) will apply.

5.10 Preliminary Earthquake Site Risk Classification

In accordance with AS 1170.4-2007 and based on assessment of the material encountered during this investigation, the recommended earthquake design parameters for the proposed development sites are as follows:

- Subsoil Class: **"Shallow Soil Site" (Class C_e)**.
- Earthquake Hazard Factor (Z): **0.08** (for Sydney).

5.11 Foundations

Following excavation depths to the FFLs of the proposed development and based on the boreholes carried out within the site, we expect varying ground conditions comprising predominately Unit 2 (residual soils) and Unit 3 (bedrock) of variable estimated strength and weathering to be exposed at bulk excavation level (depending on the actual amount of excavation required).

The possibility for encountering higher estimated strength bedrock in areas of deeper excavation across the site should not be precluded, providing the ground conditions are confirmed by a geotechnical engineer by additional borehole drilling and rock strength testing, or during construction by inspection.

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

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

5.11.1 Geotechnical Assessment

Based on the proposed development and assessment of the subsurface conditions, a suitable foundation system comprising a combination of shallow foundations typically comprising 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/infrastructures. 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.

Installation of piles should be complemented by inspections carried out by a geotechnical engineer during construction. 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.

Confirmation of the actual subsurface conditions underlying the proposed development area should also be undertaken by a geotechnical engineer during construction to confirm ground conditions are consistent throughout and allowable bearing capacities have been achieved. All foundations should not be founded on any soft/ weak bands (i.e. clay seams and extremely weathered/fractured zones) underlying the site. Consultation should be made with specialist subcontractors to discuss the feasibility of piles for the existing ground conditions.

It should be noted that due to the potential variable bedrock conditions throughout the site following bulk excavation and underlying the proposed development, precaution should be taken for the design of the building foundation system, taking into consideration the preliminary geotechnical design parameters in Table 8 below.

Higher allowable bearing capacities may be considered and justified subject to confirmation by inspection during construction, or by additional borehole drilling and rock strength testing. Where higher estimated strength bedrock is encountered during construction, GCA should be contacted to re-assess the preliminary allowable bearing capacities provided in this report. Adoption of higher preliminary bearing capacities for the design of the proposed development outlined in Table 8 should be confirmed by a geotechnical engineer, as discussed in this report.

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 higher allowable bearing capacities, subject to confirmation by a geotechnical engineer by additional borehole drilling and rock strength testing, or by inspection during construction.

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 8 provides preliminary recommended geotechnical design parameters.

Table 8. 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
Residual Soils (Unit 2)		N/A	N/A	N/A
Bedrock (Unit 3)[2]	VL	700	50	25
	L or better[3]	1,000	100	50

[1]Minimum embedment of 0.4m for shallow foundations and 0.5m for deep foundations. Assumes the presence of shale bedrock underlying the entire site area.
[2]The composition, class, depth and estimated strength of the underlying bedrock material should be confirmed either prior to construction by further borehole drilling and rock strength testing, or during construction by inspection.
[3]Conforming to at least Class IV Shale (or better) and inferred to be present within the site at depth.

Notes:

- VL = Very Low estimated strength, L = Low estimated strength.
- VL and L bedrock should conform to at least Class V and Class IV Shale, respectively, in accordance with Pells P.J.N, Mostyn G. & Walker B.F.
- 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 has been 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.

5.11.2 Geotechnical Comments

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

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

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

It is recommended that suitable drainage and the use of impermeable surfaces be implemented as a precaution as part of the design and construction of the proposed development in order to divert surface water away from the building, and help eliminate or minimise surface water infiltration to minimise moisture within the soils. Although trees and vegetation are considered to contribute to the stability of the site, we recommend that planting of trees around the development area (i.e. in close proximity to the proposed building foundations) be limited as they can also affect moisture changes within the soil and cause significant displacement/damage within the building foundations by extensive tree root system movement.

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, it is recommended that consideration be given to other piling methods such as Continuous Flight Auger (CFA) piles.

Shaft adhesion may be applied to socketed piles adopted for foundations provided the socketed shaft lengths conform to appropriate classes of bedrock (subject to confirmation) in accordance with Pells et. al, and shaft sidewall cleanliness and roughness are to acceptable levels. Shaft adhesion should be ignored or reduced within socket lengths that are 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, which are susceptible to shrink and swell due to daily and seasonal moisture, shaft adhesion be ignored due to the potential of shrinkage cracking. Pile inspections should be complemented by downhole CCTV camera.

We recommend that geotechnical inspections of foundations be completed by an experienced geotechnical engineer to determine that the designed socket materials have been reached and the required bearing capacity has been 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”.

5.12 Filling

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

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

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

5.13 Subgrade Preparation

The following are general recommendations on subgrade preparation for earthworks, slab on ground constructions and pavements:

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

6. ADDITIONAL GEOTECHNICAL RECOMMENDATIONS

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

- Dilapidation survey report on adjacent properties and infrastructures.
- Monitoring and supervision of excavations within the site.
- The composition, class, depth and estimated strength of the underlying bedrock material should be confirmed either prior to construction by further borehole drilling and rock strength testing, or during construction by inspection, predominately in areas and at depths not assessed during the geotechnical investigation.
- Geotechnical inspections of exposed materials at bulk excavation level.
- Geotechnical inspections of shoring wall piles installations.
- Geotechnical inspections of foundations (shallow and pile foundations) to confirm the preliminary bearing capacities have been achieved.
- Monitoring of any groundwater inflows into the excavation areas within the site.
- Provision for longer term groundwater monitoring within the site.
- Classification of all excavated material transported from the site.
- A meeting to be carried out to discuss any geotechnical issues and inspection requirements.
- Final architectural and structural design drawings are provided to GCA for further assessment.

