Contamination | Remediation | Geotechnical

WARDELL BUILDING PTY LTD

Geotechnical Investigation

51 Drummond Street, Belmore NSW

E25284.G03
8 December 2021

Document Control

Report Title: Geotechnical Investigation, 51 Drummond Street, Belmore NSW

Report No: E25284.G03

Copies	Recipient
1 Soft Copy (PDF – Secured, issued by email)	Donny Kayrouz Wardell Building Pty Ltd 3a/254 Wardell Road, MARRICKVILLE NSW 2204
2 Original (Saved to Digital Archives) (Z:\07 - Projects\E25284_Wardell_Belmore_PSI_GI\05_Deliverables\Work in Progress\E25284.G03 - Geotechnical Investigation Report.docx)	EI Australia Suite 6.01, 55 Miller Street, PYRMONT NSW 2009

Author	Technical Reviewer
Kaiyu Xu **Geotechnical Engineer**	**Shahzada Rizvi** **Principal Engineering Geologist**

Revision	Details	Date	Amended By
	Original	8 December 2021	

Table of Contents

Page Number

1. INTRODUCTION **1**
1.1 Background 1
1.2 Proposed Development 1
1.3 Objectives 1
1.4 Scope of Works 2
1.5 Constraints 2

2. SITE DESCRIPTION **3**
2.1 Site Description and Identification 3
2.2 Local Land Use 4
2.3 Regional Setting 4

3. INVESTIGATION RESULTS **6**
3.1 Stratigraphy 6
3.2 Groundwater Observations 6
3.3 Test Results 7

4. RECOMMENDATIONS **8**
4.1 Geotechnical Issues 8
4.2 Dilapidation Surveys 8
4.3 Existing Footings 8
4.4 Excavation Methodology 8
4.4.1 Excavation Assessment 8
4.4.2 Excavation Monitoring 9

4.5 Groundwater Considerations 10
4.6 Excavation Retention 11
4.6.1 Support Systems 11
4.6.2 Retaining Wall Design Parameters 11

4.7 Foundations 13
4.8 Basement Floor Slab 14
4.9 Existing Fill 14

5. FURTHER GEOTECHNICAL INPUTS **15**

6. STATEMENT OF LIMITATIONS **16**

REFERENCES **17**

ABBREVIATIONS **17**

Schedule of Tables

Table 2-1 Summary of Site Information 3
Table 2-2 Summary of Local Land Use 4
Table 2-3 Topographic and Geological Information 4
Table 3-1 Summary of Subsurface Conditions 6
Table 3-2 Summary of Soil Laboratory Test Results 7
Table 4-1 Geotechnical Design Parameters 13

Appendices

FIGURES
Figure 1 Site Locality Plan
Figure 2 Borehole Location Plan

APPENDIX A – BOREHOLE LOGS AND EXPLANATORY NOTES

APPENDIX B – LABORATORY CERTIFICATES

APPENDIX C – VIBRATION LIMITS

APPENDIX D – IMPORTANT INFORMATION

1. Introduction

1.1 Background

At the request of Wardell Building Pty Ltd (the Client), EI Australia (EI) has carried out a Geotechnical Investigation (GI) for the proposed development at 51 Drummond Street, Belmore NSW (the Site).

This GI report has been prepared to provide advice and recommendations to assist in the preparation of designs for the proposed development. The investigation has been carried out in accordance with the agreed scope of works outlined in EI's proposal referenced P19644.2, dated 28 July 2021, and with the Client's signed authorisation to proceed, dated 29 July 2021.

EI has completed a Preliminary Site Investigation (PSI) Report, referenced E25284.E01_Rev0, dated 1 September 2021. This GI report should be read in conjunction with the PSI report.

1.2 Proposed Development

The following documents, supplied by the Client, were used to assist with the preparation of this GI report:

- Architectural drawings prepared by Loucas Architects – Project No. Pn-21010, Drawing Nos. A-0800, A-0900, A-1000, A-1100, A-1200, A1300, Issue A, dated 9 July 2021;

Based on the provided documents, EI understands that the proposed development involves the demolition of the existing site structures and the construction of a five-storey residential development overlying a two-level basement. The lowest basement is proposed to have a Finished Floor Level (FFL) of RL 37.25m. A Bulk Excavation Level (BEL) of RL 37.00m is assumed, which includes allowance for the construction of the basement slab. To achieve the BEL, excavation depths of up to 6.40m Below Existing Ground Level (BEGL) have been estimated. Locally deeper excavations may be required for footings, lift overrun pits, crane pads, and service trenches. The basement extends up to the northern and southern boundaries, has a setback of 3.04m to 4.20m from the western site boundary and a setback of 3.90m from the eastern site boundary.

1.3 Objectives

The objective of the GI was to assess site surface and subsurface conditions at two borehole locations, and to provide preliminary geotechnical advice and recommendations addressing the following:

- Dilapidation Surveys;
- Excavation methodologies and monitoring requirements;
- Groundwater considerations;
- Vibration considerations;
- Excavation support requirements, including preliminary geotechnical design parameters for retaining walls and shoring systems;
- Building foundation options, including;
  - Preliminary design parameters.
  - Earthquake loading factor in accordance with AS1170.4:2007.

- The requirement for additional geotechnical works.

1.4 Scope of Works

The scope of works for the GI included:

- Preparation of a Work Health and Safety Plan;
- Review of relevant geological maps for the project area;
- Site walkover inspection by a Geotechnical Engineer to assess topographical features and site conditions;
- Scanning of proposed borehole locations for buried conductive services using a licensed service locator with reference to Dial Before You Dig (DBYD) plans;
- Auger drilling of two boreholes (BH1 and BH2) by a track-mounted drill rig using solid flight augers equipped with a 'Tungsten-Carbide' (T-C) bit. The boreholes were auger drilled to depths of 3.15m (RL 39.55m) and 4.23m (RL 39.11m) BEGL, respectively;
    - Standard Penetration Testing (SPT) was carried out (as per AS 1289.6.3.1-2004), where possible, during auger drilling of the boreholes to assess soil strength/relative densities;
    - Measurements of groundwater seepage/levels, where possible, in the augered sections of the boreholes during and shortly after completion of auger drilling;
    - The strength of the bedrock in the augered sections of the boreholes was assessed by observation of the auger penetration resistance using a T-C drill bit and examination of the recovered rock cuttings. It should be noted that rock strengths assessed from augered boreholes are approximate and strength variances can be expected;
    - The approximate surface levels shown on the borehole logs were interpolated from spot levels shown on the supplied survey plan. Approximate borehole locations are shown on **Figure 2**;
- Continuation of BH1 and BH2 using NMLC diamond coring techniques to termination depths of 10.38m (RL 32.32m) and 10.20m (RL 33.14m) BEGL, respectively. The rock core photographs are presented in **Appendix A**;
- All boreholes were backfilled with drilling spoils and capped with concrete upon completion;
- Soil and rock samples were sent to STS Geotechnics Pty Ltd (STS) and SGS Australia (SGS), which are National Australian Testing Authority (NATA) accredited laboratories, for testing and storage; and
- Preparation of this GI report.

EI's Geotechnical Engineer was present full-time onsite to set out the borehole locations, direct the testing and sampling, log the subsurface conditions and record groundwater levels.

1.5 Constraints

The GI was limited by the intent of the investigation and the presence of existing site structures. The discussions and advice presented in this report are preliminary and intended to assist in the preparation of initial designs for the proposed development. Further geotechnical inspections should be carried out during construction to confirm the geotechnical and groundwater models, and the preliminary design parameters provided in this report.

2. Site Description

2.1 Site Description and Identification

The site identification details and associated information are presented in **Table 2-1** below while the site locality is shown on **Figure 1.** An aerial photograph of the site is presented in **Plate 1** below.

Table 2-1 Summary of Site Information

Information	Detail
Street Address	51 Drummond Street, Belmore NSW
Lot and Deposited Plan (DP) Identification	Lot 200 in DP 1062028
Brief Site Description	At the time of our investigation, the site was occupied by a two-storey brick commercial building. The building structure appeared to be in fair condition based on a cursory inspection of the exterior walls. The front eastern portion of the site was occupied by an asphalt-paved carpark. The asphalt pavement was in fair condition, with no cracking observed throughout the site.
Site Area	The site area is approximately 1086m^2 (based on the provided architectural drawing referenced above).

Plate 1: Aerial photograph of the site (source: SIX Maps, accessed 13/9/21)

2.2 Local Land Use

The site is situated within an area of commercial use. Current uses on surrounding land at the time of our presence on site are described in **Table 2-2** below. For the sake of this report, the site boundary adjacent to Drummond Street shall be adopted as the eastern site boundary.

Table 2-2 Summary of Local Land Use

Direction Relative to Site	Land Use Description
North	Property at No. 49 Drummond Street, a two-storey brick rendered commercial building with concrete paved carpark at the front. The main building and car park abuts the northern site boundary and appeared to be in fair condition based on inspection of the external walls. No basements were observed at this property.
East	Drummond Street, a two lane, asphalt-paved road. Beyond this is a carpark of a commercial warehouse with no basements.
South	Drummond Lane, a two lane, asphalt paved road. Beyond this are one to two storey commercial buildings with no basements.
West	Property at No. 721 Canterbury Road, a five-storey residential building with basement carpark. The main building has an offset of about 12m from the western site boundary and appeared to be in good condition on inspection of the external walls.

2.3 Regional Setting

The site topography and geological information for the locality is summarised in **Table 2-3** below**.**

Table 2-3 Topographic and Geological Information

Attribute	Description
Topography	The site is located on the slightly higher west side of the road within almost flat topography with site levels varying from RL 42.78m at the eastern site boundary to RL 43.43m at the western site boundary.
Regional Geology	Information on regional sub-surface conditions, referenced from the Department of Mineral Resources Geological Map Sydney 1:100,000 Geological Series Sheet 9130 (DMR 1983) indicates the site to be underlain by Ashfield Shale, which consists of black to dark grey shale and laminite.

Plate 2: Excerpt of geological map showing location of site.

3. Investigation Results

3.1 Stratigraphy

For the development of a site-specific geotechnical model, the stratigraphy observed in the GI has been grouped into four geotechnical units. A summary of the subsurface conditions across the site, interpreted from the assessment results, is presented in **Table 3-1** below. More detailed descriptions of subsurface conditions at each borehole location are available on the borehole logs presented in **Appendix A**. The details of the methods of soil and rock classifications, explanatory notes and abbreviations adopted on the borehole logs are also presented in **Appendix A.**

Table 3-1 Summary of Subsurface Conditions

Unit	Material[2]	Depth to Top of Unit (m BEGL)[1]	RL of Top of Unit (m AHD)[1]	Observed Thickness (m)	Comments
1	**Fill**	0.10 to 0.13	42.60 to 43.21	0.87 to 0.90	Concrete and asphalt pavements of 100mm to 130mm thickness, underlain by silty / gravelly clay fill with traces of fine sand. Fill was assessed, based on our observations during drilling and SPT N Values to be poorly compacted;
2	**Residual Soil**	1.00	41.70 to 42.34	2.15 to 3.23	Medium plasticity, hard silty clay with trace ironstone gravels. SPT values ranged from 30 to refusal indicated by hammer bounced;
3	**Very Low to Low Strength Laminite / Shale**	3.15 to 4.23	39.11 to 39.55	3.11 to 4.25	Distinctly weathered, very low to low strength laminite / shale; The laminite and shale generally consisted of closely to moderately spaced defects consisting of sub-horizontal bedding partings, and fractured / decomposed zones;
4	**Medium to High Strength Shale**	7.34 to 7.40	35.30 to 36.00	-[3]	Fresh, medium to high strength shale;

Note 1 Approximate depth and level at the time of our assessment. Depths and levels may vary across the site.
Note 2 For more detailed descriptions of the subsurface conditions, reference should be made to the borehole logs attached to **Appendix A.**
Note 3 Observed up to termination depth in all boreholes.

