Triple Eight Corporation Pty Ltd

Geotechnical Investigation & Preliminary Groundwater Assessment Report

Proposed Development at:
241-245 Pennant Hills Road
Carlingford NSW 2118

G25239-1-Rev A

16th September 2025

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

Report Distribution

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GCA Report No.:	G25239-1-Rev A
Date:	16th September 2025

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Report Revision	Details	Report No.	Date	Amended By
0	Original Report	G25239-1	11th August 2025	-
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Issued By:			**Joe Nader**	

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

SEARs Declaration

Declaration		
Name	Joe Nader	
Qualifications	B.E. (Civil – Construction), Dip.Eng.Prac., MIEAust., CPEng, RPEng, NER (3943418), RPEQ (25570) Cert. IV in Building and Construction *Member of AGS and ISSMGE* NSW Fair Trading DPR No.: DEP0000184 NSW Fair Trading PER No.: PRE0000174	
	The undersigned declares that this geotechnical engineering report has been prepared in response to the following SEARs requirements issued for the Project on 23rd May 2025 for SSD- SSD-84699461:	
SEARs Item No.	**SEARs Requirement**	**Relevant Section of this Report**
12.	Geotechnical Assessment	Section 2, Section 3, Section 4, Section 5 and Section 6
12.	Preliminary Groundwater Impact Assessment	Section 3.2, Section 3.3 and Section 5.7
Signed	Jnader	
Dated	16th September 2025	

TABLE OF CONTENTS

TABLES

APPENDICES

1. INTRODUCTION

1.1 Background

This geotechnical engineering report has been prepared by Geotechnical Consultants Australia Pty Ltd (GCA) on behalf of Triple Eight Corporation Pty Ltd (the client) at the Carlingford Unit for the construction and operation of a mixed-use development including infill affordable housing on land at nos. 241-245 Pennant Hills Road Carlingford NSW 2118 (the site).

Fieldwork was carried out on 3rd, 4th and 5th July 2025, in general accordance with our approved proposal (P1308-25.1, 27th May 2025).

This report has been prepared to assess the subsurface conditions of the site at the selected borehole locations, and to provide necessary recommendations from a geotechnical perspective for the proposed development.

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

This report accompanies a State Significant Development Application that seeks approval for the demolition and construction of a mixed-use residential and commercial development including infill affordable housing, which involves the following:

- Site preparation works, including demolition of existing structures, vegetation removal as necessary, and bulk excavation.
- Construction and operation of a mixed-use development comprising commercial premises, gymnasium, child care centre and shop top housing as follows:
    - One (1), 18-storey building (Building A) in the north-east corner of the site; and two (2), 4-5 storey buildings (Building B & C) along the western edge of the site.
    - A publicly accessible through site link between Pennant Hills Road and Felton Road.
    - Six (6) commercial tenancies at ground floor.
    - A three (3) storey gymnasium within the southern portion of Building C.
    - A ninety-eight (98) place child care centre located on level 01 of Building A.
    - 2,356m^2 of communal open space in the following locations.
        - The roof of Building A, B and C,
        - The two (2) storey podium of Building A.
        - The publicly accessible through site link
    - A total of 136 residential apartments across the three buildings, including 25 infill affordable apartments,
- Provision of three levels of basement parking with a total of 263 car parking spaces comprising:
    - 151 residential car parking spaces (including 24 accessible spaces);
    - 30 commercial car parking spaces (including 4 accessible spaces);
    - 25 child care parking spaces (including 4 accessible spaces);
    - 26 gymnasium parking spaces (including 2 accessible spaces);
    - 26 visitor car parking spaces (including 12 accessible spaces); and
    - 3 car share spaces.
- Street tree planting and landscaping, extension and augmentation of services and infrastructure as required. For a detailed project description, refer to the Environmental Impact Statement prepared by Planning Direction.

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

1.2 Provided Information

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

- Architectural drawings prepared by Kennedy Associates Architects, titled "Proposed Development, 241-245 Pennant Hills Road Carlingford NSW", and referenced No. 1804.
- Site survey plan taken from architectural drawings prepared by Kennedy Associates Architects, titled "Proposed Development 241-245 Pennant Hills Road Carlingford NSW", referenced drawing No. 1804-SD0050, revision No. P4 and dated 5th May 2025.

1.3 Proposed Development

Information provided by the client indicates that the proposed development comprises demolition of the existing infrastructure onsite, followed by construction of a multi-storey residential building complex, overlying three (3) basement levels. The three (3) basement levels are denoted as Level – 01, Level – 02 and Level – 03 on the architectural drawings listed in Section 1.2.

Finished Floor Levels (FFL)s of the proposed development from the architectural drawings are shown in Table 1, with Reduced Levels (RL)s relative to the Australian Height Datum (AHD).

Table 1: Summary of Finished Floor and Bulk Excavation Levels

Building Floor Level	Building Finished Floor Levels RL (m AHD)
Level 00 (Ground Floor)	106.895 to 108.700
Level – 01 (Basement Level 1)	103.575
Level – 02 (Basement Level 2)	100.575
Level – 03 (Basement Level 3)	97.575
Estimated Bulk Excavation Level[1]	~97.27

[1]Assumes a 300mm slab for Basement Level 3 for construction of the proposed development.

Based on this information and the existing site levels and topography, maximum anticipated excavation depths of approximately 12.6m to approximate level of RL97.27m AHD are expected to be required for construction of the proposed development. Locally deeper excavations for the lift shafts, building footings and service trenches are also anticipated as part of the proposed development.

It is noted that excavation depths are expected to vary across the site and are inferred from the FFLs shown on the architectural drawings, referenced in Section 1.2 above. The maximum anticipated excavation depth given above presumes an additional 300mm excavation for preparation of a concrete slab at the Level – 03, with bulk excavation level estimated at ~RL97.27m AHD.

1.4 Geotechnical Assessment Objectives

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

- General assessment of any potential geotechnical issues that may affect the proposed development and any surrounding infrastructure, buildings, council assets, etc.
- Excavation conditions and recommendations on excavation methods in soil and rock to restrict any ground vibrations.
- Vibration control and recommendations to restrict ground vibrations.
- Recommendations on suitable shoring (retention) systems for the site.
- Design parameters based on the ground conditions within the site for retaining walls, cantilever shoring walls and propped shoring.
- Recommendations on suitable foundation types and design for the site.
- End bearing capacities and shaft adhesion for shallow and deep foundations based on ground conditions within the site.
- Groundwater levels which may be determined during the site investigation and from additional groundwater readings within the installed standpipe piezometers, along with the effects on the proposed development.
- Approximate permeability rates of underlying soils and/or bedrock.
- Recommendations on groundwater maintenance and limiting inflow.
- Preliminary soil aggressivity and salinity assessment within the site based on laboratory test results at the selected borehole locations.
- Subsoil class for earthquake design for the site in accordance with Australian Standard (AS) 1170.4-2007.
- 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 and those given in the proposal (referenced GCA Proposal No. P1308-25.1). The scope of works included:

- Request and review Before You Dig Australia (BYDA) plans and any other plans provided by the client on existing buried services within the site.
- Service locating carried out by a specialist subcontractor using electromagnetic detection equipment to ensure the area is free of any underground services at the selected borehole locations.
- Review of site plans and drawings to determine appropriate testing locations, and to 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. The boreholes are identified as BH1, BH2 and BH3. The drilling rig is owned and operated by a specialist subcontractor.
    - All three (3) boreholes were advanced using NMLC diamond coring techniques from depths of approximately 2.96m to 15.08m in BH1, 2.49m to 15.56m in BH2 and from 5.85m to 25.00m in BH3, below the existing ground level within the site (bgl).
    - Standard Penetrometer Testing (SPT) was undertaken within the augered sections of the boreholes at an initial 0.5m depth, then at 1.5m depth to assess the approximate in-situ soil strength.

        - Engineering borehole logs are presented in **Appendix D** with accompanying explanatory notes in **Appendix C** and the rock core photograph in **Appendix E**.
- Installation of three (3) standpipe piezometers in boreholes BH1, BH2 and BH3, and identified as GW1, GW2 and GW3, respectively. The standpipe piezometers were installed to take groundwater measurements and for future groundwater monitoring and permeability testing.
    - The approximate locations of the boreholes and standpipe piezometers are shown on **Figure 1 and Figure 2, Appendix B** of this report.
- Collection of soil and rock samples during drilling, for testing as follows:
    - Nine (9) selected soil samples collected during auger drilling of the boreholes were sent for testing to a National Association of Testing Authorities, Australia (NATA) accredited laboratory (ALS Environmental) to determine the pH, chloride content, sulphate content and electrical conductivity of the selected samples.
    - Rock core recovered from boreholes BH1, BH2 and BH3 were boxed, logged and sent to our affiliate NATA accredited laboratory, Geo-Teknix Solutions Pty Ltd (Geo-Teknix Solutions), for rock strength testing at approximately 1.0m intervals to assess the point load strength index (Is_{50}) values. The rock core photographs and laboratory point load test results certificate are presented in **Appendix E** and **Appendix F**, respectively.
- Preparation of this geotechnical engineering report.

The borehole drilling depths are summarised in Table 2 below.

Table 2: Approximate Borehole Drilling Depths

Borehole ID	Date Drilled	Augering Depth/Thickness (m bgl)	NMLC Diamond Coring Technique Depth/Thickness (m bgl)	Total Borehole Depth (m bgl)
BH1/GW1	3rd July 2025	0.0 – 2.96	2.96 – 15.08	15.08
BH2/GW2	3rd July 2025	0.0 – 2.49	2.49 – 15.56	15.56
BH3/GW3	4th to 5th July 2025	0.0 – 5.85	5.85 – 25.00	25.00

1.6 Constraints

The discussions and recommendations provided in this report are based on the results obtained during borehole drilling onsite. It is recommended that an experienced geotechnical engineer undertake further geotechnical inspections during construction to confirm the subsurface conditions across the site and that the foundation bearing capacities are achieved.

2. SITE DESCRIPTION

2.1 Overall Site Description

The overall site description and its surrounding are presented in Table 3 below.

Table 3: Overall Site Description and Site Surroundings

Information	Details
Overall Site Location	The site is identified as nos. 241-245 Pennant Hills Road, Carlingford within the City of Parramatta Local Government Area (LGA). It is situated in the western portion of the Carlingford Local Centre and is located approximately 18km north-west of the Sydney Central Business District (CBD). The site is an irregular triangular shape and comprises approximate frontages of 140m to Pennant Hills Road to the south-east, 100m to Felton Road to the north and 135m to the adjoining properties to the west. The site comprises a total area of 6,331m^2, with the entirety of the site under single ownership.
Site Address	241-245 Pennant Hills Road Carlingford NSW 2118
Approximate Site Area[1]	6,331m^2
Local Government Authority	City of Parramatta
Site Description	The site contained a multi-storey commercial building with some scattered mature trees. The areas surrounding the footprint of the existing development were mostly paved.
Approximate Distances to Nearest Watercourses (i.e. rivers, lakes, creeks, etc.)	• The Ponds Creek tributary – 670m south-east of the site, at a lower elevation. • Vineyard Creek – 940km south-west of the site, at a lower elevation. • Hunts Creek – 650m north-west of the site, at a lower elevation.
Site Surroundings	The site is located within an area of residential use and is bounded by: • Felton Road carriageway to the north. • Pennant Hills Road thoroughfare to the east and south. • Residential property at nos. 237-239 Pennant Hills Road and vacated site at No. 6 Felton Road to the west.

[1] Site area is approximate and obtained from the site survey plan referenced in Section 1.2.

Figure 3: Site Aerial Map (Nearmap, edits by Kennedy Associates Architects)

A summary of the site description in Figure 3 is given below, for a total site area of approximately 6,331m^2:

1. 241-245 Pennant Hills Road, Lot 1 DP805059.
2. 241-245 Pennant Hills Road, Lot 1 DP805059.
3. 241-245 Pennant Hills Road, Lot 1 DP805059.
4. 241-245 Pennant Hills Road, Lot 1 DP805059.

2.2 Topography

The local and site topography are based on spot heights and contour lines on the site survey plan in Section 1.2, reference to Mecone Mosaic and NSW SDT Explorer, and from onsite observations during the fieldwork.

The site is on a local high point, with the ground surface gently to moderately sloping away from the site on all sides. Within the site, the ground generally slopes moderately to the south, from ~RL109.6m AHD in the northern portion of the site to ~RL106m AHD in the southern portion of the site.

It should be noted that the site topography, levels and slopes are approximate and based on observations made during the geotechnical investigation and the site survey plan referenced in Section 1.2. The actual topography in areas inaccessible during the site investigation, along with the site and local topography and levels are expected to vary from those outlined in this report.

2.3 Regional Geology

Information obtained on the local regional subsurface conditions, referenced from the Department of Mineral Resources, Sydney 1:100,000 Geological Series Sheet 9130 Edition 1, dated 1983, by the Geological Survey of New South Wales, indicates that the site is located within a region underlain by the Ashfield Shale (Rwa) of the Wianamatta Group. The Ashfield Shale (Rwa) typically consists of "black to dark-grey Shale and laminite".

GCA notes that the site is approximately 390m south of a lithological contact with the underlying Triassic Aged Hawkesbury Sandstone (Rh) consisting of "medium to coarse grained quartz lithic Sandstone, very minor Shale and laminite lenses". The contact is at a lower elevation at an approximate level of RL90m AHD, approximately 19m lower than the north end of the site.

Similarly, reference made to MinView 2025 (minview.geoscience.nsw.gov.au) indicates that the site is underlain by the Ashfield Shale (Twia) and is close to the Hawkesbury Sandstone (Tuth).

A review of the regional maps by the NSW Government Environment and Heritage shows that the site is set within the Glenorie (gn) landscape group, which is normally recognised by undulating low hills on Wianamatta Group Shales.

Soils of the Glenorie group typically have a high soil erosion hazard, localised impermeable highly plastic soil, and are moderately reactive. Local reliefs are approximately 50m to 80m and slopes of approximately 5% to 20% in gradient. The dominant soil types associated with the Glenorie landscape group typically contains either a loam/sandy loam or a clayey or silty clay material. This landscape group commonly contains shale rock fragments and gravels and may also contain charcoal fragments. Soils of the Glenorie group are generally slightly acidic (pH 6.5) to strongly acidic (pH 4.0).

GCA notes the site lies close to the West Pennant Hills (wp) soil landscape group. This landscape group is generally recognised by rolling to steep side slopes on Wianamatta Group Shales and shale colluvium. Local reliefs are typically between forty (40) metres and two hundred (200) metres, with slopes grading at greater than 20%. Limitations of this soil landscape group include the potential for mass movement hazard, steep slopes, high soil erosion hazard, localised seasonal waterlogging, impermeable plastic shrink-swell of the subsoil. The dominant soil types of this landscape group typically consist of clayey materials and weathered shale. Fine-grained sandstone and ironstone fragments are common. Soils of the West Pennant Hills Group are usually extremely acidic (pH 4.0) to slightly acidic (pH 6.0).

The Glenorie (gn) landscape and West Pennant Hills (wp) group reports are attached in **Appendix I**.

3. SUBSURFACE CONDITIONS AND ASSESSMENT RESULTS

3.1 Stratigraphy

Surface and subsurface conditions from the geotechnical investigation are summarised in Table 4 below. Reference should be made to the detailed engineering borehole logs presented in **Appendix D**, in conjunction with the geotechnical explanatory notes in **Appendix C** and the rock core photographs presented in **Appendix E**.

Fill, soil and rock description provided in the detailed engineering borehole logs are based on AS 1726-2017 “Geotechnical Site Investigations”, with rock classification in Table 4 in accordance with Pells P.J.N, Mostyn G. & Walker B.F., Foundations on Sandstone and Shale in the Sydney Region, Australian Geomechanics Journal, December (1998.

The process of determining the rock classification involves considering the estimated rock strength, spacing between defects (such as bedding planes, joints etc.) and percentage thickness of seams present in the underlying bedrock.

Estimated soil consistency and strength is assessed by observing the auger drilling penetration resistance, by tactile assessment and from the SPT results. The soil consistency and strengths given are approximate. Soil strength variation is expected across the site, predominantly in areas and at depths not encountered during this geotechnical investigation.

In addition, rock strength is estimated from observing the auger drilling penetration resistance and tactile assessment of auger cuttings, and from the results of point load strength index (Is_{50}) tests carried out at approximately 1.0m intervals on rock core obtained from boreholes BH1, BH2 and BH3. Rock strength in the underlying bedrock is expected to vary throughout the site, and it is noted that auger penetration rates within certain bedrock formations vary from drilling rig to drilling rig. The rock strength values provided in this report are approximate.

Clay seams, defects and fractured or extremely weathered zones are expected to occur across the site within the underlying bedrock, including 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.

It is recommended that an experienced geotechnical engineer confirm the subsurface materials exposed during excavation and construction. It should also be noted that ground conditions within the site may differ from those encountered and inferred in this report, since no geotechnical or geological exploration program, no matter how comprehensive, can reveal and identify all subsurface conditions underlying the site.

Table 4: Summary of Subsurface Conditions

Borehole ID			**BH1**	**BH2**	**BH3**
Approximate Surface RL at Borehole Location (m AHD)[4]			RL106.4	RL109.3	RL109.1
Unit	**Unit Type**	**Estimated Consistency/ Strength**	**Depth/Thickness of Unit (m bgl) (RL m AHD)**		
1a	FILL: Silty Clay low to medium plasticity	N/A	0.0 – 0.5 (RL106.4 – 105.9)	–	0.05 – 0.6 (RL109.05 – 108.5)
1b	FILL: Silty Sand, fine to medium grained.	N/A	–	0.0 – 0.5 (RL109.3 – 108.8)	–
2a	NATURAL SOILS. Silty CLAY, medium to high plasticity	Stiff	0.5 – 0.9 (RL105.9 – 105.5)	0.5 – 1.9 (RL108.8 – 107.4)	0.6 – 1.0 (RL108.5 – 108.1)
2b	Extremely Weathered Material (Shale)[1] Silty CLAY, medium to high plasticity, pale grey	Soil Strength (Very Stiff)	0.9 – 1.8 (RL105.5 – 104.6)	–	–
3	SHALE, with extremely weathered clay seams (inferred Class V Shale)[2, 3]	VL – L	1.8 – 3.0 (RL104.6 – 103.4)	1.9 – 2.5 (RL107.4 – 106.8)	1.0 – 2.5? (RL108.1 – 106.6)
4	Class V Shale[2]	VL – L	3.0 – 6.9 (RL103.4 – 99.5)	2.5 – 6.8 (RL106.8 – 102.5)	2.5? – 4.5? (RL106.6 – 104.6) 13.8 – 14.9 (RL95.3 - 94.2)
5	Class IV Shale[2]	L – M	6.9 – 9.2 (RL99.5 – 97.2)	–	4.5? – 8.0 (RL104.6 – 101.10)
6	Class III Shale[2]	M	–	6.8 – 10.2 (RL102.5 – 99.1)	–
7	Class III/II Shale[2, 5]	M – H	9.2 – 15.08 (RL97.2 – 91.32)	10.2 – 15.56 (RL99.1 – 93.74)	8.0 – 13.8 (RL101.1 – 95.3) 14.9 – 25.00 (RL94.2 – 84.10)

[1] Extremely weathered material (Shale) is generally of soil strength, has been logged as a soil in accordance with AS1726:2017 and is grouped with the natural clay soils.

[2] An experienced geotechnical engineer should confirm the underlying bedrock composition, class, depth and estimated strength by further cored borehole drilling and rock strength testing, and during excavation by inspection and appropriate testing (i.e. spoon testing, rock strength testing, etc.).

[3] This material could not be classified as it was drilled with auger only.

[4] RLs for each borehole are approximate and have been interpreted from the site survey plan referenced in Section 1.2.

[5] Additional cored boreholes are required prior to construction to confirm the presence of Class III/II Shale bedrock across the site. Spoon testing is also required during construction of the proposed development to confirm the underlying Class III/II Shale bedrock.

Notes:

- N/A = Not applicable, VL = Very low estimated strength, L = Low estimated strength, M = Medium estimated strength, H = High estimated strength.
- Clay seams, defects and fractured/extremely weathered zones are expected to be present throughout the underlying bedrock, predominantly at depths and locations unobserved during the geotechnical investigation.

- Rock strength is approximate and based on observations made during auger penetration resistance at the time of drilling, tactile assessment of rock core and auger cuttings and point load strength index (Is_{50}) tests carried out at approximately 1.0m intervals.
- Ground conditions are expected to vary across the site and should be confirmed by a geotechnical engineer during excavation by inspection, predominantly in areas not assessed during the geotechnical investigation.

3.2 Groundwater

No groundwater was observed during auger drilling of the boreholes to depths of approximately 2.49m bgl, 2.96m bgl and 5.85m bgl in boreholes BH1, BH2 and BH3 respectively.

Water introduced during the NMLC coring process in the boreholes below these augering depths, precluded any groundwater level indications.

Following completion of drilling of boreholes BH1, BH2 and BH3, standpipe piezometers identified as GW1, GW2 and GW3, were installed to depths of approximately 15.08m, 15.56m and 25.00m bgl, respectively, to allow for groundwater level monitoring and permeability testing.

After installation of the standpipe piezometers, the water present in the wells was purged using a bailer, removing most of the water introduced for the NMLC coring process. GW2 was purged dry; however, GW1 and GW3 were unable to be purged dry, with the drillers noting water loss from around ~13.5m bgl in GW1.

It is noted that although all efforts were made to purge all of the water present within the standpipe piezometers completely, there is the possibility of groundwater being present at lower depths.

A summary of the well construction details for each standpipe piezometer is presented in Table 5 below.

Table 5: Standpipe Piezometer Construction Details

Standpipe Piezometer	Well Stickup (m Above Ground Level)	Blank Sections Depth/Thickness (m bgl)	Slotted Section Depth/Thickness (m bgl)	Total Well Depth (m bgl)
BH1/GW1	0.13m	0.0 – 1.58m	1.58 – 15.08m	15.08m
BH2/GW2	-0.04m	0.04 – 9.56m	9.56 – 15.56m	15.56m
BH3/GW3	0.00m	0.0 - 8.50m 14.50 – 25.00m	8.50 – 14.50m	25.00m

Groundwater level measurements were taken within each of the standpipe piezometers installed at the site and were measured on five (5) occasions on 9th July 2025, 16th July 2025, 23rd July 2025, 28th July 2025 and 6th August 2025.

The approximate groundwater level measurements are shown in Table 6 below.

Table 6: Approximate Groundwater Levels

Borehole ID (Surface Level)	Date	Approximate Groundwater Depth (m bgl/RL m AHD)
BH1/GW1 (RL106.4m AHD)	9th July 2025	9.27m bgl/ RL97.1m AHD
	16th July 2025	9.33m bgl/ RL97.1m AHD
	23rd July 2025	9.33m bgl/ RL97.1m AHD
	28th July 2025	9.3m bgl/ RL97.1 m AHD
	6th August 2025	9.15m bgl/ RL97.3m AHD
BH2/GW2 (RL109.3m AHD)	9th July 2025	3.58m bgl/ RL105.7m AHD
	16th July 2025	4.39m bgl/ RL104.9m AHD
	23rd July 2025	5.14m bgl/ RL104.2m AHD
	28th July 2025	5.48m bgl/ RL103.8m AHD
	6th August 2025	4.81m bgl/ RL104.5m AHD
BH3/GW3 (RL109.1m AHD)	9th July 2025	7.38m bgl/ RL101.7m AHD
	16th July 2025	7.64m bgl/ RL101.5m AHD
	23rd July 2025	7.6m bgl/ RL101.5m AHD
	28th July 2025	7.58m bgl/ RL101.5m AHD
	6th August 2025	7.4m bgl/ RL101.7m AHD

Thus, based on information available at the time of the investigation and geological position of the site, groundwater which may be present within the site is *expected* to be in the form of seepage through voids within the underlying fill material and pore spaces between particles of unconsolidated natural soils, and predominantly through networks of fractures and solution openings in consolidated Shale bedrock underlying the site. Seepage may also occur at the fill to natural soils and natural soils to bedrock contacts, predominantly following heavy rainfall events.

From the architectural plans, bulk excavation level is estimated to be at ~RL97.27m AHD (300mm below Basement Level 3 FFL of RL97.575m AHD). The measured groundwater levels in Table 6 above indicate that groundwater inflow is very likely to be encountered during excavation, above bulk excavation level, and is *expected* to be in the form of seepage.

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

It is recommended that groundwater monitoring is performed prior to and during construction to assess any groundwater inflows within the excavation areas. Further discussion and groundwater recommendations are provided in Section 3.3 below and Section 5.7. Where groundwater conditions vary from those outlined in this report, please contact GCA for further advice.

3.3 Preliminary Permeability Assessment (k)

GCA conducted a groundwater inflow assessment on 28th July 2025 within each of the installed standpipe piezometers to determine the approximate coefficient of permeability (k). Two (2) methods were used to calculate the coefficient of permeability (k), which are the established formulae used by Bouwer & Rice (1976) and Hvorslev (1951).

Prior to the inflow tests, the standing groundwater levels were measured at approximately:

- BH1/GW1: 9.30m bgl/RL97.1m AHD
- BH2/GW2: 5.48m bgl/RL103.8m AHD
- BH3/GW3: 7.58m bgl/RL101.5m AHD

Graphs of the results, with the calculated coefficient of permeability from Bouwer & Rice (1976) and Hvorslev (1951) methods are shown in **Appendix J,** and the coefficients of permeability calculated from these methods are presented in Table 7 below.

Table 7: Approximate Coefficient of Permeability (k)

Piezometer/ Borehole ID	Test Date	Initial Groundwater Level Reading (after bailing m bgl)	Approximate Coefficient of Permeability (k) (m/day) (Bouwer & Rice)		Approximate Coefficient of Permeability (k) (m/day) (Hvorslev)	Unit Type (Screen)[1]
BH1/GW1	28th July 2025	10.19	0 to 4 minutes	0.045	0.026	Bedrock
			4 to 15 minutes	0.0099		
BH2/GW2	28th July 2025	14.61	0.0015		N/A[2]	Bedrock
BH3/GW3	28th July 2025	21.00	0 to 5 minutes	0.038	0.032	Bedrock
			5 to 30 minutes	0.0039		

[1] Refer to the detailed engineering borehole logs in **Appendix D** for descriptions of the aquifer material.

[2] Standpipe piezometer BH2/GW2 did not achieve sufficient recharge to apply the Hvorslev equation. Groundwater levels should recover to 37% of the head ratio between the current head and the initial head to apply the Hvorslev equation to assess the coefficient of permeability (k).