7. LIMITATIONS

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

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

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

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

For and behalf of

Geotechnical Consultants Australia Pty Ltd (GCA)

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APPENDIX A

GCA
Geotechnical Consultants Australia

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 Location

GCA Geotechnical Consultants Australia	Figure 1 Site Plan	**Geotechnical Investigation**	Drawn: GN/GA	
		Krishathi Pty Ltd	Date: 28/10/2021	
	Job No.: G21674-1	225 & 227 Bungarribee Road Blacktown NSW 2148	Scale: NTS	

Image source: NSW Six Maps - https://maps.six.nsw.gov.au/, accessed 19th October 2021.

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.

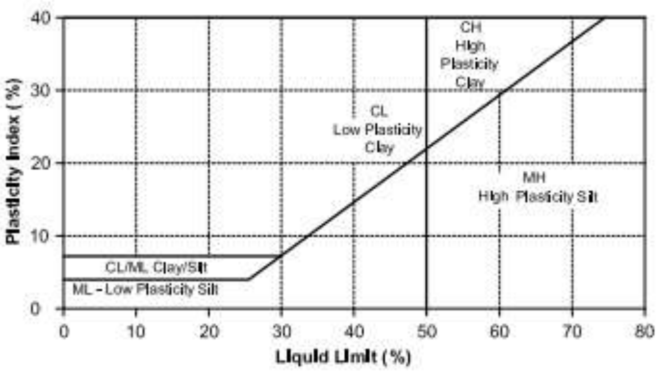
Method and Terms for Soil and Rock Descriptions Used on Borehole and Test Pit Reports

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 *DW*	Highly Weathered *Distinctly Weathered (as per AS 1726)*	The rock substance is affected by weathering to the extent that limonite staining or bleaching affects the whole rock substance and other signs of chemical or physical decomposition are evident. Porosity and strength is usually decreased compared to the fresh rock. The colour and strength of the fresh rock is no longer recognisable.
MW	Moderately Weathered	The whole of the rock substance is discoloured, usually by iron staining or bleaching, to the extent that the colour of the fresh rock is no longer recognisable
SW	Slightly Weathered	Rock is slightly discoloured but shows little or no change of strength from fresh rock
FR	Fresh	Rock shows no sign of decomposition or staining

ROCK STRENGTH (AS 1726-2017 and ISRM)

Term	Symbol	Point Load Index $I_{s(50)}$ (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.

APPENDIX D

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BOREHOLE NUMBER BH1

CLIENT Krishathi Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G21674-1
PROJECT LOCATION 225 & 227 Bungarribee Road Blacktown NSW 2148

DATE STARTED 15/8/21 **COMPLETED** 15/8/21 **R.L. SURFACE** **DATUM**
DRILLING CONTRACTOR NEO **SLOPE** 90° **BEARING** ---
EQUIPMENT Ute Mounted Drilling Rig **HOLE LOCATION** Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter **LOGGED BY** CC/GN **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
ADT	Not Encountered During Drilling					Clayey SILT, brown to dark brown, medium plasticity clay, some fine gravel, with fine grained sand, grass rootlets, moist.		FILL
			0.5		CI-CH	Silty CLAY, medium to high plasticity, brown to reddish brown, grey laminations, some fine to medium ironstone gravel, moist.		RESIDUAL SOILS
			1.0		CI-CH	Silty CLAY, medium to high plasticity, grey to pale grey, reddish brown laminations, some fine to coarse ironstone gravel, moist.	DS	
			1.5			SHALE, grey, brown, with silt, clay seams, interbedded sandstone, extremely weathered, extremely low estimated strength, moist.		BEDROCK
			2.0			SHALE, grey, brown, with silt, some clay seams, interbedded sandstone, highly weathered, very low estimated strength, moist.		
			2.5					
			3.0					
			3.5			becoming grey from 3.5m bgl.		TC bit refusal at 3.8m bgl.
						inferred low estimated strength (or better) from 3.8m bgl. Borehole BH1 terminated at 3.8m		
			4.0					
			4.5					
			5.0					

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BOREHOLE NUMBER BH2

CLIENT Krishathi Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G21674-1
PROJECT LOCATION 225 & 227 Bungarribee Road Blacktown NSW 2148

DATE STARTED 15/8/21 **COMPLETED** 15/8/21
R.L. SURFACE
DATUM
DRILLING CONTRACTOR NEO
SLOPE 90°
BEARING ---
EQUIPMENT Ute Mounted Drilling Rig
HOLE LOCATION Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter
LOGGED BY CC/GN
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
ADT	Not Encountered During Drilling		0.0			Clayey SILT, brown to dark brown, dark grey, medium plasticity clay, some fine gravel, with fine grained sand, grass rootlets, moist.		FILL
			0.5		CI-CH	Silty CLAY, medium to high plasticity, brown to reddish brown, grey laminations, some fine to medium ironstone gravel, moist.		RESIDUAL SOILS
			1.0					
			1.5		CI-CH	Silty CLAY, medium to high plasticity, brown to pale reddish brown, grey to pale grey, some fine gravel, moist.		
			2.0			SHALE, grey, brown, with silt, clay seams, extremely weathered, extremely low estimated strength, moist.		BEDROCK
			2.5					
			3.0					
			3.5			SHALE, grey, with silt, some clay seams, interbedded sandstone, highly weathered, very low estimated strength, moist.		
			4.0					
			4.5					
			5.0					

