Geotechnical Consultants Australia

Mohammed Dahar

Geotechnical Investigation Report

Proposed Development at
190 Waterloo Road
Greenacre NSW 2190

G24395-1

16th December 2024

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Report Distribution

Geotechnical Investigation Report

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TABLE OF CONTENTS

TABLES

APPENDICES

1. INTRODUCTION

1.1 Background

This geotechnical engineering report presents the results of a geotechnical investigation undertaken by Geotechnical Consultants Australia Pty Ltd (GCA) for a proposed development at No. 190 Waterloo Road Greenacre NSW 2190 (the site). The investigation was commissioned by Mr. Mohammed Dahar (the client) and the fieldwork was carried out on the 5th December 2024.

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

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 is prepared to provide preliminary geotechnical advice and recommendations to assist in the preparation of designs and construction of the ground structures for the proposed development.

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

1.2 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 Ghazi Al Ali Architect Pty Ltd, titled "190 Waterloo Rd, Greenacre", and referenced project No. 29.17.

1.3 Proposed Development

Information provided by the client indicates that the proposed development comprises demolition of existing onsite infrastructure, followed by construction of a new multi-storey residential and commercial mixed-use building, overlying two (2) basement levels.

No Finished Floor Levels (FFL)s for the proposed development have been provided to GCA during preparation of this report. However, based on the existing site levels from the topography, maximum inferred excavation depths up to 7.0m are expected to be required for construction of the proposed development, with cut and fill in certain areas of the site. Locally deeper excavations for the proposed lift shafts, building footings and service trenches are also anticipated.

It should be noted that excavation depths are expected to vary across the site. Final architectural drawings should be provided to GCA prior to construction, in order to confirm the findings and recommendations outlined in this report.

1.4 Geotechnical Assessment Objectives

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

- General assessment of any potential geotechnical issues that may affect the proposed development and any surrounding infrastructure, buildings, council assets, etc.
- Excavation conditions and recommendations on excavation methods in soil and rock to restrict any ground vibrations.
- Vibration control and recommendations to restrict ground vibrations.
- Recommendations on suitable shoring (retention) systems for the site.
- Design parameters based on ground conditions within the site for retaining walls, cantilever shoring walls and propped shoring.
- Recommendations on suitable foundation types and design parameters for the site.
- Allowable and end bearing capacities and shaft adhesion for shallow foundations based on ground conditions within the site.
- Groundwater levels which may be determined during the site investigation, along with the effects on the proposed development.
- Recommendations on groundwater management and limiting inflow, if groundwater is present.
- Preliminary subsoil class for earthquake design for the site in accordance with AS 1170.4-2007 "Structural Design Actions Part 4 Earthquake Actions in Australia".
- Preliminary site lot classification in accordance with Australian Standard (AS) 2870-2011 "Residential Slabs and Footings".
- 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 described in AS 1726-2017 "Geotechnical Site Investigations". The scope of works included:

- Submit and review Before You Dig Australia (BYDA) plans and any other plans provided by the client on existing buried services within the site.
- Service locating carried out by a specialist contractor using electromagnetic detection equipment to ensure the area is free of any underground services at the selected borehole 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 three (3) boreholes at selected locations within the site (where accessible and feasible) by a specialised Ute mounted geotechnical drilling rig, using solid flight augers equipped with a 'Tungsten Carbide' (TC) bit, and identified as boreholes BH1 to BH3 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 4.2m to 5.7m below the existing ground level within the site (bgl).
- Dynamic Cone Penetrometer (DCP) testing immediately adjacent to boreholes BH1 and BH2 using hand operated equipment to varying practical refusal/terminated depths of approximately 1.4m and 1.8m. The DCP tests are identified as DCP1 and DCP2.
- Collection of soil and rock samples during drilling for the following laboratory testing:
    - Laboratory testing by a National Association of Testing Authorities, Australia (NATA)

accredited laboratory (ALS Environmental) on six (6) selected samples collected during drilling of the boreholes, to determine the pH, chloride and sulphate content, and electrical conductivity of the selected samples.

- Preparation of this geotechnical engineering report.

1.6 Constraints

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

Furthermore, additional machine drilled boreholes with rock coring and rock strength testing is required following demolition of existing onsite infrastructure.

2. SITE DESCRIPTION

2.1 Overall Site Description

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

Table 1. Overall Site Description and Site Surroundings

Information	Details
Overall Site Location	The site is located in a residential/commercial area at the intersection of Waterloo Road and Boronia Road thoroughfares.
Site Address	190 Waterloo Road Greenacre NSW 2190
Approximate Site Area[1]	1,779m^2
Local Government Authority	City of Canterbury Bankstown Council
Site Description	At the time of the investigation, a service station and food truck occupied the site, accompanied by concrete cover throughout with some grass in the north-western portion of the site.
Approximate Distances to Nearest Watercourses (i.e. rivers, lakes, creeks, etc.)	• Canal (unnamed) – 250m west of the site. • Coxs Creek – 1.27km south-east of the site.
Site Surroundings	The site is located within an area of residential/commercial and is bounded by: • Commercial property at No. 198 Waterloo Road to the north • Waterloo Road thoroughfare to the east. • Boronia Road thoroughfare to the south. • Commercial properties at No. 9 Boronia Road and No. 198 Waterloo Road to the west.

[1]Site area is approximate and obtained from the architectural drawings referenced in Section 1.2.

2.2 Topography

The local and site topography generally slopes very gently towards to the west. Based on elevation contours from Mecone Mosaic (https://meconemosaic.au/), levels within the site vary from an estimated Reduced Level (RL) of RL50.5m to RL49.8m Australian Height Datum (AHD), at gradients of between 0° and 2°. We note that no site survey was provided to GCA during the preparation of this report.

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

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

2.3 Regional Geology

Information obtained on the local regional subsurface conditions, referenced from the Department of Mineral Resources, Sydney 1:100,000 Geological Series Sheet 9130 First Edition, dated 1983, by the Geological Survey of New South Wales, indicates the site is underlain by Bringelly Shale of the Wianamatta Group (Rwb). The Bringelly Shale (Rwb) typically consists of “Shale, carbonaceous claystone, laminite, fine to medium grained lithic Sandstone, rare coal”.

It is noted that approximately 1km south-east of the site lies a geological boundary with the Ashfield Shale (Rwa), typically consisting of “black to dark grey Shale and laminite”.

Furthermore, reference made to MinView by the State of New South Wales through Regional NSW 2024 also indicates that the site is underlain by Bringelly Shale (Twib - Bringelly Shale). The site is also situated approximately 200m east of Clastic Sediments (Q_av).

A review of the regional maps by the NSW Government Environment and Heritage shows the site is set within the Blacktown (bt) landscape group.

The Blacktown (bt) landscape group is typically recognised by gently undulating rises on Wianamatta Group Shales and Hawkesbury Shale. Local reliefs are generally up to 30m and slopes typically less than 5% in gradient. Soils of the Blacktown group typically have moderately reactive and highly plastic subsoil, low soil fertility and poor soil drainage. They usually consist of clayey or silty clay soils and can contain iron indurated gravels, charcoal fragments and tree roots.

It is noted the site sits in close proximity to the Birrong (bg) landscape group which is typically characterised by an alluvial floodplain over Wianamatta group Shales. Local reliefs are generally less than 5m and slopes typically less than 3% in gradient. These soils can be subject to localised flooding, high soil erosion hazard, saline subsoil, seasonal waterlogging and very low soil fertility and are dominated by clay and silt sized alluvial materials.

The Blacktown (bt) and Birrong (bg) landscape group reports are attached in **Appendix H**.

3. SUBSURFACE CONDITIONS AND ASSESSMENT RESULTS

3.1 Stratigraphy

A summary of the surface and subsurface conditions from across the site encountered during the geotechnical investigation is presented in Table 2 below. 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 presented in **Appendix C**.

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

It should be noted that estimated rock strength as well as soil consistency and strength assessed by observation of auger drilling penetration resistance and DCP testing, respectively, are approximate and variation should be expected throughout the site. It is worth noting that auger penetration rates within various bedrock formations vary with different drilling rigs and estimated rock strength variation across the site is expected.

Based on the geotechnical investigation at the selected borehole locations (BH1 to BH3 inclusive), along with our experience and observations made within the site and local region, it is inferred that bedrock of variable estimated strength and weathering underlies majority of the site area at depths between 3.1m and 3.4m bgl.

Fill, soil and rock units may vary between the boreholes and testing locations. Subsurface conditions between the boreholes are inferred to be similar to the subsurface conditions encountered at the borehole locations. However, fill composition, soil type and consistency/strength, and depth to the top of Shale bedrock is expected to vary across the site.

Due to the variable ground conditions throughout the site and the limited investigation, it is recommended that the subsurface materials exposed during excavation and construction, are confirmed by inspection, by an experienced geotechnical engineer. It should also be noted that ground conditions within the site could 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 should be noted that high blow counts encountered during DCP testing may be caused by factors such as gravel and ironstone bands, well consolidated soils and highly cemented sands, or other deleterious materials which may be present within the underlying soils, along with tree rootlets extending throughout the soils from trees and vegetation within the vicinity of the site. These results should be read in conjunction with the detailed engineering borehole logs in **Appendix D** and the subsurface conditions encountered during excavation and construction are confirmed by inspection, by an experienced geotechnical engineer. DCP test results are attached in **Appendix E**.