Note that these initial inflow rates do not account for the potential of the shoring walls to reduce inflow rates. It is also noted that higher permeability rates in BH3/GW3 were encountered below bulk excavation level.

These permeability results are preliminary only and further discussion on groundwater recommendations, including recommendations on additional testing is provided in Section 5.7.

The groundwater assessment presented in this report must be considered as preliminary in nature.

4. LABORATORY TEST RESULTS

4.1 Soil Aggressivity and Salinity

Nine (9) 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 8 below, with laboratory certificates presented in **Appendix G** of this report.

Table 8: Summary of Laboratory Test Results (Aggressivity and Salinity)

Borehole ID		**BH1**	**BH1**	**BH1**	**BH2**	**BH2**
Approximate Depth (m bgl)		0.5 – 0.85	1.4 – 1.5	2.4 – 2.6	0.5 – 0.95	1.5 – 1.6
Strata Type		Natural Soils	Natural Soils	Bedrock	Natural Soils	Natural soils
Aggressivity and Salinity	**pH**	4.5	4.7	5.1	4.4	4.6
	Moisture Content (%)	26.6	9.0	14.6	20.2	18.2
	Chloride (mg/kg)	<50	<10	<10	20	30
	Sulphate SO_4 (mg/kg)	40	30	30	120	100
Electrical Conductivity (µS/cm)	**EC (µS/cm)**	31	26	27	83	83
	EC (dS/m)	0.031	0.026	0.027	0.083	0.083
	Multiplication Factor[1]	8	8	15	8	8
	Saturation Extract ECe (dS/m)	0.2	0.2	0.4	0.7	0.7

Borehole ID		**BH2**	**BH3**	**BH3**	**BH3**	–
Approximate Depth (m bgl)		2.1 – 2.3	0.5 – 0.95	1.5 - 1.8	5.6 – 5.8	
Strata Type		Bedrock	Natural Soils	Natural Soils	Bedrock	
Aggressivity and Salinity	**pH**	5.0	4.9	5.4	5.8	
	Moisture Content (%)	10.7	22.8	9.3	8.8	
	Chloride (mg/kg)	<10	<10	10	<10	
	Sulphate SO_4 (mg/kg)	80	80	20	40	
Electrical Conductivity (µS/cm)	**EC (µS/cm)**	58	56	21	39	–
	EC (dS/m)	0.058	0.056	0.021	0.039	
	Multiplication Factor[1]	15	8	8	15	
	Saturation Extract ECe (dS/m)	0.9	0.4	0.2	0.6	

[1] Multiplication 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.

Assessment of the results are presented in Section 5.3.

4.2 Rock Strength Testing

Rock strength testing was performed on thirteen (13), fourteen (14) and twenty (20) rock cores obtained from boreholes BH1, BH2 and BH3, respectively. These samples were tested by our affiliate NATA accredited laboratory, Geo-Teknix Solutions, to determine the diametral and axial point load strength index (I_{s50}).

The results are presented in Table 9, Table 10 and Table 11 below with the indicative rock strengths and unconfined compressive strengths. The point load test results are also shown on the engineering borehole logs (see **Appendix D**).

Table 9: Point Load Index (I_{s50}) Laboratory Test Results for BH1

Borehole ID	Approximate Testing Depth (m bgl)	Rock Type	Point Load Index (I_{s50})[1]		Indicative Unconfined Compressive Strength (MPa)	Estimated Rock Strength
			Axial (MPa)	Diametral (MPa)		
BH1	3.52	Shale	0.10	0.02	1.8	Very Low to Low
	4.49	Shale	0.04	0.02	0.7	Very Low
	5.37	Shale	0.08	0.05	1.4	Very Low
	6.80	Shale	0.12	0.06	2.2	Low
	7.63	Shale	0.15	0.03	2.7	Low
	8.80	Shale	0.11	0.06	2.0	Low
	9.65	Shale	0.68	0.40	12.2	Medium
	10.40	Shale	0.56	0.56	10.1	Medium
	11.63	Shale	0.46	0.33	8.3	Medium
	12.52	Shale	0.89	0.37	16.0	Medium
	13.30	Shale	0.67	0.34	12.0	Medium
	14.40	Shale	1.69	0.95	30.4	High
	15.04	Shale	2.11	0.37	38.0	High

[1] Approximate indicative Unconfined Compressive Strength (UCS) can be obtained by using a multiplication factor of eighteen (18) for Shale bedrock multiplied by the axial point load I_{s50}.

Table 10: Point Load Index (Is_{50}) Laboratory Test Results for BH2

Borehole ID	Approximate Testing Depth (m bgl)	Rock Type	Point Load Index (Is_{50})[1]		Indicative Unconfined Compressive Strength (MPa)	Estimated Rock Strength
			Axial (MPa)	Diametral (MPa)		
BH2	2.83	Shale	1.06	0.22	19.1	High
	3.15	Shale	0.31	0.02	5.6	Low to Medium
	4.9	Shale	0.06	0.05	1.1	Very Low
	5.42	Shale	0.18	0.06	3.2	Low
	6.87	Shale	0.37	0.03	6.7	Medium
	7.78	Shale	0.36	0.02	6.5	Medium
	8.64	Shale	0.44	0.05	7.9	Medium
	9.40	Shale	0.56	0.12	10.1	Medium
	10.18	Shale	0.51	0.33	9.2	Medium
	11.48	Shale	1.04	0.03	18.7	High
	12.43	Shale	0.91	0.49	16.4	Medium
	13.77	Shale	0.51	0.28	9.2	Medium
	14.54	Shale	0.67	0.06	12.1	Medium
	15.20	Shale	1.22	0.13	22.0	High

[1] Approximate indicative Unconfined Compressive Strength (UCS) can be obtained by using a multiplication factor of eighteen (18) for Shale bedrock multiplied by the axial point load Is_{50}.

Table 11: Point Load Index (Is_{50}) Laboratory Test Results for BH3

Borehole ID	Approximate Testing Depth (m bgl)	Rock Type	Point Load Index (Is_{50})[1]		Indicative Unconfined Compressive Strength (MPa)	Estimated Rock Strength
			Axial (MPa)	Diametral (MPa)		
BH3	5.91	Shale	0.12	0.06	2.2	Low
	6.42	Shale	0.28	0.15	5.0	Low
	7.75	Shale	5.63	0.70	101.3	Very High
	8.58	Shale	0.53	0.21	9.5	Medium
	9.75	Shale	0.66	0.20	11.9	Medium
	10.63	Shale	0.58	0.06	10.4	Medium
	11.50	Shale	0.62	0.10	11.1	Medium
	12.65	Shale	0.74	0.27	13.3	Medium
	13.72	Shale	0.49	0.79	8.8	Medium
	14.85	Shale	1.63	0.31	29.3	High
	15.70	Shale	1.89	0.88	34.0	High
	16.63	Shale	2.08	1.55	37.4	High
	17.35	Shale	2.55	1.67	45.9	High
	18.63	Shale	2.09	1.28	37.6	High
	19.29	Shale	2.13	1.76	38.3	High
	20.35	Shale	1.88	1.25	33.8	High
	21.55	Shale	2.61	1.40	47.0	High
	22.35	Shale	2.30	0.89	41.4	High
	23.50	Shale	1.55	0.74	27.9	High
	24.67	Shale	1.09	1.15	19.6	High

[1] Approximate indicative Unconfined Compressive Strength (UCS) can be obtained by using a multiplication factor of eighteen (18) for Shale bedrock multiplied by the axial point load Is_{50}.

It is noted that variable higher and lower strength rock bands (including clay seams) are expected to occur within the underlying bedrock throughout the site, including at locations and depths not assessed during the geotechnical investigation. GCA recommends not precluding the possibility of encountering variable higher and lower strength rock bands (including clay seams) 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 infrastructure that fall within the "zone of influence" of the proposed excavation and vicinity of the proposed development.

The "zone of influence" is defined as the zone created by drawing a 45° line above horizontal at the boundaries of the excavation at bulk excavation level, into the excavation faces, to the surface.

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

5.2 General Geotechnical Issues

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

- Soil aggressivity and salinity assessment.
- Inspection test pits and underpinning.
- Excavation conditions.
- Vibration monitoring.
- Groundwater management.
- Stability of excavation and retention of adjoining properties and infrastructure.
- Site earthquake classification.
- Foundations and founding material.
- Subgrade preparation and filling.

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

5.3 Soil Aggressivity and Salinity Assessment

In accordance with AS 2159-2009 “Piling – Design and Installation” (as outlined in Table 12 below), the results of laboratory tests and introduction of a multiplication factor for electrical conductivity indicates the following classification:

Table 12: Aggressivity and Salinity Reference Table

Reference	Element Type	Soil Condition A[1]	Soil Condition B[2]	pH	Chloride (mg/kg)	Sulphate SO_4 (mg/kg)
AS 2159-2009	**Concrete Elements**	Mild	Non	>5.5	N/A	<5,000
		Moderately	Mild	4.5 – 5.5		5,000 – 10,000
		Severely	Moderately	4.0 – 4.5		10,000 – 20,000
		Very Severely	Severely	<4.0		>20,000
	Steel Elements	Non	Non	>5.0	<5,000	N/A
		Mild	Non	4.0 – 5.0	5,000 – 20,000	
		Moderately	Mild	3.0 – 4.0	20,000 – 50,000	
		Severely	Moderately	<3.0	>50,000	
Dry Salinity 1993	Electrical Conductivity Saturation Extract ECe (dS/m) value range, based on an introduction of a multiplication factor from DNR publication.			Non-Saline <2 Slightly Saline 2 – 4 Moderately Saline 4 – 8 Very Saline 8 – 16 Highly Saline >16		

[1] Soil Condition A refers to high permeability soils that are in groundwater.

[2] Soil Condition B refers to low permeability soils or all soils above groundwater.

Applying Soil Condition B, the:

- Underlying natural soils are considered:
  - Non aggressive to buried steel structural elements.
  - Mildly to moderately aggressive to buried concrete structural elements.
  - Electrical conductivity of saturated extract (ECe) ranging from 0.2dS/m to 0.7dS/m, indicating generally “non” saline natural soils underlying the site.
- Underlying bedrock is considered:
  - Non aggressive to buried steel structural elements.
  - Mildly aggressive to buried concrete structural elements.
  - Electrical conductivity of saturated extract (ECe) ranging from 0.4dS/m to 0.9dS/m, indicating generally “non” saline bedrock underlying the site.

Note that soil aggressivity and salinity may vary throughout the site, and the assessment above is based on testing at the selected locations and depths indicated, in conjunction with multiplication factors for electrical conductivity. 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.

It is recommended to consider conducting additional machine drilled boreholes across the site and laboratory testing to confirm the preliminary findings above.

5.4 Inspection Pits and Underpinning

Consideration should be given to inspection pits for the existing adjacent residences and infrastructure, particularly where they fall within the "zone of influence" (obtained by drawing a line 45° above horizontal from the base of the proposed excavations) of the proposed development. It is recommended that inspection pits are excavated prior to any excavation or construction activities, as this 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 or rock, which will determine the need for additional support, such as underpinning, prior to installation of shoring piles, or any excavation and construction activities.

5.5 Excavation

Maximum anticipated excavation depths of approximately 12.6m to approximate level of RL97.27m AHD are expected to be required for construction of the proposed development. Locally deeper excavations for the lift shafts, building footings and service trenches are also anticipated as part of the proposed development.

Based on this information and the existing ground conditions encountered during the geotechnical investigation, it is anticipated that the excavation will extend through fill, natural soils, extremely weathered material, and Shale bedrock of variable estimated strength and weathering throughout the proposed development area, as discussed in Section 3**.**

Based on the boreholes carried out on the site, bulk excavation to an approximate level of ~RL97.27m AHD is likely to expose;

- Class IV Shale bedrock (found in BH1) and
- Class III/II Shale bedrock (found in BH2 and BH3).

Note that, in the BH1 borehole log, Class III/II Shale bedrock is encountered at approximately 300mm below bulk excavation level at the BH1 testing location.

It is recommended to refer to Section 3.1, which summarises the stratigraphy encountered in each borehole drilled at the site.

As the site is a large site at approximately 6,300m^2, additional rock cored boreholes are required prior to excavation and construction to confirm the presence of Class III/II Shale bedrock.

The possibility of encountering higher strength and class bedrock during excavation should not be precluded, predominantly in areas and at depths not assessed during the geotechnical investigation. Bedrock strength variation and higher strength rock bands are expected across the site area. It is recommended to refer to the detailed engineering borehole logs attached in **Appendix D** to determine approximate levels and depths of subsurface conditions at the site.

Therefore, GCA recommends consulting with subcontractors to discuss the feasibility and capability of excavation plant for the proposed development for the existing site conditions.

5.5.1 Excavation Assessment

Excavation through softer soils and very low to low strength Shale bedrock should be feasible using conventional earth moving excavators, typically medium to large hydraulic excavators, equipped with toothed buckets. Smaller sized excavators may face difficulty in high strength bands of soil and rock which may be encountered. Where high strengths bands are encountered, allow for rock breaking or ripping. Removal of the existing pavements and associated infrastructures within the site are also expected to require larger excavators.

Excavation of medium or better estimated strength bedrock, which is anticipated to be encountered, would require higher capacity excavators, bulldozers or similar, for effective removal of the rock. Excavating medium or higher strength bedrock would require the use of heavy ripping and rock breaking equipment or vibratory rock breaking equipment. It is recommended to conduct rock saw cutting (where required), using a mounted rock saw, around the perimeter of excavations, prior to any rock breaking commencing.

Furthermore, excavation for the proposed lift shafts, building footings and service trenches may require the use of heavy ripping and rock breaking equipment or vibratory rock breaking equipment, with the possibility of rock saw cutting.

If rock hammering is required for excavation in the underlying bedrock, these rock hammers should be used in short bursts, oriented away from the site boundaries and adjoining structures and into the proposed excavation area. It is recommended to monitor vibrations transmitted from rock hammering activities to ensure compliance with acceptable limits.

Demolition, excavation and construction activities (or the like) will generate both vibration and noise, predominantly whilst being carried out within the underlying Shale bedrock of medium or better estimated strength. Vibration control measures should be considered as part of the construction process, mainly where excavations are expected to be conducted within the underlying Shale bedrock of medium or better estimated strength, in the vicinity or within the zone of influence of any infrastructure or close to site boundaries.

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

5.6 Vibration Monitoring and Controls

Particular care will be required to ensure that adjacent buildings and infrastructure (i.e. road reserves, buildings, etc.), are not damaged during excavation and construction activities (or the like) due to excessive vibrations. Therefore, appropriate excavation and construction methods should be adopted which will limit ground vibrations to 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 to 3mm/sec.
- Residential and/or low-rise structures – 5mm/sec.
- Unreinforced and/or brick structures – 10mm/sec.
- Reinforced and/or steel structures – 25mm/sec.
- Commercial and/or industrial buildings – 25mm/sec.

Vibrations transmitted using rock hammers are unacceptable and are not recommended. To minimise vibration transmission to any adjoining infrastructure, and to ensure vibration levels remain within acceptable limits, perimeter rock saw cutting using a conventional excavator with a mounted rock saw (or similar) is required prior to any rock breaking commencing. 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.

Although rock hammering is unacceptable and not recommended, if necessary during excavation, it is recommended that hammering is conducted horizontally along pre-cut rock boulders or blocks formed by rock saw cutting.

Removal of rock using rock hammers should commence from the centre of the site, progressing towards the edges to minimise vibration transmission.

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

The effectiveness of all the above-mentioned approaches must be confirmed by the results of vibration monitoring. It is recommended to undertake an attended vibration monitoring trial, not in the zone of influence or vicinity of any sensitive asset. A geotechnical engineer should attend the site for the initial live vibration monitoring. By undertaking this vibration monitoring trial, it can inform the choice of plant and excavator equipment and distances of from sensitive receivers of the vibration, for each size of plant and rock hammer.

The limits of 5mm/sec and 10mm/sec are expected to be achievable if rock breaker equipment or other excavators are restricted to the equipment and values indicated in Table 13 below.

Table 13 provides a guide and the level of vibration induced in the ground by excavation plant chosen for the job should be checked onsite with initial live vibration monitoring.

Table 13: Rock Breaking Equipment Recommendations

Distance From Adjoining Structures (m)	Maximum PPV 5mm/sec[1]		Maximum PPV 10mm/sec	
	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 5mm/sec.

The development of a vibration monitoring plan is recommended to monitor construction activities and their effects on adjoining infrastructure, mainly where excavations are conducted within the underlying Shale bedrock of medium estimated strength (or better), within the zone of influence of, or close to adjoining infrastructure, assets and site boundaries.

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

A geotechnical engineer should be contacted immediately if vibrations measured during excavation and construction in adjacent structures exceed the values outlined above and work should immediately cease to re-assess excavation methods. Rock excavation methodology should also consider acceptable noise limits as per the "Interim Construction Noise Guideline" (NSW EPA).

It is recommended that a dilapidation report be carried out prior to any excavation or construction, as discussed in Section 5.1. This should be considered a "Hold Point".

5.7 Groundwater Management

Three (3) standpipe piezometers (GW1, GW2, and GW3) were installed in boreholes BH1, BH2, and BH3, respectively. Groundwater levels during GCA's final site visit on 6th August 2025 are listed below for reference, with the measured ground levels shown in Table 6 above.

- BH1/GW1: 9.15m bgl/RL97.3m AHD
- BH2/GW2: 4.81m bgl/RL104.5m AHD
- BH3/GW3: 7.4m bgl/RL101.7m AHD

Bulk excavation level is estimated at ~RL97.27m AHD (300mm below Basement Level 3 FFL of RL97.575m AHD) and the measured groundwater levels indicate that groundwater is very likely to flow into the excavation above bulk excavation level.

GCA anticipates that groundwater will flow into the site in the form of seepage through voids within the underlying soils and predominantly through networks of fractures and defects (such as bedding planes, joints, etc.) in the underlying weathered Shale bedrock. Seepage may also occur within the excavation areas through the fill material, at the fill/natural soil and natural soil/bedrock interfaces, predominantly after heavy rain.

The inflow testing undertaken shows that the site had a moderate to relatively high permeability. Results of the preliminary inflow testing undertaken at the site showed permeability values ranging from approximately 0.038m/day to 0.045m/day initial inflow rates in the first five (5) minutes of recharge, reducing to 0.0015m/day to 0.01m/day subsequently, based on the established methods by Bouwer & Rice (1976) and Hvorslev (1951). Calculated coefficients of permeability are given in Table 7 (see Section 3.3). It is noted the high rates of permeability, particularly those encountered in BH3/GW3, were measured below bulk excavation level.

Groundwater levels are variable across the site, with the highest average groundwater levels as follows:

- BH2. Surface level at RL109.3m AHD. Average groundwater level at RL9.28m AHD.
- BH3. Surface level at RL109.1m AHD. Average groundwater level at RL4.68m AHD.
- BH1. Surface level at RL106.2m AHD. Average groundwater level at RL7.52m AHD.

Difference in elevation of the top of groundwater between the wells is up to 4.6m.

Based on these preliminary results and our experience on the nature of groundwater within the local region, it is anticipated that groundwater is expected to flow initially as moderate to rapid seepage into the excavation, but flow rates are expected to decrease as the fractures within the underlying Shale bedrock are drained. Any groundwater seepage must be collected using conventional drainage measures such as those outlined below (not limited to):

- A conventional sump and pump system used both during construction and for permanent groundwater control below the basement level floor slabs.
- Drainage installed around the perimeter of the basement level behind all retaining walls, and below the slabs. This drainage should connect 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 are installed in conjunction with the above methods and connected to the building stormwater system.

To better assess groundwater levels and the potential for groundwater inflow into the excavation, it is recommended to continue measuring groundwater levels within the newly installed standpipe piezometers GW1, GW2 and GW3 within the site, prior to and during construction. Care must be taken to not damage standpipe piezometers GW1 and GW3 during construction.

It is also recommended to drill additional larger diameter test piles, say three (3) to four (4), of approximately ~1.0m diameter auger from the surface spread across the site, following demolition of the existing infrastructure onsite. An experienced geotechnical engineer or hydrogeologist should be onsite to direct these works and assess the results of this additional groundwater assessment.

Undertaking these additional groundwater assessments can confirm the preliminary findings mentioned in this report and provide additional information on groundwater levels, volumes and inflow rates, as it is very likely that groundwater will be intercepted during excavation. This assessment should also be carried through to ensure that a suitable drainage and retention system is implemented for the proposed development, as discussed in Section 5.8 below, and to confirm the hydrogeological characteristics of the site prior to excavation and construction.

Groundwater monitoring is recommended and should be implemented during the excavation stage to confirm the capacity of the drainage system and groundwater inflows entering the excavation area. The project geotechnical engineer, in conjunction with the project stormwater engineer, should review the results of groundwater monitoring undertaken at the site during construction.

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

5.8 Excavation Stability

Maximum anticipated excavation depths of approximately 12.6m to approximate level of RL97.27m AHD are expected to be required for construction of the proposed development. Locally deeper excavations for the lift shafts, building footings and service trenches are also anticipated as part of the proposed development.

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

Therefore, it is recommended that an experienced geotechnical engineer or engineering geologist inspect the exposed materials between any shoring pile gaps at an initial 0.5m depth, followed by 1.0m to 1.5m maximum excavation depth intervals, and then at bulk excavation level.

To maintain excavation stability, GCA recommends installing inclinometer tubes within the shoring piles to evaluate and track horizontal deflections during pile installation. The inclinometers should be installed at specific locations and to at least 3.0m below the planned basement FFL.

Regular inclinometer readings will be necessary, and survey monitoring of the excavation faces is also advised. Site-specific geotechnical input is essential for specifying inclinometer installation details and interpreting monitoring data. Care must be taken to avoid damage to the inclinometers during construction activities, such as interference from ground anchors, and to ensure they remain accessible. The use of specialist subcontractors may be required to carry out accurate deflection measurements.

5.8.1 Excavation Retention Support Systems

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

It is therefore recommended to adopt a suitable retention system such as a soldier pile wall solution, with piles sufficiently embedded into Class III Shale (or better) bedrock underlying the site, and concrete and reinforcement infill panels to support the excavation, including fill materials, natural soils and extremely weathered bedrock.

Closer spaced piles are recommended and may be required to reduce lateral movements particularly where adjacent infrastructure, such as buildings, pavements, subsurface utilities and road reserves are located near the excavation, and to prevent the collapse of unsupported materials into the excavation. The project structural engineer should analyse and design the pile spacing and should consider horizontal pressures due to surcharge loads from adjacent infrastructure (i.e. buildings, road reserves, etc.), and long-term loadings.

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

As noted, all piles should be sufficiently embedded into Class III Shale (or better) bedrock underlying the site and a suitably qualified geotechnical engineer should inspect and approve the pile installations. It is recommended to refer to the estimated levels of subsurface conditions provided in Section 3 and the detailed engineering borehole logs in **Appendix D**.

Piles should not be founded into any soft or weak bands or layers (i.e. clay seams, extremely weathered zones, fractured zones, etc) underlying the site. Furthermore, the project structural engineer should carefully select the retention system, with all structural elements also inspected and approved by a suitably qualified structural engineer.

GCA should be present to witness the initial pile boring stage, and it is recommended to assess a minimum percentage of piles (20% to 30% depending on pile spacing) distributed evenly across the site.

It should be noted that groundwater seepage may pass through shoring pile gaps during excavation. To control this groundwater seepage, it is recommended to install strip drains behind the retention system, connected to the buildings' stormwater system. Weak areas of the retention system may require using concrete or localised grouting, predominantly where groundwater seepage and loose or soft soils are visible. Shoring design should consider both short term (during construction) and permanent conditions, along with surcharge loading and footing loads from adjacent infrastructure.

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

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

As excavation of up to approximately 12.6m depth is anticipated, anchors for the shoring wall are likely to be needed. In cases where anchoring is impractical, or permission by other land-holders and asset owners is not given, then other temporary support for the adopted shoring system should be considered, which may include staged excavations and installation of struts, temporary berms, or props in front of the retaining walls.

As discussed above, GCA recommends that an experienced geotechnical engineer or engineering geologist undertake geotechnical inspections of exposed materials in any shoring pile gaps at an initial 0.5m depth, followed by1.0m to maximum 1.5m excavation depth intervals, and then at bulk excavation level.

It is recommended that the shoring wall is designed using the design parameters provided in Section 5.8.2 and ground anchors (if adopted) are designed using the parameters provided in Section 5.8.3. An experienced geotechnical engineer should inspect the materials encountered at bulk excavation and the foundations (including pile installations) of the proposed development. A structural engineer should inspect all structural elements of the proposed development. Inspections should be considered as “Hold Points” to the project.

5.8.2 Design Parameters (Earth Pressures)

Forces acting on a bored pile shoring wall or other types of retaining wall will depend on a number of factors including:

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

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

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

- A triangular earth pressure distribution may be adopted for derivation of active pressures where a simple support system (i.e. cantilevered wall or propped/anchored wall with only one row of props/anchors is 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 to model the sequence of anchor installation and excavation, which can ensure deflections in the walls remain within tolerable limits. For braced retaining walls, a uniform lateral earth pressure should be adopted as follows:

Active earth pressure:

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

Where:

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

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

Support systems designed using the earth pressure approach may be based on the parameters given in Table 14 below. Table 14 also provides preliminary coefficients of lateral earth pressure for the soil and rock horizons encountered in the site. These are based on fully drained conditions, natural soils are normally consolidated and that the ground behind the retention walls is horizontal.

Table 14: Preliminary Geotechnical Design Parameters for Retaining Structures

Material	**Fill (Unit 1a/1b)**	**Natural Soils/EWM (Unit 2a/2b)**	**Weathered Shale (Unit 3)**	**Class V Shale (Unit 4)[3, 5]**	**Class IV Shale (Unit 5)[3, 5]**	**Class III Shale (Unit 6)[3, 5]**	**Class III/II Shale (Unit 7)[3, 5]**
Unit Weight (kN/m^3)[4]	16	18	21	21	22	22	24
Effective Cohesion c' (kPa)	0	5	15	20	50	70	100
Angle of Friction ϕ' (°)	24	26	26	26	28	32	32
Modulus of Elasticity E_{sh} (MPa)	3	15 (stiff) 20 (very stiff, or better)	60	70	180	250	400
Earth Pressure Coefficient At Rest Ko[1]	0.59	0.56	0.56	0.56	0.53	0.47	0.5
Earth Pressure Coefficient Active Ka[2]	0.42	0.39	0.39	0.39	0.36	0.31	0.2
Earth Pressure Coefficient Passive Kp[2]	2.37	2.56	2.56	2.56	2.77	3.25	3.0
Poisson Ratio v	0.4	0.35	0.3	0.3	0.3	0.3	0.25

[1] Earth pressure coefficient at rest (Ko) can be calculated using Jaky's equation.
[2] Earth pressure coefficient of active (Ka) and passive (Kp) can be calculated using Rankine's or Coulomb's equation.
[3] The values for rock assume no defects of adverse dipping are present in the bedrock. All excavation rock faces should be inspected on a regular basis by an experienced engineering geologist or geotechnical engineer. The engineering geologist or geotechnical engineer should consider the presence of clay seams, extremely weathered or fractured zones present in the underlying Shale bedrock.
[4] Above groundwater levels.
[5] Subject to confirmation by a geotechnical engineer by additional cored borehole drilling and rock strength testing, and during excavation and construction by inspection. Spoon testing is required during construction to confirm the presence of Class III/II Shale bedrock underlying the site.