3.2 Groundwater Observations

Following completion of auger drilling, the boreholes were left open and free standing groundwater levels were then measured within the boreholes after a period of time. No groundwater or significant seepage was observed during or after auger drilling of the boreholes. Water circulation due to coring within the boreholes prevented further observations of groundwater levels within BH1 and BH2. We note that the groundwater levels may not have become evident or stabilised in the augered boreholes within the limited observation period. No long term groundwater monitoring was carried out.

3.3 Test Results

Three soil samples were selected for laboratory testing to assess the following:

- Atterberg Limits and Linear Shrinkage; and
- Soil aggressivity (pH, chloride and sulfate content and electrical conductivity).

A summary of the soil test results is provided in **Table 3-2** below. Laboratory test certificates are presented in **Appendix B**.

Table 3-2 Summary of Soil Laboratory Test Results

Test/ Sample ID		BH1 0.5-0.95m	BH2 1.5-1.8	BH2 3.0-3.45
Unit		2	2	2
Material Description [1]		Silty CLAY	Silty CLAY	Silty CLAY
Aggressivity	Chloride Cl (ppm)	6.6	4.7	-
	Sulfate SO_4 (ppm)	190	23	-
	pH	6.1	5.2	-
	Electrical Conductivity (µS/cm)	140	21	-
	Moisture Content (%)	17.9	16.6	17.8
Atterberg Limits	Liquid Limit (%)	-	-	41
	Plastic Limit (%)	-	-	21
	Plasticity Index (%)	-	-	20
	Linear Shrinkage (%)	-	-	10.5

Note 1 More detailed descriptions of the subsurface conditions at each borehole location are available on the borehole logs presented in **Appendix A**.

The Atterberg Limits result on the selected clay sample indicated clays to be of medium plasticity and of medium shrink-swell potential.

The assessment indicated low permeability soil was present above the groundwater table. In accordance with Tables 6.4.2(C) and 6.5.2(C) of AS 2159:2009 'Piling – Design and Installation', the results of the pH, chloride and sulfate content and electrical conductivity of the soil provided the following exposure classifications:

- 'Mild' for buried concrete structural elements; and
- 'Non-Aggressive' for buried steel structural elements.

14 selected rock core samples were tested by STS to estimate the Point Load Strength Index (Is_{50}) values to assist with rock strength assessment. The results of the testing are summarised on the attached borehole logs.

The point load strength index tests correlated reasonably well with our field assessments of rock strength. The approximate Unconfined Compressive Strength (UCS) of the rock core, estimated from correlations with the point load strength index test results, varied from <1 MPa to 62 MPa.

4. Recommendations

4.1 Geotechnical Issues

Based on the results of the assessment, we consider the following to be the main geotechnical issues for the proposed development:

- Basement excavation and retention to limit lateral deflections and ground loss as a result of excavations, resulting in damage to nearby structures;
- Rock excavation;
- Groundwater within the depth of the excavation;
- Existing footings of neighbouring properties; and
- Foundation design for building loads.

4.2 Dilapidation Surveys

Prior to excavation and construction, we recommend that detailed dilapidation surveys be carried out on all structures and infrastructures surrounding the site that falls within the zone of influence of the excavation to allow assessment of the recommended vibration limits and protect the client against spurious claims of damage. The zone of influence of the excavation is defined by a distance back from the excavation perimeter of twice the total depth of the excavation. The reports would provide a record of existing conditions prior to commencement of the work. A copy of each report should be provided to the adjoining property owner who should be asked to confirm that it represents a fair assessment of existing conditions. The reports should be carefully reviewed prior to demolition and construction.

4.3 Existing Footings

Prior to any excavation, we recommend that at least two test pits be excavated adjacent to the existing neighbouring footings to the northern site boundary and be inspected by the geotechnical and structural engineers to inspect and assess the in-situ ground conditions at the founding level and footing details. The purpose of these test pits is to assess the requirement of underpinning of these neighbouring footings adjoining the site.

4.4 Excavation Methodology

4.4.1 Excavation Assessment

Prior to any excavation commencing, we recommend that reference be made to the Safe Work Australia Excavation Work Code of Practice, dated January 2020.

EI assumes that the proposed development will require a BEL of RL 37.00m for the basement, or an excavation depth up to 6.40m BEGL. Locally deeper excavations for footings, service trenches, crane pads and lifts overrun pits may be required.

Based on the borehole logs, the proposed basement excavations will therefore extend through Units 1, 2 and 3 as outlined in **Table 3-1** above. As such, an engineered retention system must be installed prior to excavation commencing.

Units 1 and 2 could be excavated using buckets of large earthmoving Hydraulic Excavators, particularly if fitted with ‘Tiger Teeth’. Excavation of Units 3 and 4 (where encountered) may present hard or heavy ripping, or “hard rock” excavation conditions. Ripping would require a high capacity and heavy bulldozer for effective production. Wear and tear should also be allowed for. The use of a smaller size bulldozer will result in lower productivity and higher wear and tear, and this should be allowed for. Alternatively, hydraulic rock breakers, rock saws, ripping hooks or rotary grinders could be used, though productivity would be lower and equipment wear increased, and this should be allowed for.

Should rock hammers be used for the excavation of the bedrock, excavation should commence away from the adjoining structures and the transmitted vibrations monitored to assess how close the hammer can operate to the adjoining structures while maintaining transmitted vibrations within acceptable limits. To fall within these limits, we recommend that the size of rock hammers do not exceed a medium sized rock hammer, say 900 kg, such as a Krupp 580, and be trialled prior to use. The transmitted vibrations from rock hammers should be measured to determine how close each individual hammer can operate to the adjoining buildings.

The vibration measurements can be carried out using either an attended or an unattended vibration monitoring system. An unattended vibration monitoring system must be fitted with an alarm in the form of a strobe light or siren or alerts sent directly to the site supervisor to make the plant operator aware immediately when the vibration limit is exceeded. The vibration monitor must be set to trigger the alarm when the overall Peak Particle Velocity (PPV) exceeds set limits outlined by a vibration monitoring plan. Reference should be made to **Appendix C** for a guide to acceptable limits of transmitted vibrations.

If it is found that the transmitted vibrations by the use of rock hammers are unacceptable, then it would be necessary to change to a smaller excavator with a smaller rock hammer, or to a rotary grinder, rock saws, jackhammers, ripping hooks, chemical rock splitting and milling machines. Although these are likely to be less productive, they would reduce or possibly eliminate risks of damage to adjoining properties through vibration effects transmitted via the ground. Such equipment would also be required for detailed excavation, such as footings or service trenches, and for trimming of faces. Final trimming of faces may also be completed using a grinder attachment rather than a rock breaker in order to assist in limiting vibrations. The use of rotary grinders generally generates dust and this may be supressed by spraying with water.

To assist in reducing vibrations and over-break of the shale bedrock, we recommend that initial saw cutting of the excavation perimeters through the bedrock may be provided using rock saw attachments fitted to the excavator. Rock sawing of the excavation perimeter has several advantages as it often reduces the need for rock bolting as the cut faces generally remain more stable and require a lower level of rock support than hammer cut excavations, ground vibrations from rock saws are minimal and the saw cuts will provide a slight increase in buffer distance for use of rock hammers. However, the effectiveness of such approach must be confirmed by the results of vibration monitoring.

Groundwater seepage monitoring should be carried out during bulk excavation works and prior to finalising the design of a pump out facility. Outlets into the stormwater system will require Council approval.

Furthermore, any existing buried services, which run below the site, will require diversion prior to the commencement of excavation or alternatively be temporarily supported during excavation, subject to permission or other instructions from the relevant service authorities. Enquiries should also be made for further information and details, such as invert levels, on the buried services.

4.4.2 Excavation Monitoring

Consideration should be made to the impact of the proposed development upon neighbouring structures, roadways and services. Basement excavation retention systems should be designed so as to limit lateral deflections.

Contractors should also consider the following limits associated with carrying out excavation and construction activities:

- Limit lateral deflection of temporary or permanent retaining structures;
- Limit vertical settlements of ground surface at common property boundaries and services easement; and
- Limit Peak Particle Velocities (PPV) from vibrations, caused by construction equipment or excavation, experienced by any nearby structures and services.

Monitoring of deflections of retaining structures and surface settlements should be carried out by a registered surveyor at agreed points along the excavation boundaries and along existing building foundations / services/ pavements and other structures located within or near the zone of influence of the excavation. Owners of existing services adjacent to the site should be consulted to assess appropriate deflection limits for their infrastructures. Measurements should be taken in the following sequence:

- Before commencing installation of retaining structures where appropriate to determine the baseline readings. Two independent sets of measurements must be taken confirming measurement consistency;
- After installation of the retaining structures, but before commencement of excavation;
- After excavation to the first row of supports or anchors, but prior to installation of these supports or anchors;
- After excavation to any subsequent rows of supports or anchors, but prior to installation of these supports or anchors;
- After excavation to the base of the excavation;
- After de-stressing and removal of any rows of supports or anchors; and
- One month after completion of the permanent retaining structure or after three consecutive measurements not less than a week apart showing no further movements, whichever is the latter.

4.5 Groundwater Considerations

Due to the low permeability of the bedrock profile, any groundwater inflows into the excavation should not have an adverse impact on the proposed development or on the neighbouring sites and should be manageable. However, we expect that some groundwater inflows into the excavation along the soil/rock interface and through any defects within the shale bedrock (such as jointing, and bedding planes, etc.) particularly following a period of heavy rainfall. The initial flows into the excavation may be locally high, but would be expected to decrease considerably with time as the bedding seams/joints are drained. We recommend that monitoring of seepage be implemented during the excavation works to confirm the capacity of the drainage system.

We expect that any seepage that does occur will be able to be controlled by a conventional sump and pump system. We recommend that a sump-and-pump system be used both during construction and for permanent groundwater control below the basement floor slab.

In the long term, drainage should be provided behind all basement retaining walls, around the perimeter of the basement and below the basement slab. The completed excavation should be inspected by the hydraulic engineer to confirm that adequate drainage has been allowed for. Drainage should be connected to the sump-and-pump system and discharging into the stormwater system. The permanent groundwater control system should take into account any possible soluble substances in the groundwater which may dictate whether or not groundwater can be pumped into the stormwater system.

The design of drainage and pump systems should take the above issues into account along with careful ongoing inspections and maintenance programs.

4.6 Excavation Retention

4.6.1 Support Systems

From a geotechnical perspective, it is critical to maintain the stability of all adjacent structures and infrastructures during demolition, excavation and construction works.

Based on the provided architectural plans, the basement extends up to the northern and southern boundaries, has a setback of 3.04m to 4.20m from the western site boundary and a setback of 3.90m from the eastern site boundary. Based on the above, the close proximity of the surrounding buildings, the encountered subsurface conditions, the shallow groundwater, and the required excavation depth, temporary batters are not recommended for this site. Unsupported vertical cuts of the soil are not recommended for this site as these carry the risk of potential slumping/collapse especially after a period of wet weather. Slumping/Collapse of the material may result in injury to personnel and/or damage to nearby structures/infrastructures and equipment.

The retention system will need to be installed to depths which satisfy stability, piping, founding and groundwater cut-off considerations. Anchors/props and shotcrete must be installed progressively as excavation proceeds.

Working platforms may also be required. We can complete the design of the working platform, if commissioned to do so.

A suitable retention system will be required for the support Units 1, 2 and 3. For this site, EI recommends an anchored and/or propped soldier pile wall with mass concrete in between the piles be founded into medium to high strength shale (Unit 4). Consideration may be made for some piers, which are not supporting the vertical structural loads of the building, to be terminated at least 0.50m, into Unit 3 material or better, above the base of the bulk excavation levels. Anchors/props and mass concrete must be installed progressively as excavation proceeds.

Bored piles are considered to be the most suitable for this site. Tremie pumps may be required where high groundwater seepage inflows are present during the drilling of the bored piles. However, relatively large capacity piling rigs will be required for drilling through the shale bedrock. The proposed pile locations should take into account the presence of buried services. Further advice should be sought from prospective piling contractors who should be provided with a copy of this report.