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BOREHOLE NUMBER BH2

CLIENT Krishathi Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G21674-1
PROJECT LOCATION 225 & 227 Bungarribee Road Blacktown NSW 2148

DATE STARTED 15/8/21 **COMPLETED** 15/8/21 **R.L. SURFACE** **DATUM**
DRILLING CONTRACTOR NEO **SLOPE** 90° **BEARING** ---
EQUIPMENT Ute Mounted Drilling Rig **HOLE LOCATION** Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter **LOGGED BY** CC/GN **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
ADT			5.5, 6.0			SHALE, grey to dark grey, with silt, highly weathered, very low estimated strength, moist.		TC bit refusal at 6.0m bgl.
			6.0			inferred low estimated strength (or better) from 6.0m bgl.	DS	
						Borehole BH2 terminated at 6m		
			6.5, 7.0, 7.5, 8.0, 8.5, 9.0, 9.5, 10.0					

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BOREHOLE NUMBER BH3

CLIENT Krishathi Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G21674-1
PROJECT LOCATION 225 & 227 Bungarribee Road Blacktown NSW 2148

DATE STARTED 15/8/21 **COMPLETED** 15/8/21
R.L. SURFACE
DATUM
DRILLING CONTRACTOR NEO
SLOPE 90°
BEARING ---
EQUIPMENT Ute Mounted Drilling Rig
HOLE LOCATION Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter
LOGGED BY CC/GN
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
ADT	Not Encountered During Drilling		0.0–0.5			Clayey SILT, brown to dark brown, medium plasticity clay, some fine gravel, with fine grained sand, grass rootlets, moist.		FILL
			0.5–1.8		CI-CH	Silty CLAY, medium to high plasticity, brown to reddish brown, grey to pale grey, some fine to medium gravel, moist.		RESIDUAL SOILS
			1.8–2.8			SHALE, brown, with silt, clay seams, extremely weathered, extremely low estimated strength, moist.	DS	BEDROCK
			2.8–4.0			SHALE, brown to pale reddish brown, grey laminations, with silt, clay seams, extremely weathered, extremely low estimated strength, moist.		
			4.0–5.0			SHALE, grey, with silt, some clay seams, highly weathered, very low estimated strength, moist.		

Depth scale (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0

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BOREHOLE NUMBER BH3

CLIENT Krishathi Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G21674-1
PROJECT LOCATION 225 & 227 Bungarribee Road Blacktown NSW 2148

DATE STARTED 15/8/21 **COMPLETED** 15/8/21 **R.L. SURFACE** **DATUM**
DRILLING CONTRACTOR NEO **SLOPE** 90° **BEARING** ---
EQUIPMENT Ute Mounted Drilling Rig **HOLE LOCATION** Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter **LOGGED BY** CC/GN **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
ADT			5.5, 6.0			SHALE, grey, with silt, some clay seams, highly weathered, very low estimated strength, moist. *(continued)*		TC bit refusal at 6.0m bgl.
			6.5, 7.0, 7.5, 8.0, 8.5, 9.0, 9.5, 10.0			inferred low estimated strength (or better) from 6.0m bgl. Borehole BH3 terminated at 6m		

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BOREHOLE NUMBER BH4

CLIENT Krishathi Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G21674-1
PROJECT LOCATION 225 & 227 Bungarribee Road Blacktown NSW 2148

DATE STARTED 15/8/21 **COMPLETED** 15/8/21 **R.L. SURFACE** **DATUM**
DRILLING CONTRACTOR NEO **SLOPE** 90° **BEARING** ---
EQUIPMENT Ute Mounted Drilling Rig **HOLE LOCATION** Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter **LOGGED BY** CC/GN **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
ADT	Not Encountered During Drilling		0.0–0.4			Clayey SILT, brown to dark brown, medium plasticity clay, some fine gravel, with fine grained sand, grass rootlets, moist.		FILL
			0.4–1.0		CI-CH	Silty CLAY, medium to high plasticity, brown to reddish brown, some fine to medium ironstone gravel, moist.		RESIDUAL SOILS
			1.0–1.9		CI	Shaly CLAY, medium plasticity, brown to pale brown, some fine to coarse gravel, some fine grained sand, interbedded shale, moist.		
			1.9–2.8			SHALE, brown, grey, clay seams, with silt, extremely weathered, extremely low estimated strength, moist.		BEDROCK
			2.8–3.2			SHALE, grey, with silt, some clay seams, highly weathered, very low estimated strength, moist.	DS	TC bit refusal at 3.2m bgl.
			3.2			inferred low estimated strength (or better) from 3.2m bgl. Borehole BH4 terminated at 3.2m		