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Table 2. Summary of Subsurface Conditions

Borehole/DCP ID				**BH1/DCP1**	**BH2/DCP2**	**BH3**
Unit	**Unit Type**	**Description**	**Estimated Consistency/ Strength**	**Depth/ Thickness of Unit (m bgl)**		
Concrete Pavement				0.0 – 0.12	0.0 – 0.1	0.0 – 0.12
1	Fill[1]	Sand and Clayey Sand, fine to medium grained, medium plasticity, with gravel.	N/A	0.12 – 0.6	0.1 – 0.3	0.12 – 0.3
2	Natural Soils[2]	Silty CLAY, medium to high plasticity, with gravel.	Firm	–	0.3 – 0.5	0.3 – 3.1
			Stiff	0.6 – 1.0	0.5 – 0.9	
			Very Stiff	1.0 – 1.2	0.9 – 1.1	
			Hard	1.2 – 3.4	1.1 – 3.1	
3	Bedrock	SHALE, highly to moderately weathered.	VL	3.4 – 4.2	3.1 – 5.7[4]	3.1 – 5.5[4]
			Inferred L (or better)[3]	4.2+	5.7+	5.5+

[1]Thickness of the fill layer is expected to vary from the layer thicknesses indicated in Table 2.
[2]Estimated soil consistency/strength is based on DCP testing to the maximum practical refusal/terminated depths at the selected testing locations within the site. The potential for weak or softer layers throughout the site should be considered.
[3]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 and is to be confirmed by subsequent borehole drilling and rock strength testing, and during construction by inspection.
[4]Occasional EL to VL in layer.

Notes:

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

3.2 Groundwater

No groundwater was encountered or observed during and shortly after (<15 minutes) machine drilling of the boreholes to maximum depths of approximately 5.7m bgl (BH2).

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

Thus, based on the above observations and data available at the time of reporting, groundwater is expected to be in the form of seepage through voids and pore spaces between particles of unconsolidated soil within the fill material, at the fill to natural soil contact, and through networks of fractures and solution openings in consolidated bedrock underlying the site. Seepage may also occur at the natural soil/rock contact.

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

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

As there is the potential for seepage or groundwater to be above bulk excavation level, GCA recommends longer term groundwater monitoring by installation of at least two (2) groundwater monitoring wells at the site (during a more comprehensive investigation). Installing groundwater monitoring wells will provide a more accurate assessment of groundwater levels, contaminant concentrations, permeability (groundwater inflow assessment), and inflow rates and volumes within the site (see also Section 5.7 and Section 6).

4. LABORATORY TEST RESULTS

4.1 Soil Aggressivity and Salinity

Six (6) 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**	**BH1**	**BH2**	**BH2**	**BH3**	**BH3**
Approximate Depth (m bgl)		1.6 – 1.7	3.4 – 3.5	0.7 – 0.8	5.6 – 5.7	1.0 – 1.1	4.2 – 4.3
Strata Type		Natural Soils	Bedrock	Natural Soils	Bedrock	Natural Soils	Bedrock
Aggressivity and Salinity	**pH**	8.1	8.0	6.7	7.9	6.7	7.7
	Moisture Content (%)	13.7	10.9	19.1	8.6	14.9	6.5
	Chloride (mg/kg)	90	150	120	170	70	190
	Sulphate SO_4 (mg/kg)	50	80	30	140	20	140
Electrical Conductivity (µS/cm)	**EC (µS/cm)**	108	70	122	146	86	142
	EC (dS/m)	0.108	0.070	0.122	0.146	0.086	0.142
	Multiplication Factor[1]	8	15	8	15	8	15
	Saturation Extract ECe (dS/m)	0.9	1.1	1.0	2.2	0.7	2.2

[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, subsurface utilities including Sydney Water mains on Waterloo Road and Boronia Road, road reserves and infrastructure that fall within the "zone of influence" in the vicinity of the proposed development. The "zone of influence" is defined as the zone created by drawing a 45° line above horizontal from the boundary at the base of the excavation, into the excavation faces, to the surface.

A dilapidation survey will record the condition of existing defects prior to any works being carried out within the site. Preparation of a dilapidation report should constitute as a "Hold Point".

5.2 General Geotechnical Issues

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

- Aggressivity and salinity assessment.
- Preliminary site lot classification.
- Excavation conditions.
- Vibrations.
- Groundwater management.
- Stability of excavation and retention of adjoining properties and infrastructure.
- Preliminary earthquake site risk 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 Aggressivity and Salinity Assessment

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

Table 4. Aggressivity and Salinity Reference Table

Reference	Element Type	Soil Condition A[1]	Soil Condition B[2]	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	

[1]Soil Condition A refers to high permeability soils or soils that are in groundwater.
[2]Soil Condition B refers to low permeability soils or all soils above groundwater.

When applying Soil Condition B to the site, it is determined that the:

- Underlying natural soils are:
  - Non-aggressive to buried steel structural elements.
  - Non-aggressive to buried concrete structural elements.
  - Electrical conductivity of saturated extract (ECe) ranging from approximately 0.7ds/m to 1.1ds/m, indicating generally non saline natural soils underlying the site.

- Underlying bedrock is:
  - Non-aggressive to buried steel structural elements.
  - Non-aggressive to buried concrete structural elements.
  - Electrical conductivity of saturated extract (ECe) ranging from approximately 1.1ds/m to 2.2ds/m, indicating generally slightly saline bedrock underlying the site.

It should be noted that soil aggressivity and salinity may vary throughout the site and the results are based on testing at the selected boreholes and testing 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 in order to confirm the preliminary findings presented above.

5.4 Inspection Pits and Underpinning

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

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

5.5 Excavation

Maximum inferred excavation depths up to 7.0m are expected to be required for construction of the proposed development, with cut and fill in certain areas of the site. Locally deeper excavations for the proposed lift shafts, building footings and service trenches are also anticipated.

Based on this information and existing ground conditions as encountered during the geotechnical investigation, it is anticipated that excavations will extend through fill material, natural soils and Shale bedrock throughout majority of the proposed development area, as discussed in Section 3 above. Shale bedrock is expected to be exposed at basement bulk excavation level.

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

Therefore, consultation should be made with subcontractors to discuss the feasibility and capability of their machinery for excavation of the proposed development with the existing site conditions.

5.5.1 Excavation Assessment

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

Excavation of medium to high estimated strength bedrock which will most likely be encountered during construction would require higher capacity excavators, bulldozers or similar, for effective removal of the rock. This excavation will require the use of heavy ripping and rock breaking equipment or vibratory rock breaking equipment. Furthermore, excavation for the proposed lift shafts, 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 monitoring of transmitted vibrations to ensure they remain within acceptable limits (refer to Section 5.6 below). Rock saw cutting should be carried out (where required) around the perimeter of excavations, prior to any rock breaking commencing.

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

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

5.6 Vibration Monitoring and Controls

Particular care will be required to ensure that adjacent buildings and infrastructure (i.e. road reserves, buildings, etc.), are not damaged during 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 less than the following maximum Peak Particle Velocity (PPV) for adjacent structures, as outlined in AS 2187.2-2006:

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

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

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

Rock hammering and rock sawing activities 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 excavators are restricted to the equipment and 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.

GCA recommends developing a vibration monitoring plan to monitor construction activities, given the close proximity to adjacent buildings, underground utilities and other infrastructure. The effects on adjoining infrastructure, should be monitored, particularly where excavations are expected to be conducted within the underlying bedrock of higher estimated strength (medium to high to very high) and fall within the "zone of influence" of adjoining or sensitive infrastructure at the site.

A vibration monitoring plan may be carried out attended or unattended. An unattended vibration monitoring system must be fitted with alarms in the form of strobe lights, sirens or live alerts sent to the vibration monitoring supervisor, which are activated when the vibration limit is exceeded. If adopted or considered, consultation should be made with appropriate subcontractors or consultants for the installation of vibration monitoring 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. A geotechnical engineer familiar with the site be contacted. Rock excavation methodology should also consider acceptable noise limits in accordance with the "Interim Construction Noise Guideline" (NSW EPA). It is recommended that dilapidation reports be carried out prior to any demolition, excavation or construction, as discussed in Section 5.1. This should be considered a "Hold Point".

5.7 Groundwater Management

Anticipated groundwater seepage levels within the site are expected to be above the proposed basement FFLs.

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

Based on our experience within the local region and from nearby projects, GCA expects any groundwater inflows into the excavation to be in the form of seepage through voids within the underlying soils and defects in the underlying weathered bedrock (such as bedding planes, joints, etc.). Seepage may also occur within the excavation areas through fill material, and at the fill/natural soils and natural soils/bedrock interfaces, predominantly following heavy rain.

The rate of flow which may enter the excavation may initially be rapid, but is expected to decrease over time as the voids in the natural soils and defects in the underlying bedrock are drained, and as local water ingress decreases. As noted, groundwater levels may be subject to fluctuation 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 proposed development. The amount of seepage into the excavation will also depend on the shoring system being adopted.