4.6.2 Retaining Wall Design Parameters

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

- Conventional free-standing cantilever walls which support areas where movement is of little concern (i.e. where only gardens or open areas are to be retained), may be designed using a triangular lateral earth pressure distribution and an ‘active’ earth pressure coefficient, K_a, as shown in **Table 4-1**;
- For progressively anchored or propped walls where minor movements can be tolerated (provided there are no buried movement sensitive services), we recommend the use of a

trapezoidal earth pressure distribution of 5H kPa for soil, where H is the retained height in meters. These pressures should be assumed to be uniform over the central 50% of the support system, tapering to nil at top and bottom;

- For progressively anchored or propped walls which support areas which are highly sensitive to movement (such as areas where movement sensitive structures or infrastructures or buried services are located in close proximity), we recommend the use of a trapezoidal earth pressure distribution of 8H kPa for soil, where ‘H’ is the retained height in meters. These pressures should be assumed to be uniform over the central 50% of the support system, tapering to nil at top and bottom;
- All surcharge loading affecting the walls (including from construction equipment, construction loads, adjacent high level footings, etc.) should be adopted in the retaining wall design as an additional surcharge using an ‘at rest’ earth pressure coefficient, Ko;
- The retaining walls should be designed as drained and measures are to be taken to provide complete and permanent drainage behind the walls. Strip drains protected with a non-woven geotextile fabric should be used behind the shotcrete infill panels for soldier pile walls. Alternatively, for the contiguous pile walls, weepholes comprising 20mm diameter, slotted PVC pipes installed into holes or gaps between adjacent piles at 1.2m centres (horizontal and vertical), may be used. The embedded pipes must, however, be wrapped with a non-woven geotextile fabric (such as Bidim A34) to act as a filter against subsoil erosion;
- For piles embedded into Unit 4 or better, the allowable lateral toe resistance values outlined in **Table 4-1** below may be adopted. These values assume excavation is not carried out within the zone of influence of the wall toe and the rock does not contain adverse defects etc. The upper 0.3m depth of the socket should not be taken into account to allow for tolerance and disturbance effects during excavation;
- If temporary anchors extend beyond the site boundaries, then permission from the neighbouring properties would need to be obtained prior to installation. Also, the presence of neighbouring basements and/or services and their levels must be confirmed prior to finalising anchor design;
- Anchors should have their bond length within Unit 3 or better. For the design of anchors bonded into Unit 3 or better, the allowable bond stress value outlined in **Table 4-1** below may be used, subject to the following conditions:
    1. Anchor bond lengths of at least 3m behind the ‘active’ zone of the excavation (taken as a 45 degree zone above the base of the excavation) is provided;
    2. Overall stability, including anchor group interaction, is satisfied;
    3. All anchors should be proof loaded to at least 1.33 times the design working load before locked off at working load. Such proof loading is to be witnessed by and engineer independent of the anchoring contractor. We recommend that only experienced contractors be considered for anchor installation with appropriate insurances;
    4. If permanent anchors are to be used, these must have appropriate corrosion provisions for longevity.

Table 4-1 Geotechnical Design Parameters

Material [1]		**Unit 1** **Fill**	**Unit 2** **Residual Soil**	**Unit 3** **Very Low to Low Strength Laminite/Shale**	**Unit 4** **Medium to High Strength Shale**
RL of Top of Unit (m AHD) [2]		**42.60 to 43.21**	**41.70 to 42.34**	**39.11 to 39.55**	**35.30 to 36.00**
Bulk Unit Weight (kN/m^3)		**18**	**20**	**24**	**24**
Friction Angle, ϕ' (°)		**25**	**25**	**34**	**40**
Earth Pressure Coefficients	At rest, K_o [3]	0.58	0.58	0.43	-
	Active, K_a [3]	0.41	0.41	0.27	-
	Passive, K_p [3]	-	-	3.69	-
Allowable Bearing Pressure (kPa) [5]		-	-	700	3500
Allowable Shaft Adhesion (kPa) [4, 5]	in Compression	-	-	70	350
	in Uplift	-	-	35	175
Allowable Toe Resistance (kPa)		-	-	70	350
Allowable Bond Stress (kPa)		-	-	50	350
Earthquake Site Risk Classification		▪ AS 1170.4:2007 indicates an earthquake subsoil class of Class C_e.(Shallow Soil) ▪ AS 1170.4:2007 indicates that the hazard factor (z) for Sydney is 0.08.			

Notes:

1 More detailed descriptions of subsurface conditions are available on the borehole logs presented in **Appendix A.**
2 Approximate levels of top of unit at the time of our investigation. Levels may vary across the site.
3 Earth pressures are provided on the assumption that the ground behind the retaining walls is horizontal.
4 Side adhesion values given assume there is intimate contact between the pile and foundation material and should achieve a clean socket roughness category R2 or better. Design engineer to check both 'piston pull-out' and 'cone liftout' mechanics in accordance with AS4678-2002 Earth Retaining Structures.
5 To adopt these parameters we have assumed that:
  - Footings have a nominal socket of at least 0.3m, into the relevant founding material;
  - For piles, there is intimate contact between the pile and foundation material (a clean socket roughness category of R2 or better);
  - Potential soil and groundwater aggressivity will be considered in the design of piles and footings;
  - Piles should be drilled in the presence of a Geotechnical Engineer prior to pile construction to verify that ground conditions meet design assumptions. Where groundwater ingress is encountered during pile excavation, concrete is to be placed as soon as possible upon completion of pile excavation. Pile excavations should be pumped dry of water prior to pouring concrete, or alternatively a tremmie system could be used;
  - The bases of all pile, pad and strip footing excavations are cleaned of loose and softened material and water is pumped out prior to placement of concrete;
  - The concrete is poured on the same day as drilling, inspection and cleaning.
  - The allowable bearing pressures given above are based on serviceability criteria of settlements at the footing base/pile toe of less than or equal to 1% of the minimum footing dimension (or pile diameter).

4.7 Foundations

Following bulk excavation to RL 37.00m, we expect Unit 3 "Very Low to Low Strength Shale" bedrock to be exposed at BEL.

It is recommended that all footings for the building be founded within the shale bedrock of similar strength of at least Unit 3 or better to provide uniform support and reduce the potential for differential settlements.

Pad or strip footings founded within Unit 3 may be preliminarily designed for an allowable bearing capacity of 700kPa, based on serviceability.

Geotechnical inspections of foundations are recommended to determine that the required bearing capacity has been achieved and to determine any variations that may occur between the boreholes and inspected locations.

Alternatively, the proposed development may be supported on deep foundations, such as piles, founded into shale bedrock.

For piles founded on Unit 4 “Medium to High Strength Shale” bedrock, these must be embedded a minimum of 0.50m into shale, and can be designed for a maximum allowable bearing pressure of 3500kPa. The allowable shaft adhesion in shale bedrock may be designed as 10% of the allowable bearing pressure (or 5% for uplift) for the socket length in excess of 0.5m.

At least the initial drilling of piles should be completed in the presence of a geotechnical engineer to verify that ground conditions meet design assumptions.

Where groundwater ingress is encountered during pile excavation, concrete is to be placed as soon as possible upon completion of pile excavation. Pile excavations should be pumped dry of water prior to pouring concrete, or alternatively a tremmie system could be used. Concrete must be poured on the same day as drilling, inspection and drilling.

4.8 Basement Floor Slab

Following bulk excavations for the proposed basement, Unit 3 “Very Low to Low Strength Shale” bedrock is expected to be exposed at the basement floor BEL.

Following the removal of all loose and softened materials, we recommend that underfloor drainage be provided and should comprise a strong, durable, single sized washed aggregate such as ‘blue metal gravel’. Joints in the concrete floor slab should be designed to accommodate shear forces but not bending moments by using dowelled and keyed joints. The basement floor slab should be isolated from columns. The completed excavation should be inspected by the hydraulic engineer to confirm the extent of the drainage required.

In addition, a system of sub-soil drains comprising a durable single sized aggregate with perforated drains/pipes leading to sumps should be provided. The basement floor slab should be isolated from columns.

Permission may need to be obtained from the NSW Department of Primary Industries (DPI) and possibly Council for any permanent discharge of seepage into the drainage system. Given the subsurface conditions, we expect that seepage volumes would be low and within the DPI limits. However, if permission for discharge is not obtained, the basement may need to be designed as a tanked basement.

4.9 Existing Fill

Based on the investigation results, the site is covered by a layer of fill between 0.87m and 0.90m deep. Based on SPT tests within the fill, it appears that it has generally been poorly compacted. However, the SPT tests do not give a precise determination of in-situ densities, since they are affected by friction during driving, the presence of gravel, and the changes in moisture content. Based on available information, the fill on site is not considered to be ‘controlled fill’. AS2870 defines ‘controlled’ fill as material that has been placed and compacted in layers by compaction equipment within a defined moisture range, to a defined density requirement, and placed in accordance with AS3798.

5. Further Geotechnical Inputs

Below is a summary of the previously recommended additional work that needs to be carried out:

- Dilapidation surveys;
- Long term groundwater monitoring and seepage modelling for finalization of suitable retention system;
- Design of working platforms (if required) for construction plant;
- Classification of all excavated material transported off site;
- Witnessing installation of support measures and proof-testing of anchors (if required).
- Geotechnical inspections of all new footings/piles by an experienced geotechnical professional before concrete or steel are placed to verify their bearing capacity and the in-situ nature of the founding strata; and
- Ongoing monitoring of groundwater inflows into the bulk excavation.

We recommend that a meeting be held after initial structural design has been completed to confirm that our recommendations have been correctly interpreted. We also recommend a meeting at the commencement of construction to discuss the primary geotechnical issues and inspection requirements.

6. Statement of Limitations

This report has been prepared for the exclusive use of Donny Kayrouz and Wardell Building Pty Ltd who is the only intended beneficiary of EI’s work. The scope of the assessment carried out for the purpose of this report is limited to those agreed with Donny Kayrouz and Wardell Building Pty Ltd

No other party should rely on the document without the prior written consent of EI, and EI undertakes no duty, or accepts any responsibility or liability, to any third party who purports to rely upon this document without EI's approval.

EI has used a degree of care and skill ordinarily exercised in similar investigations by reputable members of the geotechnical industry in Australia as at the date of this document. No other warranty, expressed or implied, is made or intended. Each section of this report must be read in conjunction with the whole of this report, including its appendices and attachments.

The conclusions presented in this report are based on a limited investigation of conditions, with specific sampling and test locations chosen to be as representative as possible under the given circumstances.

EI's professional opinions are reasonable and based on its professional judgment, experience, training and results from analytical data. EI may also have relied upon information provided by the Client and other third parties to prepare this document, some of which may not have been verified by EI.

EI's professional opinions contained in this document are subject to modification if additional information is obtained through further investigation, observations, or validation testing and analysis during construction. In some cases, further testing and analysis may be required, which may result in a further report with different conclusions.

We draw your attention to the document “Important Information”, which is included in **Appendix D** of this report. The statements presented in this document are intended to advise you of what your realistic expectations of this report should be. The document is not intended to reduce the level of responsibility accepted by EI, but rather to ensure that all parties who may rely on this report are aware of the responsibilities each assumes in so doing.

Should you have any queries regarding this report, please do not hesitate to contact EI.

References

AS1289.6.3.1:2004, *Methods of Testing Soils for Engineering Purposes,* Standards Australia.

AS1726:2017, *Geotechnical Site Investigations*, Standards Australia.

AS2159:2009, *Piling – Design and Installation*, Standards Australia.

AS3600:2009, *Concrete Structures,* Standards Australia

Safe Work Australia Excavation Work Code of Practice, dated January 2020 – WorkCover NSW

NSW Department of Finance and Service, Spatial Information Viewer, maps.six.nsw.gov.au.