APPENDIX E

Foundation Maintenance and Footing Performance: A Homeowner's Guide

CSIRO

BTF 18 replaces Information Sheet 10/91

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

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

Soil Types

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

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

Causes of Movement

Settlement due to construction

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

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

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

Erosion

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

Saturation

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

Seasonal swelling and shrinkage of soil

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

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

Shear failure

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

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

GENERAL DEFINITIONS OF SITE CLASSES

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

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

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

Unevenness of Movement

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

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

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

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

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

Effects of Uneven Soil Movement on Structures

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

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

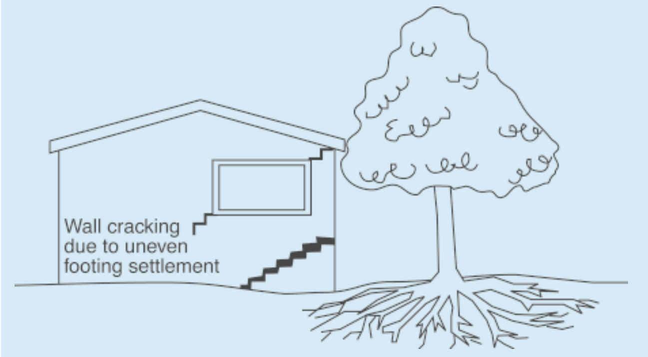
Isolated piers affected by erosion or saturation of foundations will eventually lose contact with the bearers they support and may tilt or fall over. The floors that have lost this support will become bouncy, sometimes rattling ornaments etc.

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

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

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

Trees can cause shrinkage and damage



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

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

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

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

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

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

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

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

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

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

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

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

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

Water Service and Drainage

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

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

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

Seriousness of Cracking

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

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

Prevention/Cure

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

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

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

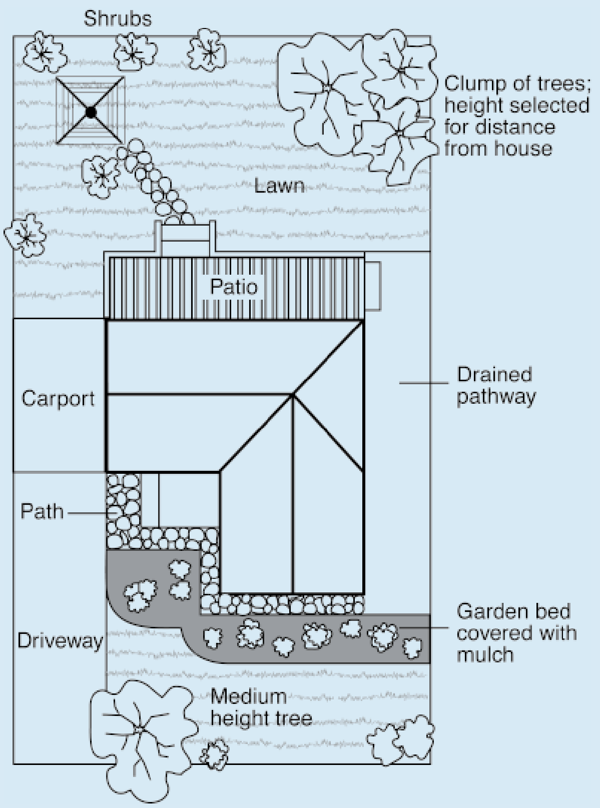
Protection of the building perimeter
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

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

CLASSIFICATION OF DAMAGE WITH REFERENCE TO WALLS

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

Gardens for a reactive site



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

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

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

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

Condensation

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

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

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

The garden

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

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

Existing trees

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

Information on trees, plants and shrubs

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

Excavation

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

Remediation

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

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

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

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APPENDIX F

ALS Environmental

CERTIFICATE OF ANALYSIS

Work Order	: **ES2137491**	Page	: 1 of 2
Client	: **GEOTECHNICAL CONSULTANTS AUSTRALIA PTY LTD**	Laboratory	: Environmental Division Sydney
Contact	: JOE NADER	Contact	: Customer Services ES
Address	: Suite 5, 5-7 Villiers Street Parramatta NSW 2151	Address	: 277-289 Woodpark Road Smithfield NSW Australia 2164
Telephone	: ----	Telephone	: +61-2-8784 8555
Project	: G21674-1	Date Samples Received	: 18-Oct-2021 16:30
Order number	:	Date Analysis Commenced	: 20-Oct-2021
C-O-C number	: ----	Issue Date	: 28-Oct-2021 09:29
Sampler	: LB		
Site	: 227 Bungarribee Road Blacktown NSW 2148		
Quote number	: EN/333		
No. of samples received	: 4		
No. of samples analysed	: 4		

NATA
Accreditation No. 825
Accredited for compliance with
ISO/IEC 17025 - Testing

This report supersedes any previous report(s) with this reference. Results apply to the sample(s) as submitted, unless the sampling was conducted by ALS. This document shall not be reproduced, except in full.

This Certificate of Analysis contains the following information:

- General Comments
- Analytical Results

Additional information pertinent to this report will be found in the following separate attachments: Quality Control Report, QA/QC Compliance Assessment to assist with Quality Review and Sample Receipt Notification.

Signatories

This document has been electronically signed by the authorized signatories below. Electronic signing is carried out in compliance with procedures specified in 21 CFR Part 11.

Signatories	*Position*	*Accreditation Category*
Ankit Joshi	Inorganic Chemist	Sydney Inorganics, Smithfield, NSW
Ivan Taylor	Analyst	Sydney Inorganics, Smithfield, NSW

General Comments

The analytical procedures used by ALS have been developed from established internationally recognised procedures such as those published by the USEPA, APHA, AS and NEPM. In house developed procedures are fully validated and are often at the client request.