GCA recommends that precautionary drainage measures are adopted. These include, but are not limited to:

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

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

A service station occupies the site and contains underground fuel storage tanks. We understand that one (1) of the underground storage tanks contained water, was not VR1 compliant and recommended it was decommissioned. GCA recommends sampling of groundwater at the site, before and during construction, to determine contaminant concentration levels, particularly hydrocarbons and heavy metals. These contaminant concentrations should be reported in accordance with the NSW Environment Protection Authority (EPA) protocols and levels. If contaminant concentrations are excessive or above NSW EPA human health or environmental investigations levels, they pose significant WHS risk to human and ecological receptors. GCA also recommends undertaking an environmental Detailed Site Investigation (DSI) to investigate contamination of soil, soil vapour and groundwater at the site before construction commences.

As there is the potential for seepage or groundwater to be above bulk excavation level, GCA recommends longer term groundwater monitoring by installation of at least two (2) groundwater monitoring wells at the site (during a more comprehensive investigation). Installing groundwater monitoring wells will provide a more accurate assessment of groundwater levels, contaminant concentrations, permeability (groundwater inflow assessment), and inflow rates and volumes within the site (see also Section 6).

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

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

5.8 Excavation Stability

Maximum inferred excavation depths up to 7.0m are expected to be required for construction of the proposed development, with cut and fill in certain areas of the site. Locally deeper excavations for the proposed lift shafts, building footings and service trenches are also anticipated.

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

It is recommended either installation of inclinometers and/or survey monitoring points are performed to measure movements and deflections into the excavation. Inclinometers should be socketed at least 3.0m below the RLs of the base of the shoring piles, to ensure their effectiveness.

5.8.1 Excavation Retention Support Systems

Based on planned excavation depths within the site, assessment of the subsurface conditions and potential for elevated groundwater, and the presence of adjoining properties and infrastructure, it is assessed that the use of temporary or permanent batter slopes are not suitable for the proposed development.

Since batter slopes are considered unsuitable, 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.

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

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

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

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

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

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

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

If considered, the shoring wall (and other retaining walls) can be designed using the recommended design parameters provided in Section 5.8.2. Bulk excavation and foundations (including pile installations) should be supervised, monitored and inspected by a geotechnical engineer, with all structural elements of the development by a structural engineer. Inspections should be considered as “Hold Points” to the project.

5.8.2 Design Parameters (Earth Pressures)

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

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

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

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

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

Lateral active or “at rest” earth pressure:

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

Passive earth pressure:

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

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

  Active earth pressure:

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

Where:

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

- Support systems and retaining structures 'should be designed to withstand hydrostatic pressures, lateral earth pressures and earthquake pressures (if applicable). The applied surcharge loads in their “zone of influence” should also be considered as part of the design.

Support systems designed using the earth pressure approach may be based on the parameters given in Table 6 below for soils and rock horizons underlying the site. Table 6 also provides preliminary coefficients of lateral earth pressure for the soils and rock horizons encountered in the site. These are based on fully drained conditions and that the ground behind the retention walls is horizontal.

Table 6. Preliminary Geotechnical Design Parameters

Material	Fill (Unit 1)	Natural Soils (Unit 2)	Bedrock[3, 5] (Unit 3)	
			VL	L (or better)[6]
Unit Weight (kN/m^3)[4]	16	18	21	22
Effective Cohesion c' (kPa)	0	3 (firm) 5 (stiff, or better)	20	40
Angle of Friction ϕ' (°)	24	26	30	32
Modulus of Elasticity E_{sh} (MPa)	3	10 (firm) 15 (stiff) 20 (very stiff, or better,)	70	160
Earth Pressure Coefficient At Rest Ko[1]	0.59	0.56	0.5	
Earth Pressure Coefficient Active Ka[2]	0.42	0.39	0.33	
Earth Pressure Coefficient Passive Kp[2]	2.37	2.56	3.0	
Poisson Ratio *v*	0.4	0.35	0.3	

[1]Earth pressure coefficient at rest (Ko) can be calculated using 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 or geotechnical engineer.
[4]Above groundwater levels.
[5]Subject to confirmation by a geotechnical engineer by further borehole drilling and rock strength testing, and during construction by inspection.
[6]Conforming to at least Class IV Shale (or better).

Notes:

- VL = Very low estimated strength, L = Low estimated strength,
- VL and L strength 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. (1998).
- 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 should be confirmed prior to construction by further borehole drilling and rock strength testing, and during excavation and construction by inspection.

5.9 Preliminary Earthquake Site Risk Classification

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

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

5.10 Foundations

Following excavation depths to approximate levels of 300mm below the FFLs of the lower basement level, and based on the boreholes carried out within the site, we expect predominantly Unit 3 (bedrock) to be exposed at bulk excavation level (depending on the actual amount of excavation required). Bedrock is expected to be of variable estimated strength and weathering, and should be confirmed prior to construction by further borehole drilling and rock strength testing, and during excavation and construction by inspection. Fill material and natural clayey soils are also expected to be exposed within the north-western corner of the site, where cut and fill is required.

Foundations bearing on fill material or natural soils are likely to result in total and differential settlement under working load, and to 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 could 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 additional borehole drilling with rock coring and rock strength testing, by an experienced geotechnical engineer.

5.10.1 Geotechnical Assessment

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

Shallow foundations should include local slab thickening to support internal walls and columns, with consideration given to settlement reducing piles.

Foundation construction (including piling) should be complemented by inspections carried out by a geotechnical engineer during construction to confirm ground conditions are consistent throughout and the required allowable bearing capacities are achieved. Foundations should not be founded on any soft/weak bands (i.e. clay seams and/or extremely weathered/fractured zones) underlying the site.

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 bulk excavation levels. These piles should be socketed into consistent and appropriate bedrock underlying the site. For cases where resistance against lateral loading induced by earthquakes or winds, and to achieve higher bearing capacities, piles may also be required.

Piles sufficiently socketed into higher strength bedrock may achieve greater allowable bearing capacities, subject to confirmation by a geotechnical engineer 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 7 provides preliminary recommended geotechnical design parameters.

Table 7. Preliminary Recommended Geotechnical Design Parameters

Unit Type/Material		Maximum Allowable (Serviceability) Values (kPa)		
		End Bearing Pressure[1]	**Shaft Adhesion (Compression)**	**Shaft Adhesion (Tension)**
Fill (Unit 1)		N/A	N/A	N/A
Natural Soils (Unit 2)		N/A	N/A	N/A
Bedrock (Unit 3)[2]	**VL**	700	30	15
	L or better[3]	1,000	70	35

[1]Minimum embedment of 0.4m for shallow 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 prior to construction by further borehole drilling and rock strength testing, and during excavation and construction by inspection.
[3]Conforming to at least Class IV Shale (or better).
Notes:

- VL = Very low estimated strength, L = Low estimated strength.
- VL and L strength 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. (1998).
- Higher allowable bearing capacities may be attained for higher estimated strength rock assessed and confirmed by a geotechnical engineer.
- All shaft adhesion parameters are based on adequately clean and rough sockets of category "R2", or better.
- N/A = Not Applicable. Not recommended for the proposed development.
- It is recommended that geotechnical inspections on the foundations are completed by an experienced geotechnical engineer to determine the founding material and confirm the required bearing capacity is achieved.

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

5.10.2 Geotechnical Comments

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

Specific geotechnical advice should be obtained for footing designs and design of the foundation system (shallow and pile foundations) should be carried out in accordance with AS 2870-2011 and AS 2159-2009. It is recommended that reference be made to the CSIRO document "Guide to Home Owners on Foundation Maintenance and Footing Performance", attached as **Appendix G,** to the NCC (National Construction Code) and to Australian Standard AS2870-2011 Residential Slabs and Footings.

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 to help eliminate or minimise surface water infiltration to minimise moisture change within the soils.

GCA recommends that planting of any 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 can cause significant displacement and damage within the building foundations by extensive tree root system movement.

The design and construction of the foundations should take into consideration the potential for flooding and elevated groundwater levels. 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 or placement of reinforcement in the foundations. The base of the foundation excavations should be level prior to pouring concrete or placement of reinforcement in the foundations. Due to the possibility of groundwater being encountered and possible groundwater seepage during installation of bored piles within the site, concrete must be tremie poured from the base of the excavation. It is important to pour concrete without delay to prevent slaking of bedrock once exposed. It is recommended that using a liner in the section of fill material to avoid collapse of the fill/soil. Other alternatives may be the use of Continuous Flight Auger (CFA) piles (including for groundwater control).

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

GCA recommends that geotechnical inspections of foundations be completed by an experienced geotechnical engineer to determine that the design socket materials are reached, the base of the foundation excavation is clean and that the required allowable bearing capacity is achieved. The geotechnical engineer should also note any variations between the subsurface conditions encountered during construction of the piled foundations and those inferred and outlined in this report. 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 of bearing capacities of both deep and shallow foundations, should constitute as a “Hold Point”.