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

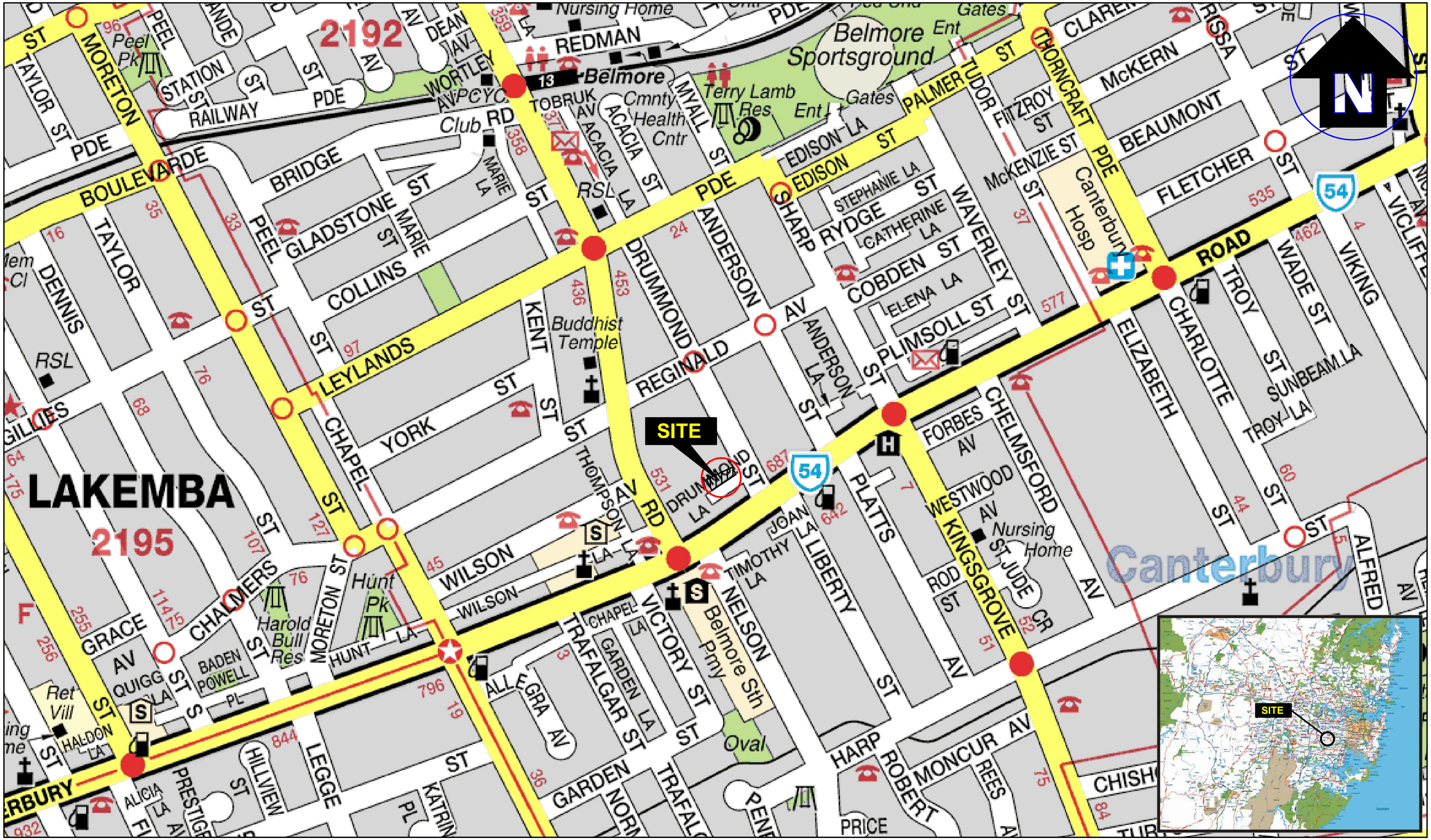
Abbreviations

AHD	Australian Height Datum
AS	Australian Standard
BEL	Bulk Excavation Level
BEGL	Below Existing Ground Level
BH	Borehole
DBYD	Dial Before You Dig
DP	Deposited Plan
EI	EI Australia
GI	Geotechnical Investigation
NATA	National Association of Testing Authorities, Australia
RL	Reduced Level
SPT	Standard Penetration Test
T-C	Tungsten-Carbide
UCS	Unconfined Compressive Strength

Figures

Figure 1	Site Locality Plan

Figure 2	Borehole Location Plan



eiaustralia

Practical Solutions for Built Environments

Suite 6.01, 55 Miller Street, PYRMONT 2009

Ph (02) 9516 0722 Fax (02) 9518 5088

Drawn:	AM.H.
Approved:	K.X.
Date:	8-12-21
Scale:	Not To Scale

Wardell Building Pty Ltd

Geotechnical Investigation

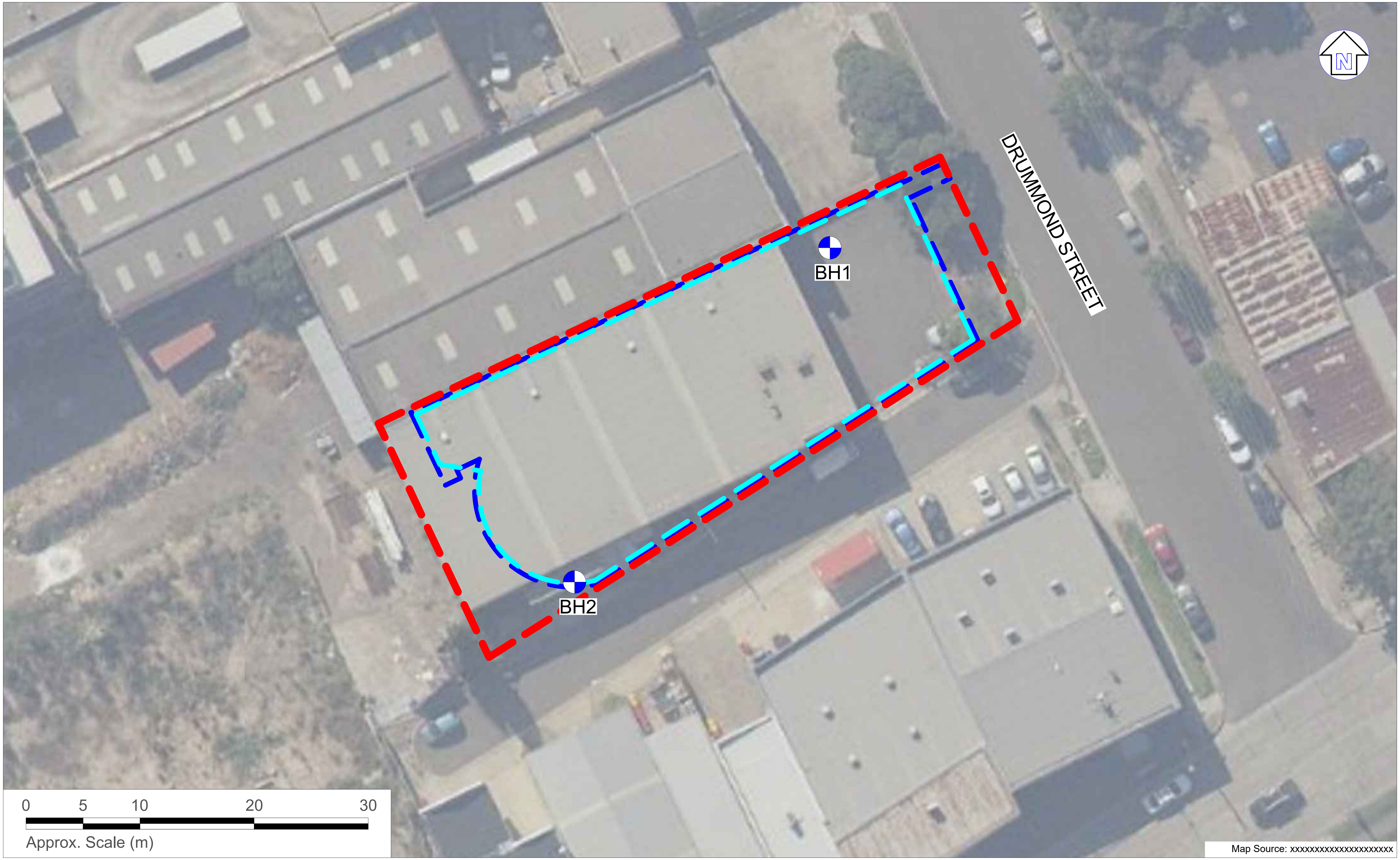
51 Drummond Street, Belmore NSW

Site Locality Plan

Figure:

1

Project: E25284.G03



LEGEND (All Locations are Approximate)

- Site boundary
- Basement 1 boundary
- Basement 2 boundary
- Borehole location

eiaustralia
Practical Solutions for Built Environments
Suite 6.01, 55 Miller Street, PYRMONT 2009
Ph (02) 9516 0722 Fax (02) 9518 5088

Drawn:	AM.H.
Approved:	K.X.
Date:	8-12-21

Wardell Building Pty Ltd
Geotechnical Investigation
51 Drummond Street, Belmore NSW
Borehole Location Plan

Figure: 2

Project: E25284.G03

Appendix A – Borehole Logs And Explanatory Notes

eiaustralia
Practical Solutions for Built Environments

BOREHOLE LOG

BH NO. BH1

Project Proposed Development
Location 51 Drummond Street, Belmore NSW
Position Refer to Figure 2
Job No. E25284.G03
Client Wardell Building Pty Ltd

Sheet 1 of 3
Date Started 26/08/2021
Date Completed 26/08/2021
Logged By KX **Date** 26/08/2021
Reviewed By SR **Date** 08/12/2021

Drilling Contractor Geosense Drilling Pty Ltd
Drill Rig Comacchio Geo 205
Surface RL ≈42.70 m AHD
Inclination -90°

Drilling					Sampling		Field Material Description					
METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	GROUP SYMBOL	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY REL. DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
DT			0	0.10 42.60			■	-	ASPHALT; 100 mm thick.	-	-	PAVEMENT
AD/T	-	GWNE		1.00 41.70	SPT 0.50-0.95 m 2,6,4 N=10	■	⊠	-	FILL: Silty CLAY; low plasticity, dark brown, trace fine grained sand, trace fine to medium gravels.	M (=PL)	-	FILL
			1 2 3	3.15	SPT 1.50-1.73 m 9,11/80mm HB	■	▨	CI	Silty CLAY; medium plasticity, orange mottled grey.	M (<PL)	H	RESIDUAL SOIL
			4 5 6 7 8 9 10						Continued as Cored Borehole			

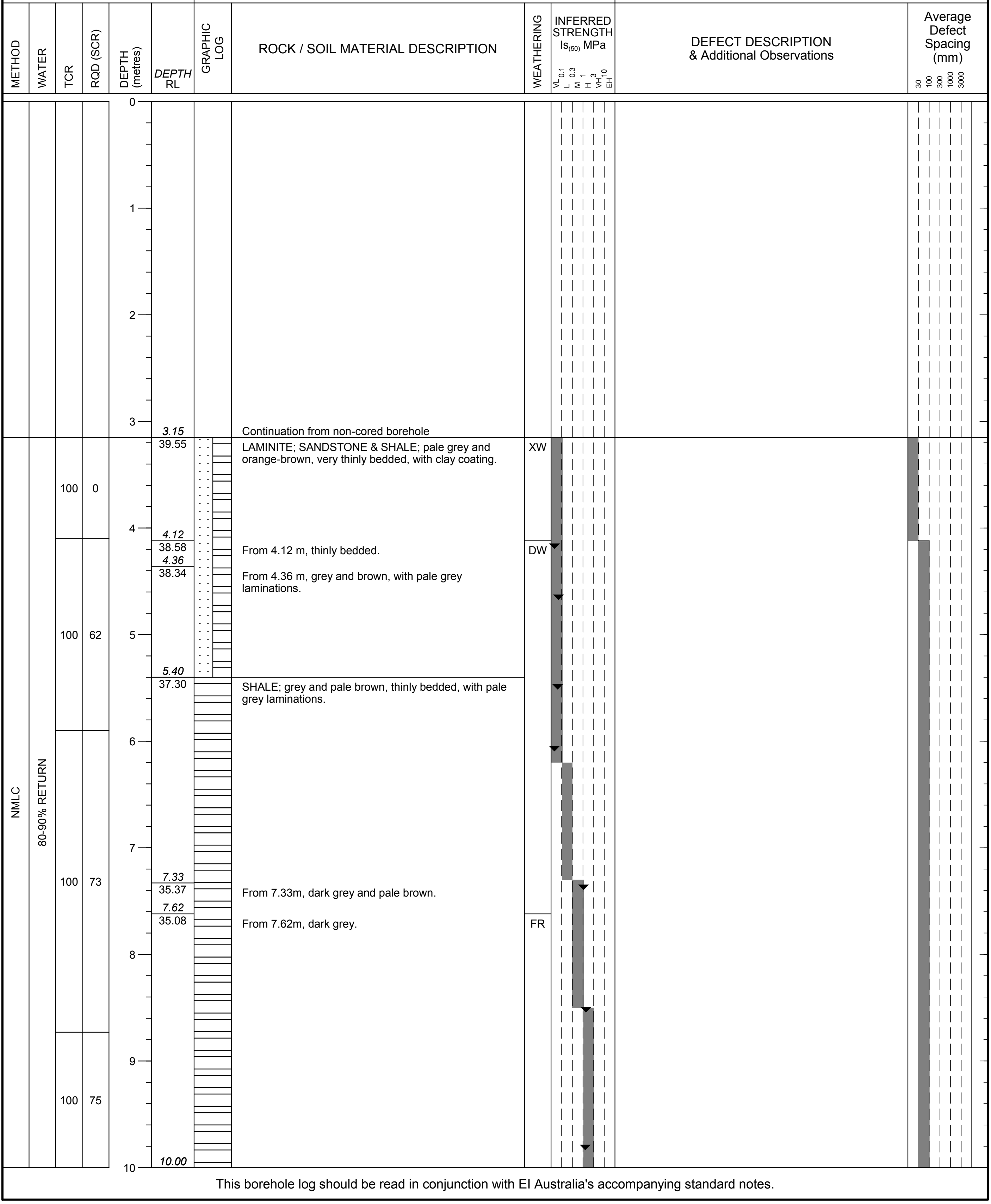
This borehole log should be read in conjunction with EI Australia's accompanying standard notes.