Where moisture determination has been performed, results are reported on a dry weight basis.

Where a reported less than (<) result is higher than the LOR, this may be due to primary sample extract/digestate dilution and/or insufficient sample for analysis.

Where the LOR of a reported result differs from standard LOR, this may be due to high moisture content, insufficient sample (reduced weight employed) or matrix interference.

When sampling time information is not provided by the client, sampling dates are shown without a time component. In these instances, the time component has been assumed by the laboratory for processing purposes.

Where a result is required to meet compliance limits the associated uncertainty must be considered. Refer to the ALS Contact for details.

Key : CAS Number = CAS registry number from database maintained by Chemical Abstracts Services. The Chemical Abstracts Service is a division of the American Chemical Society.
LOR = Limit of reporting
^ = This result is computed from individual analyte detections at or above the level of reporting
ø = ALS is not NATA accredited for these tests.
~ = Indicates an estimated value.

Analytical Results

Sub-Matrix: SOIL (Matrix: SOIL)			Sample ID	BH1 1.0m	BH2 6.0m	BH3 2.0m	BH4 3.0m	----
			Sampling date / time	15-Oct-2021 00:00	15-Oct-2021 00:00	15-Oct-2021 00:00	15-Oct-2021 00:00	----
Compound	CAS Number	LOR	Unit	ES2137491-001	ES2137491-002	ES2137491-003	ES2137491-004	--------
				Result	Result	Result	Result	----
EA002: pH 1:5 (Soils)								
pH Value	----	0.1	pH Unit	5.2	8.0	4.9	5.3	----
EA010: Conductivity (1:5)								
Electrical Conductivity @ 25°C	----	1	µS/cm	384	378	620	405	----
EA055: Moisture Content (Dried @ 105-110°C)								
Moisture Content	----	1.0	%	17.2	9.5	11.5	11.0	----
ED040S : Soluble Sulfate by ICPAES								
Sulfate as SO4 2-	14808-79-8	10	mg/kg	180	60	250	240	----
ED045G: Chloride by Discrete Analyser								
Chloride	16887-00-6	10	mg/kg	460	490	830	380	----

ALS Environmental

QUALITY CONTROL REPORT

Work Order	: **ES2137491**	Page	: 1 of 3
Client	: **GEOTECHNICAL CONSULTANTS AUSTRALIA PTY LTD**	Laboratory	: Environmental Division Sydney
Contact	: JOE NADER	Contact	: Customer Services ES
Address	: Suite 5, 5-7 Villiers Street Parramatta NSW 2151	Address	: 277-289 Woodpark Road Smithfield NSW Australia 2164
Telephone	: ----	Telephone	: +61-2-8784 8555
Project	: G21674-1	Date Samples Received	: 18-Oct-2021
Order number	:	Date Analysis Commenced	: 20-Oct-2021
C-O-C number	: ----	Issue Date	: 28-Oct-2021
Sampler	: LB		
Site	: 227 Bungarribee Road Blacktown NSW 2148		
Quote number	: EN/333		
No. of samples received	: 4		
No. of samples analysed	: 4		

Accreditation No. 825
Accredited for compliance with
ISO/IEC 17025 - Testing

This report supersedes any previous report(s) with this reference. Results apply to the sample(s) as submitted, unless the sampling was conducted by ALS. This document shall not be reproduced, except in full.

This Quality Control Report contains the following information:

- Laboratory Duplicate (DUP) Report; Relative Percentage Difference (RPD) and Acceptance Limits
- Method Blank (MB) and Laboratory Control Spike (LCS) Report; Recovery and Acceptance Limits
- Matrix Spike (MS) Report; Recovery and Acceptance Limits

Signatories

This document has been electronically signed by the authorized signatories below. Electronic signing is carried out in compliance with procedures specified in 21 CFR Part 11.

Signatories	*Position*	*Accreditation Category*
Ankit Joshi	Inorganic Chemist	Sydney Inorganics, Smithfield, NSW
Ivan Taylor	Analyst	Sydney Inorganics, Smithfield, NSW

General Comments

The analytical procedures used by ALS have been developed from established internationally recognised procedures such as those published by the USEPA, APHA, AS and NEPM. In house developed procedures are fully validated and are often at the client request.

Where moisture determination has been performed, results are reported on a dry weight basis.

Where a reported less than (<) result is higher than the LOR, this may be due to primary sample extract/digestate dilution and/or insufficient sample for analysis. Where the LOR of a reported result differs from standard LOR, this may be due to high moisture content, insufficient sample (reduced weight employed) or matrix interference.

Key : Anonymous = Refers to samples which are not specifically part of this work order but formed part of the QC process lot
CAS Number = CAS registry number from database maintained by Chemical Abstracts Services. The Chemical Abstracts Service is a division of the American Chemical Society.
LOR = Limit of reporting
RPD = Relative Percentage Difference
# = Indicates failed QC

Laboratory Duplicate (DUP) Report

The quality control term Laboratory Duplicate refers to a randomly selected intralaboratory split. Laboratory duplicates provide information regarding method precision and sample heterogeneity. The permitted ranges for the Relative Percent Deviation (RPD) of Laboratory Duplicates are specified in ALS Method QWI-EN/38 and are dependent on the magnitude of results in comparison to the level of reporting: Result < 10 times LOR: No Limit; Result between 10 and 20 times LOR: 0% - 50%; Result > 20 times LOR: 0% - 20%.

