GCA

Geotechnical Consultants Australia

Dana Bina Pty Ltd &

Midpoint Investments Pty Ltd

Geotechnical Investigation Report

Proposed Development at:

31 Santley Cresent & 2A Bringelly Road

Kingswood NSW 2747

G21551-1

29th September 2021

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Report Distribution

Geotechnical Investigation Report

Address: 31 Santley Crescent & 2A Bringelly Road Kingswood NSW 2747

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1. INTRODUCTION

1.1 Background

This geotechnical engineering report presents the results of a geotechnical investigation undertaken by Geotechnical Consultants Australia Pty Ltd (GCA) for a proposed development at No. 31 Santley Crescent and No. 2A Bringelly Road Kingswood NSW 2747 (the site). The investigation was commissioned by Dana Bina Pty Ltd & Midpoint Investments Pty Ltd (the client) and was carried out on the 13th September 2021.

The purpose of the investigation was to assess the subsurface conditions over the site at the selected borehole testing locations (where accessible and feasible), and provide necessary recommendations from a geotechnical perspective for the proposed 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 has been prepared to provide geotechnical advice and recommendations to assist in the preparation of preliminary designs and construction of the ground structures for the proposed development.

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

1.2 Proposed Development

Information provided by the client indicates the proposed development comprises demolition of the existing infrastructures onsite, followed by construction of a multi-storey boarding house building, overlying two (2) to three (3) basement levels.

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

- Basement 3 level: RL34.50m to RL34.80m Australian Height Datum (AHD).
- Basement 2 level: RL37.50m to RL37.80m AHD.
- Basement 1 level: RL40.50m to RL40.80m AHD.
- Ground floor level: RL43.50m to RL44.45m AHD.

Based on this information and the existing site levels and topography, maximum excavation depths varying from approximately 6.5m to 9.7m (varying throughout) are expected to be required for construction of the proposed development. Locally deeper excavations for the proposed lift shafts, and building footings and service trenches are also projected to be required as part of the planned development.

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

1.3 Provided Information

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

- Architectural drawings prepared by Gus Fares Architects, titled "Proposed Boarding House at 31 Santley Crescent & 2A Bringelly Rd Kingswood NSW", referenced project No. 2020-22 and included drawing nos. A001, A101, to A106 inclusive, A202 and A302.

1.4 Geotechnical Assessment Objectives

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

- General assessment of any potential geotechnical issues that may affect any surrounding infrastructures, buildings, council assets, etc., along with the proposed development.
- Excavation conditions and recommendations on excavation methods in soils and rocks to restrict any ground vibrations.
- Recommendations on suitable shoring (retention) systems for the site.
- Design parameters based on the ground conditions within the site, for retaining walls, cantilever shoring walls and propped shoring.
- Recommendations on suitable foundation types and design for the site.
- End bearing capacities and shaft adhesion for shallow and deep foundations based on the ground conditions within the site (for ultimate limit state and serviceability loads).
- Groundwater levels which may be determined during the geotechnical investigation and during an additional site visit for groundwater level measurements, along with the effects on the proposed development construction.
- Recommendations on groundwater maintenance and limiting.
- Preliminary subsoil class for earthquake design for the site in accordance with Australian Standards (AS) 1170.4-2007.
- Preliminary aggressivity and salinity assessment within the site based on laboratory testing results at the selected borehole locations.
- General geotechnical advice on site preparation, filling and subgrade preparation.

1.5 Scope of Works

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

- Submit and review Dial Before You Dig (DBYD) plans and any other plans provided by the client on existing buried services within the site.
- Service locating carried out using electromagnetic detection equipment to ensure the area is free of any underground services at the selected borehole testing locations.
- Review of site plans and drawings to determine appropriate testing locations (where accessible and feasible), and identify any relevant features of the site.
- Machine drilling of three (3) boreholes at selected locations within the site (where accessible and feasible) by a specialised track mounted 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 7.0m to 7.7m below the existing ground level within the site (bgl).
    - Following auger termination in borehole BH1, drilling commenced using NMLC diamond coring technique to the final depth outlined in Table 1 below.
- Installation of one (1) standpipe piezometer, identified as GW1 and installed to a depth of approximately 14.1m bgl (RL30.4m AHD) in borehole BH1. The standpipe piezometer was installed for groundwater measurements and any future groundwater monitoring which may be required.
    - The approximate locations of the boreholes and standpipe piezometer are shown on **Figure 1, Appendix B** of this report
- Collection of soil and rock samples during drilling for the following laboratory testing required:

- Laboratory testing by a National Association of Testing Authorities, Australia (NATA) accredited laboratory (ALS Environmental) on four (4) selected samples collected during drilling of the boreholes to determine the pH, chloride and sulphate content, and electrical conductivity of the selected samples. Laboratory analysis was undertaken for the purpose of a preliminary aggressivity and salinity assessment within the site.
- Rock cores recovered from borehole BH1 were boxed, logged and sent to our affiliate NATA accredited laboratory, Geologic Solutions Group Pty Ltd (Geologic Solutions), for rock strength testing to estimate the point load strength index (Is_{50}) values. The rock core photographs and laboratory point load test results certificates are presented in **Appendix E** and **Appendix F**, respectively.

- Preparation of this geotechnical engineering report.

Table 1. Approximate Borehole Drilling Depths

Borehole ID	Augering Depth/Thickness (m bgl)	NMLC Diamond Coring Technique Depth/Thickness (m bgl)	Total Borehole Depth (m bgl)
BH1	0.0 – 7.7	7.7 – 14.11	14.11
BH2	0.0 – 7.5	–	7.5
BH3	0.0 – 7.0	–	7.0

1.6 Constraints

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

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

2. SITE DESCRIPTION

2.1 Overall Site Description

The overall site description and its surrounding are presented in Table 2 below.

Table 2. Overall Site Description and Site Surroundings

Information	Details
Overall Site Location	The site comprises amalgamation of two (2) properties, being No. 31 Santley Crescent and No. 2A Bringelly Road, and located within a residential area approximately 60m south of the Great Western Highway.
Site Address	31 Santley Crescent & 2A Bringelly Road Kingswood NSW 2747
Approximate Site Area[1]	1,350m^2
Local Government Authority	Penrith City Council
Site Description	At the time of the investigation, a residential dwelling was present within each property, accompanied by associated concrete pavements and detached sheds. The remaining area of the site was covered in grass, vegetation and some mature trees scattered throughout.
Approximate Distances to Nearest Watercourses (i.e. rivers, lakes, creeks, etc.)	• Werrington Creek – 750m east of the site. • Unnamed Stream – 80m south-east and 150m south-west of the site.
Site Surroundings	The site is located within an area of residential use and is bounded by: • Residential properties at No. 2 Bringelly Road and No. 176 Great Western Highway to the north. • Residential property at No. 29 Santley Crescent to the east. • Santley Crescent carriageway and commercial property at No. 33 Santley Crescent to the south. • Bringelly Road thoroughfare to the west.

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

2.2 Topography

The local and site topography generally falls towards the south to south-east. Levels within the site vary from approximately RL43.4m to RL45.2m AHD.

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

2.3 Regional Geology

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

The site is also situated approximately 330m north-west of a geological boundary/region generally underlain by Quaternary Aged Holocene Deposits (Qal). The Quaternary Aged Holocene Deposits (Qal) typically comprise "fine grained sand, silt and clay".

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

A review of the regional maps by the NSW Government Environment and Heritage indicates the site is generally located within the Luddenham (lu) landscape group which is largely recognised by undulating low hills on Wianamatta Group shales, often associated with Minchinbury sandstone. Soils of the Luddenham group typically have water erosion hazard, localised steep slopes, mass movement hazard, shallow soils, surface movement potential, impermeable highly plastic subsoil and are moderately reactive. Local reliefs are approximately 50m to 80m and slopes of approximately 5% to 20% in gradient. Soils of the Luddenham group are generally neutral (pH 7.0) to strongly (pH 4.0) acidic.

The site is also noted to be approximately 280m north-west of the South Creek (sc) landscape group. The South Creek (sc) landscape group is generally recognised by floodplains, valley flats and drainage depressions of the channels on the Cumberland Plain, which are usually flat with incised channels and mainly cleared. Soils of the South Creek group typically have flood hazard, seasonal waterlogging, localised permanently high water tables, localised water erosion hazard and localised surface movement potential. Soils of the South Creek group are also generally neutral (pH 7.0) to extremely (pH 3.0) acidic.

The Luddenham (lu) and South Creek (sc) landscape group reports are attached in **Appendix I**.

3. SUBSURFACE CONDITIONS AND ASSESSMENT RESULTS

3.1 Stratigraphy

A summary of the surface and subsurface conditions from across the site during this geotechnical investigation are summarised in Table 3 below and are interpreted from the assessment results. It should be noted that Table 3 presents a summary of the overall site conditions and reference should be made to the detailed engineering borehole logs presented in **Appendix D**, in conjunction with the geotechnical explanatory notes detailed in **Appendix C**. Rock description has been based on Pells P.J.N, Mostyn G. & Walker B.F. Foundations on Sandstone and Shale in the Sydney Region, Australian Geomechanics Journal, December 1998.

It should be noted that estimated rock strengths assessed by observation during auger penetration resistance in the boreholes are approximate and variances should be expected throughout the site. In addition, estimated rock strengths are also estimated from the point load strength index (Is_{50}) carried out at the selected depths within the boreholes, and are also expected to vary throughout the site. It is worth noting that auger penetration within various bedrock formations vary from each drilling rig, and estimated rock strength variances across the site are expected.

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

Based on the geotechnical investigation at the selected testing location, along with our experience and observations made within the site and local region, it is inferred that bedrock of variable composition, strength and weathering is underlying majority of the site area at varying depths of approximately 3.2m to 5.0m bgl.

Furthermore, assessment of the underlying soils indicates the possibility of variable composition and consistency/strength natural soils to be present throughout the site.