CORED BOREHOLE LOG

BH NO. BH1

Project	Proposed Development	**Sheet**	2 OF 3
Location	51 Drummond Street, Belmore NSW	**Date Started**	26/08/2021
Position	Refer to Figure 2	**Date Completed**	26/08/2021
Job No.	E25284.G03	**Logged By** KX	**Date** 26/08/2021
Client	Wardell Building Pty Ltd	**Reviewed By** SR	**Date** 08/12/2021

Drilling Contactor	Geosense Drilling Pty Ltd	**Surface RL**	≈42.70 m AHD
Drill Rig	Comacchio Geo 205	**Inclination**	-90°



METHOD	WATER	TCR	RQD (SCR)	DEPTH RL	ROCK / SOIL MATERIAL DESCRIPTION	WEATHERING
				3.15	Continuation from non-cored borehole	
NMLC	80-90% RETURN	100	0	39.55	LAMINITE; SANDSTONE & SHALE; pale grey and orange-brown, very thinly bedded, with clay coating.	XW
		100	62	4.12 38.58	From 4.12 m, thinly bedded.	DW
				4.36 38.34	From 4.36 m, grey and brown, with pale grey laminations.	
				5.40 37.30	SHALE; grey and pale brown, thinly bedded, with pale grey laminations.	
		100	73			
				7.33 35.37	From 7.33m, dark grey and pale brown.	
				7.62 35.08	From 7.62m, dark grey.	FR
		100	75			
				10.00		

This borehole log should be read in conjunction with EI Australia's accompanying standard notes.

CORED BOREHOLE LOG

BH NO. BH1

Project	Proposed Development	**Sheet**	3 OF 3
Location	51 Drummond Street, Belmore NSW	**Date Started**	26/08/2021
Position	Refer to Figure 2	**Date Completed**	26/08/2021
Job No.	E25284.G03	**Logged By** KX	**Date** 26/08/2021
Client	Wardell Building Pty Ltd	**Reviewed By** SR	**Date** 08/12/2021

Drilling Contractor	Geosense Drilling Pty Ltd	**Surface RL**	≈42.70 m AHD
Drill Rig	Comacchio Geo 205	**Inclination**	-90°

Drilling						Field Material Description									Defect Information					
METHOD	WATER	TCR	RQD (SCR)	DEPTH (metres)	DEPTH RL	GRAPHIC LOG	ROCK / SOIL MATERIAL DESCRIPTION	WEATHERING	INFERRED STRENGTH $Is_{(50)}$ MPa VL 0.1	L 0.3	M 1	H 3	VH 10	EH	DEFECT DESCRIPTION & Additional Observations	Average Defect Spacing (mm) 30	100	300	1000	3000
NMLC		100	75	10	32.70 *10.38*		SHALE; dark grey, thinly bedded, with pale grey laminations.	FR				▨				▨				
					32.32		Hole Terminated at 10.38 m Target Depth Reached.													
				11																
				12																
				13																
				14																
				15																
				16																
				17																
				18																
				19																
				20																

This borehole log should be read in conjunction with EI Australia's accompanying standard notes.

CORE PHOTOGRAPH OF BOREHOLE: BH1

Project	Proposed Development			**Depth Range**	3.15m to 10.38m BEGL		
Location	51 Drummond Street, Belmore NSW			**Contractor**	Geosense Drilling Engineers Pty Ltd		
Position	See Figure 2	**Surface RL**	≈ 42.70m	**Drill Rig**	Comacchio Geo 205		
Job No.	E25284.G03	**Inclination**	-90°	**Logged**	KX	**Date**	26 / 08 / 2021
Client	Wardell Building Pty Ltd	**Box**	1-2 of 2	**Checked**	SR	**Date**	08 / 12 / 2021

BOREHOLE LOG

eiaustralia
Practical Solutions for Built Environments

BH NO. BH2

Project	Proposed Development	**Sheet**	1 of 3
Location	51 Drummond Street, Belmore NSW	**Date Started**	26/08/2021
Position	Refer to Figure 2	**Date Completed**	26/08/2021
Job No.	E25284.G03	**Logged By** KX	**Date** 26/08/2021
Client	Wardell Building Pty Ltd	**Reviewed By** SR	**Date** 08/12/2021

Drilling Contractor	Geosense Drilling Pty Ltd	**Surface RL**	≈43.34 m AHD
Drill Rig	Comacchio Geo 205	**Inclination**	-90°

Drilling					Sampling		Field Material Description					
METHOD	PENETRATION RESISTANCE	WATER	DEPTH (metres)	DEPTH RL	SAMPLE OR FIELD TEST	RECOVERED	GRAPHIC LOG	GROUP SYMBOL	SOIL/ROCK MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY REL. DENSITY	STRUCTURE AND ADDITIONAL OBSERVATIONS
DT			0	*0.13* 43.21			▦	-	CONCRETE; 130 mm thick.	-	-	PAVEMENT
AD/T	-	GWNE			SPT 0.50-0.95 m 4,3,2 N=5	■	⊠	-	FILL: Gravelly CLAY; low plasticity, grey, fine to medium gravels, trace sand.	M (=PL)	-	FILL
			1	*1.00* 42.34	SPT 1.50-1.80 m 8,23/150mm HB	■	▨	CI	Silty CLAY; medium plasticity, orange-brown.			RESIDUAL SOIL
			2	*2.50* 40.84	SPT 3.00-3.45 m 8,15,15 N=30	■	▨		From 2.5 m, pale grey, trace medium to coarse, sub-angular ironstone gravels.	M (<PL)	H	
			3									
			4	*4.23*								
			5						Continued as Cored Borehole			
			6									
			7									
			8									
			9									
			10									

This borehole log should be read in conjunction with EI Australia's accompanying standard notes.

eiaustralia
Practical Solutions for Built Environments

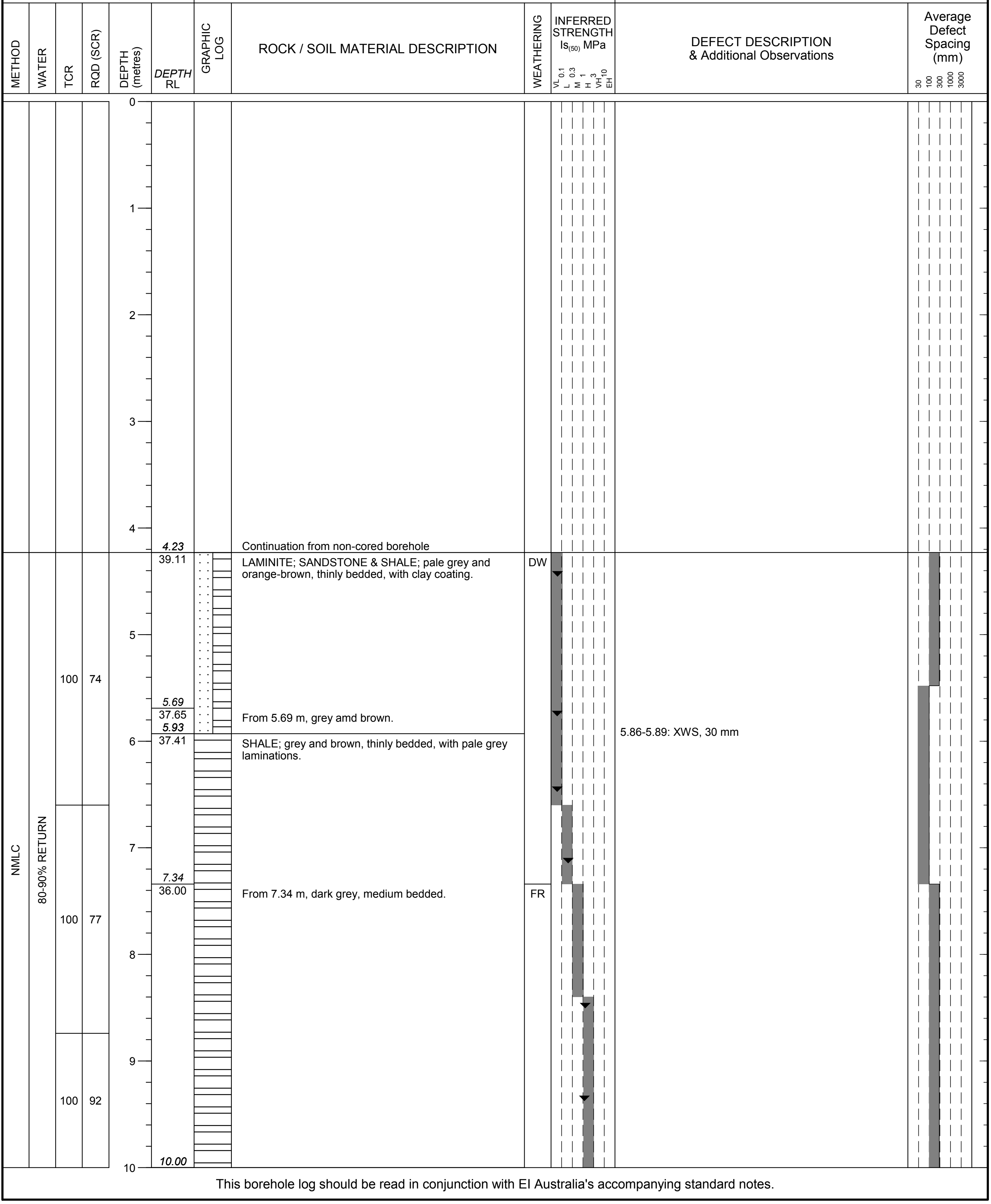
CORED BOREHOLE LOG

BH NO. BH2

Project Proposed Development
Location 51 Drummond Street, Belmore NSW
Position Refer to Figure 2
Job No. E25284.G03
Client Wardell Building Pty Ltd

Sheet 2 OF 3
Date Started 26/08/2021
Date Completed 26/08/2021
Logged By KX **Date** 26/08/2021
Reviewed By SR **Date** 08/12/2021

Drilling Contractor Geosense Drilling Pty Ltd
Drill Rig Comacchio Geo 205
Surface RL ≈43.34 m AHD
Inclination -90°



Drilling						Field Material Description				Defect Information	
METHOD	WATER	TCR	RQD (SCR)	DEPTH (metres)	DEPTH RL	GRAPHIC LOG	ROCK / SOIL MATERIAL DESCRIPTION	WEATHERING	INFERRED STRENGTH Is(50) MPa	DEFECT DESCRIPTION & Additional Observations	Average Defect Spacing (mm)
					4.23		Continuation from non-cored borehole				
NMLC	80-90% RETURN	100	74		39.11		LAMINITE; SANDSTONE & SHALE; pale grey and orange-brown, thinly bedded, with clay coating.	DW			
					5.69 37.65		From 5.69 m, grey amd brown.			5.86-5.89: XWS, 30 mm	
					5.93 37.41		SHALE; grey and brown, thinly bedded, with pale grey laminations.				
		100	77								
					7.34 36.00		From 7.34 m, dark grey, medium bedded.	FR			
		100	92								
					10.00						

This borehole log should be read in conjunction with EI Australia's accompanying standard notes.