Sub-Matrix: **SOIL**

				Laboratory Duplicate (DUP) Report					
Laboratory sample ID	*Sample ID*	*Method: Compound*	*CAS Number*	*LOR*	*Unit*	*Original Result*	*Duplicate Result*	*RPD (%)*	*Acceptable RPD (%)*
EA002: pH 1:5 (Soils) (QC Lot: 3964717)									
ES2137230-001	Anonymous	EA002: pH Value	----	0.1	pH Unit	6.3	6.0	4.5	0% - 20%
ES2137491-003	BH3 2.0m	EA002: pH Value	----	0.1	pH Unit	4.9	4.9	0.0	0% - 20%
EA010: Conductivity (1:5) (QC Lot: 3964720)									
ES2137491-003	BH3 2.0m	EA010: Electrical Conductivity @ 25°C	----	1	µS/cm	620	625	0.8	0% - 20%
EA055: Moisture Content (Dried @ 105-110°C) (QC Lot: 3964721)									
ES2137301-001	Anonymous	EA055: Moisture Content	----	0.1	%	12.5	14.3	13.0	0% - 50%
ED040S: Soluble Major Anions (QC Lot: 3964719)									
ES2137491-003	BH3 2.0m	ED040S: Sulfate as SO4 2-	14808-79-8	10	mg/kg	250	260	0.0	0% - 20%
ED045G: Chloride by Discrete Analyser (QC Lot: 3964718)									
ES2137230-001	Anonymous	ED045G: Chloride	16887-00-6	10	mg/kg	<10	<10	0.0	No Limit

Method Blank (MB) and Laboratory Control Sample (LCS) Report

The quality control term Method / Laboratory Blank refers to an analyte free matrix to which all reagents are added in the same volumes or proportions as used in standard sample preparation. The purpose of this QC parameter is to monitor potential laboratory contamination. The quality control term Laboratory Control Sample (LCS) refers to a certified reference material, or a known interference free matrix spiked with target analytes. The purpose of this QC parameter is to monitor method precision and accuracy independent of sample matrix. Dynamic Recovery Limits are based on statistical evaluation of processed LCS.

Sub-Matrix: **SOIL**

				Method Blank (MB) Report	Laboratory Control Spike (LCS) Report			
					Spike	Spike Recovery (%)	Acceptable Limits (%)	
Method: Compound	CAS Number	LOR	Unit	Result	Concentration	LCS	Low	High
EA010: Conductivity (1:5) (QCLot: 3964720)								
EA010: Electrical Conductivity @ 25°C	----	1	µS/cm	<1	1412 µS/cm	105	92.0	108
ED040S: Soluble Major Anions (QCLot: 3964719)								
ED040S: Sulfate as SO4 2-	14808-79-8	10	mg/kg	<10	750 mg/kg	99.0	80.0	120
ED045G: Chloride by Discrete Analyser (QCLot: 3964718)								
ED045G: Chloride	16887-00-6	10	mg/kg	<10	250 mg/kg	95.4	75.0	125
				<10	5000 mg/kg	94.8	79.0	117

Matrix Spike (MS) Report

The quality control term Matrix Spike (MS) refers to an intralaboratory split sample spiked with a representative set of target analytes. The purpose of this QC parameter is to monitor potential matrix effects on analyte recoveries. Static Recovery Limits as per laboratory Data Quality Objectives (DQOs). Ideal recovery ranges stated may be waived in the event of sample matrix interference.

Sub-Matrix: **SOIL**

				Matrix Spike (MS) Report			
				Spike	SpikeRecovery(%)	Acceptable Limits (%)	
Laboratory sample ID	Sample ID	Method: Compound	CAS Number	Concentration	MS	Low	High
ED045G: Chloride by Discrete Analyser (QCLot: 3964718)							
ES2137230-001	Anonymous	ED045G: Chloride	16887-00-6	250 mg/kg	116	70.0	130

APPENDIX G

bt	BLACKTOWN	Residual

Landscape—gently undulating rises on Wianamatta Group shales. Local relief to 30 m, slopes usually >5%. Broad rounded crests and ridges with gently inclined slopes. Cleared Eucalypt woodland and tall open-forest (dry schlerophyll forest).

Soils—shallow to moderately deep (>100 cm) hardsetting mottled texture contrast soils, Red and Brown Podzolic Soils (Dr3.21, Dr3.31, Db2.11, Db2.21) on crests grading to Yellow Podzolic Soils (Dy2.11, Dy3.11) on lower slopes and in drainage lines.

Limitations—localised seasonal waterlogging, localised water erosion hazard, moderately reactive highly plastic subsoil, localised surface movement potential.

LOCATION

Occurs extensively on the Cumberland Lowlands. Examples include Blacktown, Mount Druitt, Glossodia and Leppington.

Isolated examples are found at Bilpin on the Blue Mountains plateau surface and along the Silverdale Road south of Wallacia.

LANDSCAPE

Geology

Wianamatta Group—Ashfield Shale consisting of laminite and dark grey siltstone, Bringelly Shale which consists of shale with occasional calcareous claystone, laminite and infrequent coal, and Minchinbury Sandstone consisting of fine to medium-grained quartz lithic sandstone.