Table 3. Summary of Subsurface Conditions

			Borehole ID	**BH1**	**BH2**	**BH3**
Unit	**Unit Type**	**Description**	**Estimated Consistency/ Strength**	**Depth/Thickness of Unit (m bgl)**		
Approximate RL at Borehole Location (m AHD)				RL44.5	RL45.1	RL43.5
1	Fill	Clayey SILT, medium to high plasticity clay, gravel inclusions.	N/A	0.0 – 0.5	0.0 – 0.2	0.0 – 0.5
2	Residual Soils	Silty CLAY, medium to high plasticity, gravel inclusions.	Firm to stiff, very stiff with depth	0.5 – 4.0	0.2 – 3.2	0.5 – 3.2
		Shaly CLAY, low plasticity, interbedded shale.	Very stiff to hard	4.0 – 5.0	3.2 – 4.0	–
3	Class V Shale[1]	SHALE, clay seams, with silt, extremely to highly weathered.	EL	5.0 – 6.8	4.0 – 5.2	3.2 – 4.6
			VL	6.8 – 7.7	5.2 – 6.3	4.6 – 6.6
4	Class IV Shale[1]	SHALE, clay seams, interbedded laminite, highly to slightly weathered.	L	7.7 – 14.11	6.3 – 7.5	6.6 – 7.0
			L – M (or better)[2]		7.5	7.0
			M		–	–

[1]Confirmation of the underlying bedrock composition, class, depth and estimated strength should be made by further borehole drilling and rock strength testing, and during construction by inspection and appropriate testing (i.e. spoon testing, rock strength testing, etc.).
[2]Higher estimated strength and/or class bedrock (i.e. low to medium estimated strength, or better) is inferred to be present below the auger refusal depth indicated in Table 3. This is based on high auger resistance during drilling and reference to the rock core samples recovered during the NMLC process in borehole BH1 below the auger high TC bit resistance depth. Confirmation should be made by a geotechnical engineer by further borehole drilling and rock strength testing.

Notes:

- N/A = Not Applicable, EL = Extremely Low estimated strength, VL = Very Low estimated strength, L = Low estimated strength, M = Medium estimated strength, H = High estimated strength.
- Rock strengths are based on observations made during auger penetration resistance at the time of drilling and point load strength index (Is_{50}) carried out at selected depths within the boreholes.
- Confirmation of the actual composition, continuity, strength and depth of the underlying bedrock should be carried out by a geotechnical engineer by additional borehole drilling and rock strength testing, and by inspection during construction.
- Ground conditions are expected to vary across the site and should be confirmed by a geotechnical engineer, predominately in areas unobserved during the geotechnical investigation.

It is worth noting the presence of core loss layers and fractured zones within the underlying shale bedrock which were encountered during drilling of borehole BH1 at varying depths throughout (refer to the detailed engineering borehole logs).

These layers along with extremely weathered zones should not be precluded across the site, predominately at locations and depths not assessed during the geotechnical investigation. Precaution should be taken during construction and at bulk excavation level as these layers are not suitable as founding materials for the proposed development.

3.2 Groundwater

No groundwater was encountered or observed during and shortly after drilling (<30 minutes) of the boreholes to a maximum depth of approximately 7.7m bgl (BH1). Water introduced during the NMLC coring process in borehole BH1 from below the auger termination depth at approximately 7.7m bgl further precluded any groundwater level indications

Following completion of drilling in borehole BH1, a standpipe piezometer (GW1) was installed to a depth of about 14.1m bgl (RL30.4m AHD).

After installation of the standpipe piezometer, water generated during the NMLC coring process which was present in the standpipe piezometer was purged using a bailer. It should be noted that although all efforts were made to purge all of the water present within the standpipe piezometer completely, the possibility of groundwater being present at lower depths should not be precluded.

Groundwater measurements carried out on the 22nd September 2021 within standpipe piezometer GW1 indicated groundwater levels to be present at a depth of approximately 1.2m bgl, at the measured location and at the time of the measurement.

Subsequent to the groundwater measurement on the 22nd September 2021, water within the standpipe piezometer was purged using a bailer to a depth of approximately 10.5m bgl. An additional groundwater measurement was carried out after a period of approximately 15 minutes and indicated the groundwater level within the standpipe piezometer to gradually rise to a depth of about 9.6m bgl.

Thus, based on information available at the time of the investigation and position of the site in the local region, groundwater which may be present within the site is expected to be in the form of seepage through the voids within the underlying fill material and through the pore spaces between particles of unconsolidated natural soils, or through networks of fractures and solution openings in consolidated bedrock underlying the site (subject to confirmation).

It should be noted that groundwater levels have the potential to elevate during daily or seasonal influences such as tidal fluctuations, heavy rainfall, damaged services, flooding, etc., and moisture content within soils may be influenced by events within the site and adjoining properties. Groundwater monitoring should be carried out prior to and during construction to assess any groundwater inflow throughout the excavation areas. We note that no provision was made for longer term groundwater monitoring within the site and it would be prudent to allow for this.

Where groundwater conditions vary from those outlined in this report, GCA should be contacted for further advice.

4. LABORATORY TEST RESULTS

4.1 Aggressivity and Salinity

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

Table 4. Summary of Laboratory Test Results (Aggressivity and Salinity)

Borehole ID		BH1	BH2	BH2	BH3
Approximate Depth (m bgl)		1.9 – 2.0	3.2 – 3.3	4.4 – 4.5	6.9 – 7.0
Strata Type		Natural Soils	Natural Soils	Bedrock	Bedrock
Aggressivity and Salinity	**pH**	5.7	7.1	7.6	8.6
	Moisture Content (%)	14.3	13.0	10.2	8.5
	Chloride (mg/kg)	440	1,000	970	600
	Sulphate SO_4 (mg/kg)	150	140	150	100
Electrical Conductivity (µS/cm)	**EC (µS/cm)**	372	652	663	472
	EC (dS/m)	0.372	0.652	0.663	0.472
	Multiplication Factor[1]	8	8	15	15
	Saturation Extract ECe (dS/m)	2.98	5.22	9.95	7.08

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

4.2 Rock Samples

A total of six (6) samples selected from the collected rock cores from borehole BH1 were tested by our affiliate NATA accredited laboratory, Geologic Solutions, for diametral and axial point load strength index (Is_{50}). The results are outlined in Table 5 below with the indicative approximate rock strengths.

Table 5. Point Load Index (Is_{50}) Laboratory Test Results

Borehole ID	Approximate Testing Depth (m bgl)	Point Load Index (Is_{50})		Approximate Indicative Strength
		Diametral (MPa)	Axial (MPa)	
BH1	7.85	0.28	1.46	High
	8.88	0.08	0.20	Low
	9.60	1.20	0.99	Medium
	10.05	0.24	1.21	High
	11.63	0.01	0.01	Extremely Low
	12.78	0.14	0.39	Medium
	13.33	0.39	0.44	Medium

Test results ranged between a point load index (Is_{50}) from 0.01MPa to 1.46MPa for diametral testing and from 0.01MPa to 1.24MPa for axial testing.

It is noted that variable higher strength rock bands (and possible softer rock bands) are expected to be encountered within the underlying bedrock throughout the site, and at locations and depths not assessed during the geotechnical investigation. This should not be precluded during construction. The point load test results laboratory certificates are presented in **Appendix F**.

5. GEOTECHNICAL ASSESSMENT AND RECOMMENDATIONS

5.1 Dilapidation Survey

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

5.2 General Geotechnical Issues

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

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

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

5.3 Preliminary Aggressivity and Salinity Assessment

In accordance to AS 2159-2009 "Piling – Design and Installation" (as outlined in Table 6 below), the results of the laboratory tests and introduction of a multiplication factor for electrical conductivity on the selected samples pH, chloride and sulphate content, and electrical conductivity indicates the following classification:

Table 6. Aggressivity and Salinity Reference Table

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

- Underlying natural soils (from boreholes BH1 and BH2):
  - **Non** aggressive for buried steel structural elements in low and high permeability soils.
  - **Non** aggressive for buried concrete structural elements in low permeability soils.
  - **Mildly** aggressive for buried concrete structural elements in high permeability soils.
  - Electrical conductivity of saturated extract (ECe) ranging from approximately 2.99ds/m to 5.22ds/m, indicating generally "**moderately**" saline natural soils underlying the site.

- Underlying bedrock (from boreholes BH2 and BH3):
  - **Non** aggressive for buried steel structural elements in low and high permeability soils.
  - **Non** aggressive for buried concrete structural elements in low permeability soils.
  - **Mildly** aggressive for buried concrete structural elements in high permeability soils.
  - Electrical conductivity of saturated extract (ECe) ranging from approximately 7.08ds/m to 9.95ds/m, indicating generally "**very**" saline bedrock underlying the site.

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

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

5.4 Inspection Pits and Underpinning

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

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

An excavation monitoring report/plan should be implemented for the subject site prior to excavation and construction activities (mainly for adjoining infrastructures and road reserves).

5.5 Excavation

Maximum excavation depths varying from approximately 6.5m to 9.7m (varying throughout) are expected to be required for construction of the proposed development. Locally deeper excavations for the proposed lift shafts, and building footings and service trenches are also projected to be required as part of the planned development.

Based on this information and existing ground conditions as encountered during the geotechnical investigation, it is anticipated that excavations will extend through Unit 1 (fill) to Unit 4 (Class IV Shale) inclusive, throughout majority of the proposed development area, as discussed in Section 3 above (depending on the actual amount of excavation required).

The possibility for encountering higher estimated strength and/or class bedrock should not be precluded during excavation, predominately in areas and at depths not assessed during the geotechnical investigation. Estimated bedrock strength variances and higher strength rock bands are expected across the site area.

Consultation should be made with subcontractors to discuss the feasibility and capability of machinery for the proposed development for the existing site conditions.

5.5.1 Excavation Assessment

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

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

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

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

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

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

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

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

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

The effectiveness of all the above-mentioned approaches must be confirmed by the results of vibration monitoring. The limits of 5mm/sec and 10mm/sec are expected to be achievable if rock breaker equipment or other excavations are restricted to the values indicated in Table 7 below.

Table 7. 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.

A vibration monitoring plan is recommended to be considered/developed to monitor construction activities and their effects on adjoining infrastructures, mainly where excavations are expected to be conducted within the underlying bedrock of higher estimated strength and vicinity of adjoining infrastructures.

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

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

5.7 Groundwater Management

Based on the geotechnical investigation and groundwater measurements carried out within the site (summarised in Section 3.2), *inferred* groundwater seepage which may be encountered during construction is anticipated 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 the presence of groundwater should not be precluded during construction and in the long-term design life of the proposed building. It should also be noted that these groundwater levels have the potential to elevate during daily or seasonal influences such as tidal fluctuations, heavy rainfall, damaged services, flooding, etc.

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

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

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

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

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

It is recommended that groundwater levels and recharge rates within the standpipe piezometer are monitored prior to construction in order to confirm approximate groundwater levels and nature of groundwater within the site, as outlined in this report. A groundwater inflow assessment should be carried out to determine approximate inflow flow rates and permeability of the underlying soils and bedrock, where groundwater levels are expected to be above the proposed basement FFLs.

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

This assessment should be carried through to ensure a suitable drainage and retention system has been implemented for the proposed development, as discussed in Section 5.8 below and to provide confirmation of the hydrogeological characteristics prior to construction.

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

5.8 Excavation Stability

Maximum excavation depths are expected to vary within the site from approximately 6.5m to 9.7m for construction of the proposed development. Locally deeper excavations for the proposed lift shafts, and building footings and service trenches are also projected to be required as part of the planned development.