Notes:

- EWM = Extremely weathered material. Refer to borehole logs and Section 3.1.

5.8.3 Ground Anchors

The basement floor slabs are considered incorporated into the long-term design of the construction and will provide permanent restraint 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 should have permission from adjacent property owners and relevant authorities (i.e. Sydney Water assets, TfNSW assets and adjoining properties, etc.). The design of excavation support should be carried out by a suitably qualified and experienced structural or civil engineer. Anchors require sufficient embedment into the underlying Class III Shale (or better) bedrock, and a geotechnical engineer should inspect and approve the rock support requirements during excavation. Care must be taken to not damage any inclinometer tube installations or subsurface utilities, road reserves and associated infrastructure.

As noted, a geotechnical engineer should inspect and approve the installation of ground anchors during construction.

Table 15 below provides the preliminary allowable bond stresses for temporary anchors.

Table 15: Preliminary Allowable Bond Stresses for Temporary Anchors

Unit Type/Material	Allowable Bond Stress (kPa)[1]
Class V Shale (Unit 4)	N/A
Class IV Shale (Unit 5)	N/A
Class III Shale (Unit 6)	250
Class III/II Shale (Unit 7)	350

[1] An experienced geotechnical engineer should confirm the composition, class, depth and estimated strength of the underlying bedrock material prior to construction by further cored borehole drilling and rock strength testing, and during excavation and construction by inspection and appropriate testing (i.e. spoon testing, rock strength testing, etc.).

Notes:

- N/A = Not applicable. Not recommended for the proposed development.

It is recommended to embed anchors into Class III Shale (or better) bedrock underlying the site.

The parameters listed above are derived from the UCS values for the rock classes (refer Pells, et. al., 2019). Note also that for bond lengths of greater than about 4.0m in Shale or Sandstone bedrock, the distal part of the bond zone contributes nothing of substance to ultimate or serviceability performance of the anchors (Pells, et. al., 2019).

Further guidance on design of anchors in Shale in the Sydney area can be found in the papers by Pells, Mostyn, Bertuzzi and Wong (2019) and Pells, Mostyn and Walker (1998) (see Section 8 - References).

These parameters also assume that the anchor installation holes are drilled using percussive techniques, which generally provide adequate side-wall roughness (R2 roughness category or better – see **Appendix C**, Explanatory Notes), and that the drilled holes are adequately flushed with water and have clean sidewalls to provide good rock-grout contact. The following recommendations listed below require consideration 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 Earthquake Site Risk Classification

In accordance with AS 1170.4-2007 "Structural Design Actions – Part 4: Earthquake Actions in Australia", and based on assessment of the subsurface materials encountered during this investigation and proposed development, the recommended earthquake design parameters for the proposed development site are as follows for all foundations founded into the underlying Shale bedrock, as outlined in this report:

- Subsoil Class: "Rock Site" (Class B_e).
- Earthquake Hazard Factor (Z): 0.08 (for Sydney).

5.10 Foundations

Based on the boreholes carried out on the site, bulk excavation to an approximate level of RL97.275m AHD is likely to expose Class IV Shale (found in BH1) and Class III/II Shale (found in BH2 and BH3). It is noted in the BH1 borehole log, Class III/II Shale bedrock is encountered at approximately 300mm below bulk excavation level at the location of BH1.

It is also noted that a section of "No Core" and a clay seam were encountered in BH3 between 14.13m to 14.60m bgl (~RL94.97 to RL94.50m AHD). Based on joints either side, this zone is interpreted as a preferentially weathered fracture zone, and piles should not be founded in this zone.

It is also not recommended to place footings on Class IV Shale bedrock or Shale bedrock of low (or lower) estimated strength, as this will likely result in total and differential settlement under working load, and may not adequately support the proposed development within the site. Footings placed on highly weathered Shale (Unit 3), Class V Shale (Unit 4) as well as variable composition and strength fill, natural clayey soil and extremely weathered material are also not recommended.

It is recommended to refer to Section 3.1 (Table 4), which summarises the stratigraphy encountered in each borehole drilled at the site and to Table 16 (below) for rock classes and intervals, and bearing capacity and shaft adhesion values.

As the site is a large site at approximately 6,300m^2, additional cored boreholes are required prior to excavation and construction to confirm the presence and depth of Class III/II Shale bedrock across the site.

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

5.10.1 Geotechnical Assessment

Based on the proposed development and our assessment of the subsurface conditions, a suitable foundation system could comprise a combination of shallow foundations typically containing pad and/or strip footings, and a piled foundation system.

Foundations should be sufficiently embedded into Class III Shale (or better) bedrock of at least medium estimated strength (minimum 1.0m below bulk excavation level and at least 0.5m socket into Class III Shale (or better) bedrock).

Shallow foundations should only be considered in areas where Class III Shale (or better) bedrock is exposed at bulk excavation level and should include local slab thickening to support internal walls and columns. Piled foundations are required if Class III Shale (or better) bedrock is not exposed at bulk excavation level. These piles require a socket (minimum 0.5m) into Class III Shale (or better) bedrock of at least medium estimated strength. Piled foundations are also needed where greater resistance against lateral loading induced by earthquakes or wind is required, to resist the axial and working loads transmitted through the building walls and columns and to achieve higher bearing capacities.

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

It is recommended to refer to the estimated levels of subsurface conditions given in the detailed engineering borehole logs in **Appendix D** and Section 3 of this report. Foundations should not be placed on any soft or weak bands or layers (i.e. clay seams, extremely weathered zones, fractured zones, etc) underlying the site.

An experienced geotechnical engineer should inspect the foundations during excavation and construction, to confirm:

- That the ground conditions are consistent throughout;
- The materials exposed in the foundation excavations;
- The cleanliness of the base of the foundations, and that;
- The preliminary allowable bearing capacities are achieved.

In addition, for piled foundations, the geotechnical engineer should also confirm the design socket materials, socket length, depth to the pile toe and if the required shaft adhesion (if applicable) is achieved.

The actual depth and embedment of the piles should be assessed by the project structural engineer, with all structural elements of the proposed development also inspected and approved by a suitably qualified structural engineer.

Table 16 provides preliminary recommended geotechnical design parameters for foundations.

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 “Piling - Design and Installation”, 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 “Earth Retaining Structures”.

Table 16: Preliminary Geotechnical Design Parameters for Foundations

Unit Type/Material	Maximum Allowable (Serviceability) Values (kPa)		
	End Bearing Pressure[1]	Shaft Adhesion (Compression)[3]	Shaft Adhesion (Tension)[3]
Fill (Unit 1a/1b)	N/A	N/A	N/A
Natural Soils/EWM (Units 2a/2b)	N/A	N/A	N/A
Weathered Shale (Unit 3)	N/A	N/A	N/A
Class V Shale (Unit 4)	N/A	N/A	N/A
Class IV Shale (Unit 5)	N/A	N/A	N/A
Class III Shale (Unit 6)[2]	Up to 3,500[4]	200	100
Class III/II Shale (Unit 7)[2]	3,500[4] (maximum 6,000)	300	150

[1] Minimum embedment of 0.5m into Class III Shale (or better) bedrock for shallow foundations. All piles to be a minimum of 1.0m below bulk excavation level and embedded at least 0.5m into the design class of Shale bedrock (Class III Shale, or better). The allowable end bearing capacities provided above are from Pells et al (1998, 2019) and are given to limit settlement to <1% of the minimum footing dimensions. End bearing capacities for foundations also rely on base cleanliness.

[2] An experienced geotechnical engineer should confirm composition, class, depth and estimated strength of the underlying bedrock material prior to construction by further cored borehole drilling and rock strength testing, and during excavation and construction by inspection and appropriate testing (i.e. spoon testing, rock strength testing, etc.). Spoon testing is required to confirm the presence of Class III/II Shale bedrock.

[3] Shaft adhesion depends entirely on the roughness of the rock socket and needs to adhere to category R2 or better (Pells, 1999 State of Practice for the design of Socketed Piles in Rock). Where the side walls of the pile have not been cleaned or grooved, or have clay smear from the boring process, then shaft adhesion should be taken as zero (0).

[4] Subject to completion of additional rock cored boreholes across the site to confirm Class III/II Shale. Adopting allowable bearing capacities exceeding 3,500kPa (i.e. Class III Shale of >3,500kPa) will require spoon testing by an experienced geotechnical engineer on at least 50% of all footings.

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.
- An experienced geotechnical engineer should inspect the foundations during construction to confirm the materials exposed and the design allowable bearing capacities are achieved.
- Values for serviceability bearing pressure and shaft adhesion values are from Pells, Mostyn and Walker 1998 paper on classification of Sydney rock types (see references).

5.10.2 Geotechnical Comments

Bearing capacity and settlement behaviour varies according to foundation depth, shape and dimensions, including method of installation for piles. Foundations achieving the allowable bearing pressures provided above should expect to experience settlements of less than 1% of the minimum footing dimension (Pells et al, 1998, 2019). It is noted that end bearing capacities for foundations also rely on base cleanliness.

Consultation should be made with specialist subcontractors to discuss the feasibility of excavation of shallow or piled foundations given the existing site conditions. It is noted that an experienced geotechnical engineer may justify higher bearing capacities for the proposed foundations subject to confirmation by additional rock cored boreholes onsite, and by inspection during excavation and construction.

Specific geotechnical advice should be obtained for footing designs and end bearing capacities, and design of the foundation system (shallow and pile foundations) should be carried out in accordance with AS 2159-2009 "Piling - Design and Installation". Foundations for structures built at ground level of the proposed development should also refer to AS 2870-2011 "Residential Slabs and Footings". 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.

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. Furthermore, GCA recommends that any works in the vicinity of Sydney Water assets, including excavation and piling works are undertaken in accordance with the "Sydney Water Guidelines for Building Over and Adjacent to Pipe Assets".

GCA recommends that planting of trees around the development area (i.e. close to the proposed building foundations) be limited as they can also affect moisture content of 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 consider the potential of flooding, groundwater seepage and elevated groundwater levels. All foundation excavations should be free of any loose debris, fill materials and wet soils. For shallow foundations it is recommended to remove any water present in the foundation excavation prior to placing reinforcement and pouring concrete. If piled foundations are adopted for the proposed development, then due to the possibility of groundwater being encountered and possible groundwater seepage flowing into the pile excavation within the site, it is recommended that either all water in the bored pile excavations is removed prior to placing reinforcement and pouring concrete, or concrete is poured using a tremie line to displace the water. It is important to pour concrete without delay once the pile excavation is exposed to prevent ponding of groundwater, and softening and slaking of bedrock, which will reduce the allowable bearing capacity. Other alternatives may include using Continuous Flight Auger (CFA) piles.

Shaft adhesion may apply to socketed piles adopted for foundations provided that the socketed shaft lengths conform to appropriate classes of bedrock (subject to confirmation) in accordance with Pells et. al, (1998, 2019) and shaft sidewall cleanliness and roughness are to acceptable levels (minimum R2 category, refer to Pells (1999) and **Appendix C**). Shaft adhesion should be ignored or reduced within socket lengths that are clay smeared or fail to satisfy cleanliness requirements (i.e. at least 80%). Pile inspections should be complemented by using a downhole CCTV camera for inspection of sidewall cleanliness.

It is also noted that shaft adhesion and allowable end bearing pressures are not additive (Pells, Bertuzzi and Wong, 2019), and that significant pile movement (i.e. full shaft adhesion mobilisation and utilisation is necessary to invoke the end bearing capacity).

An experienced geotechnical engineer should confirm the ground conditions encountered in the foundation excavations, cleanliness of the base of the foundations and if the preliminary allowable bearing capacities are achieved. In addition, for piled foundations, the geotechnical engineer should determine that the depth to the pile toe, shaft adhesion (if applicable) and the design socket length and socket material are achieved. The geotechnical engineer should also determine any variation between the boreholes carried out and assessed 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 of the foundations should constitute as a "Hold Point".

5.11 Subgrade Preparation and Filling

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

5.11.1 Subgrade Preparation

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

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

5.11.2 Filling Specifications

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

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

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

6. ADDITIONAL GEOTECHNICAL RECOMMENDATIONS

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

- Dilapidation survey reports on adjacent properties and infrastructure.
- Monitoring and supervision of excavations within the site. Geotechnical inspections of exposed materials in any shoring pile gaps are also recommended at an initial 0.5m depth, followed by 1.0m to maximum 1.5m excavation depth intervals, and then at bulk excavation level, with inspections undertaken by an experienced geotechnical engineer or engineering geologist.
- As the site is a large site at approximately 6,300m^2, additional rock cored boreholes are required prior to excavation and construction to confirm the presence of Class III/II Shale bedrock at or near bulk excavation level across the site. An experienced geotechnical engineer should confirm the composition, class, depth and estimated strength of the underlying bedrock material prior to excavation and construction by further cored borehole drilling and rock strength testing, and during excavation and construction by inspection and appropriate testing (i.e. spoon testing, rock strength testing, etc.), predominantly in areas and at depths not assessed during the geotechnical investigation. Spoon testing is required if allowable end bearing capacities greater than 3,500kPa are adopted for the design of the proposed foundation system (and to confirm the presence of Class III/II Shale bedrock).
- Development of a vibration monitoring plan, including a provision for an attended vibration monitoring trial.
- Inclinometer tubes are installed within the shoring piles to assess and monitor horizontal deflections into the excavation (during installation of the piles). Inclinometer monitoring at regular intervals will be required.
- Geotechnical inspections of shoring wall pile boring and installation, and ground anchors.
- Geotechnical inspections of exposed materials at bulk excavation level.
- Geotechnical inspections of foundations (shallow and/or pile foundations) to confirm that the preliminary allowable bearing capacities and shaft adhesion (if applicable) are achieved.
- Further assessment of groundwater levels by boring additional holes for groundwater monitoring using an approximately ~1.0m diameter auger from the surface. A piling rig or a powerful excavator will be required to undertake this work.
- Care must be taken to not damage the existing monitoring wells if these are to be used for monitoring during excavation and construction.
- Monitoring of any groundwater inflows or seepage into the excavation areas within the site. In addition, GCA recommends continual monitoring of groundwater levels in GW1, GW2 and GW3 within the site prior to and during construction.
- Waste classification of all excavated material transported from the site.
- If required, a meeting to discuss any geotechnical issues and inspection requirements.
- Any changes to final architectural and structural design drawings are provided to GCA for further assessment.

7. LIMITATIONS

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

It is recommended that if for any reason, the site surface, subsurface and geotechnical conditions (including groundwater conditions) encountered during the site 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 were not observed, and were out of the inspection or test areas, or were in inaccessible areas during the time of the investigation.

This report and any associated information and documentation are prepared solely for **Triple Eight Corporation Pty Ltd**, and any interpretation or misinterpretation of this report or reliance on this report by third parties shall be at their own risk. Any legal or other liabilities resulting from the use of this report by other parties cannot be transferred to GCA.

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

For and on behalf of

Geotechnical Consultants Australia Pty Ltd (GCA)

Angus Nelson
BE (Hons I) & BCom
GradIEAust (8361097)
Member of AGS
Geotechnical Engineer

Jnader

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

R. Furniss

Rafael Furniss
BSc (Applied Geology), Hons, MSc
RPGeo (10338)(Geotechnical and Engineering)
Member of AGS, AIG and ISSMGE
Principal Engineering Geologist

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APPENDIX A

Important Information About Your

Geotechnical Report

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

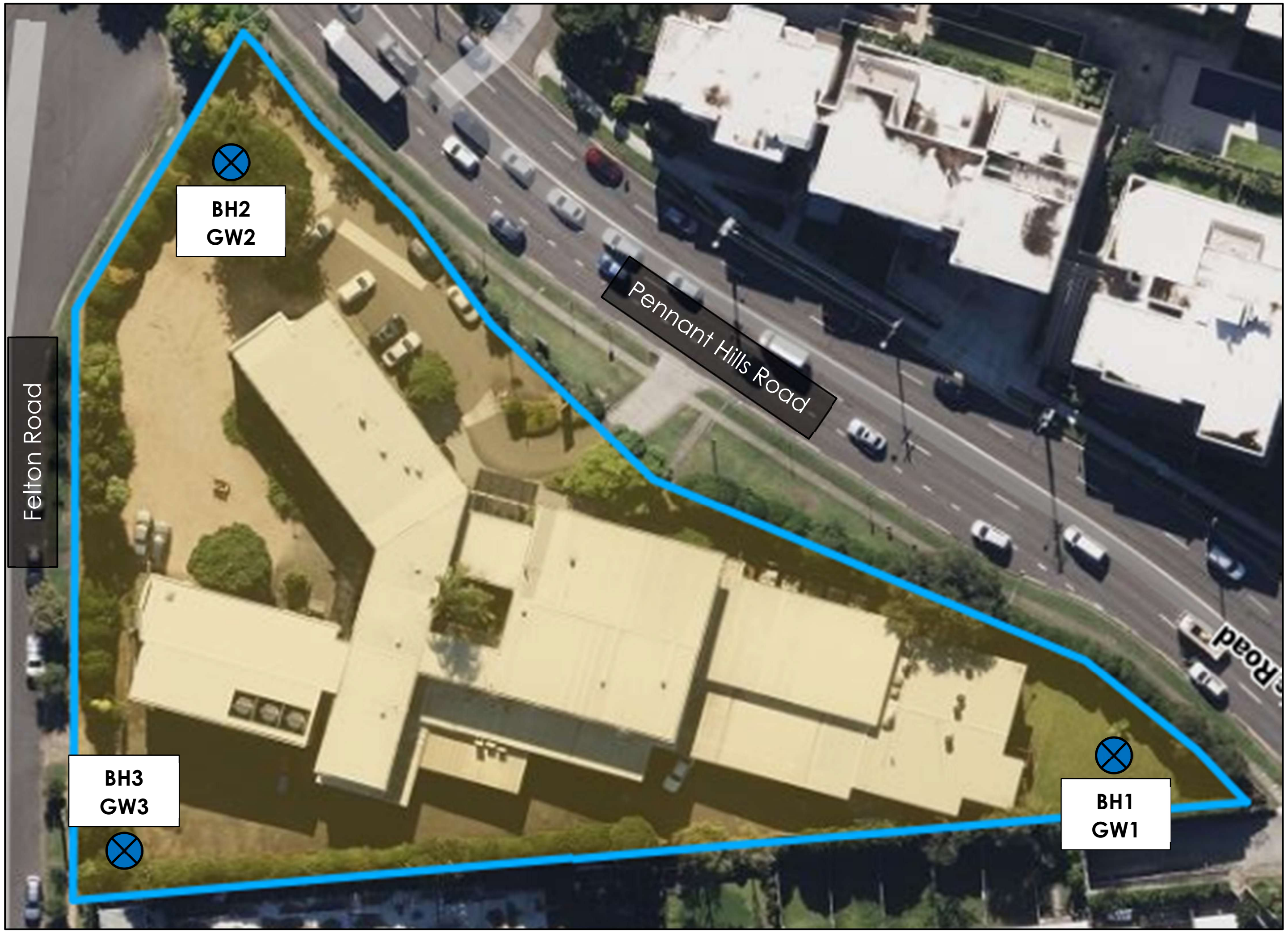
Understanding Limitation Provisions
As some clients, contractors and design professionals do not recognise geotechnical engineering is much broader and less exact than other engineering disciplines, this creates unrealistic expectations that lead to claims, disputs and other disappointments. As part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

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

APPENDIX B

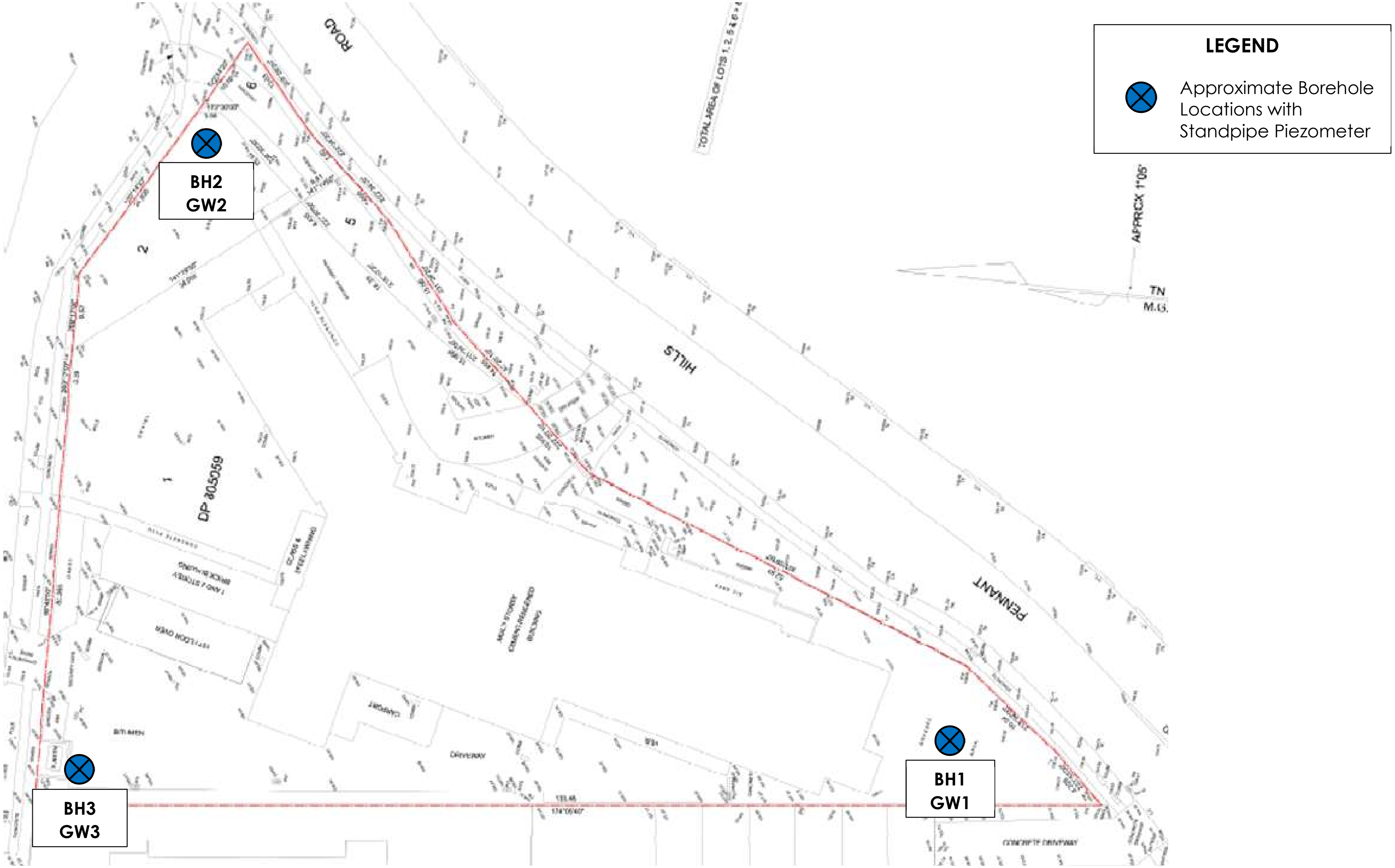
LEGEND

⊗ Approximate Borehole Locations with Standpipe Piezometer



GCA Geotechnical Consultants Australia	Figure 1 Site Plan	**Geotechnical Investigation & Preliminary Groundwater Assessment**	Drawn: OF/AN
		Triple Eight Corporation Pty Ltd	Date: 16/09/2025
	Job No.: G25239-1-Rev A	241-245 Pennant Hills Road Carlingford NSW 2118	Scale: NTS

Image source: Nearmap. Accessed 7th July 2025.



GCA
Geotechnical Consultants Australia

Figure 2
Site Plan

Job No.:
G25239-1-Rev A

Geotechnical Investigation & Preliminary Groundwater Assessment

Triple Eight Corporation Pty Ltd

241-245 Pennant Hills Road
Carlingford NSW 2118

Drawn: OF/AN

Date: 16/09/2025

Scale: NTS

Image source: Site survey plan taken from architectural drawings prepared by Kennedy Associates Architects, titled “Proposed Development 241-245 Pennant Hills Road Carlingford NSW”, referenced drawing No. 1804-SD0050, revision No. P4 and dated 5th May 2025.