Topography

Gently undulating rises on Wianamatta Shale with local relief 10–30 m and slopes generally >5% but occasionally up to 10%. Crests and ridges are broad (200–600 m) and rounded with convex upper slopes grading into concave lower slopes. Outcrops of shale do not occur naturally on the surface. They may occur, however, where soils have been removed.

Vegetation

Almost completely cleared open-forest and open-woodland (dry sclerophyll forest). The original woodland and open-forest were dominated by *Eucalyptus tereticornis* (forest red gum), *E. crebra* (narrow-leaved ironbark), *E. moluccana* (grey box) and *E. maculata* (spotted gum) (Benson, 1981).

Further west near Penrith remnant stands of *E. punctata* (grey gum) occur. Between Liverpool and St Marys, the dominant species are *E. globoidea* (white stringybark) and *E. fibrosa* (broad-leaved ironbark), with *E. longifolia* (woollybutt) as an understorey species. Individual trees or small stands of *E. sideroxylon* (mugga ironbark) are occasionally found on crests.

Landuse

The dominant landuses are intensive residential (Fairfield, Blacktown and Mt Druitt), horticulture and animal husbandry (Vineyard, Scheyville and Leppington) and light and heavy industry (Yennora and Moorebank).

Existing Erosion

No appreciable erosion occurs on this unit. Minor sheet and gully erosion may be found where surface vegetation is not maintained.

Associated Soil Landscapes

South Creek (**sc**) soil landscape occurs along drainage depressions. Picton (**pn**) soil landscape occurs on steeper south and southeast facing slopes. Small areas of Luddenham (**lu**) soil landscape may also occur.

SOILS

Dominant Soil Materials

bt1—Friable brownish black loam.

This is a friable brownish black loam to clay loam with moderately pedal subangular blocky structure and rough-faced porous ped fabric. This material occurs as topsoil (A horizon).

Peds are well defined subangular blocky and range in size from 2–20 mm. Surface condition is friable. Colour is brownish black (10YR 2/2) but can range from dark reddish brown (5YR 3/2) to dark yellowish brown (10YR 3/4). The pH varies from moderately acid (pH 5.5) to neutral (pH 7.0). Rounded iron indurated fine gravel-sized shale fragments and charcoal fragments are sometimes present. Roots are common.

bt2—Hardsetting brown clay loam.

This is a brown clay loam to silty clay loam which is hardsetting on exposure or when completely dried out. It has apedal massive to weakly pedal structure and slowly porous earthy fabric. It occurs as an A2 horizon.

Peds when present are weakly developed, subangular blocky and are rough faced and porous. They range in size between 20–50 mm. This material is water repellent when extremely dry.

Colour is dark brown (7.5YR 4/3) but can range from dark reddish brown (2.5YR 3/3) to dark brown (10YR 3/3). The pH varies from moderately acid (pH 5.0) to slightly acid (pH 6.5). Platy, iron indurated gravel-sized shale fragments are common. Charcoal fragments and roots are rarely present.

bt3—Strongly pedal, mottled brown light clay.

This is a brown light to medium clay with strongly pedal polyhedral or sub-angular to blocky structure and smooth-faced dense ped fabric. This material usually occurs as subsoil (B horizon).

Texture often increases with depth. Peds range in size from 5–20 mm. Colour is brown (7.5YR 4/6) but may range from reddish brown (2.5YR 4/6) to brown (10YR 4/6). Frequent red, yellow or grey mottles occur often becoming more numerous with depth. The pH varies from strongly acid (pH 4.5) to slightly acid (pH 6.5). Fine to coarse gravel-sized shale fragments are common and often occur in stratified bands. Both roots and charcoal fragments are rare.

bt4—Light grey plastic mottled clay.

This is a plastic light grey silty clay to heavy clay with moderately pedal polyhedral to subangular blocky structure and smoothfaced dense ped fabric. This material usually occurs as deep subsoil above shale bedrock (B3 or C horizon).

Peds range in size from 2–20 mm. Colour is usually light grey (10YR 7/1) or, less commonly, greyish yellow (2.5YR 6/2). Red, yellow or grey mottles are common. The pH varies from strongly acid (pH 4.0) to moderately acid (pH 5.5). Strongly weathered ironstone concretions and rock fragments are common. Gravel-sized shale fragments and roots are occasionally present. Charcoal fragments are rare.

Occurrence and Relationships

Crests. On crests and ridges up to 30 cm of friable brownish black loam (**bt1**) overlies 10–20 cm of hardsetting brown clay loam (**bt2**) and up to 90 cm of strongly pedal, brown mottled light clay (**bt3**) [red podzolic soils (Dr3.21, 3.11) and brown podzolic soils (Db2.11)]. **bt1** is occasionally absent. Boundaries between the soil materials are usually clear. Total soil depth is <100 cm.

Upper slopes and Midslopes. Up to 30 cm of **bt1** overlies 10–20 cm of **bt2** and 20–50 cm of **bt5**. This in turn overlies up to 100 cm of a light grey plastic mottled clay (**bt4**) [Red Podzolic Soils (Dr3.21), Brown Podzolic Soils (Db2.21). Occasionally **bt1** is absent. The boundaries between the soil materials are usually clear. Total soil depth is<200 cm.

Lower sideslopes. Up to 30 cm of **bt1** overlies 10–30 cm of **bt2** and 40–100 cm of **bt3**. Below **bt3** there is usually >100 cm of **bt4** [Yellow Podzolic Soils (Dy2.11, Dy3.11)]. The boundaries between the soil materials are clear. Total soil depth is >200 cm.