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

5.8.1 Excavation Retention Support Systems

Based on the proposed development, assessment of the subsurface conditions within the site and potential for elevated groundwater, adjoining properties and infrastructures, and extent of excavation varying across the site, it is assessed that the use of temporary or permanent batter slopes are not suitable for the proposed development, and consideration should be given to a suitable retention system such as a soldier pile wall solution, with piles sufficiently embedded into appropriate strength and competent shale bedrock underlying the site, and concrete and reinforcement infill panels for the support of the excavation and soils.

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

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

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

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

Where groundwater is deemed to be relatively high and permeability rates are excessive, it is recommended that consideration be given to a contiguous pile wall with strip drains installed behind the piles and shotcreting in weak areas susceptible to groundwater inflow. This should be confirmed by measures discussed in Section 5.7 of this report.

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

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

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

If considered, the shoring wall can be designed using the recommended design parameters provided in Section 5.8.2. 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)

Excavation pressures acting on the support will depend on a number of factors including external forces from surcharge loading, the stiffness of the support, varying groundwater levels within the site, and the construction sequence of the proposed development. Therefore, the following parameters may be used for the design of temporary and permanent retaining walls at the subject site:

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

Lateral active or "at rest" earth pressure:

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

Passive earth pressure:

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

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

Active earth pressure:

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

Where:

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

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

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

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

Table 8. Preliminary Geotechnical Design Parameters

Material	Fill (Unit 1)	Residual Soils (Unit 2)	Class V Shale (Unit 3)[3, 5]	Class IV Shale (Unit 4)[3, 5]
Unit Weight (kN/m^3)[4]	16	18	21	22
Effective Cohesion c' (kPa)	0	5	20	50
Angle of Friction ϕ' (°)	24	24	26	28
Modulus of Elasticity E_{sh} (MPa)	3	12	50	150
Earth Pressure Coefficient At Rest Ko[1]	0.59	0.59	0.56	0.53
Earth Pressure Coefficient Active Ka[2]	0.42	0.42	0.39	0.36
Earth Pressure Coefficient Passive Kp[2]	2.37	2.37	2.56	2.77
Poisson Ratio *v*	0.4	0.35	0.3	0.3

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

Notes:

- For undrained (temporary) clay soils, higher earth pressures (K=1) will apply.

5.8.3 Ground Anchors

The basement floor slabs are considered to be incorporated into the long term design of the construction and will provide permanent restraints to the walls (as a bracing system). Anchors are therefore considered to be temporary, however, it may be necessary to incorporate the temporary anchors into the finished work of the development to control deflections.

Anchors which extend outside the site boundaries and which are adopted for the development should have permission from adjacent property owners and/or relevant authorities (i.e. RMS asset, adjoining properties, etc.). The design of excavation support should be carried out by a suitably qualified and experienced structural or civil engineer. Anchors should be embedded into the underlying rock, and requirements for rock support should be inspected/approved by a geotechnical engineer during excavation.

Preliminary allowable bond stresses may be adopted for temporary anchors as detailed in Table 9 below.

Table 9. Preliminary Allowable Bond Stresses for Temporary Anchors

Unit Type/Material	Allowable Bond Stress (kPa)
Class V Shale (Unit 3)	50
Class IV Shale (Unit 4)	100

The parameters provided in Table 9 assume that the drilled holes are clean and adequately flushed. The following should also be noted during anchor design and construction:

- Anchor ground interaction and overall stability.
- Anchor bond length of at least 3.0m behind the "active" zone of the excavation.
- Permanent anchors must have appropriate corrosion provisions for longevity.
- "Lift-off" tests should be carried out to confirm the anchor capacities.
- Anchors should be proof loaded to at least 1.33 times their design working loads prior to being locked off at working loads. This should constitute as a "Hold Point".

5.9 Preliminary Earthquake Site Risk Classification

In accordance with AS 1170.4-2007 and based on assessment of the material 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 the FFLs of the proposed development and based on the boreholes carried out within the site, we expect varying ground conditions comprising predominately shale bedrock of variable estimated strength and weathering to be exposed at bulk level excavation.

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

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

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

5.10.1 Geotechnical Assessment

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

Shallow foundations should include local slab thickening to support internal walls and columns. The use of settlement reduction piles with increased socket depths may also be considered in order to increase the resistance against lateral loading induced by earthquake or winds, and to achieve higher bearing capacities than at the proposed developments FFLs.

Installation of piles (where adopted) should be complemented by inspections carried out by a geotechnical engineer during construction. The actual depth and embedment of the piles should be assessed by the project structural engineer with all structural elements of the proposed development also inspected and approved by a suitably qualified structural engineer.

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

It should be noted that due to the potential variable bedrock conditions throughout the site following bulk excavation and underlying the proposed development, precaution should be taken for the design of the building foundation system taking into consideration the preliminary geotechnical design parameters in Table 10 below.

Higher allowable bearing capacities may be considered and justified subject to confirmation by inspection during construction, and by additional borehole drilling and rock strength testing. Where higher estimated strength bedrock is encountered during construction, GCA should be contacted to re-assess the preliminary allowable bearing capacities provided in this report. Adoption of higher preliminary bearing capacities for the design of the proposed development outlined in Table 10 should be confirmed by a geotechnical engineer, as discussed in this report.

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

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

Piles sufficiently socketed into higher strength bedrock may achieve higher allowable bearing capacities, subject to confirmation by a geotechnical engineer by additional borehole drilling and rock strength testing, or by inspection during construction.

Where higher estimated strength bedrock is present within the site, or where ground conditions vary from those encountered during the geotechnical investigation, GCA should be contacted for further advice.

Table 10 provides preliminary recommended geotechnical design parameters.

Table 10. Preliminary Recommended Geotechnical Design Parameters

Unit Type/Material	Maximum Allowable (Serviceability) Values (kPa)		
	End Bearing Pressure[1]	Shaft Adhesion (Compression)	Shaft Adhesion (Tension)
Fill (Unit 1)	N/A	N/A	N/A
Residual Soils (Unit 2)	N/A	N/A	N/A
Class V Shale (Unit 3)[2]	700	50	25
Class IV Shale (Unit 4)[2, 3]	1,000	100	50

[1]Minimum embedment of 0.4m for shallow foundations and 0.5m for deep foundations. Assumes the presence of shale bedrock underlying the entire site area.
[2]The composition, class, depth and estimated strength of the underlying bedrock material should be confirmed prior to construction by further borehole drilling and rock strength testing, and during construction by inspection.
[3]Subject to confirmation by a geotechnical engineer during construction by appropriate testing (i.e. spoon testing, rock strength testing, etc.).

Notes:

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

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

5.10.2 Geotechnical Comments

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

Specific geotechnical advice should be obtained for footing deigns and end bearing capacities, and design of the foundation system (shallow and pile foundations) should be carried out in accordance with AS 2870-2011 and AS 2159-2009. It is also recommended that reference is made to the recommendations provided by CSIRO "Guide to Home Owners on Foundation Maintenance and Footing Performance", attached as **Appendix H**.

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

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

The design and construction of the foundations should take into consideration the potential of flooding. All foundation excavations should be free of any loose debris and wet soils, and if groundwater seepage or runoff is encountered dewatering should be carried out prior to pouring concrete in the foundations. Due to the possibility of groundwater being encountered and possible groundwater seepage during installation of bored piles within the site, it is recommended that consideration be given to other piling methods such as Continuous Flight Auger (CFA) piles.

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

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

5.11 Filling

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

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

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

5.12 Subgrade Preparation

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

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

6. ADDITIONAL GEOTECHNICAL RECOMMENDATIONS

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

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

7. LIMITATIONS

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

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

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

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

For and behalf of

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APPENDIX A

Geotechnical Consultants Australia

Important Information About Your

Geotechnical Report

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

APPENDIX B

Legend:

Approximate Borehole Location

Approximate Borehole/Standpipe Piezometer Location

GCA Geotechnical Consultants Australia	Figure 1 Site Plan	**Geotechnical Investigation**	Drawn: GA/PG
		Dana Bina Pty Ltd & Midpoint Investments Pty Ltd	Date: 29/09/2021
	Job No.: G21551-1	31 Santley Cresent & 2A Bringelly Road Kingswood NSW 2747	Scale: NTS

Image source: NSW Six Maps - https://maps.six.nsw.gov.au/, accessed 17th September 2021.

APPENDIX C

GCA
Geotechnical Consultants Australia

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

DRILLING/EXCAVATION METHOD

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

PENETRATIION/EXCAVATION RESISTANCE

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

L **Low Resistance.** Rapid penetration possible with little effort from the equipment used.
M **Medium Resistance.** Excavation possible at an acceptable rate with moderate effort required from the equipment used.
H **High Resistance.** Further penetration is possible at a slow rate and required significant effort from the equipment.
R **Refusal or Practical Refusal.** No further progress possible within the risk of damage or excessive wear to the equipment used.

WATER

▽ Water level at date shown ◁ Partial water loss

▷ Water inflow ◀ Complete water loss

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

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

MOISTURE CONDITION (AS 1726-2017)

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

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

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

For cohesive soils the following codes may also be used:

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

SAMPLING AND TESTING

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

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

ROCK QUALITY

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

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

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

ROCK STRENGTH TEST RESULTS

● Diametral Point Load Index test

■ Axial Point Load Index test

SOIL ORIGINS

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

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

Transported soils may be further subdivided into:

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

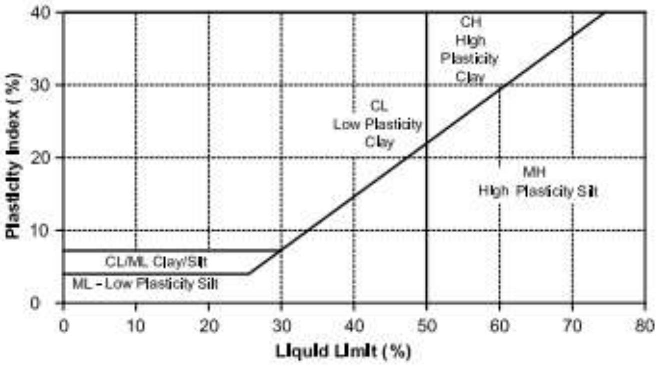
Method and Terms for Soil and Rock Descriptions Used on Borehole and Test Pit Reports

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

COHESIONLESS SOILS PARTICLE SIZE DESCRIPTIVE TERMS

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

PLASTICITY PROPERTIES



COHESIVE SOILS – CONSISTENCY (AS 1726-2017)

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

PLASTICITY

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

COHESIONLESS SOILS - RELATIVE DENSITY

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

UNIFIED SOIL CLASSIFICATION

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

ROCK MATERIAL WEATHERING

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

ROCK STRENGTH (AS 1726-2017 and ISRM)