APPENDIX C

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

DRILLING/EXCAVATION METHOD

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

PENETRATIION/EXCAVATION RESISTANCE

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

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

WATER

▽ Water level at date shown

◁ Partial water loss

▷ Water inflow

◀ Complete water loss

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

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

MOISTURE CONDITION (AS 1726-2017)

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

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

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

For cohesive soils the following codes may also be used:

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

SAMPLING AND TESTING

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

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

ROCK QUALITY

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

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

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

ROCK STRENGTH TEST RESULTS

● Diametral Point Load Index test

■ Axial Point Load Index test

SOIL ORIGINS

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

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

Transported soils may be further subdivided into:

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

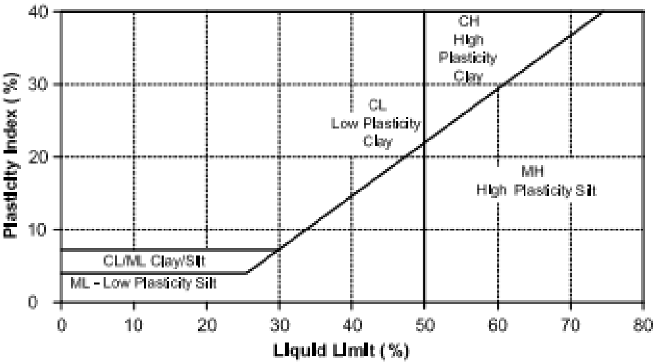
Method and Terms for Soil and Rock Descriptions Used on Borehole and Test Pit Reports

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

COHESIONLESS SOILS PARTICLE SIZE DESCRIPTIVE TERMS

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

PLASTICITY PROPERTIES



COHESIVE SOILS – CONSISTENCY (AS 1726-2017)

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

PLASTICITY

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

COHESIONLESS SOILS - RELATIVE DENSITY

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

UNIFIED SOIL CLASSIFICATION

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

ROCK MATERIAL WEATHERING

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

ROCK STRENGTH (AS 1726-2017 and ISRM)

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

ABREVIATIONS FOR DEFECT TYPES AND DECRIPTIONS

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

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

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

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

ROCK SOCKET ROUGHNESS CLASSIFICATION AND CATEGORIES

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

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

APPENDIX D

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

BOREHOLE NUMBER BH1

PAGE 1 OF 5

CLIENT Triple Eight Corporation Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G25239-1
PROJECT LOCATION 241-245 Pennant Hills Road Carlingford NSW 2118

DATE STARTED 3/7/25 **COMPLETED** 3/7/25
R.L. SURFACE 106.2 **DATUM** m AHD
DRILLING CONTRACTOR BG Drilling
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 AN/SS **CHECKED BY** JN/RF
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 Augering	░	106.0		▩		FILL: Silty Clay, low to medium plasticity, brown to dark brown, trace of fine sandstone and igneous gravel, moist (w>PL), rootlets, grass covering.		FILL Well stickup length 0.13m above ground level.
		■	105.5	0.5	▨	CI-CH	Silty CLAY, medium to high plasticity, brown, moist (w>PL), estimated stiff.	SPT 2, 4, 12 N=16 Agg/Sal	NATURAL SOILS
		■ ☰	105.0 104.5	1.0 1.5	× ×		Extremely Weathered Material (Shale): Auger cuttings returned as Silty CLAY, medium to high plasticity, pale grey, dark red and orange, with ironstone bands, moist (w<PL), estimated very stiff.	Agg/Sal SPT 15, 23, 5/30mm Bouncing	EXTREMELY WEATHERED MATERIAL
		☰	104.0 103.5	2.0 2.5	☰		SHALE, pale brown to brown, laminated, highly weathered, very low estimated strength, moist.	Agg/Sal	BEDROCK
			103.0 102.5 102.0 101.5	3.0 3.5 4.0 4.5 5.0			Borehole BH1 continued as cored hole		

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HOLE SIZE 100mm Diameter
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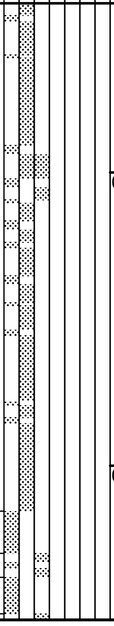
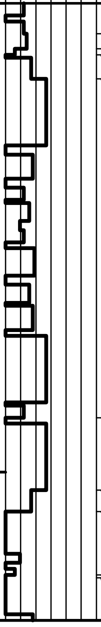
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	Water	Well Details	RL (m)	Depth (m)	Graphic Log	Material Description	Weathering	Is$_{(50)}$ MPa D- diam-etral A- axial	RQD %	Defect Description
			106.0							
				0.5						
			105.5							
				1.0						
			105.0							
				1.5						
			104.5							
				2.0						
			104.0							
				2.5						
			103.5			Continued from non-cored borehole				
NMLC				3.0		SHALE, pale grey, orange, laminated at 0°, with pale grey extremely weathered clay seams, and red-brown ironstone bands, Ashfield Shale.	HW			3.00 - 3.02m: EW Clay Seam, 20mm. 3.06m: B, 0°, S, P, Fe Stain. 3.11m: B, 5°, S, P, Fe Stain. 3.13 - 3.14m: EW Clay Seam, 10mm. 3.21m: B, 0°, S, P, Cl Coating.
			103.0							
				3.5				D A 0.02 0.1		3.43 - 3.46m: EW Clay Seam, 30mm. 3.54 - 3.57m: EW Clay Seam, 30mm. 3.61 - 3.62m: EW Clay Seam, 10mm. 3.68 - 3.71m: EW Clay Seam, 30mm. 3.75 - 3.77m: EW Clay Seam, 20mm.
			102.5						39	3.86 - 3.89m: EW Clay Seam, 30mm. 3.95 - 3.96m: EW Clay Seam, 10mm.
				4.0						4.04 - 4.06m: EW Clay Seam, 20mm. 4.13m: MB.
			102.0							4.28 - 4.29m: EW Clay Seam, 10mm. 4.33 - 4.35m: EW Clay Seam, 20mm.
				4.5				D A 0.02 0.04		4.55m: MB.
						SHALE, dark grey-pale grey and orange, laminated at 0°, with extremely weathered clay seams, Ashfield Shale.				4.57m: B, 0°, S, P, Cl Coating.
			101.5				EW		0	4.64 - 4.78m: EW Clay Seam, 140mm.
							HW			4.81 - 4.83m: EW Clay Seam, 20mm. 4.85m: B, 0°, S, P, Fe Stain.
				5.0			EW			4.86 - 4.98m: EW Clay Seam, 120mm.

Estimated Strength



Defect Spacing mm



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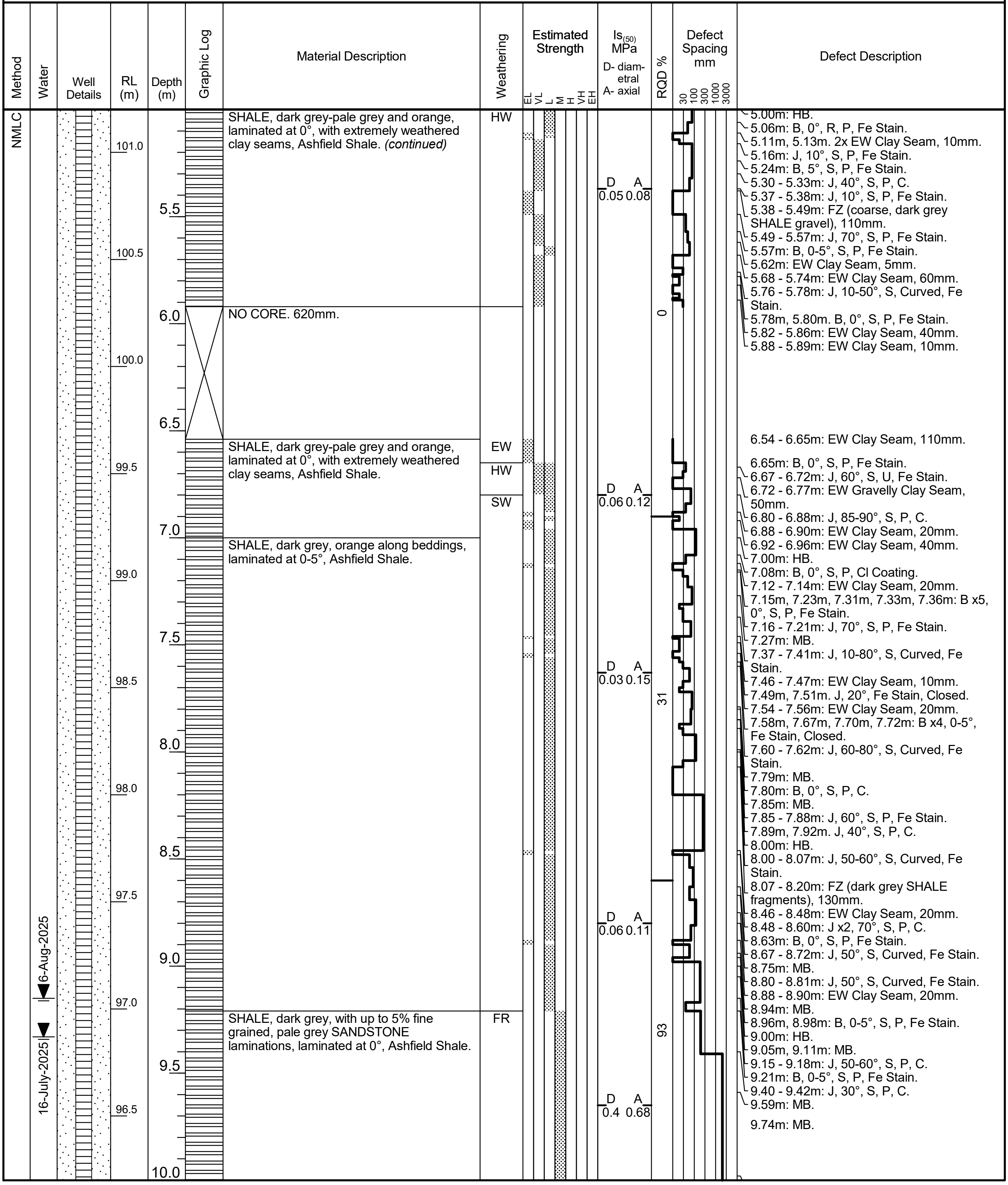

Material Description

SHALE, dark grey-pale grey and orange, laminated at 0°, with extremely weathered clay seams, Ashfield Shale. *(continued)*

NO CORE. 620mm.

SHALE, dark grey-pale grey and orange, laminated at 0°, with extremely weathered clay seams, Ashfield Shale.

SHALE, dark grey, orange along beddings, laminated at 0-5°, Ashfield Shale.

SHALE, dark grey, with up to 5% fine grained, pale grey SANDSTONE laminations, laminated at 0°, Ashfield Shale.

Defect Description

5.00m: HB.
5.06m: B, 0°, R, P, Fe Stain.
5.11m, 5.13m. 2x EW Clay Seam, 10mm.
5.16m: J, 10°, S, P, Fe Stain.
5.24m: B, 5°, S, P, Fe Stain.
5.30 - 5.33m: J, 40°, S, P, C.
5.37 - 5.38m: J, 10°, S, P, Fe Stain.
5.38 - 5.49m: FZ (coarse, dark grey SHALE gravel), 110mm.
5.49 - 5.57m: J, 70°, S, P, Fe Stain.
5.57m: B, 0-5°, S, P, Fe Stain.
5.62m: EW Clay Seam, 5mm.
5.68 - 5.74m: EW Clay Seam, 60mm.
5.76 - 5.78m: J, 10-50°, S, Curved, Fe Stain.
5.78m, 5.80m. B, 0°, S, P, Fe Stain.
5.82 - 5.86m: EW Clay Seam, 40mm.
5.88 - 5.89m: EW Clay Seam, 10mm.
6.54 - 6.65m: EW Clay Seam, 110mm.
6.65m: B, 0°, S, P, Fe Stain.
6.67 - 6.72m: J, 60°, S, U, Fe Stain.
6.72 - 6.77m: EW Gravelly Clay Seam, 50mm.
6.80 - 6.88m: J, 85-90°, S, P, C.
6.88 - 6.90m: EW Clay Seam, 20mm.
6.92 - 6.96m: EW Clay Seam, 40mm.
7.00m: HB.
7.08m: B, 0°, S, P, Cl Coating.
7.12 - 7.14m: EW Clay Seam, 20mm.
7.15m, 7.23m, 7.31m, 7.33m, 7.36m: B x5, 0°, S, P, Fe Stain.
7.16 - 7.21m: J, 70°, S, P, Fe Stain.
7.27m: MB.
7.37 - 7.41m: J, 10-80°, S, Curved, Fe Stain.
7.46 - 7.47m: EW Clay Seam, 10mm.
7.49m, 7.51m. J, 20°, Fe Stain, Closed.
7.54 - 7.56m: EW Clay Seam, 20mm.
7.58m, 7.67m, 7.70m, 7.72m: B x4, 0-5°, Fe Stain, Closed.
7.60 - 7.62m: J, 60-80°, S, Curved, Fe Stain.
7.79m: MB.
7.80m: B, 0°, S, P, C.
7.85m: MB.
7.85 - 7.88m: J, 60°, S, P, Fe Stain.
7.89m, 7.92m. J, 40°, S, P, C.
8.00m: HB.
8.00 - 8.07m: J, 50-60°, S, Curved, Fe Stain.
8.07 - 8.20m: FZ (dark grey SHALE fragments), 130mm.
8.46 - 8.48m: EW Clay Seam, 20mm.
8.48 - 8.60m: J x2, 70°, S, P, C.
8.63m: B, 0°, S, P, Fe Stain.
8.67 - 8.72m: J, 50°, S, Curved, Fe Stain.
8.75m: MB.
8.80 - 8.81m: J, 50°, S, Curved, Fe Stain.
8.88 - 8.90m: EW Clay Seam, 20mm.
8.94m: MB.
8.96m, 8.98m: B, 0-5°, S, P, Fe Stain.
9.00m: HB.
9.05m, 9.11m: MB.
9.15 - 9.18m: J, 50-60°, S, P, C.
9.21m: B, 0-5°, S, P, Fe Stain.
9.40 - 9.42m: J, 30°, S, P, C.
9.59m: MB.
9.74m: MB.

CLIENT Triple Eight Corporation Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G25239-1
PROJECT LOCATION 241-245 Pennant Hills Road Carlingford NSW 2118

DATE STARTED 3/7/25 **COMPLETED** 3/7/25
R.L. SURFACE 106.2 **DATUM** m AHD
DRILLING CONTRACTOR BG Drilling
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 AN/SS **CHECKED BY** JN/RF
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
NMLC						SHALE, dark grey, with up to 5% fine grained, pale grey SANDSTONE laminations, laminated at 0°, Ashfield Shale. *(continued)*	FR	M				9.98m: MB.
												10.00m: HB.
												10.02m: MB.
			96.0									
												10.24m: MB.
									D 0.56 A 0.56			
				10.5								
												10.65m: MB.
			95.5									
										93		10.86m: MB.
				11.0								
												11.00m: HB.
												11.03m: MB.
			95.0									
												11.40m: B, 0°, S, P, C.
				11.5								11.42m: B, 0-5°, Ir, P, C.
												11.54m: MB.
			94.5						D 0.33 A 0.46			
												11.84m: MB.
												11.90m: MB.
				12.0								12.00m: HB.
												12.13m: MB.
			94.0									
												12.36m: J, 10-15°, S, P, C.
				12.5								12.48m: MB.
									D 0.37 A 0.89			
			93.5									12.68m: MB.
												12.79m: MB.
												12.84m: MB.
												12.93m: MB.
				13.0								12.96 - 13.00m: J, 40-70°, S, Curved, C.
												13.00m: HB.
												13.03 - 13.04m: J, 35-40°, S, P, C.
			93.0									
									D 0.34 A 0.67	84		13.34m: MB.
				13.5		SHALE, dark grey, with up to 10-20% fine grained, pale grey SANDSTONE laminations, laminated at 5-10°, Ashfield Shale.	FR	H				13.55 - 13.77m: FZ (13.55m, 13.61m, 13.67m: J x3, 30°, S, P, C, 13.74m: J, 20-25°, S, P, C, 13.77m: B, 10°, S, P, C), 220mm.
			92.5									
												13.83 - 13.86m: J, 40°, S, P, C.
												13.91m: B, 0-5°, S, P, C.
				14.0								14.00m: HB.
												14.03m: MB.
			92.0									14.19m: B, 10°, S, P, C.
									D 0.95 A 1.69			
				14.5								14.47m: B, 5-10°, S, P, C.
												14.54 - 14.57m: J, 50°, S, P, C.
			91.5									
												14.76m: B, 5-10°, S, P, C.
										100		
				15.0								14.97 - 15.00m: J, 40°, S, P, C.

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BOREHOLE NUMBER BH1

CLIENT Triple Eight Corporation Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G25239-1
PROJECT LOCATION 241-245 Pennant Hills Road Carlingford NSW 2118

DATE STARTED 3/7/25 **COMPLETED** 3/7/25
R.L. SURFACE 106.2 **DATUM** m AHD
DRILLING CONTRACTOR BG Drilling
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 AN/SS **CHECKED BY** JN/RF
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
							FR	▨	D 0.37 A 2.11			
						BH1 terminated at 15.08m						
			91.0									
				15.5								
			90.5									
				16.0								
			90.0									
				16.5								
			89.5									
				17.0								
			89.0									
				17.5								
			88.5									
				18.0								
			88.0									
				18.5								
			87.5									
				19.0								
			87.0									
				19.5								
			86.5									
				20.0								

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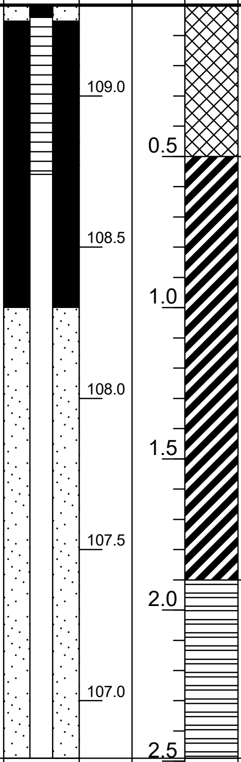
BOREHOLE NUMBER BH2

CLIENT Triple Eight Corporation Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G25239-1
PROJECT LOCATION 241-245 Pennant Hills Road Carlingford NSW 2118

DATE STARTED 3/7/25
COMPLETED 4/7/25
R.L. SURFACE 109.3
DATUM m AHD
DRILLING CONTRACTOR BG Drilling
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 AN/SS
CHECKED BY JN/RF
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 Augering		109.0				FILL: Silty Sand, fine to medium grained, dark grey, with medium to coarse, subrounded to rounded sandstone gravel, moist, brick fragments.		FILL Well stickup length 0.04m below ground level.
			108.5	0.5		CI-CH	Silty CLAY, medium to high plasticity, red-brown, trace of yellow, moist (w<PL), estimated stiff.	SPT 1, 4, 6 N=10 Agg/Sal	NATURAL SOILS
			108.0	1.0			Becoming dark grey to pale brown from 1.0m bgl.		
			107.5	1.5				SPT 2, 9, 15/50mm Agg/Sal Bouncing	
			107.0	2.0			SHALE, pale grey to pale brown, laminated, highly weathered, very low to low estimated strength, moist.	Agg/Sal	BEDROCK
			106.5	2.5			Borehole BH2 continued as cored hole		
			106.0	3.0					
			105.5	3.5					
			105.0	4.0					
			104.5	4.5					
				5.0					

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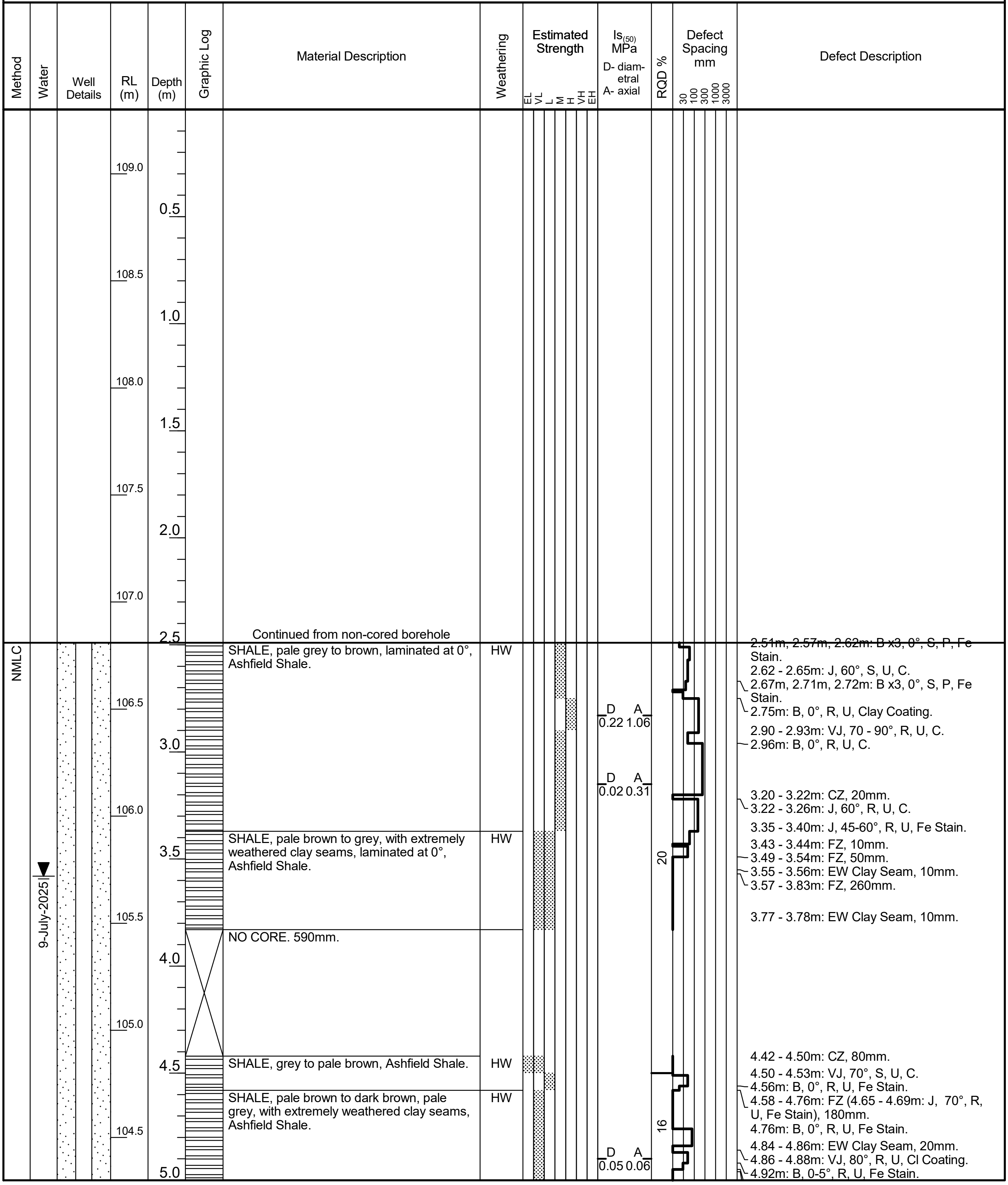
BOREHOLE NUMBER BH2

CLIENT Triple Eight Corporation Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G25239-1
PROJECT LOCATION 241-245 Pennant Hills Road Carlingford NSW 2118

DATE STARTED 3/7/25
COMPLETED 4/7/25
R.L. SURFACE 109.3
DATUM m AHD
DRILLING CONTRACTOR BG Drilling
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 AN/SS
CHECKED BY JN/RF
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
			109.0									
				0.5								
			108.5									
				1.0								
			108.0									
				1.5								
			107.5									
				2.0								
			107.0									
				2.5		Continued from non-cored borehole						
NMLC			106.5 106.0	3.0		SHALE, pale grey to brown, laminated at 0°, Ashfield Shale.	HW		D A 0.22 1.06 D A 0.02 0.31			2.51m, 2.57m, 2.62m: B x3, 0°, S, P, Fe Stain. 2.62 - 2.65m: J, 60°, S, U, C. 2.67m, 2.71m, 2.72m: B x3, 0°, S, P, Fe Stain. 2.75m: B, 0°, R, U, Clay Coating. 2.90 - 2.93m: VJ, 70 - 90°, R, U, C. 2.96m: B, 0°, R, U, C. 3.20 - 3.22m: CZ, 20mm. 3.22 - 3.26m: J, 60°, R, U, C. 3.35 - 3.40m: J, 45-60°, R, U, Fe Stain.
	9-July-2025		105.5	3.5		SHALE, pale brown to grey, with extremely weathered clay seams, laminated at 0°, Ashfield Shale.	HW			20		3.43 - 3.44m: FZ, 10mm. 3.49 - 3.54m: FZ, 50mm. 3.55 - 3.56m: EW Clay Seam, 10mm. 3.57 - 3.83m: FZ, 260mm. 3.77 - 3.78m: EW Clay Seam, 10mm.
			105.0	4.0		NO CORE. 590mm.						
				4.5		SHALE, grey to pale brown, Ashfield Shale.	HW					4.42 - 4.50m: CZ, 80mm. 4.50 - 4.53m: VJ, 70°, S, U, C. 4.56m: B, 0°, R, U, Fe Stain. 4.58 - 4.76m: FZ (4.65 - 4.69m: J, 70°, R, U, Fe Stain), 180mm.
			104.5	5.0		SHALE, pale brown to dark brown, pale grey, with extremely weathered clay seams, Ashfield Shale.	HW		D A 0.05 0.06	16		4.76m: B, 0°, R, U, Fe Stain. 4.84 - 4.86m: EW Clay Seam, 20mm. 4.86 - 4.88m: VJ, 80°, R, U, Cl Coating. 4.92m: B, 0-5°, R, U, Fe Stain.

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info@geoconsultants.com.au
www.geoconsultants.com.au
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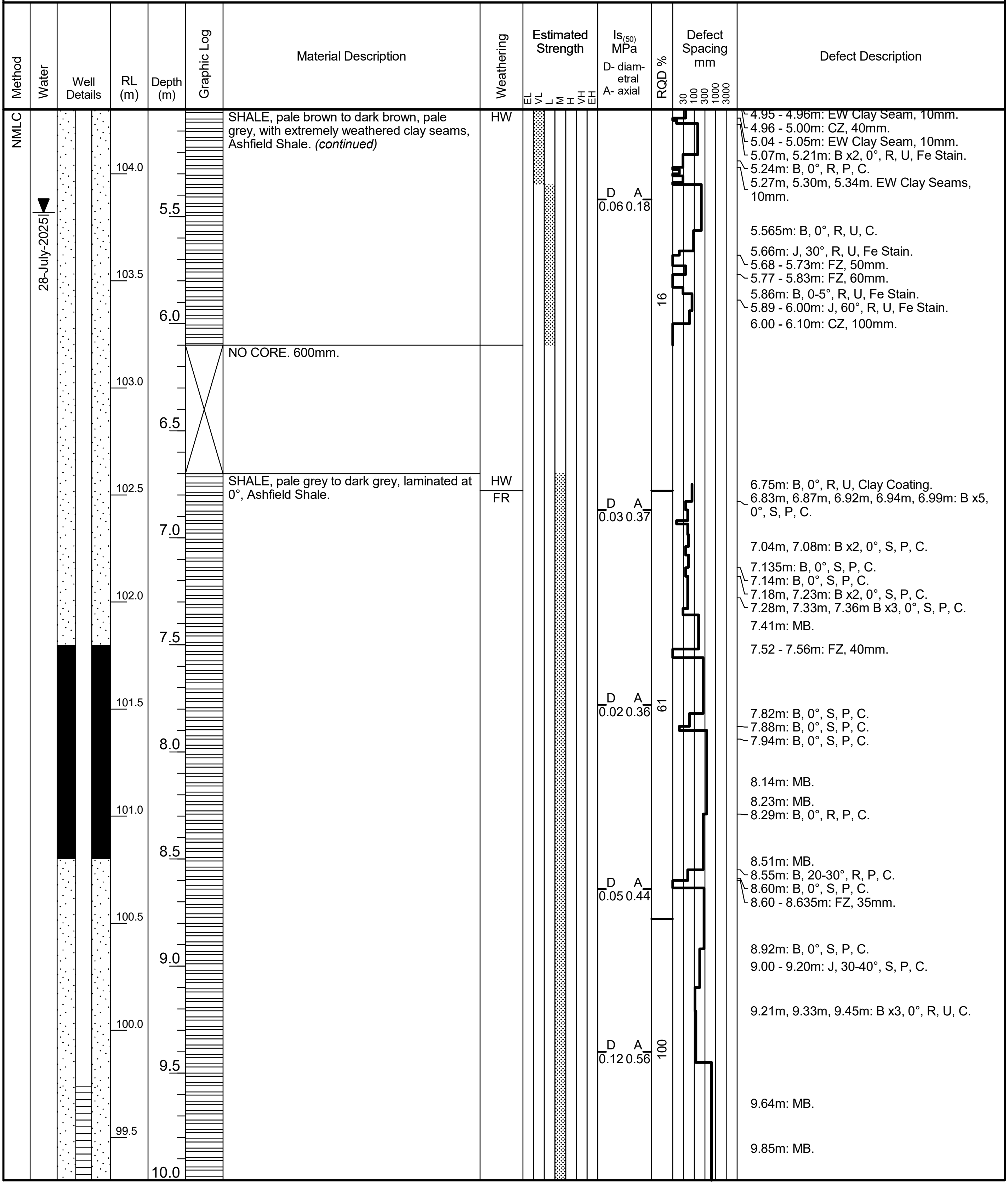
BOREHOLE NUMBER BH2

CLIENT Triple Eight Corporation Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G25239-1
PROJECT LOCATION 241-245 Pennant Hills Road Carlingford NSW 2118

DATE STARTED 3/7/25
COMPLETED 4/7/25
R.L. SURFACE 109.3
DATUM m AHD
DRILLING CONTRACTOR BG Drilling
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 AN/SS
CHECKED BY JN/RF
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate



SHALE, pale brown to dark brown, pale grey, with extremely weathered clay seams, Ashfield Shale. *(continued)*

NO CORE. 600mm.