LIMITATIONS TO DEVELOPMENT

Soil Limitations

btl Strongly acid

bt2 Hardsetting
Low fertility
Strongly acid
High aluminium toxicity

bt3 High shrink-swell (localised)
Low wet strength
Low permeability

Low available water capacity
Salinity (localised)
Sodicity (localised)
Very low fertility
Very strongly acid
Very high aluminium toxicity

bt4 High shrink-swell (localised)
Low wet strength
Stoniness
Low available water capacity
Low permeability
Salinity (localised)
Sodicity (localised)
Low fertility
Strongly acid
Very high aluminium toxicity
High erodibility (localised)

Fertility

General fertility is low to moderate. Soil materials have low to moderate available water capacity, low CEC values, hardsetting surfaces (**bt2**), very low phosphorus and low to very low nitrogen levels. The subsoils (**bt3**, **bt4**) may be locally sodic with low permeability. When **bt1** is present its higher organic matter content and moderate nitrogen levels result in higher general fertility.

Erodibility

Blacktown soil materials have moderate erodibility. The topsoils (**bt1**, **bt2**) are often hardsetting and they have high fine sand and silt content, but they also have high to moderate organic matter content. The subsoils (**bt3**, **bt4**) are very low in organic matter. Where they are also highly dispersible and occasionally sodic the erodibility is high.

Erosion Hazard

The erosion hazard for non-concentrated flows is slight to moderate but ranges from low to very high. Calculated soil loss during the first twelve months of urban development for topsoil and exposed subsoil tends to be low (7–11 t/ha). Soil erosion hazard for concentrated flows is moderate to high.

Surface Movement Potential

The deep clay soils are moderately reactive. These are generally found on side-slopes and footslopes. Shallower soils on forests are slightly reactive.

Landscape Limitations

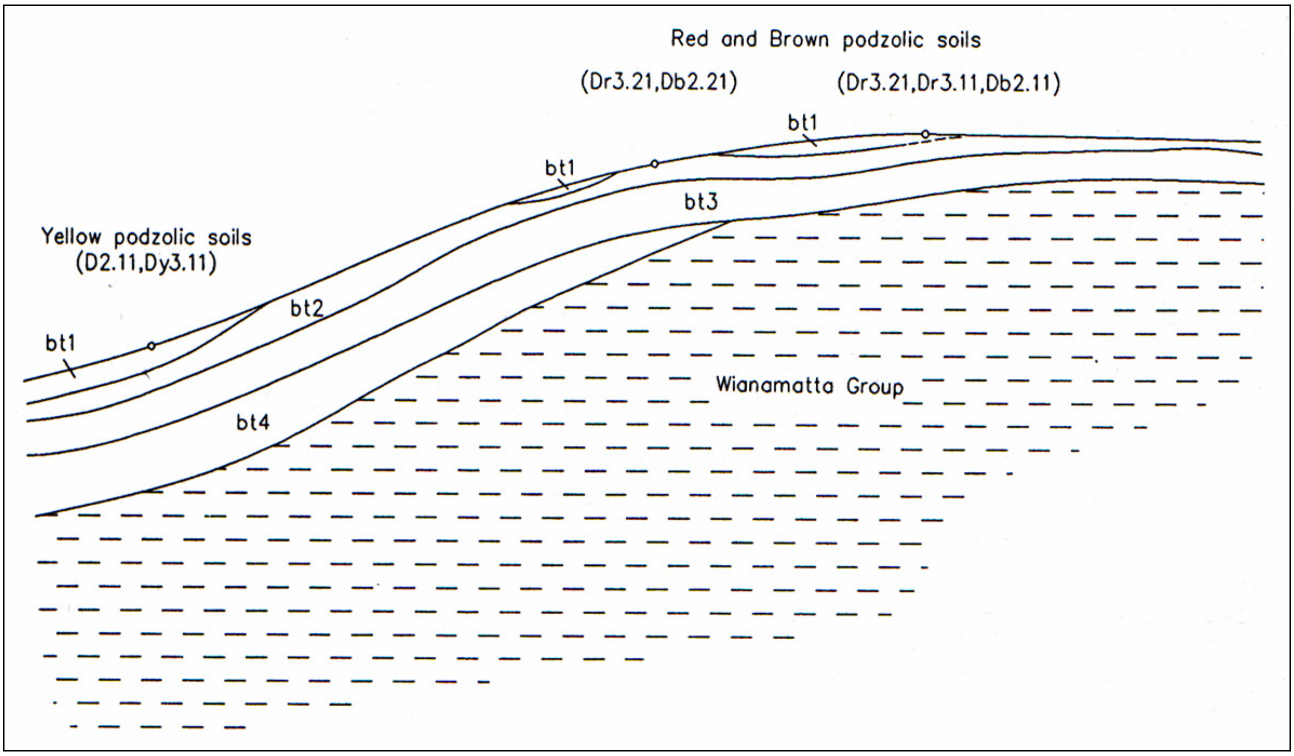
Seasonal waterlogging (localised), water erosion hazard (localised), surface movement potential (localised).

Urban Capability

High capability for urban development with appropriate foundation design.

Rural Capability

Small portions of this soil landscape which have not been urbanised are capable of sustaining regular cultivation and grazing.



Distribution diagram of the Blacktown soil landscape showing the occurrence and relationship of dominant soil materials.