5.11 Subgrade Preparation and Filling

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

5.11.1 Subgrade Preparation

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

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

5.11.2 Filling Specifications

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

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

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

6. ADDITIONAL GEOTECHNICAL RECOMMENDATIONS

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

- Dilapidation survey report on adjacent properties and infrastructure.
- Monitoring and supervision of any excavations within the site.
- Development of a vibration monitoring and control plan, featuring either attended or unattended monitoring.
- The composition, class, depth and estimated strength of the underlying bedrock material should be confirmed prior to construction by further borehole drilling with rock coring and rock strength testing, and during excavation and construction by inspection, by an experienced geotechnical engineer, predominantly in areas and at depths not assessed during the geotechnical investigation.
- Geotechnical inspections of shoring wall pile boring and installation.
- Geotechnical inspections of foundations (shallow and pile foundations) to confirm that the preliminary allowable bearing capacities are achieved.
- Monitoring of any groundwater inflows into the excavation areas within the site.
- Provision for longer term groundwater monitoring within the site by installation of at least two (2) groundwater monitoring wells and period monitoring (during a more comprehensive investigation). Installing groundwater monitoring wells will provide a more accurate assessment of groundwater levels, contaminant concentrations, permeability
- Classification of all excavated material transported from the site.
- If required, a meeting to be carried out to discuss any geotechnical issues and inspection requirements.
- Final architectural and structural design drawings are provided to GCA for further assessment.

7. LIMITATIONS

Geotechnical Consultants Australia Pty Ltd (GCA) has based its geotechnical assessment on available information obtained prior to and during the geotechnical site investigation. The geotechnical assessment and recommendations provided in this report, along with the surface, subsurface and geotechnical conditions are limited to the inspection and test areas during the site 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 is completed.

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

GCA does not accept any liability for any varying site conditions which were not observed, and were out of the inspection or test areas, or were inaccessible during the time of the investigation. This report and any associated information and documentation are prepared solely for **Mr. Mohammed Dahar**, and any misinterpretation of this report or reliance on this report by third parties shall be at their own risk. Any legal or other liabilities resulting from the use of this report by other parties cannot be transferred to GCA.

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

For and behalf of

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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.

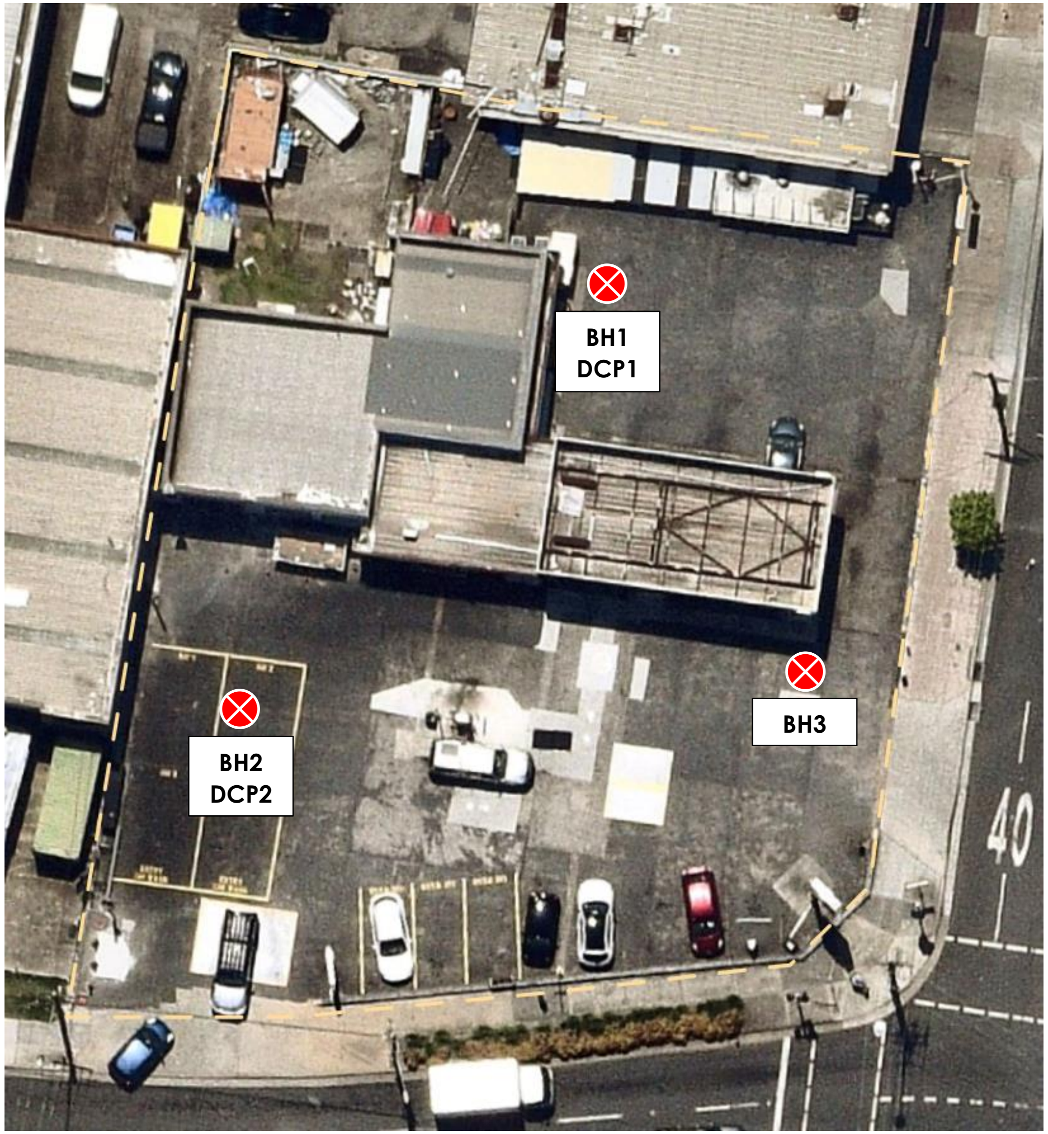
Understanding Limitation Provisions
As some clients, contractors and design professionals do not recognise geotechnical engineering is much broader and less exact than other engineering disciplines, this creates unrealistic expectations that lead to claims, disputs and other disappointments. As part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



GCA Geotechnical Consultants Australia	Figure 1 Site Plan	**Geotechnical Investigation**	Drawn: GN/AS	
		Mohammed Daher	Date: 16/12/2024	
	Job No.: G24395-1	190 Waterloo Road Greenacre NSW 2190	Scale: NTS	

Image source: Nearmap - https://apps.nearmap.com/maps/, accessed 9th December 2024.

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:

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

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

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.

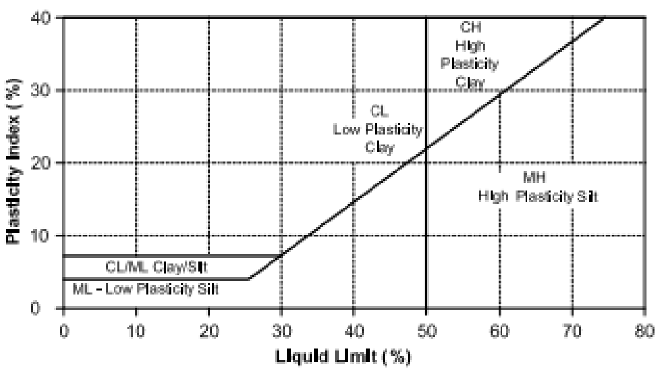
Method and Terms for Soil and Rock Descriptions Used on Borehole and Test Pit Reports

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

ABREVIATIONS FOR DEFECT TYPES AND DECRIPTIONS

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

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

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

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

ROCK SOCKET ROUGHNESS CLASSIFICATION AND CATEGORIES

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

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

APPENDIX D

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(02) 9788 2829

BOREHOLE NUMBER BH1

CLIENT Mohammed Daher
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G24395-1
PROJECT LOCATION 190 Waterloo Road Greenacre NSW 2190

DATE STARTED 6/12/24 **COMPLETED** 6/12/24
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 GN
CHECKED BY JN/RF
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					Concrete Pavement 120mm.		PAVEMENT
						FILL: Sand, fine to medium grained, brown, moist.		FILL
			0.5			FILL: Clayey Sand, fine grained, brown, medium plasticity clay, moist.		
					CH	Silty CLAY, high plasticity, brown, trace of red, trace of fine gravel, moist, estimated stiff.		NATURAL SOILS
			1.0			1.0m bgl. Becoming estimated very stiff.		
						1.2m bgl. Becoming estimated hard.		
			1.5		CI	Silty CLAY, medium plasticity, brown, grey, trace of red brown, with fine to medium gravel, moist.		Occasional high drilling resistance from 1.4m bgl.
							Agg/Sal	
			2.0					
			2.5					
			3.0					
			3.5			SHALE, grey to dark grey, brown, highly to moderately weathered, very low estimated strength, moist.	Agg/Sal	BEDROCK
			4.0			SHALE, grey to dark grey, brown to dark brown, with sand, with ironstone, highly weathered, very low estimated strength, moist.		
						Low estimated strength (or better) at 4.2m bgl. Borehole BH1 terminated at 4.2m		Practical TC bit refusal at 4.2m bgl.
			4.5					
			5.0					

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CLIENT Mohammed Daher
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G24395-1
PROJECT LOCATION 190 Waterloo Road Greenacre NSW 2190