SHALE, pale grey to dark grey, laminated at 0°, Ashfield Shale.

4.95 - 4.96m: EW Clay Seam, 10mm.
4.96 - 5.00m: CZ, 40mm.
5.04 - 5.05m: EW Clay Seam, 10mm.
5.07m, 5.21m: B x2, 0°, R, U, Fe Stain.
5.24m: B, 0°, R, P, C.
5.27m, 5.30m, 5.34m. EW Clay Seams, 10mm.
5.565m: B, 0°, R, U, C.
5.66m: J, 30°, R, U, Fe Stain.
5.68 - 5.73m: FZ, 50mm.
5.77 - 5.83m: FZ, 60mm.
5.86m: B, 0-5°, R, U, Fe Stain.
5.89 - 6.00m: J, 60°, R, U, Fe Stain.
6.00 - 6.10m: CZ, 100mm.
6.75m: B, 0°, R, U, Clay Coating.
6.83m, 6.87m, 6.92m, 6.94m, 6.99m: B x5, 0°, S, P, C.
7.04m, 7.08m: B x2, 0°, S, P, C.
7.135m: B, 0°, S, P, C.
7.14m: B, 0°, S, P, C.
7.18m, 7.23m: B x2, 0°, S, P, C.
7.28m, 7.33m, 7.36m B x3, 0°, S, P, C.
7.41m: MB.
7.52 - 7.56m: FZ, 40mm.
7.82m: B, 0°, S, P, C.
7.88m: B, 0°, S, P, C.
7.94m: B, 0°, S, P, C.
8.14m: MB.
8.23m: MB.
8.29m: B, 0°, R, P, C.
8.51m: MB.
8.55m: B, 20-30°, R, P, C.
8.60m: B, 0°, S, P, C.
8.60 - 8.635m: FZ, 35mm.
8.92m: B, 0°, S, P, C.
9.00 - 9.20m: J, 30-40°, S, P, C.
9.21m, 9.33m, 9.45m: B x3, 0°, R, U, C.
9.64m: MB.
9.85m: MB.

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BOREHOLE NUMBER BH2

CLIENT Triple Eight Corporation Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G25239-1
PROJECT LOCATION 241-245 Pennant Hills Road Carlingford NSW 2118

DATE STARTED 3/7/25 **COMPLETED** 4/7/25
R.L. SURFACE 109.3 **DATUM** m AHD
DRILLING CONTRACTOR BG Drilling
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 AN/SS **CHECKED BY** JN/RF
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
NMLC						SHALE, pale grey to dark grey, laminated at 0°, Ashfield Shale. *(continued)*	FR			100		10.10 - 10.16m: FZ, 60mm.
									D 0.33 A 0.51			
			99.0									
												10.43m: B, 0°, S, P, C. 10.44m: B, 0°, R, U, C.
				10.5								
												10.67m: B, 0°, S, P, C.
			98.5									
												10.86m, 10.87m, 10.92m: B x3, 0°, S, P, C.
				11.0								
												11.20m: B, 0°, S, P, C.
			98.0									
				11.5					D 0.03 A 1.04	93		11.45m: B, 0°, S, P, C. 11.51m: B, 0°, S, P, C.
			97.5									
												11.90m: MB. 11.97m: MB.
				12.0								
												12.12m: MB.
			97.0									
				12.5					D 0.49 A 0.91			
												12.57m: MB.
			96.5									
						SHALE, dark grey, with up to 20% fine-grained, pale grey SANDSTONE laminations, laminated at 0-5°, Ashfield Shale.						12.89m: MB.
				13.0								13.00m: HB. 13.08m: MB. 13.13m: B, 0-5°, S, P, C. 13.15m: MB. 13.27m: MB.
			96.0									
				13.5								13.54m: MB. 13.62m: MB.
			95.5						D 0.28 A 0.51			
										100		13.84m: MB. 13.87m: MB.
				14.0								14.00m: HB.
												14.13m: MB.
			95.0									14.34m: MB.
				14.5					D 0.06 A 0.67			14.50m: MB. 14.62m: MB.
			94.5									
				15.0								14.88m: MB.

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BOREHOLE NUMBER BH2

CLIENT Triple Eight Corporation Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G25239-1
PROJECT LOCATION 241-245 Pennant Hills Road Carlingford NSW 2118

DATE STARTED 3/7/25 **COMPLETED** 4/7/25
R.L. SURFACE 109.3 **DATUM** m AHD
DRILLING CONTRACTOR BG Drilling
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 AN/SS **CHECKED BY** JN/RF
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
NMLC			94.0	15.5		SHALE, dark grey, with up to 20% fine-grained, pale grey SANDSTONE laminations, laminated at 0-5°, Ashfield Shale. *(continued)*	FR		D 0.13 A 1.22	100		15.00m: HB. 15.04m: MB. 15.09m: MB. 15.16m: MB. 15.32m: MB. 15.36m: MB. 15.53m: MB.
			93.5			BH2 terminated at 15.56m						
			93.0	16.0								
			92.5	16.5								
			92.0	17.0								
			91.5	17.5								
			91.0	18.0								
			90.5	18.5								
			90.0	19.0								
			89.5	19.5								
				20.0								

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BOREHOLE NUMBER BH3

CLIENT Triple Eight Corporation Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G25239-1
PROJECT LOCATION 241-245 Pennant Hills Road Carlingford NSW 2118

DATE STARTED 4/7/25
COMPLETED 5/7/25
R.L. SURFACE 109.1
DATUM m AHD
DRILLING CONTRACTOR BG Drilling
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 AN/SS
CHECKED BY JN/RF
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 Augering						PAVEMENT. 50mm.		PAVEMENT
			109.0 108.5	0.5			FILL: Silty Clay, low to medium plasticity, dark brown and orange-brown, trace of igneous gravel, moist (w>PL).		FILL
				1.0		CI-CH	Silty CLAY, medium to high plasticity, red, moist (w>PL), estimated stiff.	SPT 3, 5, 6 N=11 Agg/Sal	NATURAL SOILS
			108.0 107.5 107.0	1.5 2.0 2.5			SHALE, pale grey, orange and dark grey, laminated, highly weathered, very low to low estimated strength, moist.	SPT 9, 25 Agg/Sal Bouncing	BEDROCK
			106.5 106.0 105.5 105.0	3.0 3.5 4.0 4.5			SHALE, dark grey and orange, laminated, highly weathered, low estimated strength, moist.		
			104.5	5.0			SHALE, dark grey and brown, laminated, highly to moderately weathered, low to medium estimated strength, moist.		

BOREHOLE NUMBER BH3

CLIENT Triple Eight Corporation Pty Ltd
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PROJECT LOCATION 241-245 Pennant Hills Road Carlingford NSW 2118

DATE STARTED 4/7/25
COMPLETED 5/7/25
R.L. SURFACE 109.1
DATUM m AHD
DRILLING CONTRACTOR BG Drilling
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 AN/SS
CHECKED BY JN/RF
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			104.0				SHALE, dark grey and brown, laminated, highly to moderately weathered, low to medium estimated strength, moist. *(continued)*		
				5.5					
			103.5					Agg/Sal	
							Borehole BH3 continued as cored hole		
				6.0					
			103.0						
				6.5					
			102.5						
				7.0					
			102.0						
				7.5					
			101.5						
				8.0					
			101.0						
				8.5					
			100.5						
				9.0					
			100.0						
				9.5					
			99.5						
				10.0					

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www.geoconsultants.com.au
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BOREHOLE NUMBER BH3

CLIENT Triple Eight Corporation Pty Ltd
PROJECT NAME Geotechnical Investigation
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PROJECT LOCATION 241-245 Pennant Hills Road Carlingford NSW 2118

DATE STARTED 4/7/25 **COMPLETED** 5/7/25
R.L. SURFACE 109.1 **DATUM** m AHD
DRILLING CONTRACTOR BG Drilling
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 AN/SS **CHECKED BY** JN/RF
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	Water	RL (m)	Depth (m)	Material Description	Weathering	Is$_{(50)}$ MPa D- diametral A- axial	RQD %	Defect Description
		104.0 103.5	5.5	Continued from non-cored borehole				
NMLC		103.0 102.5	6.0 6.5	SHALE, dark grey and orange-brown, laminated at 0°, Ashfield Shale.	SW	D 0.06 A 0.12 D 0.15 A 0.28	0	5.85 - 5.87m: EW Clay Seam, 20mm. 5.88m: J, 30°, S, Curved, Fe Stain. 5.92m, 5.97m, 6.05m: B x3, 0°, S, P, Fe Stain. 5.93 - 5.95m: EW Clay Seam, 20mm. 6.00m, 6.07m. HB. 6.11m: B, 0°, S, P, Cl Coating. 6.13m: MB. 6.15m: B, 0°, S, P, Fe Stain. 6.17m, 6.23m: MB. 6.24 - 6.30m: B x4, 0°, S, P, Fe Stain. 6.34 - 6.47m: B x4, 0°, S, P, Fe Stain to Cl Veneer. 6.52m, 6.58m, 6.64m. B, 0-5°, R, P, Fe Stain. 6.67 - 7.13m: CZ (J, 80°, R, P, Cl Coating), 460mm.
	9-July-2025 16-July-2025	102.0 101.5	7.0 7.5			D 0.7 A 5.63	55	7.13 - 7.27m: J, 30°, S, Ir, Fe Stain. 7.22m: MB. 7.32m: MB. 7.33m: B, 5°, S, P, C. 7.33 - 7.55m: VJ, 85°, S, Ir, Fe Stain. 7.58m, 7.62m. 2x J, 40°, S, Curved, C. 7.67m: B, 0°, S, P, Fe Stain. 7.75m: MB. 7.80m, 7.84m: B, 0°, S, P, Fe Stain. 7.85 - 7.88m: J, 40°, S, P, C. 7.88 - 8.00m: VJ, S, P, Fe Stain.
		101.0	8.0 8.5	SHALE, dark grey with up to 10% fine grained, pale grey SANDSTONE laminations, thinly laminated at 0°, Ashfield Shale.	FR	D 0.21 A 0.53		8.07m: MB. 8.23m: MB. 8.42m: MB.
		100.5 100.0 99.5	9.0 9.5 10.0			D 0.2 A 0.66	100	8.89m: MB. 9.00m: HB. 9.02m: HB. 9.07m: MB. 9.07 - 9.22m: J, 80°, S, Curved, C. 9.27 - 9.33m: J, 60°, S, P, C. 9.68m: MB. 9.80m: MB. 9.88 - 9.90m: J, 40-50°, S, P, C. 9.89m: MB.

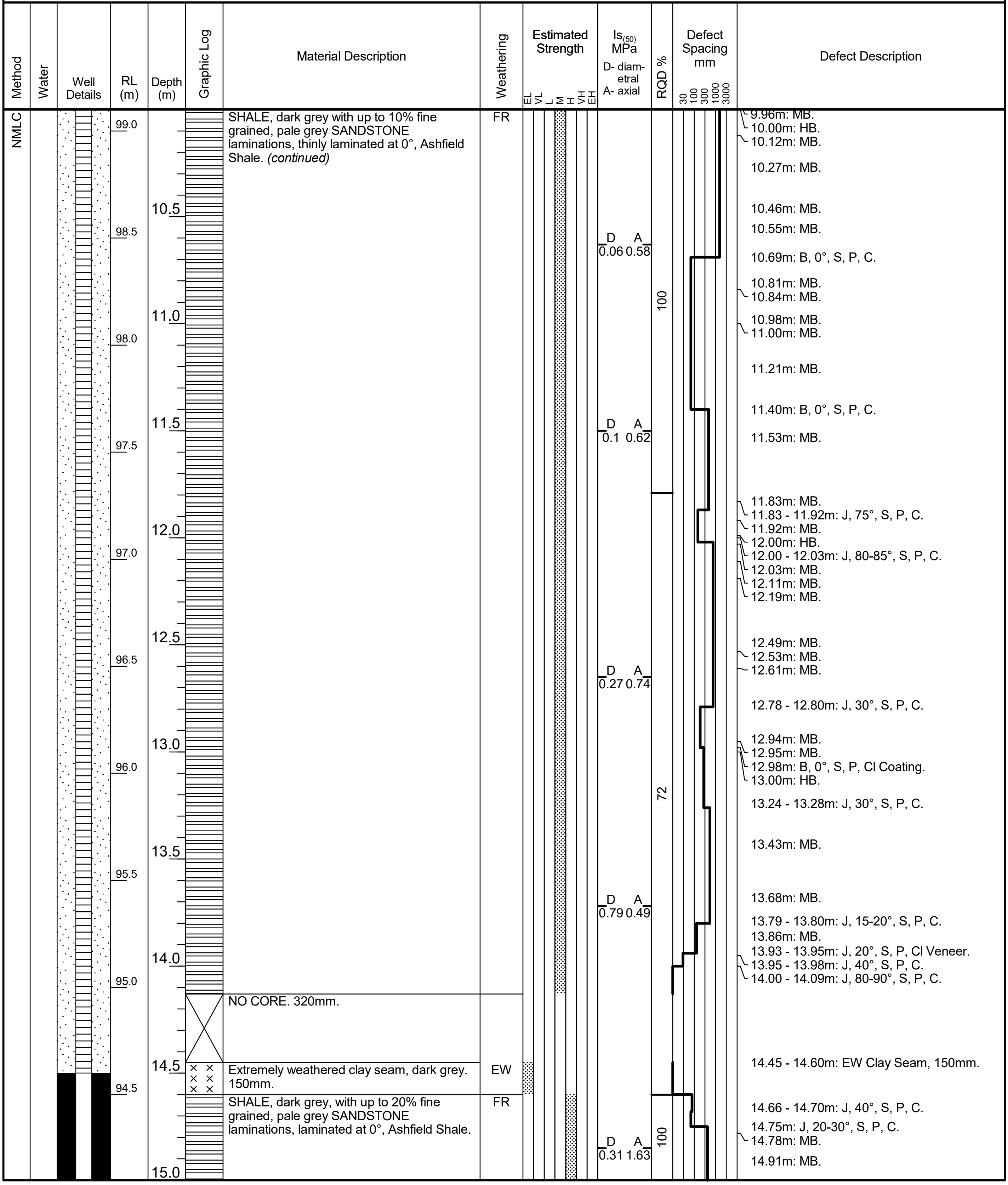
Estimated Strength: EL VL L M H VH EH

Defect Spacing mm: 30 100 300 1000 3000

BOREHOLE NUMBER BH3

CLIENT Triple Eight Corporation Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G25239-1
PROJECT LOCATION 241-245 Pennant Hills Road Carlingford NSW 2118

DATE STARTED 4/7/25 **COMPLETED** 5/7/25
R.L. SURFACE 109.1 **DATUM** m AHD
DRILLING CONTRACTOR BG Drilling
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 AN/SS **CHECKED BY** JN/RF
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate



Depth (m)	Material Description	Weathering
to 14.12	SHALE, dark grey with up to 10% fine grained, pale grey SANDSTONE laminations, thinly laminated at 0°, Ashfield Shale. *(continued)*	FR
14.12 – 14.45	NO CORE. 320mm.	
14.45 – 14.60	Extremely weathered clay seam, dark grey. 150mm.	EW
14.60 – 15.0	SHALE, dark grey, with up to 20% fine grained, pale grey SANDSTONE laminations, laminated at 0°, Ashfield Shale.	FR

Is(50) MPa D- diametral	A- axial
0.06	0.58
0.1	0.62
0.27	0.74
0.79	0.49
0.31	1.63

RQD %: 100; 72; 100

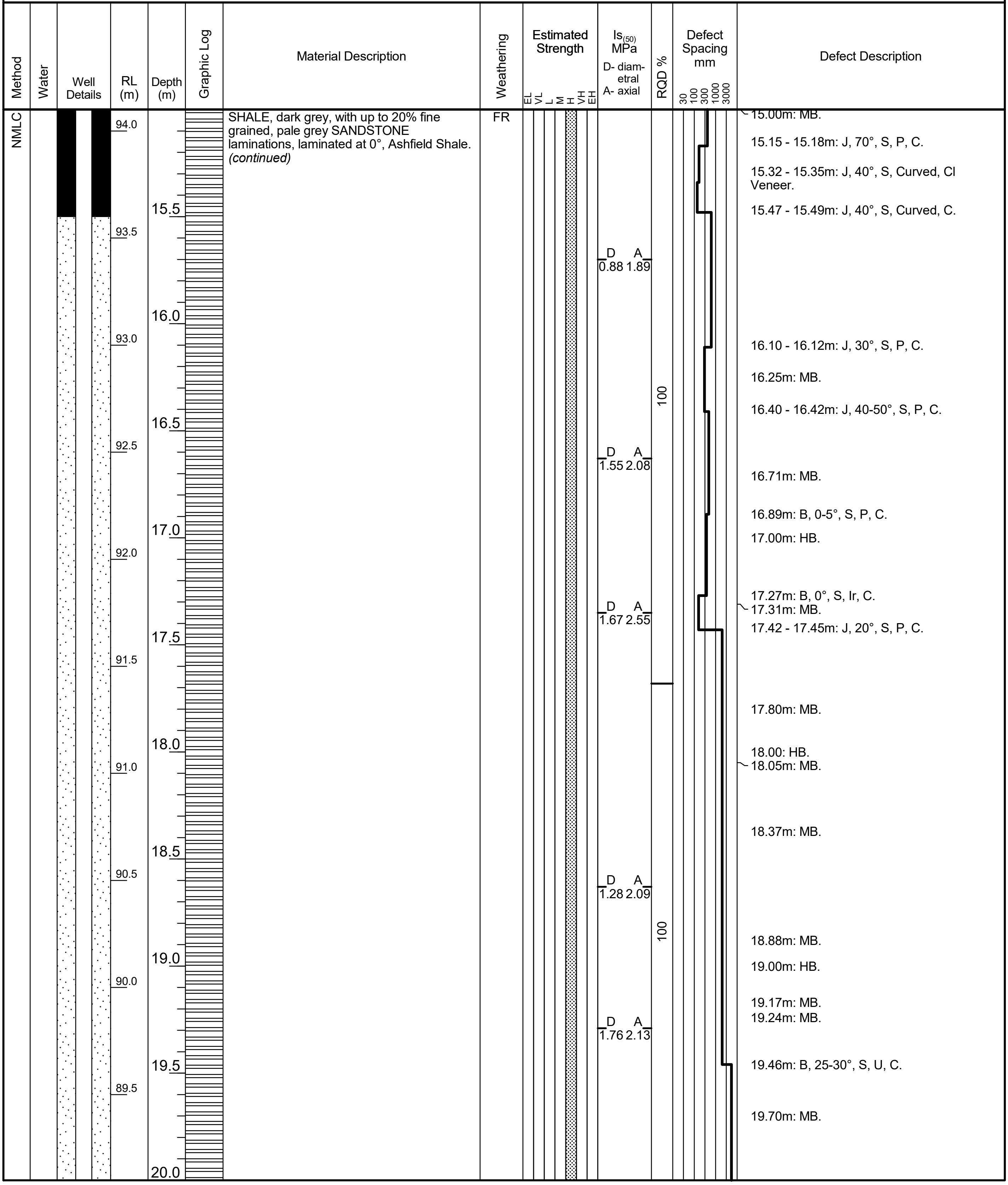
Defect Description

- 9.96m: MB.
- 10.00m: HB.
- 10.12m: MB.
- 10.27m: MB.
- 10.46m: MB.
- 10.55m: MB.
- 10.69m: B, 0°, S, P, C.
- 10.81m: MB.
- 10.84m: MB.
- 10.98m: MB.
- 11.00m: MB.
- 11.21m: MB.
- 11.40m: B, 0°, S, P, C.
- 11.53m: MB.
- 11.83m: MB.
- 11.83 - 11.92m: J, 75°, S, P, C.
- 11.92m: MB.
- 12.00m: HB.
- 12.00 - 12.03m: J, 80-85°, S, P, C.
- 12.03m: MB.
- 12.11m: MB.
- 12.19m: MB.
- 12.49m: MB.
- 12.53m: MB.
- 12.61m: MB.
- 12.78 - 12.80m: J, 30°, S, P, C.
- 12.94m: MB.
- 12.95m: MB.
- 12.98m: B, 0°, S, P, Cl Coating.
- 13.00m: HB.
- 13.24 - 13.28m: J, 30°, S, P, C.
- 13.43m: MB.
- 13.68m: MB.
- 13.79 - 13.80m: J, 15-20°, S, P, C.
- 13.86m: MB.
- 13.93 - 13.95m: J, 20°, S, P, Cl Veneer.
- 13.95 - 13.98m: J, 40°, S, P, C.
- 14.00 - 14.09m: J, 80-90°, S, P, C.
- 14.45 - 14.60m: EW Clay Seam, 150mm.
- 14.66 - 14.70m: J, 40°, S, P, C.
- 14.75m: J, 20-30°, S, P, C.
- 14.78m: MB.
- 14.91m: MB.

BOREHOLE NUMBER BH3

CLIENT Triple Eight Corporation Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G25239-1
PROJECT LOCATION 241-245 Pennant Hills Road Carlingford NSW 2118

DATE STARTED 4/7/25 **COMPLETED** 5/7/25
R.L. SURFACE 109.1 **DATUM** m AHD
DRILLING CONTRACTOR BG Drilling
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 AN/SS **CHECKED BY** JN/RF
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
NMLC			94.0 93.5 93.0 92.5 92.0 91.5 91.0 90.5 90.0 89.5	15.5 16.0 16.5 17.0 17.5 18.0 18.5 19.0 19.5 20.0		SHALE, dark grey, with up to 20% fine grained, pale grey SANDSTONE laminations, laminated at 0°, Ashfield Shale. *(continued)*	FR		D A 0.88 1.89 D A 1.55 2.08 D A 1.67 2.55 D A 1.28 2.09 D A 1.76 2.13	100 100		15.00m: MB. 15.15 - 15.18m: J, 70°, S, P, C. 15.32 - 15.35m: J, 40°, S, Curved, Cl Veneer. 15.47 - 15.49m: J, 40°, S, Curved, C. 16.10 - 16.12m: J, 30°, S, P, C. 16.25m: MB. 16.40 - 16.42m: J, 40-50°, S, P, C. 16.71m: MB. 16.89m: B, 0-5°, S, P, C. 17.00m: HB. 17.27m: B, 0°, S, Ir, C. 17.31m: MB. 17.42 - 17.45m: J, 20°, S, P, C. 17.80m: MB. 18.00: HB. 18.05m: MB. 18.37m: MB. 18.88m: MB. 19.00m: HB. 19.17m: MB. 19.24m: MB. 19.46m: B, 25-30°, S, U, C. 19.70m: MB.