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

ABREVIATIONS FOR DEFECT TYPES AND DECRIPTIONS

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

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

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

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

APPENDIX D

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BOREHOLE NUMBER BH1

CLIENT Dana Bina Pty Ltd & Midpoint Investments Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G21551-1
PROJECT LOCATION 31 Santley Cresent & 2A Bringelly Road Kingswood N

DATE STARTED 13/9/21 **COMPLETED** 13/9/21 **R.L. SURFACE** 44.5 **DATUM** m AHD
DRILLING CONTRACTOR BG Drilling Pty Ltd **SLOPE** 90° **BEARING** ---
EQUIPMENT Track Mounted Drilling Rig **HOLE LOCATION** Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter **LOGGED BY** GA **CHECKED BY** JN
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	Water	Well Details	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT	Not Encountered During Drilling			0.0			Clayey SILT, brown to dark brown, high plasticity clay, with fine gravel, grass rootlets, moist.		FILL
			44.0	0.5		CI-CH	Silty CLAY, medium to high plasticity, brown to brownish orange, some fine to medium gravel, moist.		RESIDUAL SOILS
	22/09/2021		43.5	1.0					
			43.0	1.5		CI-CH	Silty CLAY, medium to high plasticity, brown to pale reddish brown, grey laminations, some fine to coarse gravel, moist.		
			42.5	2.0				DS	
			42.0	2.5					
			41.5	3.0		CI-CH	Silty CLAY, medium to high plasticity, grey to pale grey, reddish brown laminations, some fine gravel, moist.		
			41.0	3.5		CI-CH	Silty CLAY, medium to high plasticity, brown to pale brown, grey laminations, some fine gravel, moist.		
			40.5	4.0		CL	Shaly CLAY, low plasticity, grey to dark grey, interbedded shale, moist.		
			40.0	4.5					
			39.5	5.0					

BOREHOLE / TEST PIT BOREHOLE LOGS.GPJ GINT STD AUSTRALIA.GDT 29/9/21

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BOREHOLE NUMBER BH1

CLIENT Dana Bina Pty Ltd & Midpoint Investments Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G21551-1
PROJECT LOCATION 31 Santley Cresent & 2A Bringelly Road Kingswood N

DATE STARTED 13/9/21 **COMPLETED** 13/9/21 **R.L. SURFACE** 44.5 **DATUM** m AHD
DRILLING CONTRACTOR BG Drilling Pty Ltd **SLOPE** 90° **BEARING** ---
EQUIPMENT Track Mounted Drilling Rig **HOLE LOCATION** Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter **LOGGED BY** GA **CHECKED BY** JN
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	Water	Well Details	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT							SHALE, grey, brown, clay seams, with silt, extremely weathered, extremely low estimated strength, moist.		BEDROCK
			39.0	5.5					
			38.5	6.0			grey laminations from 6.0m bgl.		
			38.0	6.5					
							SHALE, grey, clay seams, with silt, highly weathered, very low estimated strength, moist.		
			37.5	7.0					
			37.0	7.5					TC bit refusal at 7.7m bgl.
							Borehole BH1 continued as cored hole		
			36.5	8.0					
			36.0	8.5					
			35.5	9.0					
			35.0	9.5					
			34.5	10.0					

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PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G21551-1
PROJECT LOCATION 31 Santley Cresent & 2A Bringelly Road Kingswood N

DATE STARTED 13/9/21 **COMPLETED** 13/9/21
R.L. SURFACE 44.5 **DATUM** m AHD
DRILLING CONTRACTOR BG Drilling Pty Ltd
SLOPE 90° **BEARING** ---
EQUIPMENT Track Mounted Drilling Rig
HOLE LOCATION Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter
LOGGED BY GA **CHECKED BY** JN
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	Water	Well Details	RL (m)	Depth (m)	Graphic Log	Material Description	Weathering	Estimated Strength (EL VL L M H VH EH)	$Is_{(50)}$ MPa (D- diametral, A- axial)	RQD %	Defect Spacing mm (30 100 300 1000 3000)	Defect Description
			39.0	5.5								
			38.5	6.0								
			38.0	6.5								
			37.5	7.0								
			37.0	7.5		Continued from non-cored borehole						
NMLC			36.5	8.0		SHALE, grey to dark grey, interbedded pale grey laminite.	SW		D 1.46, A 0.28	47		7.80m, J, S, C, U, 5 deg 7.92m, J, S, C, U, 5 deg 7.96m, FZ, 80mm 8.06m, J, S, C, U, 5 deg 8.09m, J, S, C, U, 5 deg 8.21m, J, S, C, U, 5 deg 8.27m, J, S, C, U, 5 deg 8.35m, J, S, C, U, 5 deg
			36.0	8.5		Core Loss 300m	EW					8.46m, Core Loss, 300mm
			35.5	9.0		SHALE, grey to dark grey, interbedded pale grey laminite.	SW		D 0.2, A 0.08	48		8.82m, J, S, C, U, 5 deg 9.00m, HB 9.04m, HB 9.06m, J, S, C, U, 5-10 deg 9.20m, J, S, C, U, 5 deg 9.23m, J, S, C, U, 5 deg 9.28m, FZ, 20mm 9.34m, J, S, C, U, 5-10 deg 9.41m, FZ, 20mm
			35.0	9.5					D 0.99, A 1.2			9.48m, J, S, C, U, 5 deg 9.53m, J, S, C, U, 5 deg 9.69m, J, S, C, U, 5 deg 9.72m, J, S, C, U, 5 deg 9.77m, J, S, C, U, 5 deg 9.82m, J, S, C, U, 5 deg 9.88m, J, S, C, U, 5 deg
			34.5	10.0								

BOREHOLE NUMBER BH1

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CLIENT Dana Bina Pty Ltd & Midpoint Investments Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G21551-1
PROJECT LOCATION 31 Santley Cresent & 2A Bringelly Road Kingswood N

DATE STARTED 13/9/21 **COMPLETED** 13/9/21 **R.L. SURFACE** 44.5 **DATUM** m AHD
DRILLING CONTRACTOR BG Drilling Pty Ltd **SLOPE** 90° **BEARING** ---
EQUIPMENT Track Mounted Drilling Rig **HOLE LOCATION** Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter **LOGGED BY** GA **CHECKED BY** JN
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	RL (m)	Depth (m)	Material Description	Weathering	$Is_{(50)}$ MPa (D- diametral, A- axial)	RQD %	Defect Description
NMLC			SHALE, grey to dark grey, interbedded pale grey laminite. *(continued)*	SW	D 1.21 A 0.24	48	9.94m, J, S, C, U, 5 deg 10.00m, J, S, C, U, 5 deg 10.08m, HB 10.11m, FZ, 60mm
						32	10.24m, J, S, C, U, 5 deg 10.25m, J, S, C, U, 5 deg 10.35m, J, S, C, U, 5 deg 10.46m, J, S, C, U, 5 deg
	34.0	10.5					10.61m, J, S, C, U, 5 deg 10.65m, J, S, C, U, 5 deg 10.71m, B, S, C, U, 5 deg 10.76m, B, S, C, U, 5 deg
				MW/HW		50	
	33.5	11.0					11.00m, J, S, C, U, 5 deg 11.07m, J, S, C, U, 5 deg 11.12m, J, S, C, U, 5 deg 11.24m, J, S, C, U, 5 deg 11.27m, J, S, C, Cu, 5-15 deg 11.31m, FZ, 50mm 11.41m, J, S, Cl, U, 5 deg
	33.0	11.5			D 0.01 A 0.01	32	11.57m, J, S, Cl, U, 5 deg 11.61m, J, S, C, U, 5 deg 11.63m, J, S, C, U, 5 deg 11.67m, J, S, Cl, U, 5 deg 11.75m, J, S, Cl, U, 5-10 deg 11.77m, J, S, Cl, U, 5 deg 11.89m, B, S, Cl, U, 5 deg 11.92m, Core Loss, 290mm
	32.5	12.0	Core Loss 700m	EW			
	32.0	12.5					
			SHALE, grey to dark grey, interbedded pale grey laminite.	SW	D 0.12 A 0.14		12.77m, HB 12.81, FZ, 40mm 12.92, FZ, 80mm
	31.5	13.0					13.06m, J, S, C, U, 5 deg 13.09m, J, S, C, U, 5 deg 13.14m, J, S, C, U, 5 deg 13.20m, J, S, C, U, 5 deg 13.26m, J, S, C, U, 5 deg 13.38m, J, S, C, U, 5 deg 13.38m, J, S, Cl, U, 5 deg 13.50m, J, S, C, U, 5-10 deg 13.56m, Core Loss, 290mm
				MW/HW	D 0.44 A 0.39	29	
	31.0	13.5	Core Loss 290m	EW			
			SHALE, grey to dark grey, interbedded pale grey laminite.	SW			14.00m, HB 14.06, HB
	30.5	14.0					
			BH1 terminated at 14.11m				
	30.0	14.5					
	29.5	15.0					

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BOREHOLE NUMBER BH2

CLIENT Dana Bina Pty Ltd & Midpoint Investments Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G21551-1
PROJECT LOCATION 31 Santley Cresent & 2A Bringelly Road Kingswood N

DATE STARTED 13/9/21 **COMPLETED** 13/9/21 **R.L. SURFACE** 45.1 **DATUM** m AHD
DRILLING CONTRACTOR BG Drilling Pty Ltd **SLOPE** 90° **BEARING** ---
EQUIPMENT Track Mounted Drilling Rig **HOLE LOCATION** Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter **LOGGED BY** GA **CHECKED BY** JN
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	Water	RL (m)	Depth (m)	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT	Not Encountered During Drilling	45.0	0.0–0.2		Clayey SILT, brown to dark brown, medium plasticity clay, some fine gravel, grass rootlets, mosit.		FILL
		44.5, 44.0	0.2–1.2	CI-CH	Silty CLAY, medium to high plasticity, brown to brownish orange, some fine gravel, moist, estimated firm. grey laminations from 0.8m bgl.		RESIDUAL SOILS
		43.5	1.2–1.7	CI-CH	Silty CLAY, medium to high plasticity, brown to pale brown, grey to pale grey, some fine gravel, moist, estimated firm.	SPT 8, 5, 4 N=9	
		43.0	1.7–2.4	CI-CH	Silty CLAY, medium to high plasticity, grey to pale grey, reddish brown laminations, some fine gravel, moist, estimated firm.		
		42.5, 42.0	2.4–3.2	CI-CH	Silty CLAY, medium to high plasticity, brown to pale brown, grey to pale grey, some fine gravel, moist, estimated stiff.	SPT 11, 25/70 Bouncing/DS	
		41.5	3.2–4.0	CL	Shaly CLAY, low plasticity, grey to dark grey, interbedded shale, moist, estimated hard.		
		41.0, 40.5	4.0–5.0		SHALE, grey, brown, clay seams, with silt, extremely weathered, extremely low estimated strength, moist.	DS	BEDROCK