DATE STARTED 6/12/24 **COMPLETED** 6/12/24
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 GN
CHECKED BY JN/RF
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					Concrete Pavement 100mm.		PAVEMENT
						FILL: Sand, fine to medium grained, brown orange, moist.		FILL
			0.5		CI-CH	Silty CLAY, medium to high plasticity, brown to red brown, trace of grey to pale grey, trace of fine gravel, moist, estimated firm. 0.5m bgl. Becoming estimated stiff.	Agg/Sal	NATURAL SOILS
			1.0			0.9m bgl. Becoming estimated very stiff. 1.1m bgl. Becoming estimated hard.		
			1.5					
			2.0		CI-CH	Silty CLAY, medium to high plasticity, brown to brown orange, grey to pale grey, brown red, with fine to medium gravel, moist.		Occasional high drilling resistance from 1.7m bgl.
			2.5					
			3.0					
			3.5			SHALE, brown, grey, trace of clay, trace of sand, highly weathered, very low estimated strength, moist.		BEDROCK
			4.0					
			4.5			SHALE, grey to dark grey, brown, highly weathered, very low estimated strength (occasional extremely low to very low estimated strength), moist.		
			5.0					

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BOREHOLE NUMBER BH2

CLIENT Mohammed Daher

PROJECT NAME Geotechnical Investigation

PROJECT NUMBER G24395-1

PROJECT LOCATION 190 Waterloo Road Greenacre NSW 2190

DATE STARTED 6/12/24 **COMPLETED** 6/12/24 **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** GN **CHECKED BY** JN/RF

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						SHALE, grey to dark grey, brown, highly weathered, very low estimated strength (occasional extremely low to very low estimated strength), moist. *(continued)*		
			5.5			SHALE, grey to dark grey, moderately weathered, very low estimated strength, moist.	Agg/Sal	
						Low estimated strength (or better) at 5.7m bgl. Borehole BH2 terminated at 5.7m		Practical TC bit refusal at 5.7m bgl.
			6.0					
			6.5					
			7.0					
			7.5					
			8.0					
			8.5					
			9.0					
			9.5					
			10.0					

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BOREHOLE NUMBER BH3

CLIENT Mohammed Daher
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G24395-1
PROJECT LOCATION 190 Waterloo Road Greenacre NSW 2190

DATE STARTED 6/12/24 **COMPLETED** 6/12/24
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 GN
CHECKED BY JN/RF
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	Water	RL (m)	Depth (m)	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT	Not Encountered During Drilling				Concrete Pavement 120mm.		PAVEMENT
					FILL: Sand, fine to medium grained, brown orange, moist.		FILL
			0.5	CI-CH	Silty CLAY, medium to high plasticity, grey, red brown, organics, moist.		NATURAL SOILS
			1.0	CI-CH	Silty CLAY, medium to high plasticity, grey to pale grey, brown, with fine to medium gravel, trace of fine grained sand, moist.	Agg/Sal	
			1.5				
			2.0	CI-CH	Silty CLAY, medium to high plasticity, brown, grey to pale grey, with fine to medium gravel, moist.		
			2.5				
			3.0		SHALE, brown, trace of clay, highly weathered, very low estimated strength, moist.		BEDROCK
			3.5				
			4.0		SHALE, grey to dark grey, trace of sand, moderately weathered, very low estimated strength, moist.	Agg/Sal	
			4.5		SHALE, dark grey, moderately weathered, very low estimated strength (occasional extremely low to very low estimated strength), moist.		
			5.0				

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BOREHOLE NUMBER BH3

CLIENT Mohammed Daher
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G24395-1
PROJECT LOCATION 190 Waterloo Road Greenacre NSW 2190

DATE STARTED 6/12/24 **COMPLETED** 6/12/24
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 GN
CHECKED BY JN/RF
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			SHALE, dark grey, moderately weathered, very low estimated strength (occasional extremely low to very low estimated strength), moist. *(continued)*		
						Low estimated strength (or better) at 5.5m bgl. Borehole BH3 terminated at 5.5m		Practical TC bit refusal at 5.5m bgl.
			6.0					
			6.5					
			7.0					
			7.5					
			8.0					
			8.5					
			9.0					
			9.5					
			10.0					

APPENDIX E

DYNAMIC CONE PENETROMETER RESULTS

Client:	Mohammed Daher	Test Date:	5/12/2024
Address:	190 Waterloo Road Greenacre NSW 2190	Job No.:	G24395-1

	DCP No.					DCP No.			
Surface	-	-			*Surface*				
Depths (mm bgl)	1	2			Depths (mm bgl)				
0-100	CC 120mm	CC 100mm			0-100				
100-200		2			100-200				
200-300	1	2			200-300				
300-400		2			300-400				
400-500	2	2			400-500				
500-600	2	3			500-600				
600-700	3	4			600-700				
700-800	4	5			700-800				
800-900	6	7			800-900				
900-1000	7	11			900-1000				
1000-1100	10	12			1000-1100				
1100-1200	9	16			1100-1200				
1200-1300	17	16			1200-1300				
1300-1400	20	20			1300-1400				
1400-1500	**Bouncing**	24			1400-1500				
1500-1600		22			1500-1600				
1600-1700		25			1600-1700				
1700-1800		30			1700-1800				
1800-1900		**Terminated**			1800-1900				
1900-2000					1900-2000				
2000-2100					2000-2100				
2100-2200					2100-2200				
2200-2300					2200-2300				
2300-2400					2300-2400				
2400-2500					2400-2500				
2500-2600					2500-2600				
2600-2700					2600-2700				
2700-2800					2700-2800				
2800-2900					2800-2900				
2900-3000					2900-3000				
3000-3100					3000-3100				
3100-3200					3100-3200				
3200-3300					3200-3300				
3300-3400					3300-3400				
3400-3500					3400-3500				
3500-3600					3500-3600				
3600-3700					3600-3700				
3700-3800					3700-3800				
3800-3900					3800-3900				
3900-4000					3900-4000				

Notes:
- DCP1 at BH1 location.
- DCP2 at BH2 location.

CC = Concrete

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

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Tested:	GN	©Geotechnical Consultants Australia Pty Ltd	Sheet:	1 of 1

APPENDIX F

CERTIFICATE OF ANALYSIS

Work Order	: **ES2439856**	Page	: 1 of 4
Client	: **GEOTECHNICAL CONSULTANTS AUSTRALIA PTY LTD**	Laboratory	: Environmental Division Sydney
Contact	: JOE NADER	Contact	: Customer Services ES
Address	: 2 HAROLD STREET PARRAMATTA NSW 2150	Address	: 277-289 Woodpark Road Smithfield NSW Australia 2164
Telephone	: ----	Telephone	: +61-2-8784 8555
Project	: G24395-1	Date Samples Received	: 05-Dec-2024 15:10
Order number	: ----	Date Analysis Commenced	: 09-Dec-2024
C-O-C number	: ----	Issue Date	: 16-Dec-2024 08:53
Sampler	: GEORGE N		
Site	: 190 Waterloo Road Greenacre NSW 2190		
Quote number	: EN/333		
No. of samples received	: 6		
No. of samples analysed	: 6		

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	Senior Chemist - Inorganics	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 Contract 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.

- ED045G: The presence of Thiocyanate, Thiosulfate and Sulfite can positively contribute to the chloride result, thereby may bias results higher than expected. Results should be scrutinised accordingly.

Analytical Results

Sub-Matrix: SOIL (Matrix: SOIL)			Sample ID	BH1 1.6m-1.7m	BH1 3.4m-3.5m	BH2 0.7m-0.8m	BH2 5.6m-5.7m	BH3 1.0m-1.1m
			Sampling date / time	05-Dec-2024 00:00	05-Dec-2024 00:00	05-Dec-2024 00:00	05-Dec-2024 00:00	05-Dec-2024 00:00
Compound	CAS Number	LOR	Unit	ES2439856-001	ES2439856-002	ES2439856-003	ES2439856-004	ES2439856-005
				Result	Result	Result	Result	Result
EA002: pH 1:5 (Soils)								
pH Value	----	0.1	pH Unit	**8.1**	**8.0**	**6.7**	**7.9**	**6.7**
EA010: Conductivity (1:5)								
Electrical Conductivity @ 25°C	----	1	µS/cm	**108**	**70**	**122**	**146**	**86**
EA055: Moisture Content (Dried @ 105-110°C)								
Moisture Content	----	1.0	%	**13.7**	**10.9**	**19.1**	**8.6**	**14.9**
ED040S : Soluble Sulfate by ICPAES								
Sulfate as SO4 2-	14808-79-8	10	mg/kg	**50**	**80**	**30**	**140**	**20**
ED045G: Chloride by Discrete Analyser								
Chloride	16887-00-6	10	mg/kg	**90**	**150**	**120**	**170**	**70**

Analytical Results

Sub-Matrix: SOIL
(Matrix: SOIL)

			Sample ID	Bh3 4.2m-4.3m	----	----	----	----
			Sampling date / time	05-Dec-2024 00:00	----	----	----	----
Compound	CAS Number	LOR	Unit	ES2439856-006	--------	--------	--------	--------
				Result	----	----	----	----
EA002: pH 1:5 (Soils)								
pH Value	----	0.1	pH Unit	**7.7**	----	----	----	----
EA010: Conductivity (1:5)								
Electrical Conductivity @ 25°C	----	1	µS/cm	**142**	----	----	----	----
EA055: Moisture Content (Dried @ 105-110°C)								
Moisture Content	----	1.0	%	**6.5**	----	----	----	----
ED040S : Soluble Sulfate by ICPAES								
Sulfate as SO_4 2-	14808-79-8	10	mg/kg	**140**	----	----	----	----
ED045G: Chloride by Discrete Analyser								
Chloride	16887-00-6	10	mg/kg	**190**	----	----	----	----

QUALITY CONTROL REPORT

Work Order	: **ES2439856**	Page	: 1 of 3
Client	: **GEOTECHNICAL CONSULTANTS AUSTRALIA PTY LTD**	Laboratory	: Environmental Division Sydney
Contact	: JOE NADER	Contact	: Customer Services ES
Address	: 2 HAROLD STREET PARRAMATTA NSW 2150	Address	: 277-289 Woodpark Road Smithfield NSW Australia 2164
Telephone	: ----	Telephone	: +61-2-8784 8555
Project	: G24395-1	Date Samples Received	: 05-Dec-2024
Order number	: ----	Date Analysis Commenced	: 09-Dec-2024
C-O-C number	: ----	Issue Date	: 16-Dec-2024
Sampler	: GEORGE N		
Site	: 190 Waterloo Road Greenacre NSW 2190		
Quote number	: EN/333		
No. of samples received	: 6		
No. of samples analysed	: 6		

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	Senior Chemist - Inorganics	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
* = The final LOR has been raised due to dilution or other sample specific cause; adjusted LOR is shown in brackets. The duplicate ranges for Acceptable RPD% are applied to the final LOR where applicable.