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

BOREHOLE NUMBER BH3

CLIENT Triple Eight Corporation Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G25239-1
PROJECT LOCATION 241-245 Pennant Hills Road Carlingford NSW 2118

DATE STARTED 4/7/25
COMPLETED 5/7/25
R.L. SURFACE 109.1
DATUM m AHD
DRILLING CONTRACTOR BG Drilling
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 AN/SS
CHECKED BY JN/RF
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	89.0	20.0	SHALE, dark grey, with up to 20% fine grained, pale grey SANDSTONE laminations, laminated at 0°, Ashfield Shale. *(continued)*	FR		100	20.00m: HB.
					D 1.25 A 1.88		
	88.5	20.5					
		21.0					21.00m: HB.
	88.0						21.23m: HB. 21.34m: HB.
		21.5					21.50m: MB.
	87.5				D 1.4 A 2.61		21.66m: MB. 21.87m: MB.
		22.0				100	22.00m: HB.
	87.0						22.25m: HB.
					D 0.89 A 2.3		22.42m: HB.
	86.5	22.5					22.87m: MB.
		23.0					23.00m: HB. 23.09m: HB.
	86.0						23.30m: HB.
		23.5			D 0.74 A 1.55		
	85.5						
		24.0					24.00m: HB. 24.07m: MB.
	85.0					100	24.21m: MB.
		24.5					
	84.5				D 1.15 A 1.09		24.72m: MB.
		25.0					

BH3 terminated at 25m

APPENDIX E

Borehole BH1 Rock Core From ~2.96m to 12.00m

GCA

Geotechnical Consultants Australia

Geotechnical Investigation & Preliminary Groundwater Assessment

Core Box Photograph

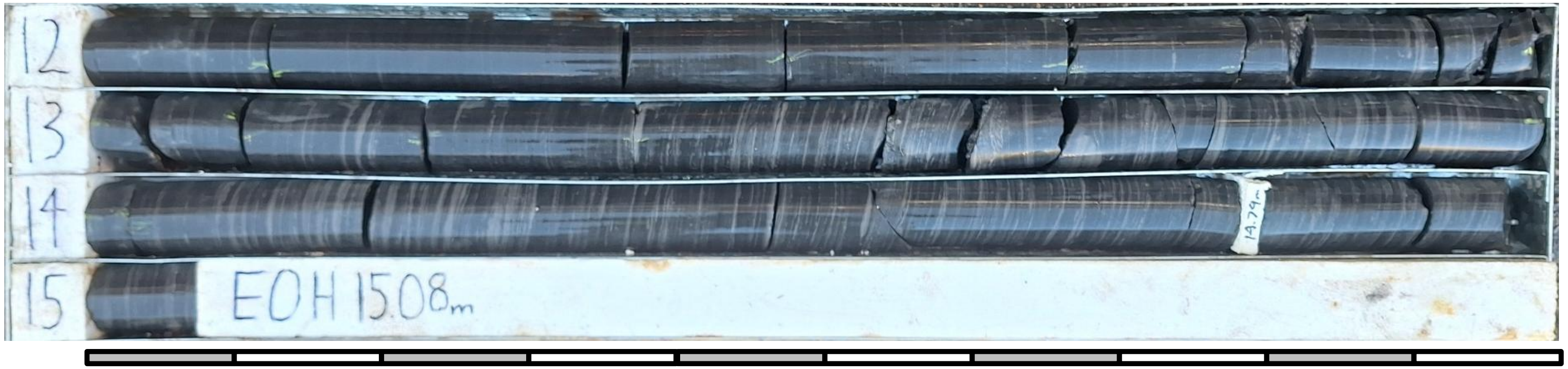
Triple Eight Corporation Pty Ltd

Job No.:
G25239-1-Rev A

241-245 Pennant Hills Road
Carlingford NSW 2118

Date:
16/09/2025

Borehole BH1 Rock Core From ~12.00m to 15.08m



GCA Geotechnical Consultants Australia	**Geotechnical Investigation & Preliminary Groundwater Assessment**	Core Box Photograph
	Triple Eight Corporation Pty Ltd	Job No.: G25239-1-Rev A
	241-245 Pennant Hills Road Carlingford NSW 2118	Date: 16/09/2025

Borehole BH2 Rock Core From ~2.49m to 12.00m

GCA Geotechnical Consultants Australia	**Geotechnical Investigation & Preliminary Groundwater Assessment**	Core Box Photograph
	Triple Eight Corporation Pty Ltd	Job No.: G25239-1-Rev A
	241-245 Pennant Hills Road Carlingford NSW 2118	Date: 16/09/2025

Borehole BH2 Rock Core From ~12.00m to 15.56m

GCA Geotechnical Consultants Australia	**Geotechnical Investigation & Preliminary Groundwater Assessment**	Core Box Photograph
	Triple Eight Corporation Pty Ltd	Job No.: G25239-1-Rev A
	241-245 Pennant Hills Road Carlingford NSW 2118	Date: 16/09/2025

Borehole BH3 Rock Core From ~5.85m to 15.00m

GCA
Geotechnical Consultants Australia

Geotechnical Investigation & Preliminary Groundwater Assessment

Core Box Photograph

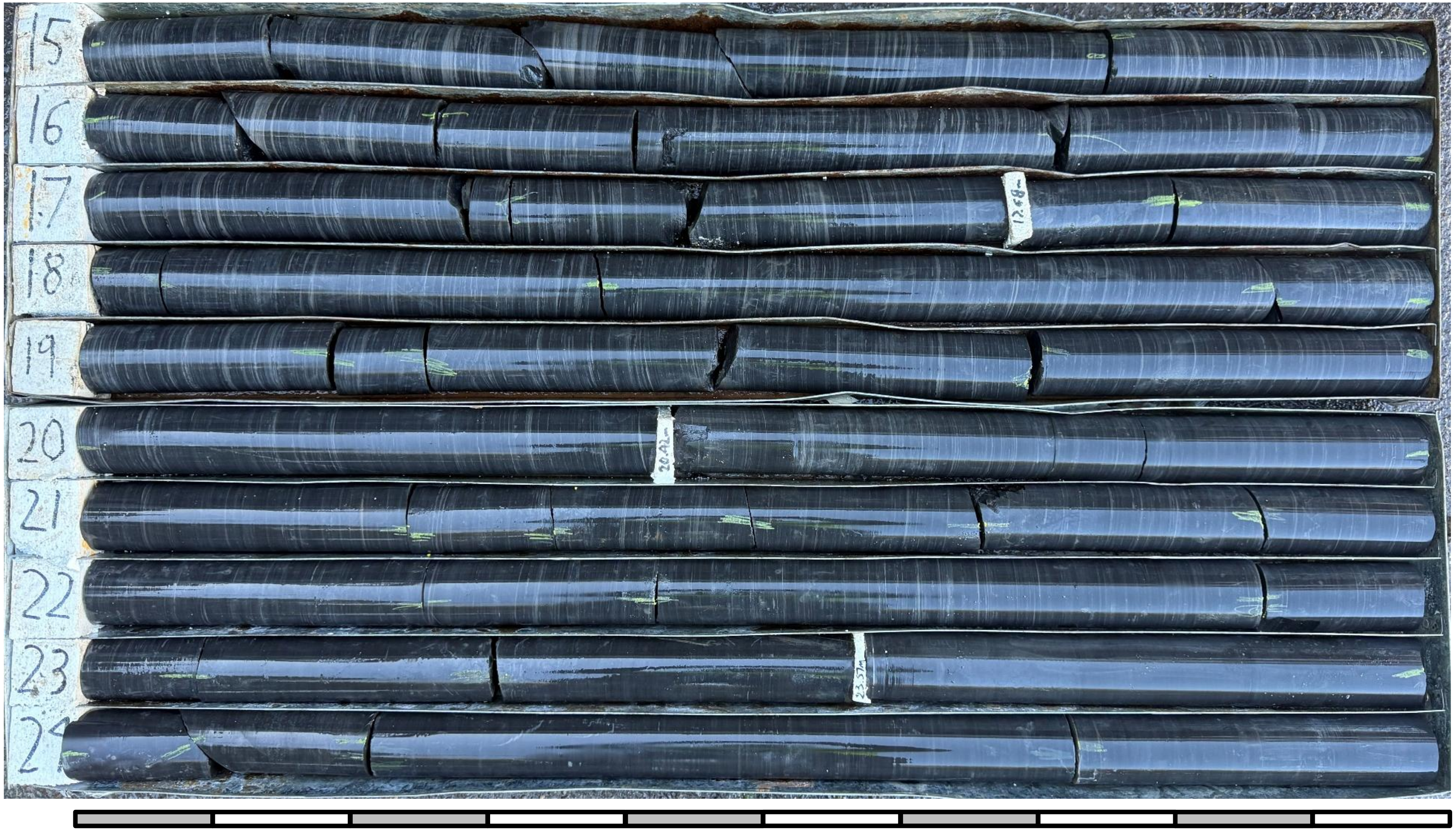
Triple Eight Corporation Pty Ltd

Job No.:
G25239-1-Rev A

241-245 Pennant Hills Road
Carlingford NSW 2118

Date:
16/09/2025

Borehole BH3 Rock Core From ~15.00m to 25.00m



GCA
Geotechnical Consultants Australia

Geotechnical Investigation & Preliminary Groundwater Assessment

Core Box Photograph

Triple Eight Corporation Pty Ltd

Job No.:
G25239-1-Rev A

241-245 Pennant Hills Road
Carlingford NSW 2118

Date:
16/09/2025

APPENDIX F

GEO-TEKNIX SOLUTIONS
Geotechnical & Environmental Testing

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

Point Load Strength Index

TEST METHOD: AS4133.4.1

Client:	**GCA**
Project :	**Material Testing**
Location:	**241-245 Pennant Hills Rd, Carlingford NSW 2118**
Project No.	**L1558 / G25239-1**
Date Reported:	**8/07/2025**
Sample Procedure	***Sampled By Client: Core Samples Supplied**

Sample Number	**BH1**	**BH1**	**BH1**	**BH1**	**BH1**
Sample Depth (m)	**3.52**	**4.49**	**5.37**	**6.80**	**7.63**
Date Sampled	**3/07/2025**	**3/07/2025**	**3/07/2025**	**3/07/2025**	**3/07/2025**
Sample Description/ Rock Type	Shale	Shale	Shale	Shale	Shale
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
P (kN)	0.04	0.06	0.12	0.16	0.08
Is - (Mpa)	0.02	0.02	0.05	0.06	0.03
Is(50) - (Mpa)	0.02	0.02	0.05	0.06	0.03
Moisture Condition	Natural	Natural	Natural	Natural	Natural
Sample Weakness Description	N/A	N/A	N/A	N/A	N/A
Test Type	Axial	Axial	Axial	Axial	Axial
P (kN)	0.30	0.09	0.20	0.31	0.41
Is - (Mpa)	0.10	0.04	0.07	0.12	0.15
Is(50) - (Mpa)	0.10	0.04	0.08	0.12	0.15
Moisture Condition	Natural	Natural	Natural	Natural	Natural
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	7/07/2025	7/07/2025	7/07/2025	7/07/2025	7/07/2025

NOTES:

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

Laboratory Approved Signatory:
Samer Ghanem
Date:
8/07/2025
Sign:

GEO-TEKNIX SOLUTIONS
Geotechnical & Environmental Testing

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

Point Load Strength Index

TEST METHOD: AS4133.4.1

Client:	**GCA**
Project :	**Material Testing**
Location:	**241-245 Pennant Hills Rd, Carlingford NSW 2118**
Project No.	**L1558 / G25239-1**
Date Reported:	**8/07/2025**
Sample Procedure	***Sampled By Client: Core Samples Supplied**

Sample Number	**BH1**	**BH1**	**BH1**	**BH1**	**BH1**
Sample Depth (m)	**8.80**	**9.65**	**10.40**	**11.63**	**12.52**
Date Sampled	**3/07/2025**	**3/07/2025**	**3/07/2025**	**3/07/2025**	**3/07/2025**
Sample Description/ Rock Type	Shale	Shale	Shale	Shale	Shale
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
P (kN)	0.16	1.04	1.48	0.86	0.97
Is - (Mpa)	0.06	0.39	0.56	0.32	0.36
Is(50) - (Mpa)	0.06	0.40	0.56	0.33	0.37
Moisture Condition	Natural	Natural	Natural	Natural	Natural
Sample Weakness Description	N/A	N/A	N/A	N/A	N/A
Test Type	Axial	Axial	Axial	Axial	Axial
P (kN)	0.31	1.53	1.50	1.20	2.45
Is - (Mpa)	0.10	0.71	0.55	0.45	0.87
Is(50) - (Mpa)	0.11	0.68	0.56	0.46	0.89
Moisture Condition	Natural	Natural	Natural	Natural	Natural
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	7/07/2025	7/07/2025	7/07/2025	7/07/2025	7/07/2025

NOTES:

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

Laboratory Approved Signatory:
Samer Ghanem
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8/07/2025
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Point Load Strength Index

TEST METHOD: AS4133.4.1

Client:	**GCA**
Project :	**Material Testing**
Location:	**241-245 Pennant Hills Rd, Carlingford NSW 2118**
Project No.	**L1558 / G25239-1**
Date Reported:	**8/07/2025**
Sample Procedure	***Sampled By Client: Core Samples Supplied**

Sample Number	**BH1**	**BH1**	**BH1**		
Sample Depth (m)	**13.30**	**14.40**	**15.04**		
Date Sampled	**3/07/2025**	**3/07/2025**	**3/07/2025**		
Sample Description/ Rock Type	Shale	Shale	Shale		
Sample size when received	Approx 50mm X 100mm Cylinder	Approx 50mm X 100mm Cylinder	Approx 50mm X 100mm Cylinder		
Test Type	Diametral	Diametral	Diametral		
P (kN)	0.89	2.50	0.99		
Is - (Mpa)	0.33	0.94	0.37		
Is(50) - (Mpa)	0.34	0.95	0.37		
Moisture Condition	Natural	Natural	Natural		
Sample Weakness Description	N/A	N/A	N/A		
Test Type	Axial	Axial	Axial		
P (kN)	1.67	4.23	5.08		
Is - (Mpa)	0.67	1.69	2.14		
Is(50) - (Mpa)	0.67	1.69	2.11		
Moisture Condition	Natural	Natural	Natural		
Sample Weakness Description	N/A	N/A	N/A		
Sample Storage History	Sealed Bag	Sealed Bag	Sealed Bag		
Date Tested	7/07/2025	7/07/2025	7/07/2025		

NOTES:

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

Laboratory Approved Signatory:
Samer Ghanem
Date:
8/07/2025

Sign:

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Point Load Strength Index

TEST METHOD: AS4133.4.1

Client:	**GCA**
Project :	**Material Testing**
Location:	**241-245 Pennant Hills Rd, Carlingford NSW 2118**
Project No.	**L1558 / G25239-1**
Date Reported:	**8/07/2025**
Sample Procedure	***Sampled By Client: Core Samples Supplied**

Sample Number	**BH2**	**BH2**	**BH2**	**BH2**	**BH2**
Sample Depth (m)	**2.83**	**3.15**	**4.90**	**5.42**	**6.87**
Date Sampled	**3/07/2025**	**3/07/2025**	**3/07/2025**	**3/07/2025**	**3/07/2025**
Sample Description/ Rock Type	Shale	Shale	Shale	Shale	Shale
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
P (kN)	0.53	0.06	0.14	0.16	0.06
Is - (Mpa)	0.22	0.02	0.05	0.06	0.03
Is(50) - (Mpa)	0.22	0.02	0.05	0.06	0.03
Moisture Condition	Natural	Natural	Natural	Natural	Natural
Sample Weakness Description	N/A	N/A	N/A	N/A	N/A
Test Type	Axial	Axial	Axial	Axial	Axial
P (kN)	2.90	0.79	0.13	0.45	0.88
Is - (Mpa)	1.03	0.31	0.06	0.18	0.38
Is(50) - (Mpa)	1.06	0.31	0.06	0.18	0.37
Moisture Condition	Natural	Natural	Natural	Natural	Natural
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	7/07/2025	7/07/2025	7/07/2025	7/07/2025	7/07/2025

NOTES:

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

Laboratory Approved Signatory:
Samer Ghanem
Date:
8/07/2025

Sign:

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www.geoteknix.com.au
ABN: 57 621 548 294
PH: 0402 597 452
Email: samer@geoteknix.com.au

Point Load Strength Index

TEST METHOD: AS4133.4.1

Client:	**GCA**
Project :	**Material Testing**
Location:	**241-245 Pennant Hills Rd, Carlingford NSW 2118**
Project No.	**L1558 / G25239-1**
Date Reported:	**8/07/2025**
Sample Procedure	***Sampled By Client: Core Samples Supplied**

Sample Number	**BH2**	**BH2**	**BH2**	**BH2**	**BH2**
Sample Depth (m)	**7.78**	**8.64**	**9.40**	**10.18**	**11.48**
Date Sampled	**3/07/2025**	**3/07/2025**	**3/07/2025**	**3/07/2025**	**3/07/2025**
Sample Description/ Rock Type	Shale	Shale	Shale	Shale	Shale
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
P (kN)	0.05	0.12	0.33	0.87	0.08
Is - (Mpa)	0.02	0.05	0.12	0.33	0.03
Is(50) - (Mpa)	0.02	0.05	0.12	0.33	0.03
Moisture Condition	Natural	Natural	Natural	Natural	Natural
Sample Weakness Description	N/A	N/A	N/A	N/A	N/A
Test Type	Axial	Axial	Axial	Axial	Axial
P (kN)	0.86	1.03	1.52	1.45	2.41
Is - (Mpa)	0.37	0.45	0.54	0.49	1.06
Is(50) - (Mpa)	0.36	0.44	0.56	0.51	1.04
Moisture Condition	Natural	Natural	Natural	Natural	Natural
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	7/07/2025	7/07/2025	7/07/2025	7/07/2025	7/07/2025

NOTES:

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

Laboratory Approved Signatory:
Samer Ghanem
Date:
8/07/2025
Sign:

GEO-TEKNIX SOLUTIONS
Geotechnical & Environmental Testing

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

Point Load Strength Index

TEST METHOD: AS4133.4.1

Client:	**GCA**				
Project :	**Material Testing**				
Location:	**241-245 Pennant Hills Rd, Carlingford NSW 2118**				
Project No.	**L1558 / G25239-1**				
Date Reported:	**8/07/2025**				
Sample Procedure	***Sampled By Client: Core Samples Supplied**				
Sample Number	**BH2**	**BH2**	**BH2**	**BH2**	
Sample Depth (m)	**12.43**	**13.77**	**14.54**	**15.20**	
Date Sampled	**3/07/2025**	**3/07/2025**	**3/07/2025**	**3/07/2025**	
Sample Description/ Rock Type	Shale	Shale	Shale	Shale	
Sample size when received	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	
P (kN)	1.30	0.69	0.15	0.30	
Is - (Mpa)	0.49	0.28	0.06	0.13	
Is(50) - (Mpa)	0.49	0.28	0.06	0.13	
Moisture Condition	Natural	Natural	Natural	Natural	
Sample Weakness Description	N/A	N/A	N/A	N/A	
Test Type	Axial	Axial	Axial	Axial	
P (kN)	2.42	1.43	1.80	2.83	
Is - (Mpa)	0.90	0.49	0.65	1.25	
Is(50) - (Mpa)	0.91	0.51	0.67	1.22	
Moisture Condition	Natural	Natural	Natural	Natural	
Sample Weakness Description	N/A	N/A	N/A	N/A	
Sample Storage History	Sealed Bag	Sealed Bag	Sealed Bag	Sealed Bag	
Date Tested	7/07/2025	7/07/2025	7/07/2025	7/07/2025	

NOTES:

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

Laboratory Approved Signatory:
Samer Ghanem
Date:
8/07/2025
Sign:

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Point Load Strength Index

TEST METHOD: AS4133.4.1

Client:	**GCA**
Project :	**Material Testing**
Location:	**241-245 Pennant Hills Rd, Carlingford NSW 2118**
Project No.	**L1559 / G25239-1**
Date Reported:	**8/07/2025**
Sample Procedure	***Sampled By Client: Core Samples Supplied**

Sample Number	**BH3**	**BH3**	**BH3**	**BH3**	**BH3**
Sample Depth (m)	**5.91**	**6.42**	**7.75**	**8.58**	**9.75**
Date Sampled	**4/07/2025**	**4/07/2025**	**4/07/2025**	**4/07/2025**	**4/07/2025**
Sample Description/ Rock Type	Shale	Shale	Shale	Shale	Shale
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
P (kN)	0.15	0.41	1.84	0.56	0.52
Is - (Mpa)	0.06	0.15	0.69	0.21	0.20
Is(50) - (Mpa)	0.06	0.15	0.70	0.21	0.20
Moisture Condition	Natural	Natural	Natural	Natural	Natural
Sample Weakness Description	N/A	N/A	N/A	N/A	N/A
Test Type	Axial	Axial	Axial	Axial	Axial
P (kN)	0.30	0.81	14.92	1.45	1.86
Is - (Mpa)	0.11	0.27	5.53	0.52	0.63
Is(50) - (Mpa)	0.12	0.28	5.63	0.53	0.66
Moisture Condition	Natural	Natural	Natural	Natural	Natural
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	8/07/2025	8/07/2025	8/07/2025	8/07/2025	8/07/2025

NOTES:

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

Laboratory Approved Signatory:
Samer Ghanem
Date:
8/07/2025
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Point Load Strength Index

TEST METHOD: AS4133.4.1

Client:	**GCA**
Project :	**Material Testing**
Location:	**241-245 Pennant Hills Rd, Carlingford NSW 2118**
Project No.	**L1559 / G25239-1**
Date Reported:	**8/07/2025**
Sample Procedure	***Sampled By Client: Core Samples Supplied**

Sample Number	**BH3**	**BH3**	**BH3**	**BH3**	**BH3**
Sample Depth (m)	**10.63**	**11.50**	**12.65**	**13.72**	**14.85**
Date Sampled	**4/07/2025**	**4/07/2025**	**4/07/2025**	**4/07/2025**	**4/07/2025**
Sample Description/ Rock Type	Shale	Shale	Shale	Shale	Shale
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
P (kN)	0.16	0.27	0.71	2.05	0.82
Is - (Mpa)	0.06	0.10	0.26	0.78	0.31
Is(50) - (Mpa)	0.06	0.10	0.27	0.79	0.31
Moisture Condition	Natural	Natural	Natural	Natural	Natural
Sample Weakness Description	N/A	N/A	N/A	N/A	N/A
Test Type	Axial	Axial	Axial	Axial	Axial
P (kN)	1.36	1.64	1.92	1.46	4.25
Is - (Mpa)	0.59	0.61	0.73	0.47	1.61
Is(50) - (Mpa)	0.58	0.62	0.74	0.49	1.63
Moisture Condition	Natural	Natural	Natural	Natural	Natural
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	8/07/2025	8/07/2025	8/07/2025	8/07/2025	8/07/2025

NOTES:

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Laboratory Approved Signatory:
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Point Load Strength Index

TEST METHOD: AS4133.4.1

Client:	**GCA**
Project :	**Material Testing**
Location:	**241-245 Pennant Hills Rd, Carlingford NSW 2118**
Project No.	**L1559 / G25239-1**
Date Reported:	**8/07/2025**
Sample Procedure	***Sampled By Client: Core Samples Supplied**

Sample Number	**BH3**	**BH3**	**BH3**	**BH3**	**BH3**
Sample Depth (m)	**15.70**	**16.63**	**17.35**	**18.63**	**19.29**
Date Sampled	**4/07/2025**	**4/07/2025**	**4/07/2025**	**4/07/2025**	**4/07/2025**
Sample Description/ Rock Type	Shale	Shale	Shale	Shale	Shale
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
P (kN)	2.33	4.12	4.38	3.36	4.65
Is - (Mpa)	0.87	1.52	1.64	1.26	1.73
Is(50) - (Mpa)	0.88	1.55	1.67	1.28	1.76
Moisture Condition	Natural	Natural	Natural	Natural	Natural
Sample Weakness Description	N/A	N/A	N/A	N/A	N/A
Test Type	Axial	Axial	Axial	Axial	Axial
P (kN)	4.85	5.39	6.17	5.11	5.88
Is - (Mpa)	1.88	2.06	2.57	2.11	2.08
Is(50) - (Mpa)	1.89	2.08	2.55	2.09	2.13
Moisture Condition	Natural	Natural	Natural	Natural	Natural
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	8/07/2025	8/07/2025	8/07/2025	8/07/2025	8/07/2025

NOTES:

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

Laboratory Approved Signatory:
Samer Ghanem
Date:
8/07/2025
Sign:

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Point Load Strength Index

TEST METHOD: AS4133.4.1

Client:	**GCA**
Project :	**Material Testing**
Location:	**241-245 Pennant Hills Rd, Carlingford NSW 2118**
Project No.	**L1559 / G25239-1**
Date Reported:	**8/07/2025**
Sample Procedure	***Sampled By Client: Core Samples Supplied**

Sample Number	**BH3**	**BH3**	**BH3**	**BH3**	**BH3**
Sample Depth (m)	**20.35**	**21.55**	**22.35**	**23.50**	**24.67**
Date Sampled	**4/07/2025**	**4/07/2025**	**4/07/2025**	**4/07/2025**	**0/01/1900**
Sample Description/ Rock Type	Shale	Shale	Shale	Shale	Shale
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
P (kN)	3.29	3.67	2.35	1.95	3.05
Is - (Mpa)	1.23	1.37	0.88	0.73	1.13
Is(50) - (Mpa)	1.25	1.40	0.89	0.74	1.15
Moisture Condition	Natural	Natural	Natural	Natural	Natural
Sample Weakness Description	N/A	N/A	N/A	N/A	N/A
Test Type	Axial	Axial	Axial	Axial	Axial
P (kN)	5.11	7.28	6.13	4.54	2.94
Is - (Mpa)	1.83	2.52	2.25	1.48	1.07
Is(50) - (Mpa)	1.88	2.61	2.30	1.55	1.09
Moisture Condition	Natural	Natural	Natural	Natural	Natural
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	8/07/2025	8/07/2025	8/07/2025	8/07/2025	8/07/2025

NOTES:

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

Laboratory Approved Signatory:
Samer Ghanem
Date:
8/07/2025
Sign:

APPENDIX G

CERTIFICATE OF ANALYSIS

Work Order	: **ES2520520**	Page	: 1 of 4
Client	: **GEOTECHNICAL CONSULTANTS AUSTRALIA PTY LTD**	Laboratory	: Environmental Division Sydney
Contact	: JOE NADER	Contact	: Customer Services ES
Address	: 2 HAROLD STREET PARRAMATTA NSW 2150	Address	: 277-289 Woodpark Road Smithfield NSW Australia 2164
Telephone	: ----	Telephone	: +61-2-8784 8555
Project	: G25239-1 Geotechnical Investigation	Date Samples Received	: 04-Jul-2025 15:00
Order number	: ----	Date Analysis Commenced	: 09-Jul-2025
C-O-C number	: ----	Issue Date	: 14-Jul-2025 12:02
Sampler	: ANGUS N, S Suraj		
Site	: 241-245 Pennant Hills Road Carlingford NSW 2118		
Quote number	: EN/333		
No. of samples received	: 9		
No. of samples analysed	: 9		

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

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

This Certificate of Analysis contains the following information:

- General Comments
- Analytical Results

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

Signatories

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

Signatories	*Position*	*Accreditation Category*
Ankit Joshi	Senior Chemist - Inorganics	Sydney Inorganics, Smithfield, NSW

General Comments

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

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

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

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

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

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

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

- ED045G: LOR raised for Chloride on sample 1 due to sample matrix.
- ED045G: The presence of Thiocyanate, Thiosulfate and Sulfite can positively contribute to the chloride result, thereby may bias results higher than expected. Results should be scrutinised accordingly.