BOREHOLE / TEST PIT BOREHOLE LOGS.GPJ GINT STD AUSTRALIA.GDT 29/9/21

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

BOREHOLE NUMBER BH2

CLIENT Dana Bina Pty Ltd & Midpoint Investments Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G21551-1
PROJECT LOCATION 31 Santley Cresent & 2A Bringelly Road Kingswood N

DATE STARTED 13/9/21 **COMPLETED** 13/9/21 **R.L. SURFACE** 45.1 **DATUM** m AHD
DRILLING CONTRACTOR BG Drilling Pty Ltd **SLOPE** 90° **BEARING** ---
EQUIPMENT Track Mounted Drilling Rig **HOLE LOCATION** Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter **LOGGED BY** GA **CHECKED BY** JN
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT		40.0				SHALE, grey, brown, clay seams, with silt, extremely weathered, extremely low estimated strength, moist. *(continued)*		
		39.5	5.5			SHALE, grey, some clay seams, with silt, highly weathered, very low estimated strength, moist.		
		39.0	6.0					
		38.5	6.5			SHALE, grey, with silt, moderately weathered, low estimated strength, moist.		
		38.0	7.0					TC bit refusal at 7.5m bgl.
		37.5	7.5			inferred low to medium estimated strength (or better) from 7.5m bgl. Borehole BH2 terminated at 7.5m		
		37.0	8.0					
		36.5	8.5					
		36.0	9.0					
		35.5	9.5					
			10.0					

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info@geoconsultants.com.au
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(02) 9788 2829

BOREHOLE NUMBER BH3

CLIENT Dana Bina Pty Ltd & Midpoint Investments Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G21551-1
PROJECT LOCATION 31 Santley Cresent & 2A Bringelly Road Kingswood N

DATE STARTED 13/9/21 **COMPLETED** 13/9/21 **R.L. SURFACE** 43.5 **DATUM** m AHD
DRILLING CONTRACTOR BG Drilling Pty Ltd **SLOPE** 90° **BEARING** ---
EQUIPMENT Track Mounted Drilling Rig **HOLE LOCATION** Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter **LOGGED BY** GA **CHECKED BY** JN
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
	Not Encountered During Drilling		0.0			Clayey SILT, brown to dark brown, medium plasticity clay, some fine gravel, grass rootlets, mosit.		FILL
		43.0	0.5		CI-CH	Silty CLAY, medium to high plasticity, brown to reddish brown, grey to pale grey, moist.		RESIDUAL SOILS
		42.5	1.0					
		42.0	1.5		CI-CH	Silty CLAY, medium to high plasticity, brown to reddish brown, some fine gravel, moist.		
		41.5	2.0		CI-CH	Silty CLAY, medium to high plasticity, grey to pale grey, reddish brown laminations, some fine gravel, moist.		
		41.0	2.5					
		40.5	3.0					
						SHALE, brown, grey, clay seams, with silt, extremely weathered, extremely low estimated strength, moist.		BEDROCK
		40.0	3.5					
		39.5	4.0					
		39.0	4.5					
						SHALE, brown, grey, some clay seams, with silt, highly weathered, very low estimated strength, moist.		
		38.5	5.0					

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(02) 9788 2829

BOREHOLE NUMBER BH3

CLIENT Dana Bina Pty Ltd & Midpoint Investments Pty Ltd **PROJECT NAME** Geotechnical Investigation
PROJECT NUMBER G21551-1 **PROJECT LOCATION** 31 Santley Cresent & 2A Bringelly Road Kingswood N

DATE STARTED 13/9/21 **COMPLETED** 13/9/21 **R.L. SURFACE** 43.5 **DATUM** m AHD
DRILLING CONTRACTOR BG Drilling Pty Ltd **SLOPE** 90° **BEARING** ---
EQUIPMENT Track Mounted Drilling Rig **HOLE LOCATION** Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter **LOGGED BY** GA **CHECKED BY** JN
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
						SHALE, brown, grey, some clay seams, with silt, highly weathered, very low estimated strength, moist. *(continued)*		
		38.0	5.5					
		37.5	6.0					
		37.0	6.5					
						SHALE, grey, with silt, moderately weathered, low estimated strength, moist.		
		36.5	7.0				DS	TC bit refusal at 7.0m bgl.
						inferred low to medium estimated strength at 7.0m bgl. Borehole BH3 terminated at 7m		
		36.0	7.5					
		35.5	8.0					
		35.0	8.5					
		34.5	9.0					
		34.0	9.5					
		33.5	10.0					

APPENDIX E

BOREHOLE BH1 CORING STARTS FROM 7.7m to 14.11m bgl

GCA Geotechnical Consultants Australia	**Geotechnical Investigation**	Borehole Core Box Photographs
	Dana Bina Pty Ltd & Midpoint Investments Pty Ltd	Job No.: G21551-1
	31 Santley Cresent & 2A Bringelly Road Kingswood NSW 2747	Date: 29/09/2021

APPENDIX F

Geo-Logic Solutions
Geotechnical & Environmental Testing

Unit 3 /112 Fairfield Street, Fairfield East NSW 2165
www.geo-logic.com.au
ABN: 57 621 548 294
PH: 0402 597 452
Email: samer@geo-logic.com.au

Point Load Strength Index

TEST METHOD: AS4133.4.1

Client:	**Geotechncial Consultants Australia P/L**
Project :	**Materials Testing**
Location:	**Kingswood NSW**
Project No.	**L662**
Date Reported:	**21/09/2021**
Report No.	**L662-Rev1**
Sample Procedure	***Sampled By Client: Core Samples Supplied**

Sample Number	BH1	BH1	BH1	BH1	BH1
Sample Depth (m)	7.85	8.88	9.60	10.05	11.63
Date Sampled	13/09/2021	13/09/2021	13/09/2021	13/09/2021	13/09/2021
Sample Description/ Rock Type	Shale - Black	Shale - Black	Shale - Black	Shale - Black	Shale - Black
Sample size when received	Approx 50mm X 100mm Cylinder	Approx 50mm X 100mm Cylinder	Approx 50mm X 100mm Cylinder	Approx 50mm X 100mm Cylinder	Approx 50mm X 100mm Cylinder
Test Type	Diametral	Diametral	Diametral	Diametral	Diametral
Is - (Mpa)	0.27	0.08	1.18	0.24	0.01
Is(50) - (Mpa)	0.28	0.08	1.20	0.24	0.01
Moisture Condition	Moist	Moist	Moist	Moist	Moist
Test Type	Axial	Axial	Axial	Axial	Axial
Is - (Mpa)	1.46	0.19	1.00	1.28	0.01
Is(50) - (Mpa)	1.46	0.20	0.99	1.21	0.01
Moisture Condition	Moist	Moist	Moist	Moist	Moist
Sample Weakness Description	N/A	N/A	N/A	N/A	N/A
Sample Storage History	Sealed Bag	Sealed Bag	Sealed Bag	Sealed Bag	Sealed Bag
Date Tested	20/09/2021	20/09/2021	20/09/2021	20/09/2021	20/09/2021

NOTES:

All equipment used in testing process has been calibrated by a NATA accredited laboratory.

Laboratory Approved Signatory:
Samer Ghanem
Date:
21/09/2021

Sign:

Geo-Logic Solutions
Geotechnical & Environmental Testing

Unit 3 /112 Fairfield Street, Fairfield East NSW 2165
www.geo-logic.com.au
ABN: 57 621 548 294
PH: 0402 597 452
Email: samer@geo-logic.com.au

Point Load Strength Index

TEST METHOD: AS4133.4.1

Client:	**Geotechncial Consultants Australia P/L**
Project :	**Materials Testing**
Location:	**Kingswood NSW**
Project No.	**L662**
Date Reported:	**21/09/2021**
Report No.	**L519-Rev1**
Sample Procedure	***Sampled By Client: Core Samples Supplied**

Sample Number	BH1	BH1			
Sample Depth (m)	12.78	13.33			
Date Sampled	13/09/2021	13/09/2021			
Sample Description/ Rock Type	Sandstone	Sandstone			
Sample size when received	Approx 50mm X 100mm Cylinder	Approx 50mm X 100mm Cylinder			
Test Type	Diametral	Diametral			
Is - (Mpa)	0.14	0.39			
Is(50) - (Mpa)	0.14	0.39			
Moisture Condition	Moist	Moist			
Test Type	Axial	Axial			
Is - (Mpa)	0.11	0.44			
Is(50) - (Mpa)	0.12	0.44			
Moisture Condition	Moist	Moist			
Sample Weakness Description	N/A	N/A			
Sample Storage History	Sealed Bag	Sealed Bag			
Date Tested	20/09/2021	20/09/2021			

NOTES:	All equipment used in testing process has been calibrated by a NATA accredited laboratory.	Laboratory Approved Signatory: ***Samer Ghanem*** **Date:** ***21/09/2021*** **Sign:**

APPENDIX G

ALS Environmental

CERTIFICATE OF ANALYSIS

Work Order	: **ES2133385**	Page	: 1 of 2
Client	: **GEOTECHNICAL CONSULTANTS AUSTRALIA**	Laboratory	: Environmental Division Sydney
Contact	: JOE NADER	Contact	: Customer Services ES
Address	: Suite 5, 5-7 Villiers Street Parramatta NSW 2151	Address	: 277-289 Woodpark Road Smithfield NSW Australia 2164
Telephone	: ----	Telephone	: +61-2-8784 8555
Project	: G21551-1 Geotechnical Investigation	Date Samples Received	: 15-Sep-2021 12:00
Order number	: ----	Date Analysis Commenced	: 20-Sep-2021
C-O-C number	: ----	Issue Date	: 28-Sep-2021 12:59
Sampler	: George A		
Site	: 31 Santley Crescent & 2A Bringelly Road Kingswood NSW 2747		
Quote number	: EN/333		
No. of samples received	: 4		
No. of samples analysed	: 4		

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

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

This Certificate of Analysis contains the following information:

- General Comments
- Analytical Results

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

Signatories

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

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

General Comments

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

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

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

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

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

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

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

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

Analytical Results

Sub-Matrix: SOIL (Matrix: SOIL)			Sample ID	BH1 1.9m-2.0m	BH2 3.2m-3.3m	BH2 4.4m-4.5m	BH3 6.9m-7.0m	----
			Sampling date / time	13-Sep-2021 00:00	13-Sep-2021 00:00	13-Sep-2021 00:00	13-Sep-2021 00:00	----
Compound	CAS Number	LOR	Unit	ES2133385-001	ES2133385-002	ES2133385-003	ES2133385-004	--------
				Result	Result	Result	Result	----
EA002: pH 1:5 (Soils)								
pH Value	----	0.1	pH Unit	5.7	7.1	7.6	8.6	----
EA010: Conductivity (1:5)								
Electrical Conductivity @ 25°C	----	1	µS/cm	372	652	663	472	----
EA055: Moisture Content (Dried @ 105-110°C)								
Moisture Content	----	1.0	%	14.3	13.0	10.2	8.5	----
ED040S : Soluble Sulfate by ICPAES								
Sulfate as SO4 2-	14808-79-8	10	mg/kg	150	140	150	100	----
ED045G: Chloride by Discrete Analyser								
Chloride	16887-00-6	10	mg/kg	440	1000	970	600	----

QUALITY CONTROL REPORT

Work Order	: **ES2133385**	Page	: 1 of 3
Client	: **GEOTECHNICAL CONSULTANTS AUSTRALIA**	Laboratory	: Environmental Division Sydney
Contact	: JOE NADER	Contact	: Customer Services ES
Address	: Suite 5, 5-7 Villiers Street Parramatta NSW 2151	Address	: 277-289 Woodpark Road Smithfield NSW Australia 2164
Telephone	: ----	Telephone	: +61-2-8784 8555
Project	: G21551-1 Geotechnical Investigation	Date Samples Received	: 15-Sep-2021
Order number	: ----	Date Analysis Commenced	: 20-Sep-2021
C-O-C number	: ----	Issue Date	: 28-Sep-2021
Sampler	: George A		
Site	: 31 Santley Crescent & 2A Bringelly Road Kingswood NSW 2747		
Quote number	: EN/333		
No. of samples received	: 4		
No. of samples analysed	: 4		

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

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

This Quality Control Report contains the following information:

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

Signatories

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

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

General Comments

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

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

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

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

Laboratory Duplicate (DUP) Report

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

Sub-Matrix: **SOIL**

				Laboratory Duplicate (DUP) Report					
Laboratory sample ID	*Sample ID*	*Method: Compound*	*CAS Number*	*LOR*	*Unit*	*Original Result*	*Duplicate Result*	*RPD (%)*	*Acceptable RPD (%)*
EA002: pH 1:5 (Soils) (QC Lot: 3910245)									
ES2133801-015	Anonymous	EA002: pH Value	----	0.1	pH Unit	6.9	6.1	11.4	0% - 20%
ES2133385-001	BH1 1.9m-2.0m	EA002: pH Value	----	0.1	pH Unit	5.7	5.6	0.0	0% - 20%
EA010: Conductivity (1:5) (QC Lot: 3910247)									
ES2133801-015	Anonymous	EA010: Electrical Conductivity @ 25°C	----	1	µS/cm	481	422	13.1	0% - 20%
ES2133385-001	BH1 1.9m-2.0m	EA010: Electrical Conductivity @ 25°C	----	1	µS/cm	372	361	3.0	0% - 20%
EA055: Moisture Content (Dried @ 105-110°C) (QC Lot: 3913015)									
ES2133385-002	BH2 3.2m-3.3m	EA055: Moisture Content	----	0.1	%	13.0	13.2	1.4	0% - 50%
ED040S: Soluble Major Anions (QC Lot: 3910246)									
ES2133385-001	BH1 1.9m-2.0m	ED040S: Sulfate as SO4 2-	14808-79-8	10	mg/kg	150	140	0.0	0% - 50%
ED045G: Chloride by Discrete Analyser (QC Lot: 3910248)									
ES2133385-001	BH1 1.9m-2.0m	ED045G: Chloride	16887-00-6	10	mg/kg	440	430	3.1	0% - 20%

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

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

Sub-Matrix: **SOIL**

				Method Blank (MB) Report	Laboratory Control Spike (LCS) Report			
					Spike	Spike Recovery (%)	Acceptable Limits (%)	
Method: Compound	CAS Number	LOR	Unit	Result	Concentration	LCS	Low	High
EA010: Conductivity (1:5) (QCLot: 3910247)								
EA010: Electrical Conductivity @ 25°C	----	1	µS/cm	<1	1412 µS/cm	99.7	92.0	108
ED040S: Soluble Major Anions (QCLot: 3910246)								
ED040S: Sulfate as SO4 2-	14808-79-8	10	mg/kg	<10	750 mg/kg	92.0	80.0	120
ED045G: Chloride by Discrete Analyser (QCLot: 3910248)								
ED045G: Chloride	16887-00-6	10	mg/kg	<10	250 mg/kg	102	75.0	125
				<10	5000 mg/kg	94.8	79.0	117

Matrix Spike (MS) Report

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

Sub-Matrix: **SOIL**

				Matrix Spike (MS) Report			
				Spike	SpikeRecovery(%)	Acceptable Limits (%)	
Laboratory sample ID	Sample ID	Method: Compound	CAS Number	Concentration	MS	Low	High
ED045G: Chloride by Discrete Analyser (QCLot: 3910248)							
ES2133385-001	BH1 1.9m-2.0m	ED045G: Chloride	16887-00-6	250 mg/kg	100	70.0	130

APPENDIX H

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).

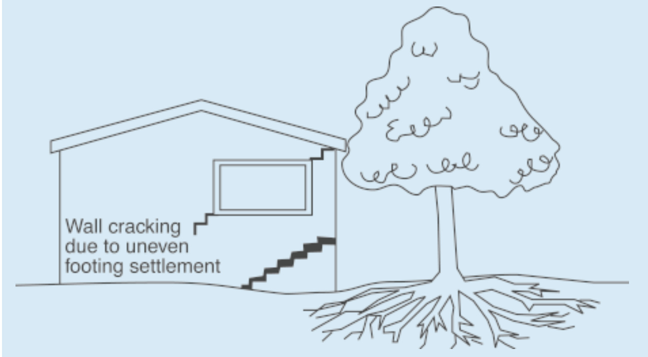
Isolated piers affected by erosion or saturation of foundations will eventually lose contact with the bearers they support and may tilt or fall over. The floors that have lost this support will become bouncy, sometimes rattling ornaments etc.

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.

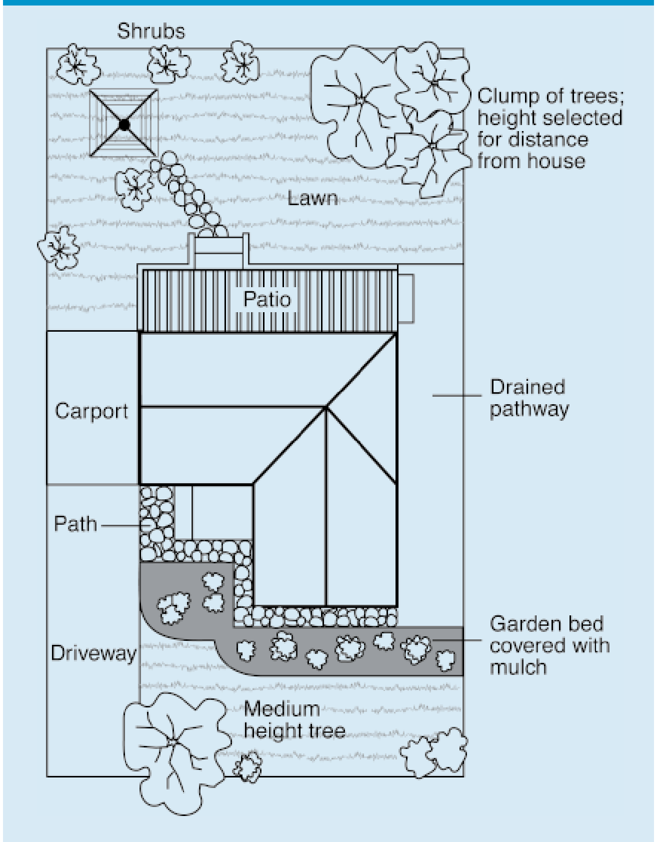
Protection of the building perimeter
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

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 I

lu	LUDDENHAM	Erosional

Landscape—undulating to rolling low hills on Wianamatta Group shales, often associated with Minchinbury Sandstone. Local relief 50–80 m, slopes 5–20%. Narrow ridges, hillcrests and valleys. Extensively cleared tall open forest (wet sclerophyll forest).

Soils—shallow (<100 cm) dark Podzolic Soils (Dd3.51) or massive Earthy Clays (Uf6.71) on crests; moderately deep (70–150 cm) Red Podzolic Soils (Dr2.11, Dr2.41, Dr3.11) on upper slopes; moderately deep (<150 cm) Yellow Podzolic Soils (Dy4.22) and Prairie Soils (Gn3.26) on lower slopes and drainage lines.

Limitations—water erosion hazard, localised steep slopes, localised mass movement hazard, localised shallow soils, localised surface movement potential; localised impermeable highly plastic subsoil, moderately reactive.

LOCATION

This unit occurs mainly towards the south and west in the Cumberland Lowland. Good examples can be found on the dissected ridges running from Denham Court north to Cecil Park. Another major occurrence lies east of the Nepean River, south of Penrith. A smaller area is found near Luddenham and minor examples occur in the north bordering the Hawkesbury Sandstone units on the Homsby Plateau.

LANDSCAPE

Geology

This soil landscape is underlain by Wianamatta Group Ashfield Shale and Bringelly Shale formations. The Ashfield Shale consists of laminite and dark grey shale. Bringelly Shale consists of shale, calcareous claystone, and laminite. Between these two shale members is the Minchinbury Sandstone consisting of fine to medium-grained lithic quartz sandstone.

Topography

Low rolling to steep low hills. Local relief 50–120 m, slopes 5–20%. Convex narrow (20–300 m) ridges and hillcrests grade into moderately inclined sideslopes with narrow concave drainage lines. Moderately inclined slopes of 10–15% are the dominant landform elements.

Vegetation

Extensively cleared open forest (dry sclerophyll forest). Dominant tree species include *Eucalyptus maculata* (spotted gum) and *E. moluccana* (grey box). Lesser occurrences of *E. fibrosa* (broad-leaved ironbark), *E. crebra* (narrow-leaved ironbark), *E. tereticornis* (forest red gum) and *E. longifolia* (woollybutt) occur. Understorey shrub species include *Bursaria spinosa* (blackthorn), *Breynia oblongifolia* (coffee bush), *Allocasuarina torulosa* (forest oak), *Acacia implexa* (hickory) and *Clerodendrum tomentosum* (hairy clerodendrum). Grasses are commonly *Aristida vagans* (speargrass), *Entolasia marginata* (bordered panic), *Eragrostis leptostachya* (paddock lovegrass) and *Themeda australis* (kangaroo grass) (Benson, 1981). Examples of natural vegetation can be found near Werombi and Floxton Park.