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: 6243815)									
ES2439843-010	Anonymous	EA002: pH Value	----	0.1	pH Unit	6.1	6.1	0.0	0% - 20%
ES2439835-001	Anonymous	EA002: pH Value	----	0.1	pH Unit	6.1	6.2	0.0	0% - 20%
EA010: Conductivity (1:5) (QC Lot: 6243818)									
ES2439843-010	Anonymous	EA010: Electrical Conductivity @ 25°C	----	1	µS/cm	39	32	19.8	0% - 20%
ES2439835-001	Anonymous	EA010: Electrical Conductivity @ 25°C	----	1	µS/cm	8	10	12.1	No Limit
EA055: Moisture Content (Dried @ 105-110°C) (QC Lot: 6243823)									
ES2439835-001	Anonymous	EA055: Moisture Content	----	0.1	%	7.4	8.3	12.0	0% - 20%
ES2439843-008	Anonymous	EA055: Moisture Content	----	0.1 (1.0)*	%	12.0	10.1	16.6	0% - 50%
ED040S: Soluble Major Anions (QC Lot: 6243816)									
ES2439856-002	BH1 3.4m-3.5m	ED040S: Sulfate as SO4 2-	14808-79-8	10	mg/kg	80	80	0.0	No Limit
ES2439857-007	Anonymous	ED040S: Sulfate as SO4 2-	14808-79-8	10	mg/kg	20	10	0.0	No Limit
ED045G: Chloride by Discrete Analyser (QC Lot: 6243817)									
ES2439835-001	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
EA002: pH 1:5 (Soils) (QCLot: 6243815)								
EA002: pH Value	----	----	pH Unit	----	4 pH Unit	101	98.8	101
				----	7 pH Unit	100	98.8	101
EA010: Conductivity (1:5) (QCLot: 6243818)								
EA010: Electrical Conductivity @ 25°C	----	1	µS/cm	<1	1412 µS/cm	98.5	92.0	108
ED040S: Soluble Major Anions (QCLot: 6243816)								
ED040S: Sulfate as SO4 2-	14808-79-8	10	mg/kg	<10	750 mg/kg	105	80.0	120
ED045G: Chloride by Discrete Analyser (QCLot: 6243817)								
ED045G: Chloride	16887-00-6	10	mg/kg	<10	250 mg/kg	99.7	75.0	125
				<10	5000 mg/kg	102	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: 6243817)							
ES2439835-001	Anonymous	ED045G: Chloride	16887-00-6	1250 mg/kg	118	70.0	130

APPENDIX G

Foundation Maintenance and Footing Performance: A Homeowner's Guide

BTF 18 replaces Information Sheet 10/91

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

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

Soil Types

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

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

Causes of Movement

Settlement due to construction

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

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

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

Erosion

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

Saturation

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

Seasonal swelling and shrinkage of soil

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

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

Shear failure

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

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

GENERAL DEFINITIONS OF SITE CLASSES

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

Tree root growth

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

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

Unevenness of Movement

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

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

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

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

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

Effects of Uneven Soil Movement on Structures

Erosion and saturation

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

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

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

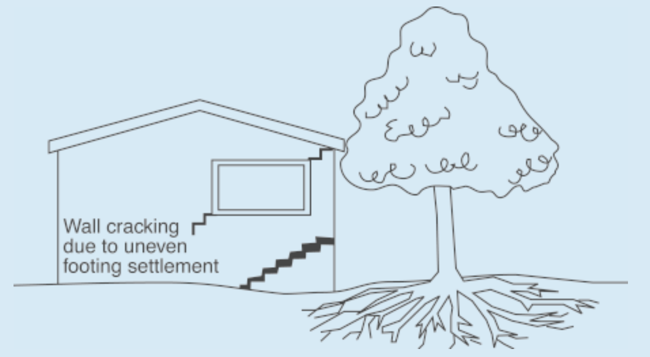
Seasonal swelling/shrinkage in clay

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

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

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

Trees can cause shrinkage and damage



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

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

Movement caused by tree roots

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

Complications caused by the structure itself

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

Effects on full masonry structures

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

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

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

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

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

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

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

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

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

Water Service and Drainage

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

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

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

Seriousness of Cracking

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

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

Prevention/Cure

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

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

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

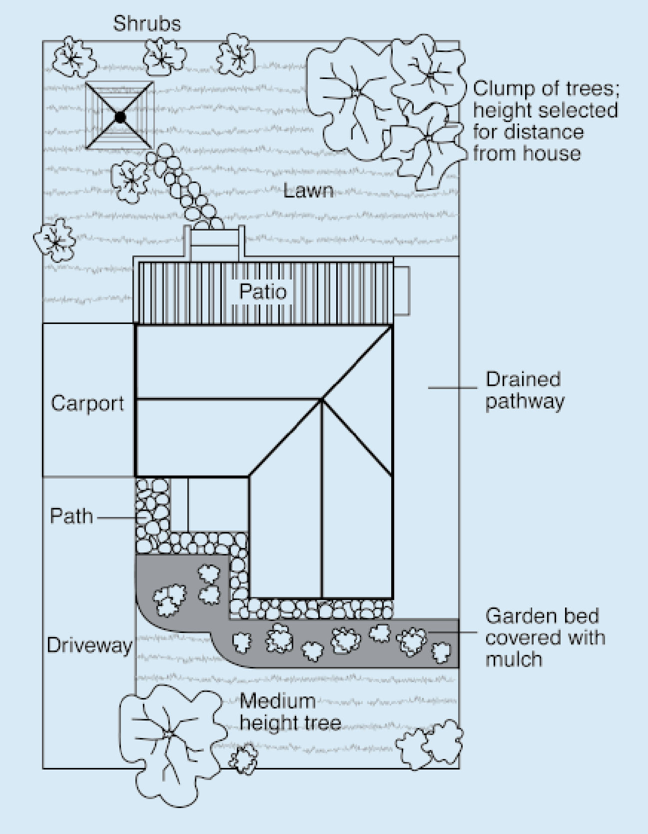
Protection of the building perimeter
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

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

CLASSIFICATION OF DAMAGE WITH REFERENCE TO WALLS

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

Gardens for a reactive site



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

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

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

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

Condensation

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

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

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

The garden

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

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

Existing trees

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

Information on trees, plants and shrubs

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

Excavation

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

Remediation

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

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

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APPENDIX H

bt	BLACKTOWN	Residual

Source: Soil Landscapes of the Penrith 1:100,000 Sheet *report*

Landscape—gently undulating rises on Wianamatta Group shales and Hawkesbury shale. Local relief to 30 m, slopes are usually <5%. Broad rounded crests and ridges with gently inclined slopes. Cleared eucalypt woodland and tall open-forest (wet sclerophyll forests).

Soils—shallow to moderately deep (<100 cm) Red and Brown Podzolic Soils (Dr3.21, Dr3.11, Db2.11) on crests, upper slopes and well-drained areas; deep (150–300 cm) Yellow Podzolic Soils and Soloths (Dy2.11, Dy3.11) on lower slopes and in areas of poor drainage.

Limitations—moderately reactive highly plastic subsoil, low soil fertility, poor soil drainage.

LOCATION

Occurs extensively on the Cumberland Lowlands between the Georges and Parramatta Rivers in the south-west. Examples include Strathfield, Auburn and Belmore. Isolated examples are found north of Parramatta River on the Hornsby Plateau at Chatswood, Crows Nest, Duffys Forest, Dundas, Naremburn, Neutral Bay, St. Ives and St. Leonards.

LANDSCAPE

Geology

Wianamatta Group— Ashfield Shale consisting of laminite and dark grey siltstone and Bringelly Shale which consists of shale, with occasional calcareous claystone, laminite and coal.

This unit is occasionally underlain by claystone and laminite lenses within the Hawkesbury Sandstone such as at Duffys Forest.

Topography

Gently undulating rises on Wianamatta Shale with local relief 10–30 m and slopes generally <5%, but up to 10%. Crests and ridges are broad (200–600 m) and rounded with convex upper slopes grading into concave lower slopes. Rock outcrop is absent.