CORED BOREHOLE LOG

BH NO. BH2

Project	Proposed Development	**Sheet**	3 OF 3
Location	51 Drummond Street, Belmore NSW	**Date Started**	26/08/2021
Position	Refer to Figure 2	**Date Completed**	26/08/2021
Job No.	E25284.G03	**Logged By** KX	**Date** 26/08/2021
Client	Wardell Building Pty Ltd	**Reviewed By** SR	**Date** 08/12/2021

Drilling Contractor	Geosense Drilling Pty Ltd	**Surface RL**	≈43.34 m AHD
Drill Rig	Comacchio Geo 205	**Inclination**	-90°

Drilling						Field Material Description				Defect Information	
METHOD	WATER	TCR	RQD (SCR)	DEPTH (metres)	*DEPTH* RL	GRAPHIC LOG	ROCK / SOIL MATERIAL DESCRIPTION	WEATHERING	INFERRED STRENGTH $Is_{(50)}$ MPa VL 0.1 L 0.3 M 1 H 3 VH 10 EH	DEFECT DESCRIPTION & Additional Observations	Average Defect Spacing (mm) 30 100 300 1000 3000
		100	92	10	*10.20* 33.14		SHALE; dark grey, medium bedded, with pale grey laminations.	FR			
							Hole Terminated at 10.20 m Target Depth Reached.				
				11							
				12							
				13							
				14							
				15							
				16							
				17							
				18							
				19							
				20							

This borehole log should be read in conjunction with EI Australia's accompanying standard notes.

CORE PHOTOGRAPH OF BOREHOLE: BH2

Project	Proposed Development			**Depth Range**	4.23m to 10.20m BEGL		
Location	51 Drummond Street, Belmore NSW			**Contractor**	Geosense Drilling Engineers Pty Ltd		
Position	See Figure 2	**Surface RL**	≈ 43.34m	**Drill Rig**	Comacchio Geo 205		
Job No.	E25284.G03	**Inclination**	-90°	**Logged**	KX	**Date**	26 / 08 / 2021
Client	Wardell Building Pty Ltd	**Box**	1-2 of 2	**Checked**	SR	**Date**	08 / 12 / 2021

eiaustralia

Contamination | Remediation | Geotechnical

EXPLANATION OF NOTES, ABBREVIATIONS & TERMS USED ON BOREHOLE AND TEST PIT LOGS

DRILLING/EXCAVATION METHOD

HA	Hand Auger	**ADH**	Hollow Auger	**NQ**	Diamond Core - 47 mm
DT	Diatube Coring	**RT**	Rotary Tricone bit	**NMLC**	Diamond Core - 52 mm
NDD	Non-destructive digging	**RAB**	Rotary Air Blast	**HQ**	Diamond Core - 63 mm
AD*	Auger Drilling	**RC**	Reverse Circulation	**HMLC**	Diamond Core - 63 mm
***V**	V-Bit	**PT**	Push Tube	**EX**	Tracked Hydraulic Excavator
***T**	TC-Bit, e.g. AD/T	**WB**	Washbore	**HAND**	Excavated by Hand Methods

PENETRATION RESISTANCE

L	**Low Resistance**	Rapid penetration/ excavation possible with little effort from equipment used.
M	**Medium Resistance**	Penetration/ excavation possible at an acceptable rate with moderate effort from equipment used.
H	**High Resistance**	Penetration/ excavation is possible but at a slow rate and requires significant effort from equipment used.
R	**Refusal/Practical Refusal**	No further progress possible without risk of damage or unacceptable wear to equipment used.

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

WATER

▽ **Standing Water Level**	◁ **Partial water loss**
▷ **Water Seepage**	◀ **Complete Water Loss**

GWNO	GROUNDWATER NOT OBSERVED - Observation of groundwater, whether present or not, was not possible due to drilling water, surface seepage or cave-in of the borehole/ test pit.
GWNE	GROUNDWATER NOT ENCOUNTERED - Borehole/ test pit was dry soon after excavation. However, groundwater could be present in less permeable strata. Inflow may have been observed had the borehole/ test pit been left open for a longer period.

SAMPLING AND TESTING

SPT	Standard Penetration Test to AS1289.6.3.1-2004
4,7,11 N=18	4,7,11 = Blows per 150mm. N = Blows per 300mm penetration following a 150mm seating drive
30/80mm	Where practical refusal occurs, the blows and penetration for that interval are reported, N is not reported
RW	Penetration occurred under the rod weight only, N<1
HW	Penetration occurred under the hammer and rod weight only, N<1
HB	Hammer double bouncing on anvil, N is not reported
Sampling	
DS	Disturbed Sample
ES	Sample for environmental testing
BDS	Bulk disturbed Sample
GS	Gas Sample
WS	Water Sample
U50	Thin walled tube sample - number indicates nominal sample diameter in millimetres
Testing	
FP	Field Permeability test over section noted
FVS	Field Vane Shear test expressed as uncorrected shear strength (sv= peak value, sr= residual value)
PID	Photoionisation Detector reading in ppm
PM	Pressuremeter test over section noted
PP	Pocket Penetrometer test expressed as instrument reading in kPa
WPT	Water Pressure tests
DCP	Dynamic Cone Penetrometer test
CPT	Static Cone Penetration test
CPTu	Static Cone Penetration test with pore pressure (u) measurement

GEOLOGICAL BOUNDARIES

———————— = Observed Boundary (position known)	– – – – – – – – – – – = Observed Boundary (position approximate)	– –?– –?– –?– – = Boundary (interpreted or inferred)

ROCK CORE RECOVERY

TCR=Total Core Recovery (%)	RQD = Rock Quality Designation (%)
$= \frac{Length\ of\ core\ recovered}{Length\ of\ core\ run} \times 100$	$= \frac{\sum Axial\ lengths\ of\ core > 100mm}{Length\ of\ core\ run} \times 100$

eiaustralia
Contamination | Remediation | Geotechnical

METHOD OF SOIL DESCRIPTION USED ON BOREHOLE AND TEST PIT LOGS

▩	FILL	⁂	ORGANIC SOILS (OL, OH or Pt)	☰	CLAY (CL, CI or CH)
⬠	COUBLES or BOULDERS	⁙	SILT (ML or MH)	⁚	SAND (SP or SW)
⁘	GRAVEL (GP or GW)		Combinations of these basic symbols may be used to indicate mixed materials such as sandy clay		

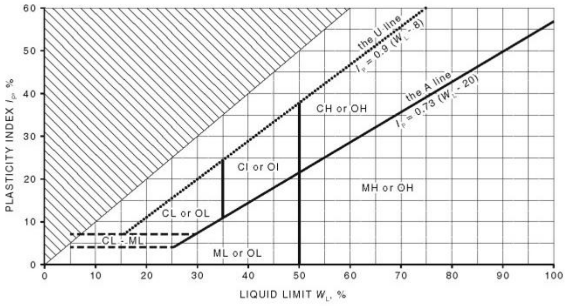
CLASSIFICATION AND INFERRED STRATIGRAPHY

Soil is broadly classified and described in Borehole and Test Pit Logs using the preferred method given in AS 1726:2017, Section 6.1 – Soil description and classification.

PARTICLE SIZE CHARACTERISTICS

Fraction	Components	Sub Division	Size mm
Oversize	BOULDERS		>200
	COBBLES		63 to 200
Coarse grained soil	GRAVEL	Coarse	19 to 63
		Medium	6.7 to 19
		Fine	2.36 to 6.7
	SAND	Coarse	0.6 to 2.36
		Medium	0.21 to 0.6
		Fine	0.075 to 0.21
Fine grained soil	SILT		0.002 to 0.075
	CLAY		<0.002

PLASTICITY PROPERTIES



GROUP SYMBOLS

Major Divisions		Symbol	Description
COARSE GRAINED SOILS More than 65% of soil excluding oversize fraction is greater than 0.075mm	**GRAVEL** More than 50% of coarse fraction is >2.36mm	GW	Well graded gravel and gravel-sand mixtures, little or no fines, no dry strength.
		GP	Poorly graded gravel and gravel-sand mixtures, little or no fines, no dry strength.
		GM	Silty gravel, gravel-sand-silt mixtures, zero to medium dry strength.
		GC	Clayey gravel, gravel-sand-clay mixtures, medium to high dry strength.
	SAND More than 50% of coarse fraction is <2.36 mm	SW	Well graded sand and gravelly sand, little or no fines, no dry strength.
		SP	Poorly graded sand and gravelly sand, little or no fines, no dry strength.
		SM	Silty sand, sand-silt mixtures, zero to medium dry strength.
		SC	Clayey sand, sandy-clay mixtures, medium to high dry strength.
FINE GRAINED SOILS More than 35% of soil excluding oversized fraction is less than 0.075mm	Liquid Limit less < 50%	ML	Inorganic silts of low plasticity, very fine sands, rock flour, silty or clayey fine sands, zero to medium dry strength.
		CL, CI	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, medium to high dry strength.
		OL	Organic silts and organic silty clays of low plasticity, low to medium dry strength.
	Liquid Limit > than 50%	MH	Inorganic silts of high plasticity, high to very high dry strength.
		CH	Inorganic clays of high plasticity, high to very high dry strength.
		OH	Organic clays of medium to high plasticity, medium to high dry strength.
Highly Organic soil		PT	Peat muck and other highly organic soils.

MOISTURE CONDITION

Symbol	Term	Description
D	Dry	Non- cohesive and free-running.
M	Moist	Soils feel cool, darkened in colour. Soil tends to stick together.
W	Wet	Soils feel cool, darkened in colour. Soil tends to stick together, free water forms when handling.

Moisture content of cohesive soils shall be described in relation to plastic limit (PL) or liquid limit (LL) for soils with higher moisture content as follows: Moist, dry of plastic limit (w < PL); Moist, near plastic limit (w ≈ PL); Moist, wet of plastic limit (w < PL); Wet, near liquid limit (w ≈ LL), Wet, wet of liquid limit (w > LL),

CONSISTENCY

Symbol	Term	Undrained Shear Strength (kPa)	SPT "N" #
VS	Very Soft	≤ 12	≤ 2
S	Soft	>12 to ≤ 25	>2 to ≤ 4
F	Firm	>25 to ≤ 50	>4 to 8
St	Stiff	>50 to ≤ 100	>8 to 15
VSt	Very Stiff	>100 to ≤ 200	>15 to 30
H	Hard	>200	>30
Fr	Friable	-	

DENSITY

Symbol	Term	Density Index %	SPT "N" #
VL	Very Loose	≤ 15	0 to 4
L	Loose	>15 to ≤ 35	4 to 10
MD	Medium Dense	>35 to ≤ 65	10 to 30
D	Dense	>65 to ≤ 85	30 to 50
VD	Very Dense	>85	Above 50

In the absence of test results, consistency and density may be assessed from correlations with the observed behaviour of the material.
SPT correlations are not stated in AS1726:2017, and may be subject to corrections for overburden pressure, moisture content of the soil, and equipment type.