Landuse

Grazing is the dominant landuse over much of this soil landscape. Examples are found east of Bents Basin and south west of Bringelly. Low density housing occurs at West Floxton and Mulgoa. Increasing pressure for home-sites is resulting in more areas of this landscape changing from semi-rural to suburban land use.

Existing Erosion

Minor gully erosion is evident along unpaved roads. Moderate sheet erosion occurs on disturbed areas (e.g. cultivated lands). Small areas of moderate to severe sheet erosion occur in overgrazed paddocks on many hobby farms. Evidence of previous erosion is commonplace, especially where eroded topsoil has been deposited against fences.

Associated Soil Landscapes

Small unmapped areas of Picton (**pn**) soil landscape occur on steeper slopes especially those facing south and east. Blacktown (**bt**) soil landscape is also associated with Luddenham soil landscape.

SOILS

Dominant Soil Materials

lu1—Friable dark brown loam.

This is a dark brown, friable loam, silt loam or silty clay loam with moderate to strong structure and porous rough-faced ped fabric. This material occurs as topsoil (A1 horizon).

Peds are commonly subangular blocky to polyhedral, 2–10 mm in size and are rough-faced and porous. In uncompacted soils these peds break down readily to very small crumbs. Surface condition is distinctly friable but may become hardsetting when compacted and dry. Colour is dark brown (10YR 3/3, 7.5YR 3/3) but can range from brownish black (5YR 3/1) to brown (10YR 4/4). This material is occasionally water repellent. The pH varies from moderately acid (pH 5.0) to slightly acid (pH 6.5). A few small, subrounded-rounded weakly weathered shale fragments occur. Roots are common to 10 cm becoming fewer with increasing depth. Charcoal fragments occur occasionally.

lu2—Hardsetting brown clay loam.

This is a clay loam to fine sandy clay loam with an apedal massive or weakly pedal structure and an earthy or porous, rough-faced ped fabric. This material occurs as an A2 horizon and is occasionally hardsetting when exposed at the surface.

Peds, when present, are sub-angular blocky, 10–50 mm in size, and are rough faced and porous. Otherwise this material has apedal massive structure with an earthy porous fabric. Colour is brown (7.5YR 4/4) but can range between dull yellowish brown (10YR 5/4) and reddish brown (5YR 4/6). The pH varies between strongly acid (pH 4.0) and slightly acid (pH 6.5). Shale rock fragments, charcoal fragments and roots are present.

lu3—Whole coloured, strongly pedal clay.

This is a medium clay with strong structure and smooth-faced, dense ped fabric. It occurs as subsoil (B horizon).

Texture is commonly medium clay bit can range from silty clay to heavy clay. The peds are sub-angular blocky or polyhedral and range in size from 5–20 mm. They are smooth-faced and dense. Cutans are also present. Colour is reddish brown (5YR 4/6-8) and can range from bright reddish brown (2.5YR 4/8) to bright yellowish brown (10YR 6/6). The pH varies from strongly acid (pH 4.0) to moderately acid (pH 5.5). Shale rock fragments are common. Roots are rare and charcoal fragments are absent.

lu4—Mottled grey plastic clay.

This is a grey, mottled, medium clay with strongly pedal structure and dense, smooth-ped fabric. It occurs as deep subsoil.

Texture ranges to heavy clay. The peds are usually sub-angular blocky, 10–20 mm in size, and are smooth-faced and dense. These can be broken down easily to smaller (2–5 mm) polyhedral peds. Colour is usually light grey (10YR 7/1) but ranges to light reddish grey (2.5YR 7/1). Yellow and red mottles are common. It is usually moist and is very plastic. The pH varies from strongly acid (pH 4.0) to moderately acid (pH 5.5). Shale rock fragments and gravels are common. Roots are rare, and other inclusions are absent.

lu5—Apedal brown sandy clay.

This is an apedal massive brown, sandy clay to light clay with dense earthy fabric. It occurs as subsoil (B horizon).

Occasionally weak subangular blocky or polyhedral structure is evident. Colour is usually brown (7.5YR 4/4–6) but ranges from dull reddish brown (5YR 4/4) to dull yellowish brown (10YR 5/4). This material is moderately acid (pH 5.0) to neutral (pH 7.0). Roots are common. Up to 10% of the volume may be small (2–6 mm) angular, well weathered shale fragments. Charcoal and other inclusions do not occur.

Associated Soil Materials

Greyish brown loamy or clayey sand.

This material occurs on lower slopes and in drainage lines as a shallow (<50 cm) surface material. It has a neutral pH (pH 7.0) and frequently contains small amounts of gravels 2–20 mm and charcoal fragments.

Occurrence and Relationships

Crests. Up to 10 cm of friable dark brown loam (**lu1**) overlies <40 cm sandy clay (**lu5**) which usually directly overlies deeply weathering shale bedrock. The boundary between materials is sharp to clear. Total soil depth <40 cm [dark Podzolic Soils (Dd3.51)]. In some places **lu1** is not present [massive earthy clays (Uf6.71)]. More rarely **lu1** and **lu5** overlie >200 cm mottled grey plastic clays (**lu4**). Boundaries between soil materials are sharp to clear. Total soil depth >200 cm [Yellow Podzolic Soils (Dy2.21)].

Upper slopes and mid-slopes. Sandy clay (**lu1**) is rare but <10 cm may occur on surface. Up to 40 cm of clay loam (**lu2**) overlies >50 cm medium or heavy clay (**lu3**) which overlies <90 cm of grey mottled clay (**lu4**) [Red Podzolic Soils (Dr2.11), Yellow Podzolic Soils (Dy3.51, Gn3.71)]. Where underlying lithology is Minchinbury Sandstone up to 60 cm **lu5** occurs between **lu2** and **lu3**. In this instance **lu4** does not often occur. Total soil depth >100 cm. Boundaries between soil materials are generally clear but can be gradual [Red Podzolic Soils (Dr2.41, Dr3.11), Chocolate Soils (Db3.11)].

Lower slopes and drainage lines. Up to 50 cm of loamy sand overlies >100 cm sandy clay (**lu5**) [yellow podzolic soils (Dy4.22)]. In other locations up to 40 cm clay loam (**lu2**) overlies <50 cm sandy clay (**lu5**) and >100 cm whole-coloured medium clay (**lu3**). This is occasionally underlain by >150 cm mottled grey plastic clay (**lu4**) [prairie soils (Gn3.26)]. The boundaries between materials are clear or, less often, gradual. Total soil depth >200 cm.

LIMITATIONS TO DEVELOPMENT

Soil Limitations

lu1 High erodibility
Stoniness (localised)

lu2 Very hardsetting surface
Stoniness (localised)
Low available water capacity

lu3 Low wet strength
Low permeability (localised)
Low fertility
High shrink-swell (localised)
Low available water capacity

lu4 Low wet strength
Low permeability
Low available water capacity
Stoniness
Low fertility
High shrink-swell (localised)

lu5 Low wet strength
Low fertility
High shrink-swell (localised)
Very high aluminium toxicity
Low available water capacity

Fertility

The general fertility is low to moderate. The topsoil (**lu1**) has moderate fertility with high available water capacity, moderate amounts of organic matter, and moderate nutrient status. **lu2** normally has low to moderate fertility with low available water capacity, moderate organic matter content, low CEC, and intrinsically low nutrient status. All the other soil materials have low fertility with low available water capacities, moderate CEC and generally low nitrogen and very low phosphorus levels (**lu3–lu5**).

Erodibility

lu1 and **lu2** have moderate erodibility as they have moderate organic matter percentage, have stable aggregates and are well graded. All the other soil materials are moderately erodible as they are finely graded with relatively stable aggregates. **lu3–lu5** clays may be locally dispersible and, in those circumstances, should be considered highly erodible.

Erosion Hazard

The erosion hazard for non-concentrated flows ranges from moderate to very high. The calculated soil loss for the first twelve months of urban development ranges up to 135 t/ha for topsoil and up to 97 t/ha for exposed subsoil. The erosion hazard for concentrated flows is high to very high.

Surface Movement Potential

Moderately reactive soil materials. Soils are deep and have high clay content. Clay often has low to moderate shrink-swell potential. Tall trees are common on this landscape.

Landscape Limitations

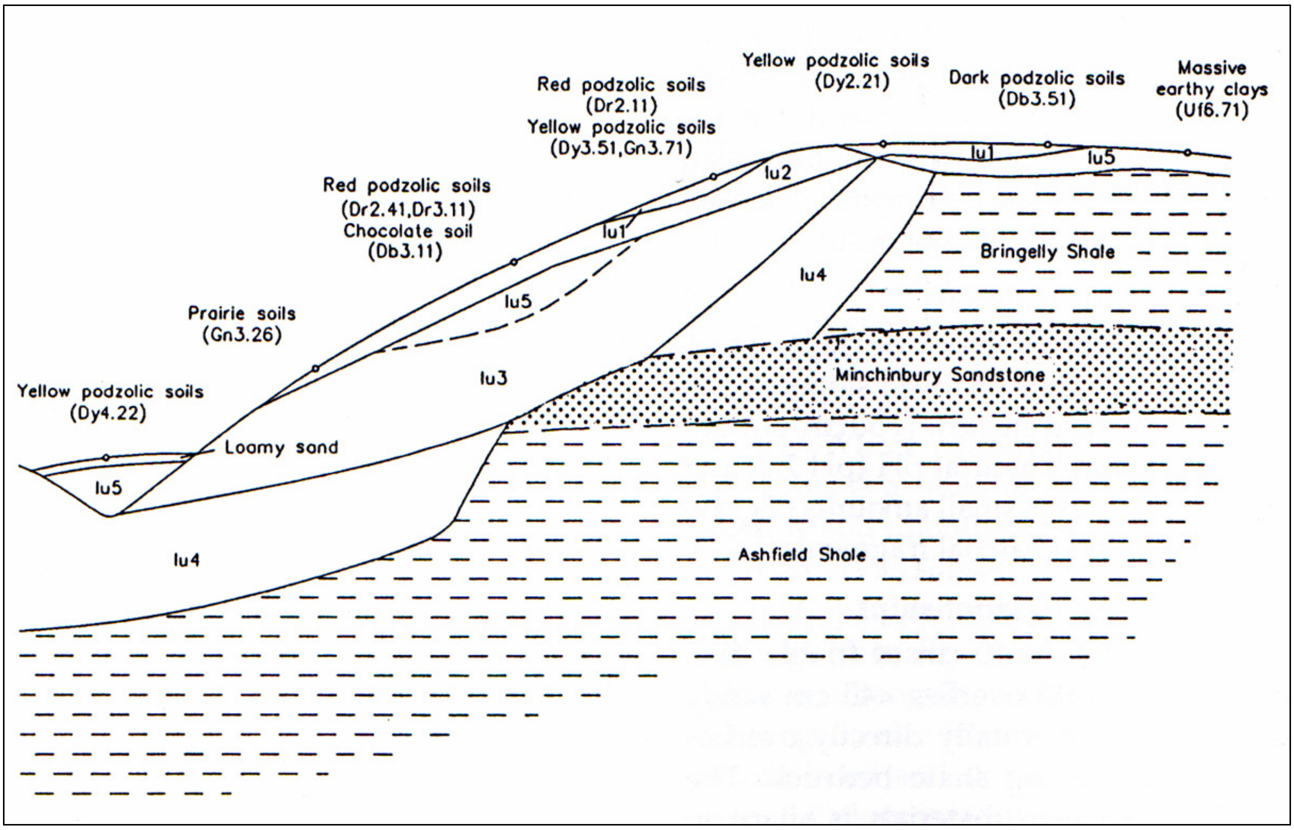
Water erosion hazard, steep slopes (localised), mass movement hazard (localised), shallow soils (localised), surface movement potential (localised).