Vegetation

Almost completely cleared tall open-forest (wet sclerophyll forest) and open-woodland (dry sclerophyll forest). Remaining traces of the original wet sclerophyll forest containing Sydney blue gum *Eucalyptus saligna* and blackbutt *E. pilularis* are located at Ashfield Park. The original woodland and open-forest in drier areas to the west were dominated by forest red gum *E. tereticornis*, narrow-leaved ironbark *E. crebra* and grey box *E. moluccana*. This has been almost completely cleared. At Duffys Forest there is an open-forest dominated by ash *E. sieberi* with a dry sclerophyll shrub understorey.

Land use

The dominant land uses are intensive residential and light and heavy industry. Examples of residential areas include Newtown, Petersham, Strathfield and Belmore. Examples of industrial areas include Enfield, Lidcombe and Clyde.

Existing Erosion

No appreciable erosion occurs on this unit as most of the surface is covered by tiles, concrete, bitumen or turf.

Associated Soil Landscapes

Birrong (**bg**) soil landscape occurs along drainage depressions.

SOILS

Dominant Soil Materials

bt1—Friable brownish-black loam. This is a friable brownish-black loam to clay loam with moderately pedal sub-angular blocky structure and rough-faced porous ped fabric. This material occurs as topsoil (A1 horizon). Peds are well defined sub-angular blocky and range in size from 2–20 mm. Surface condition is friable. Colour is commonly brownish-black (10YR 2/2) but can range from dark reddish-brown (5YR 3/2) to dark yellowish-brown (10YR 3/4). The pH ranges from slightly 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 hardsetting brown clay loam to silty clay loam with apedal massive to weakly pedal structure and slowly porous earthy fabric. It commonly occurs as an A2 horizon. Peds when present are weakly developed, sub-angular blocky and are rough faced and porous. They range in size between 20–50 mm. Colour is commonly 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 ranges from moderately acid (pH 5.0) to slightly acid (pH 6.5). Platy ironstone 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 subangular-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 usually brown (7.5YR 4/6) but may range from reddish-brown (2.5YR 4/6) to brown (10YR 4/6). Red, yellow or grey mottles are commonly present and often become more numerous with depth. The pH ranges from strongly acid (pH 4.5) to slightly acid (pH 6.5). Fine to coarse gravel-sized shale fragments are common and widespread and often occur in stratified bands. Both roots and charcoal fragments are rare.

bt4—Light grey plastic mottled clay. This is plastic light grey silty clay to heavy clay with moderately pedal polyhedral to sub-angular blocky structure and smooth-faced 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 (2.5Y 6/2). Red, yellow or grey mottles are common. The pH ranges 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 100 cm of strongly pedal, brown mottled light clay (**bt3**) [Red Podzolic Soils (Dr 3.21, 3.11) and Brown Podzolic Soils (Db 2.11)]. **bt1** material 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 **bt3**. This in turn overlies up to 100 cm of light grey plastic mottled clay (**bt4**). Occasionally the **bt1** material is absent. The boundaries between the soil materials are usually clear. Total soil depth is <200 cm [Red Podzolic Soils (Dr 3.21), Brown Podzolic Soils (Db 2.21)].

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**. The boundaries between the soil materials are clear. Total soil depth is >200 cm [Yellow Podzolic Soils (Dy 2.11, Dy 3.11)].

LIMITATIONS TO DEVELOPMENT

Urban Capability

High capability for urban development with appropriate foundation design.

Rural Capability

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

Landscape Limitations

Moderately reactive soil
Seasonal waterlogging

Soil Limitations

bt1 Low wet strength
High organic matter
Low fertility
Sodicity (localised)
Strongly acid

bt2 Low wet strength
Hardsetting
Low fertility
Sodicity (localised)
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
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)
Very low fertility
Strongly acid
Very high aluminium toxicity
High erodibility (localised)

Fertility

General fertility is low to very low. 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

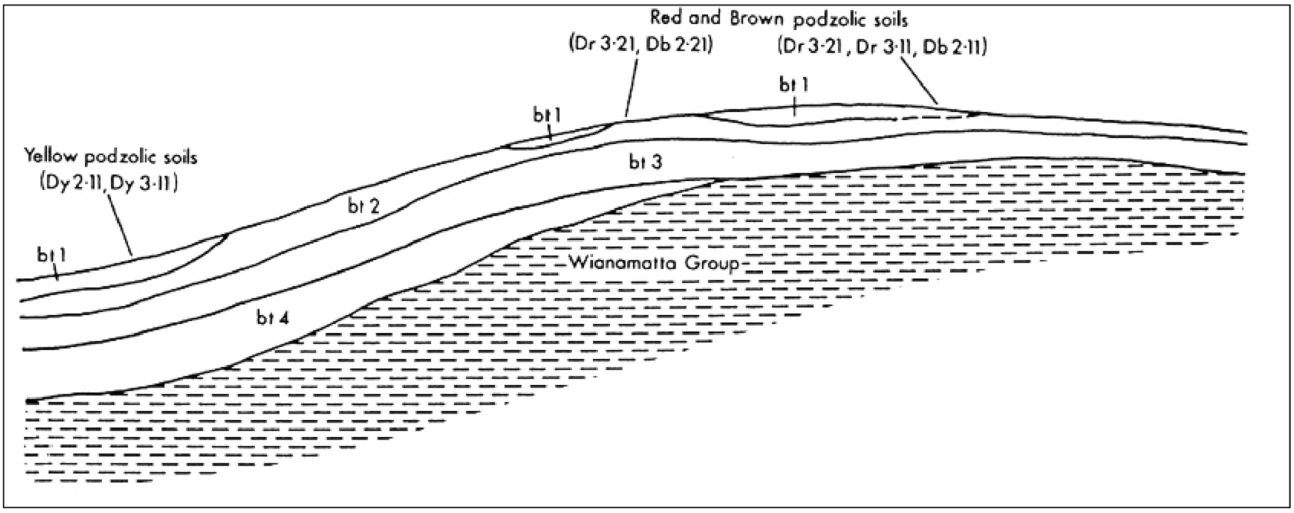
Blacktown soil materials have moderate erodibility. The topsoils (**bt1**, **bt2**) are often hardsetting and they have high fine sand and silt content, but they also have high to moderate organic matter content. The subsoils (**bt3**, **bt4**) are very low in organic matter. Where they are also highly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

Erosion Hazard

The erosion hazard for non-concentrated flows is generally moderate, but ranges from low to very high. Calculated soil loss during the first twelve months of urban development ranges up to 73 t/ha for topsoil and 68 t/ha for exposed subsoil. 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 sideslopes and footslopes. Shallower soils on crests are slightly reactive.



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

bg	BIRRONG	Fluvial

Source: Soil Landscapes of the Penrith 1:100,000 Sheet *report*

Landscape—level to gently undulating alluvial floodplain draining Wianamatta Group shales. Local relief to 5 m, slopes <3%. Broad valley flats. Extensively cleared tall open-forest and woodland.

Soils—deep (>250 cm) Yellow Podzolic Soils (Dy2.42, Dy3.12) and Yellow Solodic Soils (Dy3.42) on older alluvial terraces; deep (>250 cm) Solodic Soils (Dy3.42) and Yellow Solonetz (Dy3.43) on current floodplain.

Limitations—localised flooding, high soil erosion hazard, saline subsoil, seasonal waterlogging, very low soil fertility.

LOCATION

Floodplains of watercourses draining Wianamatta Group shales, on the Cumberland Lowlands. Examples include the catchments of Duck Creek (Sefton, Auburn, Birrong and Yagoona), Haslams Creek (Lidcombe), Upper Cooks River (Chullora, Greenacre, Campsie, Belmore and Beverly Hills) and Salt Pan Creek (Bankstown, Punchbowl and Riverwood). The lower reaches of Subiaco Creek at Rydalmere and parts of the Parramatta River flood plain at Parramatta are also included.

LANDSCAPE

Geology

Dominated by silt and clay sized alluvial materials derived from the Wianamatta Group. (The Wianamatta Group consists mostly of shale with some carbonaceous claystone, laminite, and occasional fine to medium grain lithic sandstones.)

Topography

Level to gently undulating alluvial floodplains with local relief <5m and slope gradients <3%. Broad concave valleys. Most drainage lines have been converted to lined concrete and brick channels.

Vegetation

Extensively cleared. Small relict stands of ironbark *Eucalyptus paniculata*, turpentine *Syncarpia glomulifera*, and Sydney blue gum *E. saligna* forest and woodland are present.

Land use

Major land use is recreation (playing fields parks and reserves). Landfill has occurred in many areas. Isolated locations such as Chullora have been developed for industry. Other areas, such as Campsie, are used for residential purposes. A small proportion of the land is vacant or used for horse grazing.

Existing Erosion

Most drainage lines have been artificially lined with concrete preventing most erosion. Minor streambank erosion has occurred along remaining natural drainage lines.

Associated Soil Landscapes

Lower slopes of Blacktown soil landscape (**bt**) adjoin and occasionally overlap the Birrong soil landscape.

SOILS

Dominant Soil Materials

bg1—Dark brown pedal silty clay loam. This is a dark brown silt loam or silty clay loam with moderately pedal structure and rough ped fabric. It generally occurs as topsoil (A1 horizon).