Analytical Results

Sub-Matrix: **SOIL** (Matrix: **SOIL**)			*Sample ID*	**BH1 0.5-0.85m**	**BH1 1.4-1.5m**	**BH1 2.4-2.6m**	**BH2 0.5-0.95m**	**BH2 1.5-1.6m**
			Sampling date / time	03-Jul-2025 00:00	03-Jul-2025 00:00	03-Jul-2025 00:00	03-Jul-2025 00:00	03-Jul-2025 00:00
Compound	*CAS Number*	*LOR*	*Unit*	**ES2520520-001**	**ES2520520-002**	**ES2520520-003**	**ES2520520-004**	**ES2520520-005**
				Result	Result	Result	Result	Result
EA002: pH 1:5 (Soils)								
pH Value	----	0.1	pH Unit	**4.5**	**4.7**	**5.1**	**4.4**	**4.6**
EA010: Conductivity (1:5)								
Electrical Conductivity @ 25°C	----	1	µS/cm	**31**	**26**	**27**	**83**	**83**
EA055: Moisture Content (Dried @ 105-110°C)								
Moisture Content	----	1.0	%	**26.6**	**9.0**	**14.6**	**20.2**	**18.2**
ED040S : Soluble Sulfate by ICPAES								
Sulfate as SO4 2-	14808-79-8	10	mg/kg	**40**	**30**	**30**	**120**	**100**
ED045G: Chloride by Discrete Analyser								
Chloride	16887-00-6	10	mg/kg	<50	<10	<10	**20**	**30**

Analytical Results

Sub-Matrix: **SOIL** (*Matrix*: **SOIL)**			*Sample ID*	**BH2 2.1-2.3m**	**BH3 0.5-0.95m**	**BH3 1.5-1.8m**	**BH3 5.6-5.8m**	**----**
			Sampling date / time	03-Jul-2025 00:00	04-Jul-2025 00:00	04-Jul-2025 00:00	04-Jul-2025 00:00	----
Compound	*CAS Number*	*LOR*	*Unit*	**ES2520520-006**	**ES2520520-007**	**ES2520520-008**	**ES2520520-009**	**--------**
				Result	Result	Result	Result	----
EA002: pH 1:5 (Soils)								
pH Value	----	0.1	pH Unit	**5.0**	**4.9**	**5.4**	**5.8**	**----**
EA010: Conductivity (1:5)								
Electrical Conductivity @ 25°C	----	1	µS/cm	**58**	**56**	**21**	**39**	**----**
EA055: Moisture Content (Dried @ 105-110°C)								
Moisture Content	----	1.0	%	**10.7**	**22.8**	**9.3**	**8.8**	**----**
ED040S : Soluble Sulfate by ICPAES								
Sulfate as SO4 2-	14808-79-8	10	mg/kg	**80**	**80**	**20**	**40**	**----**
ED045G: Chloride by Discrete Analyser								
Chloride	16887-00-6	10	mg/kg	<10	<10	**10**	<10	**----**

QUALITY CONTROL REPORT

Work Order : **ES2520520**

Client : **GEOTECHNICAL CONSULTANTS AUSTRALIA PTY LTD**

Contact : JOE NADER

Address : 2 HAROLD STREET
PARRAMATTA NSW 2150

Telephone : ----

Project : G25239-1 Geotechnical Investigation

Order number : ----

C-O-C number : ----

Sampler : ANGUS N, S Suraj

Site : 241-245 Pennant Hills Road Carlingford NSW 2118

Quote number : EN/333

No. of samples received : 9

No. of samples analysed : 9

Page : 1 of 3

Laboratory : Environmental Division Sydney

Contact : Customer Services ES

Address : 277-289 Woodpark Road Smithfield NSW Australia 2164

Telephone : +61-2-8784 8555

Date Samples Received : 04-Jul-2025

Date Analysis Commenced : 09-Jul-2025

Issue Date : 14-Jul-2025

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

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

This Quality Control Report contains the following information:

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

Signatories

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

Signatories	*Position*	*Accreditation Category*
Ankit Joshi	Senior Chemist - Inorganics	Sydney Inorganics, Smithfield, NSW

General Comments

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

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

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

Key : Anonymous = Refers to samples which are not specifically part of this work order but formed part of the QC process lot
CAS Number = CAS registry number from database maintained by Chemical Abstracts Services. The Chemical Abstracts Service is a division of the American Chemical Society.
LOR = Limit of reporting
RPD = Relative Percentage Difference
# = Indicates failed QC
* = The final LOR has been raised due to dilution or other sample specific cause; adjusted LOR is shown in brackets. The duplicate ranges for Acceptable RPD% are applied to the final LOR where applicable.

Laboratory Duplicate (DUP) Report

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

Sub-Matrix: **SOIL**

				Laboratory Duplicate (DUP) Report					
Laboratory sample ID	*Sample ID*	*Method: Compound*	*CAS Number*	*LOR*	*Unit*	*Original Result*	*Duplicate Result*	*RPD (%)*	*Acceptable RPD (%)*
EA002: pH 1:5 (Soils) (QC Lot: 6706553)									
ES2520527-002	Anonymous	EA002: pH Value	----	0.1	pH Unit	6.4	6.8	6.5	0% - 20%
ES2520520-001	BH1 0.5-0.85m	EA002: pH Value	----	0.1	pH Unit	4.5	4.6	0.0	0% - 20%
EA010: Conductivity (1:5) (QC Lot: 6706552)									
ES2520527-002	Anonymous	EA010: Electrical Conductivity @ 25°C	----	1	µS/cm	292	288	1.5	0% - 20%
ES2520520-001	BH1 0.5-0.85m	EA010: Electrical Conductivity @ 25°C	----	1	µS/cm	31	27	13.6	0% - 20%
EA055: Moisture Content (Dried @ 105-110°C) (QC Lot: 6706556)									
ES2520459-008	Anonymous	EA055: Moisture Content	----	0.1 (1.0)*	%	18.1	17.8	1.8	0% - 50%
ES2520520-001	BH1 0.5-0.85m	EA055: Moisture Content	----	0.1 (1.0)*	%	26.6	24.5	8.3	0% - 20%
ED040S: Soluble Major Anions (QC Lot: 6706551)									
ES2520520-004	BH2 0.5-0.95m	ED040S: Sulfate as SO4 2-	14808-79-8	10	mg/kg	120	100	19.4	0% - 50%
ES2520536-002	Anonymous	ED040S: Sulfate as SO4 2-	14808-79-8	10	mg/kg	90	100	0.0	No Limit
ED045G: Chloride by Discrete Analyser (QC Lot: 6706554)									
ES2520527-001	Anonymous	ED045G: Chloride	16887-00-6	10	mg/kg	50	40	0.0	No Limit
ES2520520-001	BH1 0.5-0.85m	ED045G: Chloride	16887-00-6	10 (50)*	mg/kg	<50	60	24.7	No Limit

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

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

Sub-Matrix: **SOIL**

				Method Blank (MB) Report	Laboratory Control Spike (LCS) Report			
					Spike	Spike Recovery (%)	Acceptable Limits (%)	
Method: Compound	*CAS Number*	*LOR*	*Unit*	*Result*	*Concentration*	*LCS*	*Low*	*High*
EA002: pH 1:5 (Soils) (QCLot: 6706553)								
EA002: pH Value	----	----	pH Unit	----	4 pH Unit	100	98.8	101
				----	7 pH Unit	101	98.8	101
EA010: Conductivity (1:5) (QCLot: 6706552)								
EA010: Electrical Conductivity @ 25°C	----	1	µS/cm	<1	1412 µS/cm	100	92.0	108
ED040S: Soluble Major Anions (QCLot: 6706551)								
ED040S: Sulfate as SO4 2-	14808-79-8	10	mg/kg	<10	750 mg/kg	97.5	80.0	120
ED045G: Chloride by Discrete Analyser (QCLot: 6706554)								
ED045G: Chloride	16887-00-6	10	mg/kg	<10	250 mg/kg	104	75.0	125
				<10	5000 mg/kg	102	79.0	117

Matrix Spike (MS) Report

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

Sub-Matrix: **SOIL**

				Matrix Spike (MS) Report			
				Spike	SpikeRecovery(%)	Acceptable Limits (%)	
Laboratory sample ID	*Sample ID*	*Method: Compound*	*CAS Number*	*Concentration*	*MS*	*Low*	*High*
ED045G: Chloride by Discrete Analyser (QCLot: 6706554)							
ES2520520-001	BH1 0.5-0.85m	ED045G: Chloride	16887-00-6	12500 mg/kg	128	70.0	130

APPENDIX H

FOUNDATION MAINTENANCE AND FOOTING PERFORMANCE

Understanding and preventing soil-related building movement

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

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 home owner 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.

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. Table 1 below is a reproduction of Table 2.1 from Australian Standard AS 2870-2011, Residential slabs and footings.

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 may be the province of the builder and should be taken into consideration as part of the preparation of the site for construction.

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, from AS 2870). 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.

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.

TABLE 1. 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, which may experience only slight ground movement from moisture changes
M	Moderately reactive clay or silt sites, which may experience moderate ground movement from moisture changes
H1	Highly reactive clay sites, which may experience high ground movement from moisture changes
H2	Highly reactive clay sites, which may experience very high ground movement from moisture changes
E	Extremely reactive sites, which may experience extreme ground movement from moisture changes

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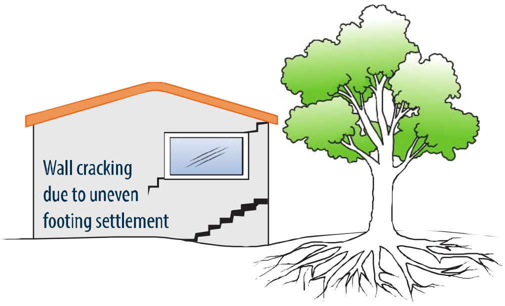

FIGURE 1 Trees can cause shrinkage and damage.

- 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 through absorption. The swelling process will usually begin at the uphill extreme of the building, or on the weather side where the land is flat. Shrinkage usually begins on the side of the building 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.

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 causes 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. Table 2 below is a reproduction of Table C1 of AS 2870-2011.

AS 2870-2011 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 AND 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 may 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 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.

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.

TABLE 2. CLASSIFICATION OF DAMAGE WITH REFERENCE TO WALLS.

Description of typical damage and required repair	Approximate crack width limit	Damage category
Hairline cracks	<0.1 mm	0 – Negligible
Fine cracks which do not need repair	<1 mm	1 – Very Slight
Cracks noticeable but easily filled. Doors and windows stick slightly.	<5 mm	2 – Slight
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 – Moderate
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 depends on number of cracks	4 – Severe

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Warning: Although this Building Technology Resource 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, and mould.
- 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.

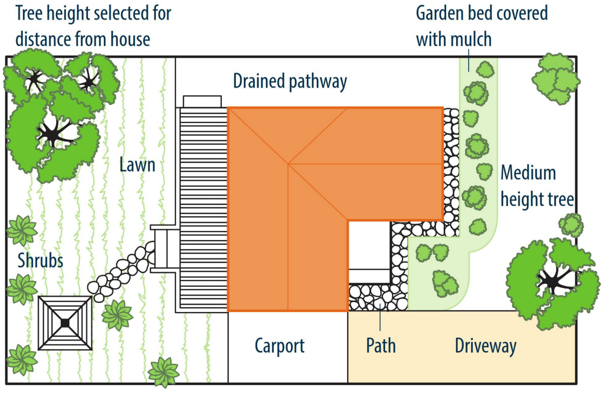

FIGURE 2 Gardens for a reactive site.

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 home owner 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

gn GLENORIE Erosional

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

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

Soils—shallow to moderately deep (<100 cm) Red Podzolic Soils (Dr2.11) on crests; moderately deep (70–150 cm) Red and Brown Podzolic Soils (Dr2.11, Dr2.21, Db1.11, Db1.21) on upper slopes; deep (>200 cm) Yellow Podzolic Soils (Dy5.11) and Gleyed Podzolic Soils (Dg4.11) along drainage lines.

Limitations—high soil erosion hazard, localised impermeable highly plastic soil, moderately reactive.

LOCATION

Glenorie soil landscape occurs north of the Parramatta River on the Hornsby Plateau in Baulkham Hills, Hornsby, Ku-ring-gai, and Ryde local government areas. Smaller isolated areas are at Condell Park, Hurstville, and on the Cumberland Lowlands at Rosehill.

LANDSCAPE

Geology

This soil landscape is underlain by Wianamatta Group Ashfield Shale and Bringelly Shale formations.

The Ashfield Shale is comprised of laminite and dark grey shale. Bringelly Shale consists of shale, calcareous claystone, laminite, fine to medium grained lithic-quartz sandstone (Herbert, 1983).

Topography

Low rolling and steep 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 tall open-forest (wet sclerophyll forest). Dominant tree species include Sydney blue gum *Eucalyptus saligna* and blackbutt *E. pilularis*. Other species include turpentine *Syncarpia glomulifera*, grey ironbark *E. paniculata*, white stringybark *E. globoidea* and rough-barked apple *Angophora floribunda*. Pittosporum *Pittosporum undulatum* and coffee bush *Breynia oblongifolia* are common understorey species (Benson, 1980). Most original vegetation has been extensively cleared, except for larger trees in many residential areas. Examples of original vegetation can be seen in parts of Dalrymple Hay Reserve at St Ives and in Blackwood Memorial Sanctuary at Beecroft.

Land use

Land is used for urban residential sites over much of this soil landscape (Turramurra, Carlingford). Rural land uses, mostly hobby farms and small rural subdivisions, occur at Glenorie, Dural and Castle Hill. These include equestrian activities, orchards, cut flower production and market gardens (Galston, Arcadia) and timber production (Cumberland State Forest).

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.

SOILS

Dominant Soil Materials

gn1—Friable dark brown loam. This is generally a dark brown, friable loam, silt loam or silty clay loam with moderately to strongly pedal structure and porous rough-faced ped fabric. This material occurs as topsoil (A1 horizon).

Peds are commonly sub-angular 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 generally dark brown (10YR 3/3, 7.5YR 3/3) and may range from brownish-black (7.5YR 2/2) to brown (10YR 4/4). This material is occasionally water repellent. The pH ranges from moderately acid (pH 5.0) to slightly acid (pH 6.0). Shale fragments occur and charcoal is occasionally present whilst roots are common.

gn2—Hardsetting brown clay loam. This is commonly 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 commonly brown (7.5YR 4/4) but may range between dull yellowish-brown (10YR 5/4) and reddish-brown (5YR 4/6). The pH ranges between strongly acid (pH 4.0) and moderately acid (pH 6.0). Shale rock fragments, charcoal fragments and roots are present.

gn3—Whole-coloured, reddish-brown, strongly pedal clay. This is medium clay with strongly pedal structure and smooth-faced, dense, ped fabric. It generally occurs as subsoil (B horizon).

Texture is generally medium clay but may range from silty clay to heavy clay. The peds are usually sub-angular blocky or polyhedral. They range in size from 5–20 mm and are smooth-faced and porous. Cutans are also present. Colour is generally reddish-brown (5YR 4/6-8) and can range from bright reddish-brown (2.5YR 4/8) to dull yellowish-brown (10YR 5/4). The pH ranges 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.

gn4—Mottled grey plastic clay. This is a grey, mottled, medium to heavy clay with strongly pedal structure and dense, smooth ped fabric. It commonly occurs as deep subsoil.

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 pale grey (5YR 7/1), but ranges from light reddish-grey (2.5YR 7/1) to brownish-grey (7.5YR 6/1). Yellow and red mottles are common. It is usually moist and is very plastic. The pH ranges from strongly acid (pH 4.0) to moderately acid (pH 5.0). Shale rock fragments and gravels are common. Roots are rare and charcoal is absent.

gn5—Brownish-grey plastic silty clay. This is commonly brownish-grey, plastic silty clay which is often saturated and exhibits apedal massive structure. It usually occurs as subsoil (B horizon).

Colour is dark brown (10YR 3/3) often becoming brownish-grey (10YR 4/1) with dark brown mottles at depth. This material is moderately sticky and very plastic when moist. The pH ranges from moderately acid (pH 5.0) to slightly acid (pH 6.5). Rock and charcoal fragments are absent and roots are rare.

Associated soil materials

Lateritic ironstone concretions. Red, concretionary, ironstone nodules are often found in the deep subsoil and are closely associated with **gn4** soil material.

Occurrence and Relationships

Crests. Up to 15 cm of friable, dark brown loam (**gn1**) overlies 10–30 cm of hardsetting, brown clay loam (**gn2**) and 30–100 cm of whole-coloured reddish-brown, strongly pedal clay (**gn3**). Occasionally **gn1** is absent. Boundaries between soil materials are usually clear. Total soil depth is commonly <150 cm [Red Podzolic Soils (Dr 2.11)].

Upper slopes and midslopes. Up to 15 cm of **gn1** overlies 5–30 cm of **gn2** and >100 cm of **gn3**. **gn3** usually overlies up to 150 cm of mottled grey, plastic, strongly structured clay (**gn4**). Occasionally the **gn1** soil material is absent. Boundaries between soil materials are usually clear. Total soil depth is 50–>100 cm [Red Podzolic Soils (Dr2.11, 2.21), Brown Podzolic Soils (Db 1.11,1.21)].

Lower slopes. Generally 10–60 cm of **gn1** overlies 100 cm of **gn3** and 20–100 cm of **gn4**. Boundaries between soil materials are usually clear. Total soil depth is >150 cm [Yellow Podzolic Soils (Dy 4.11, Dy 5.11)].

Drainage lines. Up to 100 cm of brownish-grey, plastic, silty clay (**gn5**) overlies >100 cm of **gn4**. [Humic Gleys (Uf6.6)]. Many drainage lines contain up to 100 cm of recently transported topsoil **gn1** overlying **gn4** [Gleyed Podzolic Soils (Dg4.11) and occasionally **gn3** (Yellow Podzolic Soils (Dy5.11)].

In some forested areas soils may be gradational. These soils are typically covered with up to 20 cm of forest litter [Prairie Soils (Gn 3.21)].

Occasionally ironstone concretionary nodules associated with laterite occur above and within **gn4** at depth [Red Lateritic Podzolic Soils (KS–Dr3.21)].

LIMITATIONS TO DEVELOPMENT

Urban Capability

Generally, a low to moderate capability for urban development.

Rural Capability

Land generally capable of being grazed and regularly cultivated.

Landscape Limitations

Erosion hazard
Seasonal waterlogging (localised)
Moderate surface swelling potential

Soil Limitations

gn1 Low wet strength
Very strongly acid
High aluminium toxicity

gn2 Low wet strength
Stoniness (localised)
Low fertility
Strongly acid
High aluminium toxicity

gn3 Low wet strength
Low permeability (localised)
Low available water capacity
Stoniness (localised)
Salinity (localised)
Sodicity (localised)
Low fertility
Very strongly acid
Very high aluminium toxicity
High shrink-swell (localised)

gn4 Low wet strength
Low permeability
Low available water capacity
Stoniness
Strongly acid
Very high aluminium toxicity
Low fertility (localised)
Shrink-swell (localised)

gn5 Low wet strength
Low permeability
Low available water capacity

Salinity (localised)
Sodicity (localised)
High shrink-swell (localised)

Fertility

The general fertility is low to moderate. The topsoil (**gn1**) has moderate fertility with high available water capacity, moderate amounts of organic matter, and moderate nutrient status. **gn2** normally has low to moderate fertility with moderate available water capacity, high organic matter content, low CEC, and intrinsically low to moderate nutrient status. All the other soil materials have low fertility with moderate available water capacities, low to moderate CEC and generally low to moderate nitrogen and phosphorus levels (**gn3–gn5**). All soil materials are acid and are potentially aluminium toxic.

Erodibility

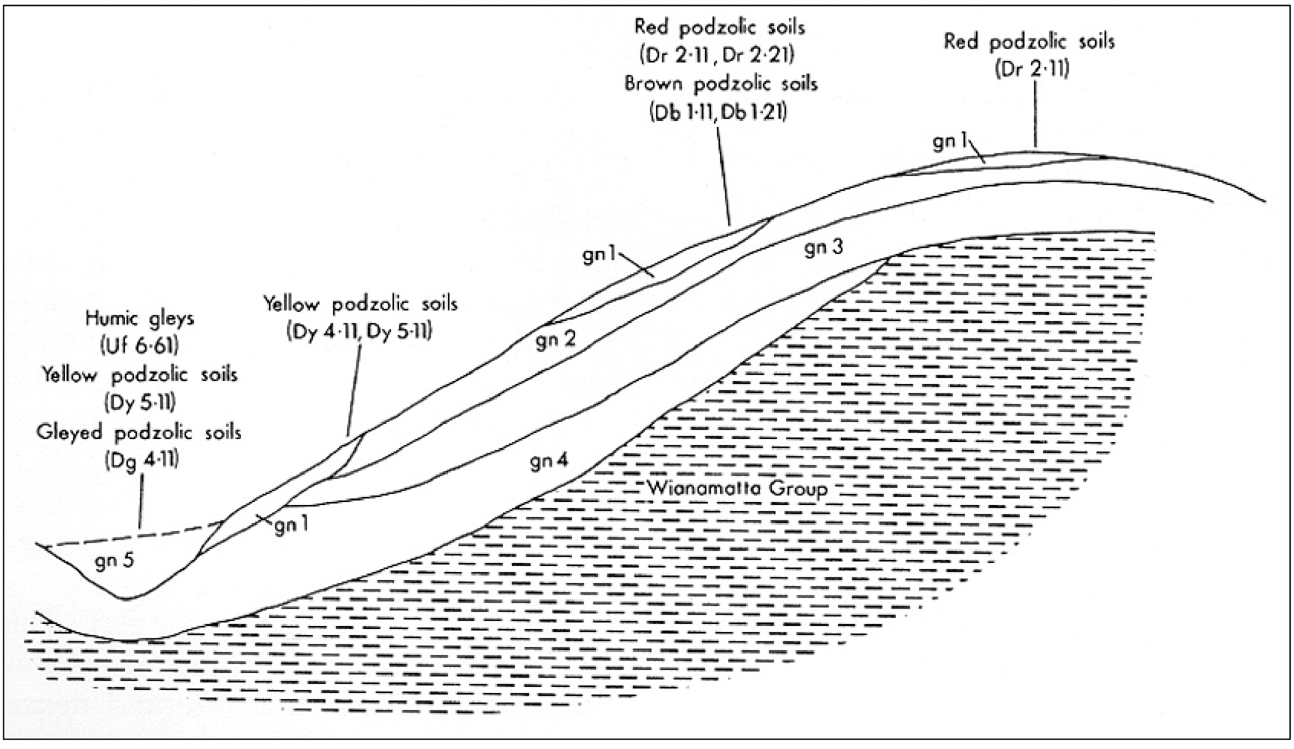
gn1 and **gn2** have low erodibility as they are generally high in organic matter have stable aggregates and are well graded. All the other soil materials are moderately erodible as they are finely graded and have relatively stable aggregates. **gn3–gn5** clays may be locally dispersible and should be considered highly erodible.

Erosion Hazard

The erosion hazard for non-concentrated flows ranges from moderate to very high. Calculated soil loss for the first twelve months of urban development ranges up to 65 t/ha for topsoil and 117 t/ha for exposed subsoil. The soil erosion hazard for concentrated flows is high.

Surface Movement Potential

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



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

wp WEST PENNANT HILLS Colluvial

Landscape—rolling to steep sideslopes on Wianamatta Group shales and shale colluvium. Local relief 40–100 m, slopes >20%. Partially cleared, tall, open-forest (wet sclerophyll).

Soils—deep (>200 cm) Red and Brown Podzolic Soils (Dr2.11, Dr3.11, Db1.11) on upper and midslopes; Yellow and Brown Podzolic Soils My 4.11, Dy5.11, Db1.11) on colluvial benches; Yellow Podzolic Soils (Dy3.11) and Gleyed Podzolic Soils (Dg4.11) in drainage lines and poorly drained areas.

Limitations—mass movement hazard, steep slopes, high soil erosion hazard, localised seasonal waterlogging, impermeable plastic shrink-swell subsoil.

LOCATION

West Pennant Hills soil landscape occurs as steep, narrow, generally south-west facing, hill slopes on the Hornsby Plateau. Examples occur along the Pacific Highway ridge at Gore Hill, Turramurra, Pymble, Wahroonga, Waitara, and to the west of Castle Hill Road at West Pennant Hills and Baulkham Hills. It also occurs at Eastwood, Denistone, Glenhaven and Carlingford.

LANDSCAPE

Geology

Wianamatta Group. Ashfield Shale formation-laminite and dark grey shale. Bringelly Shale-shale, calcareous claystone, laminite, fine to medium grained lithic quartz sandstone (Herbert, 1983).

Topography

Steep sideslopes with mass movement derived landforms. Local relief is 40–100 m with slopes generally >20% and ranging up to 40%. Steep slopes with colluvial benches with southerly and south-westerly aspects are dominant landform elements. Mass movement processes, predominantly soil creep, earthflow and slumping and tension cracks may be evident (Hird and Quilty, 1977). Perched watertables are often associated with colluvial deposits.

Vegetation

Extensively cleared tall open-forest (wet sclerophyll forest). Dominant tree species include Sydney blue gum *Eucalyptus saligna* and blackbutt E. *pilularis.* Other species include turpentine *Syncarpia glomulifera,* grey ironbark *E. paniculata* and white stringybark *E. globoidea.* Understorey shrubs include pittosporum *Pittosporum undulatum* and blackthorn *Bursaria spinosa.* The Dalrymple Hay Reserve at St Ives contains undisturbed native vegetation.

Landuse

Cleared open spaces, parks and reserves are the most common land use. High density residential development occurs at Gore Hill. Low density and strategic residential development occurs at West Pennant Hills and Pymble. The Forestry Commission manages Cumberland State Forest. Horses are grazed at West Pennant Hills.

Associated Soil Landscapes

Glenorie (**gn**) soil landscape is often adjacent and contains similar soil materials. Glenorie soil landscape is less steep, and is not subject to mass movement.

SOILS

Dominant Soil Materials

wpl—Friable, dark brown clay loam. This is a friable loam to silty clay loam with strongly pedal sub-angular blocky structure. It generally occurs as topsoil (A1 horizon).

The peds are commonly rough-faced and porous, ranging- in size between 2 mm and 10 mm. They break down readily into fine crumbs. The surface condition is usually friable. Colour is generally dark brown (l0YR 3/7, 5YR 3/3) but may range from very dark brown (7.5YR 2/2) to a dark yellowish-brown (l0YR 4/4). The pH can range from moderately acid (pH 5.0) to slightly acid (pH 6.0). Weakly weathered shale and brown, platy, iron coated fine sandstone rock fragments are often present.

Charcoal fragments are occasionally present and roots are common.

wp2—Whole-coloured, strongly pedal clay. This is a whole-coloured, silty to heavy clay with strongly pedal, sub-angular blocky structure. It often occurs as subsoil (B horizon).

Peds are commonly smooth-faced and dense, ranging in size between 10 mm and 20 mm. The colour ranges from reddishbrown (5YR 4/6-2.5YR 4/8) to dull yellowish brown (l0YR 5/4). Although not common, red and grey mottles may occur at the base of this material. The pH ranges from extremely acid (pH 4.0) to slightly acid (pH 6.5). Weakly weathered shale and iron coated rock fragments are present and charcoal fragments and roots are rare.

wp3—Mottled, light grey, highly plastic clay. This is a red and yellow mottled highly plastic, medium to heavy clay with strongly pedal, sub-angular blocky structure. It generally occurs as deep subsoil (B horizon).