MINOR COMPONENTS

Term	Assessment Guide	Proportion by Mass
Add 'Trace'	Presence just detectable by feel or eye but soil properties little or no different to general properties of primary component	Coarse grained soils: ≤ 5% Fine grained soil: ≤ 15%
Add 'With'	Presence easily detectable by feel or eye but soil properties little or no different to general properties of primary component	Coarse grained soils: 5 - 12% Fine grained soil: 15 - 30%
Prefix soil name	Presence easily detectable by feel or eye in conjunction with the general properties of primary component	Coarse grained soils: >12% Fine grained soil: >30%

TERMS FOR ROCK MATERIAL STRENGTH AND WEATHERING

CLASSIFICATION AND INFERRED STRATIGRAPHY

Rock is broadly classified and described in Borehole and Test Pit Logs using the preferred method given in AS1726 – 2017, Section 6.2 – Rock identification, description and classification.

ROCK MATERIAL STRENGTH CLASSIFICATION

Symbol	Term	Point Load Index, $Is_{(50)}$ (MPa) #	Field Guide
VL	Very Low	0.03 to 0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 30 mm can be broken by finger pressure.
L	Low	0.1 to 0.3	Easily scored with a knife; indentations 1 mm to 3 mm show in the specimen with firm blows of pick point; has dull sound under hammer. A piece of core 150 mm long by 50 mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
M	Medium	0.3 to 1	Readily scored with a knife; a piece of core 150 mm long by 50 mm diameter can be broken by hand with difficulty.
H	High	1 to 3	A piece of core 150 mm long by 50 mm diameter cannot be broken by hand but can be broken with pick with a single firm blow; rock rings under hammer.
VH	Very High	3 to 10	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
EH	Extremely High	>10	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

 # Rock Strength Test Results

▼ Point Load Strength Index, $Is_{(50)}$, Axial test (MPa)

● Point Load Strength Index, $Is_{(50)}$, Diametral test (MPa)

Relationship between rock strength test result ($Is_{(50)}$) and unconfined compressive strength (UCS) will vary with rock type and strength, and should be determined on a site-specific basis. However UCS is typically 20 x $Is_{(50)}$.

ROCK MATERIAL WEATHERING CLASSIFICATION

Symbol		Term	Field Guide
RS		Residual Soil	Soil developed on extremely weathered rock; the mass structure and substance fabric are no longer evident; there is a large change in volume but the soil has not been significantly transported.
XW		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.
DW	HW	Distinctly Weathered	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores. In some environments it is convenient to subdivide into Highly Weathered and Moderately Weathered, with the degree of alteration typically less for MW.
	MW		
SW		Slightly Weathered	Rock slightly discoloured but shows little or no change of strength relative to fresh rock.
FR		Fresh	Rock shows no sign of decomposition or staining.

ABBREVIATIONS AND DESCRIPTIONS FOR ROCK MATERIAL AND DEFECTS

CLASSIFICATION AND INFERRED STRATIGRAPHY

Rock is broadly classified and described in Borehole and Test Pit Logs using the preferred method given in AS1726 – 2017, Section 6.2 – Rock identification, description and classification.

DETAILED ROCK DEFECT SPACING

Defect Spacing

Spacing/width (mm)	Descriptor	Symbol
<20	Extremely Close	EC
20-60	Very Close	VC
60-200	Close	C
200-600	Medium	M
600-2000	Wide	W
2000-6000	Very Wide	VW

Bedding Thickness (Stratification)

Term	Spacing (mm)
Thinly laminated	<6
Laminated	6 – 20
Very thinly bedded	20 – 60
Thinly bedded	60 – 200
Medium bedded	200 – 600
Thickly bedded	600 – 2,000
Very thickly bedded	> 2,000

ABBREVIATIONS AND DESCRIPTIONS FOR DEFECT TYPES

Defect Type	Abbr.	Description
Joint	JT	Surface of a fracture or parting, formed without displacement, across which the rock has little or no tensile strength. May be closed or filled by air, water or soil or rock substance, which acts as cement.
Bedding Parting	BP	Surface of fracture or parting, across which the rock has little or no tensile strength, parallel or sub-parallel to layering/ bedding. Bedding refers to the layering or stratification of a rock, indicating orientation during deposition, resulting in planar anisotropy in the rock material.
Contact	CO	The surface between two types or ages of rock.
Sheared Surface	SSU	A near planar, curved or undulating surface which is usually smooth, polished or slickensided.
Sheared Seam/ Zone (Fault)	SS/SZ	Seam or zone with roughly parallel almost planar boundaries of rock substance cut by closely spaced (often <50 mm) parallel and usually smooth or slickensided joints or cleavage planes.
Crushed Seam/ Zone (Fault)	CS/CZ	Seam or zone composed of disoriented usually angular fragments of the host rock substance, with roughly parallel near-planar boundaries. The brecciated fragments may be of clay, silt, sand or gravel sizes or mixtures of these.
Extremely Weathered Seam/ Zone	XWS/XWZ	Seam of soil substance, often with gradational boundaries, formed by weathering of the rock material in places.
Infilled Seam	IS	Seam of soil substance, usually clay or clayey, with very distinct roughly parallel boundaries, formed by soil migrating into joint or open cavity.
Vein	VN	Distinct sheet-like body of minerals crystallised within rock through typically open-space filling or crack-seal growth.

NOTE: Defects size of <100mm SS, CS and XWS. Defects size of >100mm SZ, CZ and XWZ.

ABBREVIATIONS AND DESCRIPTIONS FOR DEFECT SHAPE AND ROUGHNESS

Shape	Abbr.	Description	Roughness	Abbr.	Description
Planar	PR	Consistent orientation	Polished	POL	Shiny smooth surface
Curved	CU	Gradual change in orientation	Slickensided	SL	Grooved or striated surface, usually polished
Undulating	UN	Wavy surface	Smooth	SM	Smooth to touch. Few or no surface irregularities
Stepped	ST	One or more well defined steps	Rough	RO	Many small surface irregularities (amplitude generally <1mm). Feels like fine to coarse sandpaper
Irregular	IR	Many sharp changes in orientation	Very Rough	VR	Many large surface irregularities, amplitude generally >1mm. Feels like very coarse sandpaper

Orientation: **Vertical Boreholes –** The dip (inclination from horizontal) of the defect.
Inclined Boreholes – The inclination is measured as the acute angle to the core axis.

ABBREVIATIONS AND DESCRIPTIONS FOR DEFECT COATING

Coating	Abbr.	Description
Clean	CN	No visible coating or infilling
Stain	SN	No visible coating but surfaces are discoloured by staining, often limonite (orange-brown)
Veneer	VNR	A visible coating of soil or mineral substance, usually too thin to measure (< 1 mm); may be patchy

DEFECT APERTURE

Aperture	Abbr.	Description
Closed	CL	Closed.
Open	OP	Without any infill material.
Infilled	-	Soil or rock i.e. clay, silt, talc, pyrite, quartz, etc.

Appendix B – Laboratory Certificates

ANALYTICAL REPORT

Accreditation No. 2562

CLIENT DETAILS

Contact	Kaiyu Xu
Client	EI AUSTRALIA
Address	SUITE 6.01 55 MILLER STREET PYRMONT NSW 2009
Telephone	61 2 9516 0722
Facsimile	(Not specified)
Email	kaiyu.xu@eiaustralia.com.au
Project	**E25284.G03 51 Drummond St, Belmore NSW**
Order Number	**E25284.G03**
Samples	2

LABORATORY DETAILS

Manager	Huong Crawford
Laboratory	SGS Alexandria Environmental
Address	Unit 16, 33 Maddox St Alexandria NSW 2015
Telephone	+61 2 8594 0400
Facsimile	+61 2 8594 0499
Email	au.environmental.sydney@sgs.com
SGS Reference	**SE223070 R0**
Date Received	30/8/2021
Date Reported	3/9/2021

COMMENTS

Accredited for compliance with ISO/IEC 17025 - Testing. NATA accredited laboratory 2562(4354).

SIGNATORIES

Dong LIANG
Metals/Inorganics Team Leader

Shane MCDERMOTT
Inorganic/Metals Chemist

Soluble Anions (1:5) in Soil/Solids by Ion Chromatography [AN245] Tested: 1/9/2021

PARAMETER	UOM	LOR	BH1 0.5-0.95m SOIL - 26/8/2021 SE223070.001	BH2 1.5-1.8m SOIL - 26/8/2021 SE223070.002
Chloride	mg/kg	0.25	6.6	4.7
Sulfate	mg/kg	5	190	23

pH in soil (1:5) [AN101] Tested: 1/9/2021

PARAMETER	UOM	LOR	BH1 0.5-0.95m SOIL - 26/8/2021 SE223070.001	BH2 1.5-1.8m SOIL - 26/8/2021 SE223070.002
pH	pH Units	0.1	6.1	5.2

Conductivity and TDS by Calculation - Soil [AN106] Tested: 1/9/2021

			BH1 0.5-0.95m SOIL - 26/8/2021	BH2 1.5-1.8m SOIL - 26/8/2021
PARAMETER	UOM	LOR	SE223070.001	SE223070.002
Conductivity of Extract (1:5 dry sample basis)	µS/cm	1	140	21

Moisture Content [AN002] Tested: 31/8/2021

PARAMETER	UOM	LOR	BH1 0.5-0.95m SOIL - 26/8/2021 SE223070.001	BH2 1.5-1.8m SOIL - 26/8/2021 SE223070.002
% Moisture	%w/w	1	17.9	16.6

METHOD	METHODOLOGY SUMMARY
AN002	The test is carried out by drying (at either 40°C or 105°C) a known mass of sample in a weighed evaporating basin. After fully dry the sample is re-weighed. Samples such as sludge and sediment having high percentages of moisture will take some time in a drying oven for complete removal of water.
AN101	pH in Soil Sludge Sediment and Water: pH is measured electrometrically using a combination electrode and is calibrated against 3 buffers purchased commercially. For soils, sediments and sludges, an extract with water (or 0.01M CaCl2) is made at a ratio of 1:5 and the pH determined and reported on the extract. Reference APHA 4500-H+.
AN106	Conductivity and TDS by Calculation: Conductivity is measured by meter with temperature compensation and is calibrated against a standard solution of potassium chloride. Conductivity is generally reported as µmhos/cm or µS/cm @ 25°C. For soils, an extract of as received sample with water is made at a ratio of 1:5 and the EC determined and reported on the extract, or calculated back to the as-received sample. Salinity can be estimated from conductivity using a conversion factor, which for natural waters, is in the range 0.55 to 0.75. Reference APHA 2510 B.
AN245	Anions by Ion Chromatography: A water sample is injected into an eluent stream that passes through the ion chromatographic system where the anions of interest ie Br, Cl, NO2, NO3 and SO4 are separated on their relative affinities for the active sites on the column packing material. Changes to the conductivity and the UV-visible absorbance of the eluent enable identification and quantitation of the anions based on their retention time and peak height or area. APHA 4110 B

FOOTNOTES

*	NATA accreditation does not cover the performance of this service.	-	Not analysed.	UOM	Unit of Measure.
		NVL	Not validated.	LOR	Limit of Reporting.
**	Indicative data, theoretical holding time exceeded.	IS	Insufficient sample for analysis.	↑↓	Raised/lowered Limit of Reporting.
		LNR	Sample listed, but not received.		
***	Indicates that both * and ** apply.				

Unless it is reported that sampling has been performed by SGS, the samples have been analysed as received.
Solid samples expressed on a dry weight basis.

Where "Total" analyte groups are reported (for example, Total PAHs, Total OC Pesticides) the total will be calculated as the sum of the individual analytes, with those analytes that are reported as <LOR being assumed to be zero. The summed (Total) limit of reporting is calculated by summing the individual analyte LORs and dividing by two. For example, where 16 individual analytes are being summed and each has an LOR of 0.1 mg/kg, the "Totals" LOR will be 1.6 / 2 (0.8 mg/kg). Where only 2 analytes are being summed, the " Total" LOR will be the sum of those two LORs.

Some totals may not appear to add up because the total is rounded after adding up the raw values.

If reported, measurement uncertainty follow the ± sign after the analytical result and is expressed as the expanded uncertainty calculated using a coverage factor of 2, providing a level of confidence of approximately 95%, unless stated otherwise in the comments section of this report.