Texture can range from loam to silty clay loam with fine sand and silt often being present. Peds are blocky and large (100–200 mm). These large peds readily break down to small (<2–10 mm) crumb and polyhedral shaped peds. Colour is commonly dark brown (10YR 3/3), brown (7.5YR 4/4) or brownish-black (10YR 2/2). The pH is usually slightly acid (pH 6.0). Roots are common to abundant, but stones and charcoal fragments are absent.

bg2—Bleached hardsetting clay loam. This is a bleached, clay loam to fine sandy clay loam with weakly pedal to apedal massive structure that is hardsetting when dry. This material occurs as an A2 horizon.

This material contains large amounts of silt and fine sand. Peds when present are rough faced, angular blocky and range in size from 100–200 mm. Otherwise an earthy, apedal massive structure is present. Colour ranges from dull yellowish-brown (10YR 5/3) to greyish yellow-brown (10YR 6/2) when moist to bleached, light grey (10YR 7/1) when dry. Pores and faunal casts are common. The pH varies from moderately acid (pH5.5) to slightly acid (pH6.5). A few small rounded, ironstone nodules are present. There are few roots and charcoal fragments are absent.

bg3—Orange mottled silty clay. This is an orange mottled fine sandy clay loam to silty clay with moderately pedal structure and smooth-faced dense ped fabric. It occurs as subsoil (B horizon).

Fine sand is commonly present in large amounts. Peds are smooth-faced, prismatic, angular blocky and range in size from 50–100 mm. Colour is orange (7.5YR 6/6) with dark orange mottles. This material is hard and brittle when dry and very sticky when wet. The pH varies from moderately acid (pH 5.5) to slightly acid (pH 6.5). Stones, roots and charcoal fragments are absent.

bg4—Brown mottled clay. This is commonly brown, mottled medium clay with strongly pedal structure and dense, rough-faced ped fabric. It occurs as deep subsoil (B horizon).

Texture can range from light to heavy clay. Peds are large (20–100 mm) and prismatic or angular blocky and readily break down to smaller (10–20 mm) polyhedral peds. Peds are mostly dense and smooth-faced with occasional cutans. Colour varies from brown (7.5YR 4/4, 10YR 4/6) to greyish-brown (7.5YR 6/2). Orange, red or grey mottles are often present. The pH varies from strongly acid (pH 4.5) to slightly acid (pH 5.5). Stones, charcoal and roots are rare.

bg5—Light grey mottled saline clay. This is light grey, mottled, light medium clay to heavy clay with moderately to strongly pedal structure and dense smooth ped fabric. It is saline and occurs as deep subsoil (C or D horizon) overlying bedrock.

Peds are dense, smooth-faced, range in size from 50–100 mm and are sub-angular blocky. Colour is predominantly light grey (10YR 7/1) and ranges from bright yellowish-brown (10YR 6/6) to light grey (10YR 8/2). Prominent dark red mottles are common. These occupy up to 40% of the material and form reticulated patterns. The red mottles often contain iron concretions that harden on exposure. The pH ranges from strongly acid (pH 4.5) to slightly acid (pH 6.0). Stones and roots are usually absent.

Associated Soil Materials

bt3—yellowish-brown strongly structured light clay. Occurs as subsoil on the lower slopes (see Blacktown soil landscape **bt3** soil material).

Occurrence and Relationships

Floodplains and drainage lines. Generally, 10–40 cm of dark brown pedal silty clay loam (**bg1**) overlies up to 35 cm of bleached hardsetting clay loam (**bg2**) and >100 cm of orange, mottled silty clay (**bg3**) and brown mottled clay (**bg4**). **bg3** usually occurs as vertical columns or pipes within **bg4**. There is a gradual change to light grey mottled salty heavy clay (**bg5**). **bg5** often overlies bedrock. Total soil depth is >200 cm. The boundaries between the soil materials are clear except for the gradual **bg4**–**bg5** boundary (Yellow Solodic Soils (Dy3.42), Yellow Solonetzic Soils (Dy3.43)).

Lower slopes and edges of floodplains. Generally, 3–30 cm of **bg1** sharply overlies 10–40 cm of **bg2** and 30–50 cm of associated soil material (**bt3**) which grades into >50 cm of **bg4** (Yellow Podzolic Soil, Yellow Solodic Soil (Dy2.42)).

In many instances, especially in very poorly drained areas, associated soil material **bt3** is absent (Yellow Podzolic Soil (Dy3.12)).

Flatter areas, especially those closer to the current floodplain, commonly have well developed, bleached (**bg2**) layers (Yellow Podzolic Soil, Yellow Solodic Soil (Dy3.42)).

LIMITATIONS TO DEVELOPMENT

Urban Capability

Despite extensive existing development this landscape is generally not capable of urban development without extensive drainage works and soil amelioration.

Rural Capability

Many areas have been urbanised. Generally the landscape is not capable of cultivation and only localised areas are capable of being grazed.

Landscape Limitations

Flood hazard
Seasonal waterlogging
Water erosion hazard

Soil Limitations

bg1 Very low fertility

bg2 Erodibility
Low available water capacity
Very hardsetting surface
Low fertility
Salinity (localised)
Low wet strength

bg3 Low wet strength
Low permeability
Low available water capacity
High erodibility
Salinity
Sodicity
Very hardsetting surface
Very low fertility

bg4 Low wet strength
Low permeability
Low available water capacity
Salinity
Strongly sodic
Hardsetting surface
Very low fertility

bg5 Low wet strength
Low permeability
Low available water capacity
Salinity
Strongly sodic
Hardsetting surface
Very low fertility

Fertility

bg1 topsoil has moderate fertility with high organic matter, moderate CEC, moderate available water capacity and moderate nutrient status. All the other soil materials (**bg2–bg5**) have low to very low fertility. They have low to very low nutrient status, low available water capacity, are often saline and have very poor structure.

Erodibility

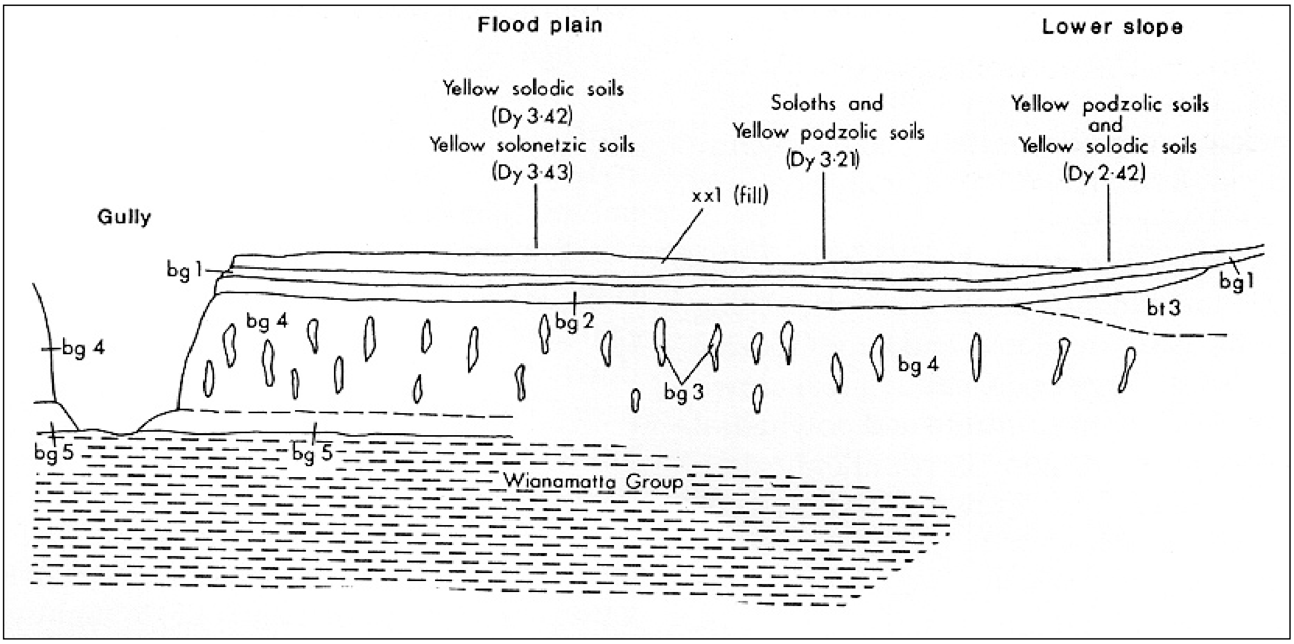
The erodibility of **bg1** is low as it consists of stable soil aggregates bound together with organic matter and clays. **bg2** and **bg3** are highly erodible as they are low in organic matter and contain a high proportion of fine sand and silt. **bg3–bg5** clays are often highly dispersible and are highly to extremely erodible in this condition.

Erosion Hazard

Generally, the erosion hazard for non-concentrated flows ranges from low to moderate. Calculated soil loss for the first twelve months of urban development ranges up to 11 t/ha of topsoil and 28 t/ha of exposed subsoil.

Surface Movement Potential

Moderately to slightly reactive. Soils are deep and have high clay content. Potential movements are offset by poor drainage and high concentrations of salt that tend to reduce soil moisture changes.



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