Urban Capability

Low to moderate capability for urban development.

Rural Capability

Land generally capable of being grazed and regularly cultivated.



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

Landscape—floodplains, valley flats and drainage depressions of the channels on the Cumberland Plain. Usually flat with incised channels; mainly cleared.

Soils—often very deep layered sediments over bedrock or relict soils. Where pedogenesis has occurred Structured Plastic Clays (Uf6.13) or Structured Loams (Um6.1) in and immediately adjacent to drainage lines; Red and Yellow Podzolic Soils (Dr5.11, Dy2.41, Dr2.21) are most common terraces with small areas of Structured Grey Clays (Gn4.54), leached clays (Uf4.42) and Yellow Solodic Soils (Dy4.42, Dy5.23).

Limitations—flood hazard, seasonal waterlogging, localised permanently high watertables, localised water erosion hazard, localised surface movement potential.

LOCATION

This soil landscape comprises the present active floodplain of many drainage networks of the Cumberland Plain. This includes the South Creek, Eastern Creek, Ricabys Creek and Prospect Creek systems. Typical profiles and landscape can be seen on South Creek between Bringelly Road and Elizabeth Drive.

LANDSCAPE

Geology

Quaternary alluvium derived from Wianamatta Group shales and Hawkesbury Sandstone.

Topography

Flat to gently sloping alluvial plain with occasional terraces or levees providing low relief. Slopes <5%. Local relief <10m.

Vegetation

The vegetation of this soil landscape reflects its frequent inundation. Common tree species include *Angophora subvelutina* (broad-leaved apple), *Eucalyptus amplifolia* (cabbage gum) and *Casuarina glauca* (swamp oak). Still water species such as *Eleocharis sphacelata* (tall spike rush), *Juncus usitatus* and *Polygonum* spp. occur where channels are silted up. On more elevated streambanks a tall shrubland of *Melaleuca* spp. (paperbarks) and *Leptospermum* spp. (tea trees) may occur. However, much of this soil landscape has been previously cleared and is now dominated by exotic species such as *Rubus vulgaris* (blackberry) and other weeds.

Landuse

Most of this land is reserved for recreational use (playing fields, parks and reserves) or left unused. Some areas in the Prospect Creek system have been altered to provide lakes and dryland recreation space.

Existing Erosion

This is a dynamic soil landscape; there are many areas of erosion and deposition. Streambank erosion and sheet erosion of floodplains are common. In depositional phases streams may be partially or completely blocked by sedimentation or vegetated bars.

Associated Soil Landscapes

Small areas of Bakers Lagoon (**ba**) soil landscape occur in areas of interrupted drainage.

SOILS

Dominant Soil Materials

sc1—Brown apedal single-grained loam.

This is a brown sandy loam to sandy clay loam with generally apedal single-grained structure and porous earthy fabric. It commonly occurs as topsoil (A horizon).

Colours range from dull reddish brown (5YR 4/3) to dull yellowish brown (10YR 4/3). This material is usually moderately acid (pH 5.5) but varies from strongly acid (pH 4.5) to slightly acid (pH 6.5). Small (2–6 mm) angular or rounded gravels may occur. Roots are abundant in surface layers, charcoal and other inclusions do not occur.

sc2—Dull brown clay loam.

This is a hardsetting dull brown clay loam to fine sandy clay loam, usually with apedal massive structure and porous earthy fabric. It occurs as topsoil (A horizon).

Occasionally, weak structure occurs with small (2–5mm) rough-faced subangular blocky peds. Colour is usually dull brown (7.5YR 5/4) but has a range from greyish brown (5YR 4/2) to yellowish brown (10YR 5/6). pH varies from moderately acid (pH 5.5) to neutral (pH 7.0). Stones and other inclusions do not occur, and roots are rarely found.

sc3—Bright brown clay.

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

Occasionally this material contains sufficient fine sand to reach the texture grade of sandy clay. Peds are smooth-faced angular blocky or polyhedral and 20–50 mm in size. This material is generally whole-coloured ranging from reddish brown (5YR 4/8) to bright yellowish brown (10YR 5/1). Mottles, when they do occur, are yellow or grey and occupy up to 15% of the volume of the material. pH is highly variable, ranging from extremely acid (pH 3.0) to neutral (pH 7.0). Roots are only present where this material occurs as topsoil. There is no charcoal but small (2–20 mm) subrounded or subangular gravels may make up to 50% of the volume.

Associated Soil Materials

Dark brown sand. This material is a sandy layer which occurs on the surface as splay deposits in some swales. Texture ranges from sand to clayey sand. It is apedal single-grained and depth varies from 50–100 cm. It is highly erodible and has a pH range of 5.0 to 6.0.

Occurrence and Relationships

In channel. Variable depth sandy clay loam (**sc1**) over bright brown mottled medium clay (**sc3**) [Brown and Yellow Podzolic Soils (Dy3.51, Db2.21, Dy4.42, Dy3.11, Db2.41)]. Soil materials reoccur down through the soils in layers which can sometimes be related to major flood events. Smaller events either remove, or remove and replace, surface material. Sedimentation has a greater influence than pedogenesis in this environment.

Near channel. 30–50 cm friable to loose sandy loam (**sc1**) overlies 15 cm apedal massive clay loam (**sc2**), and 70 cm of light-medium clay (**sc3**). Swales are sometimes filled by sand splays [Structured Plastic Clays (Uf6.12) or Structured Loams (Um6.1)].

Low terrace. 2–50 cm sandy clay loam (**sc1**) overlies 15 cm apedal massive clay loam (**sc2**) and 60–85 cm whole-coloured medium to heavy clay (sometimes medium textured sandy clay) (**sc3**) [Red and Yellow Podzolic Soils (Dr5.11, Dr2.21, Dy141)].

High terrace. Up to 190 cm of stratified clay (light to medium) (**sc3**) over shale bedrock [leached clays (Uf4.43).

LIMITATIONS TO DEVELOPMENT

Soil Limitations

sc1 High erodibility

sc2 High erodibility (localised)
Hardsetting surface
Strongly acid
Low fertility

sc3 Shrink-swell potential (localised)
Stoniness (localised)
Very high erodibility
Saline
Low fertility

Fertility

General fertility is low. The surface soil material (**sc1**) has low CEC and low nitrogen and phosphorus. It is moderately acid and has low available water capacity. **sc2** also has low CEC with very low nitrogen and phosphorus. It is strongly acid and has a potential for a low level of aluminium toxicity. The deep subsoil material (**sc3**) has a high CEC and high intrinsic nutrient storage but is sodic and saline in some locations.

Erodibility

The erodibility of these soil materials is high. The topsoil (**sc1**) is moderately dispersible and has more than 50% fine sand, but it contains moderate amounts of organic matter. The subsoils (**sc2**,

sc3) have high fine sand and silt fractions with a very low percentage of organic matter.

Erosion Hazard

The erosion hazard for South Creek soil landscape is potentially very high to extreme. This is an active floodplain and is presently being reworked by fluvial processes. Apparent stability is probably short term. Streambank and gully erosion are common results of concentrated flow.

Surface Movement Potential

Generally low. Soils are often deep with high clay content. Subsoil materials are moderately reactive in some locations, while surface soils are generally stable.

Landscape Limitations

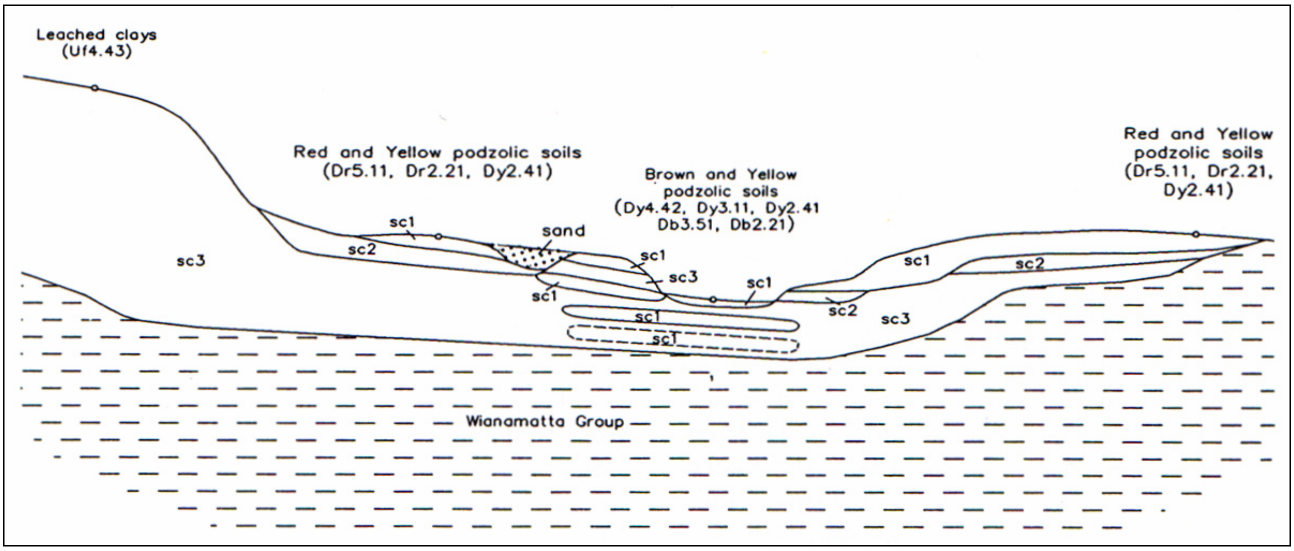
Flood hazard, seasonal waterlogging, permanently high watertables (localised), water erosion hazard (localised), surface movement potential (localised).

Urban Capability

Not capable of urban development due to flood hazard.

Rural Capability

Capable of supporting both grazing and regular cultivation.



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