Results reported for samples tested under test methods with codes starting with ARS-SOP, radionuclide or gross radioactivity concentrations are expressed in becquerel (Bq) per unit of mass or volume or per wipe as stated on the report. Becquerel is the SI unit for activity and equals one nuclear transformation per second.
Note that in terms of units of radioactivity:
a. 1 Bq is equivalent to 27 pCi
b. 37 MBq is equivalent to 1 mCi

For results reported for samples tested under test methods with codes starting with ARS-SOP, less than (<) values indicate the detection limit for each radionuclide or parameter for the measurement system used. The respective detection limits have been calculated in accordance with ISO 11929.

The QC and MU criteria are subject to internal review according to the SGS QAQC plan and may be provided on request or alternatively can be found here: www.sgs.com.au/en-gb/environment-health-and-safety.

This document is issued by the Company under its General Conditions of Service accessible at www.sgs.com/en/Terms-and-Conditions.aspx. Attention is drawn to the limitation of liability, indemnification and jurisdiction issues defined therein.

Any holder of this document is advised that information contained hereon reflects the Company's findings at the time of its intervention only and within the limits of Client's instructions, if any. The Company's sole responsibility is to its Client only. Any unauthorized alteration, forgery or falsification of the content or appearance of this document is unlawful and offenders may be prosecuted to the fullest extent of the law.

This report must not be reproduced, except in full.

GEOTECHNICS PTY LTD
CONSULTING GEOTECHNICAL ENGINEERS

STS Geotechnics Pty Ltd
14/1 Cowpasture Place, Wetherill Park NSW 2164
Phone: (02)9756 2166 | Email: enquiries@stsgeo.com.au

NATA
Accredited for Compliance with ISO/IEC 17025 - Testing No. 2750

Atterberg Limits and Linear Shrinkage Report

Project: 51 DRUMMOND ST, BELMORE NSW - E25284.G03
Client: EI Australia Pty Ltd
Address: Suite 6.01, 55 Miller Street, Pyrmont NSW 2009
Test Method: AS1289.3.1.2, .3.2.1, .3.1.1, .3.4.1

Project No.: 31380
Report No.: 21/2541
Report Date: 3/09/2021
Page: 1 OF 2

Sampling Procedure: Samples Supplied By Client (Not covered under NATA Scope of Accreditation)

STS / Sample No.	5488D-L/1					
Sample Location	Borehole 2					
Material Description	Silty Clay, grey with brown trace of gravel					
Depth (m)	3.0 - 3.45					
Sample Date	26/08/2021					
Sample History	Oven Dried					
Method of Preparation	Dry Sieved					
Liquid Limit (%)	**41**					
Plastic Limit (%)	**21**					
Plasticity Index	**20**					
Linear Shrinkage (%)	**10.5**					
Mould Size (mm)	127					
Crumbing	N					
Curling	N					

Remarks:

Approved Signatory...................... ....

Technician: ZW

Orlando Mendoza - Laboratory Manager

Moisture Content of Soil and Aggregate Samples

Project: 51 DRUMMOND ST, BELMORE NSW - E25284.G03
Project No.: 31380

Client: EI Australia Pty Ltd
Report No.: 21/2541

Address: Suite 6.01, 55 Miller Street, Pyrmont NSW 2009
Report Date: 3/09/2021

Test Method: AS1289.3.1.2, .3.2.1, .3.1.1, .3.4.1
Page: 2 OF 2

Sampling Procedure: Samples Supplied By Client (Not covered under NATA Scope of Accreditation)

STS / Sample No.	5488D-L/1					
Sample Location	Borehole 2					
Material Description	Silty Clay, grey with brown trace of gravel					
Depth (mm)	3.0 - 3.45					
Sample Date	26/08/2021					
Moisture Content (%)	**17.8**					

Remarks:

Approved Signatory...................... ....

Technician: ZW

Orlando Mendoza - Laboratory Manager

STS Geotechnics Pty Ltd
14/1 Cowpasture Place, Wetherill Park NSW 2164
Phone: (02)9756 2166 | Email: enquiries@stsgeo.com.au

Accredited for Compliance with ISO/IEC 17025 - Testing No. 2750

Point Load Strength Index Report

Project: 51 DRUMMOND ST, BELMORE NSW - E25284.G03
Client: EI Australia Pty Ltd
Address: Suite 6.01, 55 Miller Street, Pyrmont NSW 2009
Test Method: AS4133.4.1

Project No.: 31380/5488D-L
Report No.: 21/2517
Report Date: 2/09/2021
Page: 1 OF 1

Sampling Procedure: Samples Supplied By Client (Not covered under NATA Scope of Accreditation)

Date Samples Drilled / Taken: 26/08/2021

Borehole No. 1

Depth	Test Type	**Is(50) (Mpa)**	Rock Type	Failure Type	Moisture
4.17	A	**0.043**	YS	3	M
4.65	A	**0.067**	YS	3	M
5.49	A	**0.061**	SH	3	M
6.07	A	**0.043**	SH	3	M
7.37	A	**1.000**	SH	3	D
8.52	A	**1.300**	SH	3	D
9.81	A	**1.200**	SH	3	D

Sampling Procedure: Samples Supplied By Client (Not covered under NATA Scope of Accreditation)

Date Samples Drilled / Taken: 26/08/2021

Borehole No. 2

Depth	Test Type	**Is(50) (Mpa)**	Rock Type	Failure Type	Moisture
4.43	A	**0.060**	YS	3	M
5.74	A	**0.058**	YS	3	M
6.45	A	**0.058**	SH	3	M
7.12	A	**0.190**	SH	3/1	W
8.48	A	**1.200**	SH	3	D
9.35	A	**1.100**	SH	3/1	D
10.11	A	**3.100**	SH	3	D

FAILURE TYPE
1= FRACTURE THROUGH BEDDING OR WEAK PLANE
2= FRACTURE ALONG BEDDING
3= FRACTURE THROUGH ROCK MASS
4= FRACTURE INFLUENCED BY NATURAL DEFECT OR DRILLING
5= PARTIAL FRACTURE OR CHIP (INVALID RESULT)

TEST TYPE
A= AXIAL
D= DIAMETRAL
I= IRREGULAR
C= CUBE

MOISTURE CONDITION
W= WET
M= MOIST
D= DRY

ROCK TYPE
SS= SANDSTONE
ST= SILTSTONE
SH= SHALE
YS= CLAYSTONE
IG= IGNEOUS

Remarks:

Approved Signatory........... ....

Technician: ZW

Orlando Mendoza - Laboratory Manager

Appendix C – Vibration Limits

German Standard DIN 4150 – Part 3: 1999 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally considered to be conservative.

The DIN 4150 values (maximum levels measured in any direction at the foundation, OR, maximum levels measured in (x) or (y) directions, in the plane of the uppermost floor), are summarised in **Table A** below.

It should be noted that peak vibration velocities higher than the minimum figures in **Table A** for low frequencies may be quite 'safe', depending on the frequency content of the vibration and the actual conditions of the structures.

It should also be noted that these levels are 'safe limits', up to which no damage due to vibration effects has been observed for the particular class of building. 'Damage' is defined by DIN 4150 to include even minor non-structural cracking in cement render, the enlargement of cracks already present, and the separation of partitions or intermediate walls from load bearing walls. Should damage be observed at vibration levels lower than the 'safe limits', then it may be attributed to other causes. DIN 4150 also states that when vibration levels higher than the 'safe limits' are present, it does not necessarily follow that damage will occur. Values given are only a broad guide.

Table A DIN 4150 – Structural Damage – Safe Limits for Building Vibration

Group	Type of Structure	Peak Vibration Velocity (mm/s)			
		At Foundation Level at a Frequency of:			Plane of Floor of Uppermost Storey
		Less than 10 Hz	10 Hz to 50 Hz	50 Hz to 100 Hz	All Frequencies
1	Buildings used for commercial purposes, industrial buildings and buildings of similar design	20	20 to 40	40 to 50	40
2	Dwellings and buildings of similar design and/or use	5	5 to 15	15 to 20	15
3	Structures that because of their particular sensitivity to vibration, do not correspond to those listed in Group 1 and 2 and have intrinsic value (e.g. buildings that are under a preservation order)	3	3 to 8	8 to 10	8

Note: For frequencies above 100 Hz, the higher values in the 50 Hz to 100 Hz column should be used.

Appendix D – Important Information

Important Information

SCOPE OF SERVICES

The geotechnical report ("the report") has been prepared in accordance with the scope of services as set out in the contract, or as otherwise agreed, between the Client And EI Australia ("EI"). The scope of work may have been limited by a range of factors such as time, budget, access and/or site disturbance constraints.

RELIANCE ON DATA

EI has relied on data provided by the Client and other individuals and organizations, to prepare the report. Such data may include surveys, analyses, designs, maps and plans. EI has not verified the accuracy or completeness of the data except as stated in the report. To the extent that the statements, opinions, facts, information, conclusions and/or recommendations ("conclusions") are based in whole or part on the data, EI will not be liable in relation to incorrect conclusions should any data, information or condition be incorrect or have been concealed, withheld, misrepresented or otherwise not fully disclosed to EI.

GEOTECHNICAL ENGINEERING

Geotechnical engineering is based extensively on judgment and opinion. It is far less exact than other engineering disciplines. Geotechnical engineering reports are prepared for a specific client, for a specific project and to meet specific needs, and may not be adequate for other clients or other purposes (e.g. a report prepared for a consulting civil engineer may not be adequate for a construction contractor). The report should not be used for other than its intended purpose without seeking additional geotechnical advice. Also, unless further geotechnical advice is obtained, the report cannot be used where the nature and/or details of the proposed development are changed.

LIMITATIONS OF SITE INVESTIGATION

The investigation programme undertaken is a professional estimate of the scope of investigation required to provide a general profile of subsurface conditions. The data derived from the site investigation programme and subsequent laboratory testing are extrapolated across the site to form an inferred geological model, and an engineering opinion is rendered about overall subsurface conditions and their likely behaviour with regard to the proposed development. Despite investigation, the actual conditions at the site might differ from those inferred to exist, since no subsurface exploration program, no matter how comprehensive, can reveal all subsurface details and anomalies. The engineering logs are the subjective interpretation of subsurface conditions at a particular location and time, made by trained personnel. The actual interface between materials may be more gradual or abrupt than a report indicates.

SUBSURFACE CONDITIONS ARE TIME DEPENDENT

Subsurface conditions can be modified by changing natural forces or man-made influences. The report is based on conditions that existed at the time of subsurface exploration. Construction operations adjacent to the site, and natural events such as floods, or ground water fluctuations, may also affect subsurface conditions, and thus the continuing adequacy of a geotechnical report. EI should be kept appraised of any such events, and should be consulted to determine if any additional tests are necessary.

VERIFICATION OF SITE CONDITIONS

Where ground conditions encountered at the site differ significantly from those anticipated in the report, either due to natural variability of subsurface conditions or construction activities, it is a condition of the report that EI be notified of any variations and be provided with an opportunity to review the recommendations of this report. Recognition of change of soil and rock conditions requires experience and it is recommended that a suitably experienced geotechnical engineer be engaged to visit the site with sufficient frequency to detect if conditions have changed significantly.

REPRODUCTION OF REPORTS

This report is the subject of copyright and shall not be reproduced either totally or in part without the express permission of this Company. Where information from the accompanying report is to be included in contract documents or engineering specification for the project, the entire report should be included in order to minimize the likelihood of misinterpretation from logs.

REPORT FOR BENEFIT OF CLIENT

The report has been prepared for the benefit of the Client and no other party. EI assumes no responsibility and will not be liable to any other person or organisation for or in relation to any matter dealt with or conclusions expressed in the report, or for any loss or damage suffered by any other person or organisation arising from matters dealt with or conclusions expressed in the report (including without limitation matters arising from any negligent act or omission of EI or for any loss or damage suffered by any other party relying upon the matters dealt with or conclusions expressed in the report). Other parties should not rely upon the report or the accuracy or completeness of any conclusions and should make their own inquiries and obtain independent advice in relation to such matters.

OTHER LIMITATIONS

EI will not be liable to update or revise the report to take into account any events or emergent circumstances or fact occurring or becoming apparent after the date of the report.