Peds are commonly smooth-faced and dense and range in size between 10 mm and 20 mm. The colour is usually light brownish grey (5YR 7/ 1), but can range from light reddish grey (2.5YR 7/1) to brownish grey (7.5YR 6/1). Yellow and red mottles are commonly present. The pH ranges from extremely acid (pH 4.0) to very strongly acid (pH 5.0). Strongly weathered shale and iron coated fine sandstone rock fragments are common to abundant. Charcoal fragments are absent and roots are rare.

Occurrence and Relationships

Upper slopes and midslopes. Up to 50 cm of friable, dark brown clay loam (**wp1**) overlies >100 cm of whole-coloured, strongly pedal, brown clay (**wp2**). Bleached A2 horizons are rare. Up to several metres of mottled grey, highly plastic clay (**wp3**) and weathered shale occurs at depth. Occasionally the **wp1** soil material is absent. Boundaries between soil materials are usually clear to gradual. Total soil depth is commonly >200 cm [Red Podzolic Soils (Dr2.11, 2.21), Brown Podzolic Soils (Db 1.11, Db1.21)).

Colluvial shale benches. Soils on colluvial benches are highly variable over short distances. Stones are dispersed and reoriented rather than stratified as in most Wianamatta shale soils.

Up to 80 cm of **wp1** overlies >100 cm of **wp2**, which is sometimes mottled. **wp2** overlies up to several metres of **wp3**. Boundaries between soil materials are clear. Total soil depth is >200 cm [Brown Podzolic Soils (Db1.11), Yellow Podzolic Soils (Dy 4.11, Dy 5.11)].

Footslopes, drainage lines. Up to 60 cm of **wp1** overlies >200 cm of **wp3** and deeply weathered shale [Yellow Podzolic Soil (Dy3.11), Gleyed Podzolic Soils (Dg4.11)].

LIMITATIONS TO DEVELOPMENT

Urban Capability

Generally not capable of urban development.

Rural Capability

Generally not capable of being cultivated or grazed. Small areas with slopes <20% are suitable for grazing.

Landscape Limitations

Mass movement hazard
Steep slopes
Seasonal waterlogging (localised)
Erosion hazard
Run-on (localised)
Moderate to highly reactive soils

Soil Limitations

wp1	Low wet strength
wp2	Low wet strength Low permeability (localised Strongly acid High aluminium toxicity
wp3	Low wet strength Slump prone Low permeability Low available water capacity (localised) Stoniness Low fertility Very strongly acid

High shrink swell (localised)
Very high aluminium toxicity

Fertility

The general fertility is moderate. The topsoil (**wp1**) has high organic matter content, high to moderate CEC and moderate amounts of nitrogen. The other soil materials have low fertility with low to moderate nitrogen and low phosphorus levels and moderate to very high potential aluminium toxicity.

Erodibility

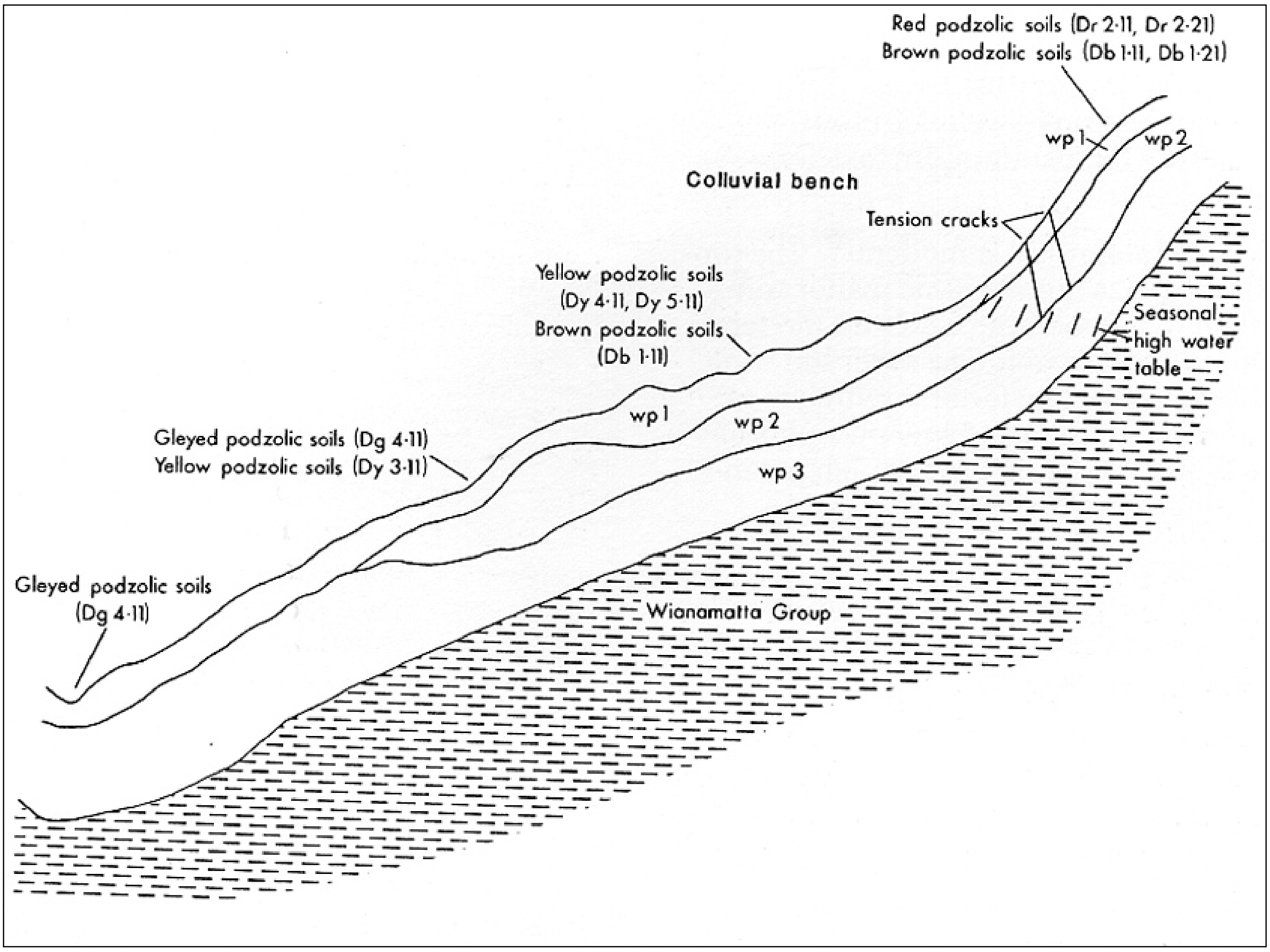
The West Pennant Hills soil materials have low soil erodibility as they are composed of relatively stable soil aggregates.

Erosion Hazard

Because slopes are steep the erosion hazard for non-concentrated flows is high to extreme. Calculated soil loss for the first twelve months of urban development ranges up to 219 t/ha of topsoil and 372 t/ha for exposed subsoil. The erosion hazard for concentrated flows is very high to extreme.

Surface Movement Potential

Moderately reactive. Deep clayey expansive soils occur on a landscape with complex drainage conditions and areas of tall trees. Isolated areas of highly reactive soils occur.



Schematic cross-section of West Pennant Hills soil landscape illustrating the occurrence and relationship of the dominant soil materials.

APPENDIX J

BH1/GW1: 0 to 4 minutes (28th July 2025)

WELL ID: BH1/GW1 No. 241-245 Pennant Hills Road, Carlingford NSW.

Local ID: BH1/GW1
Date: 30/07/2025

INPUT

Construction:

Casing dia. (d_c) 50 mm
Annulus dia. (d_w) 100 mm
Screen Length (L) 13.58 Meter

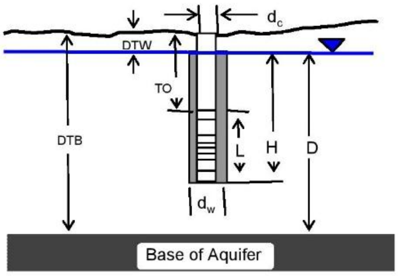
Depths to:

water level (DTW) 9.3 Meter
top of screen (TOS) 1.5 Meter
Base of Aquifer (DTB) 15.08 Meter

Annular Fill:

across screen -- Gravel
above screen -- Bentonite

Aquifer Material -- Shale



COMPUTED

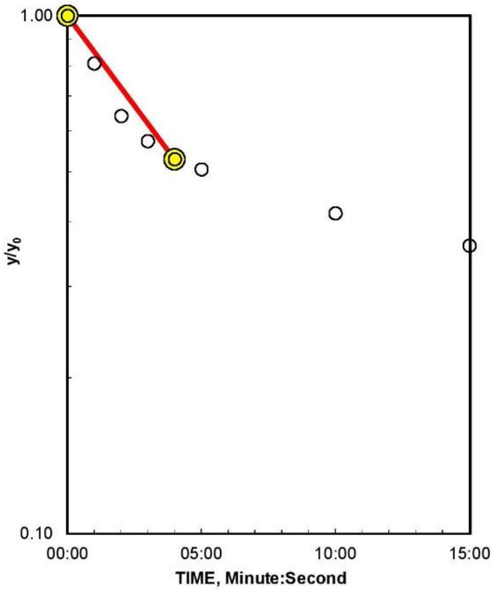
L_{wetted} 5.78 Meter
D = 5.78 Meter
H = 5.78 Meter
L/r_w = 115.60
$y_{0\text{-DISPLACEMENT}}$ = 0.89 Meter
$y_{0\text{-SLUG}}$ = 1.00 Meter
From look-up table using L/r_w

Fully penetrate C = 4.865
ln(Re/rw) = 3.654
Re = 6.34 Meter

Slope = 0.001156 $\log_{10}$/sec
$t_{90\%}$ recovery = 865 sec

Input is consistent.

K = 0.045 Meter/Day



REMARKS:

Bouwer and Rice analysis of slug test, WRR 1976

Geotechnical Consultants Australia

Geotechnical Investigation & Preliminary Groundwater Assessment

Coefficient of Permeability Calculations - Bouwer

Triple Eight Corporation Pty Ltd

Job No.: G25239-1-Rev A

241-245 Pennant Hills Road
Carlingford NSW 2118

Date: 16/09/2025

BH1/GW1: 4 to 15 minutes (28th July 2025)

WELL ID: BH1/GW1 No. 241-245 Pennant Hills Road, Carlingford NSW.

Local ID: BH1/GW1
Date: 30/07/2025

INPUT

Construction:

Casing dia. (d_c)	50	mm
Annulus dia. (d_w)	100	mm
Screen Length (L)	13.58	Meter

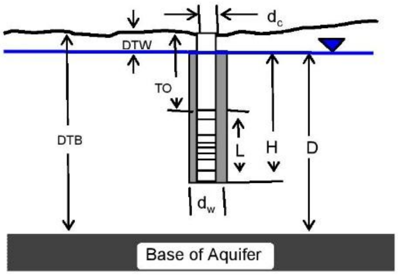
Depths to:

water level (DTW)	9.3	Meter
top of screen (TOS)	1.5	Meter
Base of Aquifer (DTB)	15.08	Meter

Annular Fill:

across screen -- Gravel
above screen -- Bentonite

Aquifer Material -- Shale



COMPUTED

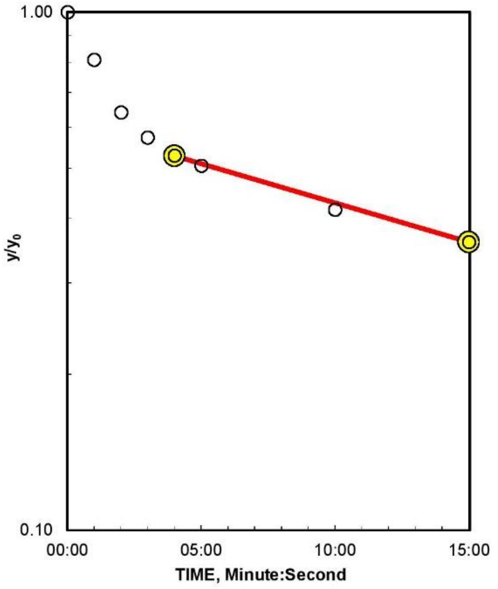
L_{wetted}	5.78	Meter
D =	5.78	Meter
H =	5.78	Meter
L/r_w =	115.60	
$y_{0\text{-DISPLACEMENT}}$ =	0.89	Meter
$y_{0\text{-SLUG}}$ =	1.00	Meter

From look-up table using L/r_w

Fully penetrate C =	4.865	
ln(Re/rw) =	3.654	
Re =	6.34	Meter
Slope =	0.000252	$\log_{10}$/sec
$t_{90\%}$ recovery =	3968	sec

Input is consistent.

K = 0.0099 Meter/Day



REMARKS:

Bouwer and Rice analysis of slug test, WRR 1976

GCA Geotechnical Consultants Australia	**Geotechnical Investigation & Preliminary Groundwater Assessment**	Coefficient of Permeability Calculations - Bouwer
	Triple Eight Corporation Pty Ltd	Job No.: G25239-1-Rev A
	241-245 Pennant Hills Road Carlingford NSW 2118	Date: 16/09/2025

BH1/GW1 (28th July 2025)

WELL ID: BH1/GW1 No. 241-245 Pennant Hills Road, Carlingford NSW

INPUT

Date: 30/07/2025

Construction:

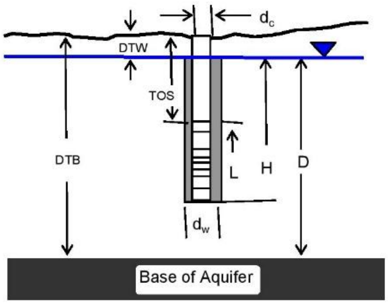
Casing rad. (r_c)	25	mm
Annulus rad. (r_w)	50	mm
Screen Length (L)	5.78	Meter

Depths to:

water level (DTW)	9.3	Meter
top of screen (TOS)	1.5	Meter
Base of Aquifer (DTB)	15.08	Meter

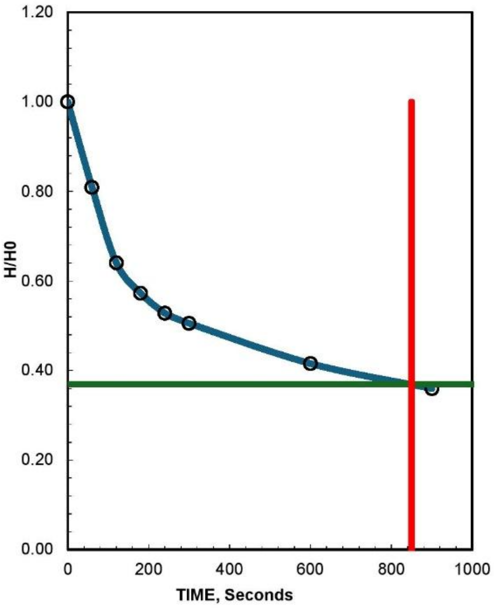
Annular Fill:

across screen --	Gravel
above screen --	Bentonite



OUTPUT

To	850	seconds
k	0.026	m/day



Comments

Refer to detailed engineering borehole logs in Appendix D for aquifer material description.

GCA Geotechnical Consultants Australia	**Geotechnical Investigation & Preliminary Groundwater Assessment**	Coefficient of Permeability Calculations - Hvorslev
	Triple Eight Corporation Pty Ltd	Job No.: G25239-1-Rev A
	241-245 Pennant Hills Road Carlingford NSW 2118	Date: 16/09/2025

BH2/GW2 (28th July 2025)

WELL ID: BH2/GW2 No. 241-245 Pennant Hills Road, Carlingford NSW.

Local ID: BH2/GW2
Date: 30/07/2025

INPUT

Construction:

Casing dia. (d_c)	50	mm
Annulus dia. (d_w)	100	mm
Screen Length (L)	6	Meter

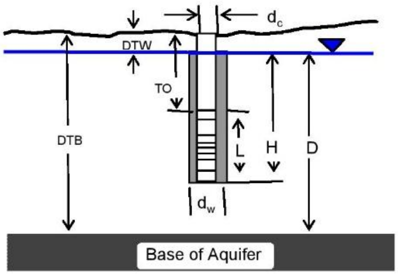
Depths to:

water level (DTW)	5.48	Meter
top of screen (TOS)	9.56	Meter
Base of Aquifer (DTB)	15.56	Meter

Annular Fill:

across screen -- Gravel
above screen -- Bentonite

Aquifer Material -- Shale



COMPUTED

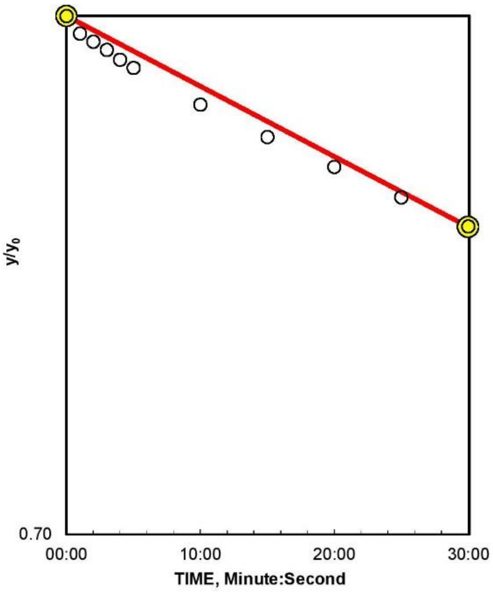
L_{wetted}	6	Meter
D =	10.08	Meter
H =	10.08	Meter
L/r_w =	120.00	
$y_{0\text{-DISPLACEMENT}}$ =	9.13	Meter
$y_{0\text{-SLUG}}$ =	6.00	Meter

From look-up table using L/r_w

Fully penetrate C =	5.011	
ln(Re/rw) =	4.015	
Re =	9.10	Meter
Slope =	3.5E-05	$\log_{10}$/sec
$t_{90\%}$ recovery =	28579	sec

Input is consistent.

K = 0.0015 Meter/Day



REMARKS:

Bouwer and Rice analysis of slug test, WRR 1976

BH3/GW3: 0 to 5 minutes (28th July 2025)

WELL ID: BH3/GW3 No. 241-245 Pennant Hills Road, Carlingford NSW.

Local ID: BH3/GW3

Date: 7/08/2025

INPUT

Construction:

Casing dia. (d_c)	50 mm
Annulus dia. (d_w)	100 mm
Screen Length (L)	6 Meter

Depths to:

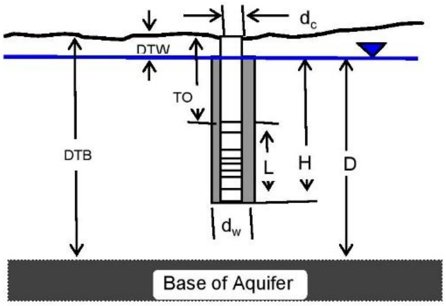
water level (DTW)	7.58 Meter
top of screen (TOS)	8.5 Meter
Base of Aquifer (DTB)	25.00 Meter

Annular Fill:

across screen -- Gravel

above screen -- Bentonite

Aquifer Material -- Shale



COMPUTED

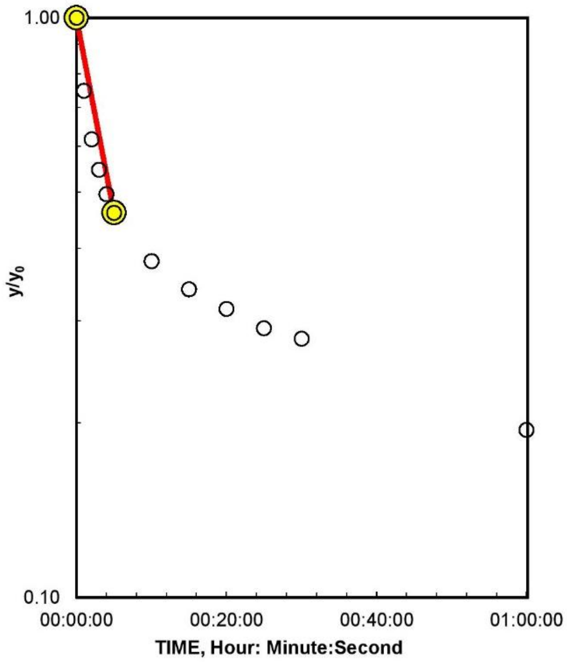
L_{wetted}	6 Meter
D =	17.42 Meter
H =	6.92 Meter
L/r_w =	120.00
$y_{0\text{-DISPLACEMENT}}$ =	9.92 Meter
$y_{0\text{-SLUG}}$ =	6.00 Meter

From look-up table using L/r_w

Partial penetrate A =	4.925
B =	0.857
ln(Re/rw) =	3.308
Re =	4.49 Meter
Slope =	0.001121 $\log_{10}$/sec
$t_{90\%}$ recovery =	892 sec

Input is consistent.

K = 0.038 Meter/Day



REMARKS:

Bouwer and Rice analysis of slug test, WRR 1976

GCA Geotechnical Consultants Australia	**Geotechnical Investigation & Preliminary Groundwater Assessment**	Coefficient of Permeability Calculations - Bouwer
	Triple Eight Corporation Pty Ltd	Job No.: G25239-1-Rev A
	241-245 Pennant Hills Road Carlingford NSW 2118	Date: 16/09/2025

BH3/GW3: 5 to 30 minutes (28th July 2025)

WELL ID: BH3/GW3 No. 241-245 Pennant Hills Road, Carlingford NSW.

Local ID: BH3/GW3
Date: 7/08/2025

INPUT

Construction:

Casing dia. (d_c)	50	mm
Annulus dia. (d_w)	100	mm
Screen Length (L)	6	Meter

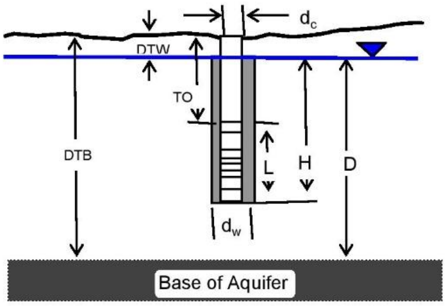
Depths to:

water level (DTW)	7.58	Meter
top of screen (TOS)	8.5	Meter
Base of Aquifer (DTB)	25.00	Meter

Annular Fill:

across screen -- Gravel
above screen -- Bentonite

Aquifer Material -- Shale



COMPUTED

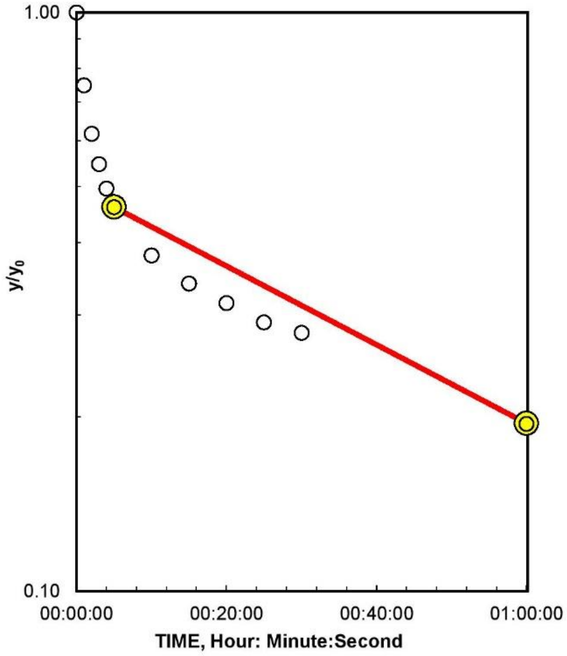
L_{wetted}	6	Meter
D =	17.42	Meter
H =	6.92	Meter
L/r_w =	120.00	
$y_{0\text{-DISPLACEMENT}}$ =	9.92	Meter
$y_{0\text{-SLUG}}$ =	6.00	Meter

From look-up table using L/r_w

Partial penetrate A =	4.925	
B =	0.857	
ln(Re/rw) =	3.308	
Re =	4.49	Meter
Slope =	0.000113	$\log_{10}$/sec
$t_{90\%}$ recovery =	8831	sec

Input is consistent.

K = 0.0039 Meter/Day



REMARKS:

Bouwer and Rice analysis of slug test, WRR 1976

BH3/GW3 (28th July 2025)

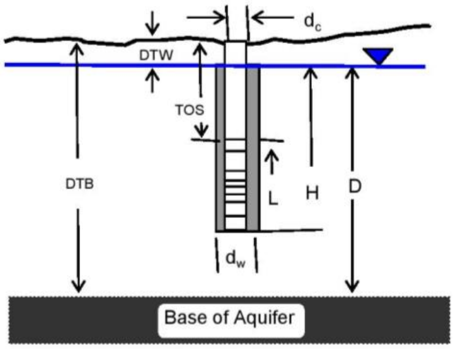
WELL ID: BH3/GW3 No. 241-245 Pennant Hills Road, Carlingford NSW

INPUT

Date: 7/08/2025

Construction:

Casing rad. (r_c)	25 mm
Annulus rad. (r_w)	50 mm
Screen Length (L)	6 Meter



Depths to:

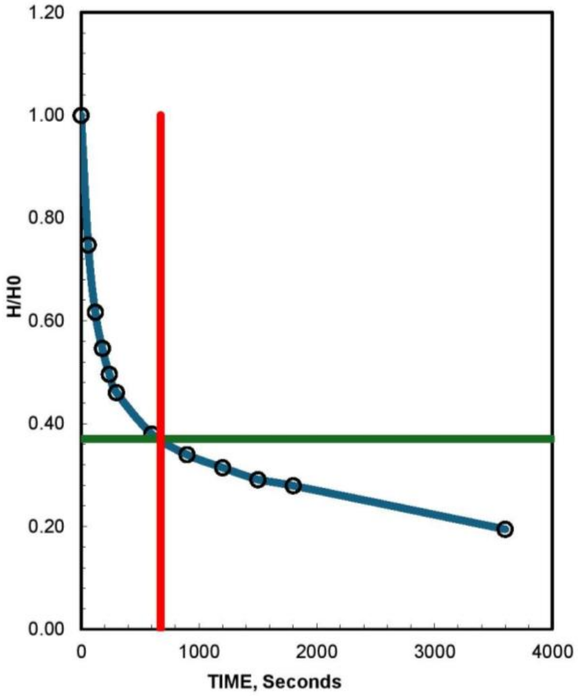
water level (DTW)	7.58 Meter
top of screen (TOS)	8.5 Meter
Base of Aquifer (DTB)	25 Meter

Annular Fill:

across screen -- Gravel
above screen -- Bentonite

OUTPUT

To	675 seconds
k	0.032 m/day



Comments

Refer to detailed engineering borehole logs in Appendix D for aquifer material description.