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URBAN APARTMENTS PTY LTD

Geotechnical Investigation

102 Burlington Road, Homebush NSW

E25324.G03_Rev1
8 October 2021

Document Control

Report Title: Geotechnical Investigation, 102 Burlington Road, Homebush NSW

Report No: E25324.G03_Rev1

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2 Original (Saved to Digital Archives) (Z:\07 - Projects\E25324_Urban Apartments_Homebush_GI_CMP\05_Deliverables\Work in Progress\E25324.G03_Rev1 - Geotechnical Investigation Report.docx)	EI Australia Suite 6.01, 55 Miller Street, PYRMONT NSW 2009

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Revision	Details	Date	Amended By
	Original	8 October 2021	
1	Updated basement setbacks	8 October 2021	KX

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1. Introduction

1.1 Background

At the request of Urban Apartments Pty Ltd (the Client), EI Australia (EI) has carried out a Geotechnical Investigation (GI) for the proposed development at 102 Burlington Road, Homebush 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 P19713.1, dated 18 August 2021, and with the Client's signed authorisation to proceed, dated 25 August 2021.

1.2 Proposed Development

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

- Draft Architectural Drawings prepared by Litera Trotta – Project No. 007, Drawing No. DA101, DA201, DA202, DA203, DA301, DA302 and DA321, dated 9 August 2021; and
- Site survey plan prepared by SDG – Referenced 8294, Issue D, dated 13 July 2021. The datum in the survey plan is in Australian Height Datum (AHD), hence all Reduced Levels (RL) mentioned in this report are henceforth in AHD.

Based on the provided documents, EI understands that the project site is home to heritage listed "Camden Lodge" which was mostly destroyed in a fire about 10 years ago. The proposed development involves partial demolition of the existing heritage building, restoration of existing heritage building for a single-storey residential building at the front portion of the site, and an extension of the existing heritage building including construction of a two-storey residential development overlying a single-level basement at rear portion of the site. The basement will extend underneath the existing building and is proposed to have a Finished Floor Level (FFL) of RL 21.21m. A Bulk Excavation Level (BEL) of RL 20.90m is assumed, which includes allowance for the construction of the basement slab. To achieve the BEL, excavation depths up to 3.50m Below Existing Ground Level (BEGL) have been estimated. The proposed basement located at the rear of the existing heritage building has a setback of about 20.30m from the eastern site boundary, 4.90m from the western site boundary, 9.00m from the southern site boundary, and the northern boundary of the basement extends below the southern end of the existing heritage building. Locally deeper excavations may be required for footings, lift overrun pits, crane pads, and service trenches.

1.3 Objectives

The objective of the GI was to assess site surface and subsurface conditions at six test 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 (BH1M and BH2) by a track-mounted drill rig using solid flight augers equipped with a 'Tungsten-Carbide' (T-C) bit. BH1M and BH2 were auger drilled to depths of 3.12m (RL 20.93m) and 3.18m (RL 20.72m) 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 BH1M and BH2 using NMLC diamond coring techniques to a common termination depth of 8.00m (RL 16.05m and 15.90m) BEGL. The rock core photographs are presented in **Appendix A**;
- Borehole BH1M was converted into a groundwater monitoring well with a depth of 4.50m BEGL (RL 19.55m) to allow for long-term groundwater monitoring.
- Boreholes BH2 was backfilled with drilling spoils upon completion;
- Excavation of three test pits (TP1, TP2 and TP3) to expose the footings at existing site structures, the test pits were excavated to depths of 0.30m (RL 23.58m), 0.35m (RL 23.63m) and 0.15m (RL 22.10m) BEGL, respectively;

- Three Dynamic Cone Penetrometer (DCP) tests were carried out, DCP1 and DCP2 at test pit locations and DCP3 at a separate location near the test pit, to refusal depths of 0.56m (RL 23.32m), 0.7m (RL 23.28m) and 1.26m (RL 22.44m) BEGL, respectively;
- 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	102 Burlington Road, Homebush NSW
Lot and Deposited Plan (DP) Identification	Lot 13, 14 and 15 in DP 400
Brief Site Description	At the time of our investigation, the site was occupied by a single-storey brick rendered residential building located at Lot 14 and 15. The building structure was damaged previously by fire and the collapsed roof and floors were removed. Internal brick piers were exposed and vegetation was observed within the building. A brick garage and wooden shed were observed adjacent to the south-west corner of the site. The remaining area of the site was unoccupied.
Site Area	The site area is approximately 2769.6m^2 (based on the provided survey plan referenced above).

Plate 1: Aerial photograph of the site (source: SIX Maps, accessed 24/9/21)

2.2 Local Land Use

The site is situated within an area of residential 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 Burlington Road shall be adopted as the northern site boundary.

Table 2-2 Summary of Local Land Use

Direction Relative to Site	Land Use Description
North	Burlington Road, a two-lane, asphalt paved road. Beyond this are one to two-storey brick residential dwellings with no basements.
East	Property at No. 96 Burlington Road, a two-storey brick rendered residential building with concrete driveway. The main house has an offset of about 13m from the eastern site boundary and appeared to be in good condition based on inspection of the external walls. No. 96 Burlington Road is on a similar elevation to the site. No basements were observed on this property.
South	Properties at No. 85, No. 87 and No. 89 Abbotsford Road, consist of single-storey brick rendered residential buildings with brick paved driveways. The closest building has an offset of about 8.50m from the southern site boundary and appeared to be in good condition based on inspection of the external walls. Nos. 85, 87 and 89 Abbotsford Road are on a similar elevation to the site. No basements were observed on these properties.
West	Property at No. 104-106 Burlington Road, contains a two-storey brick rendered residential building with brick paved driveway. The main building has an offset of about 4.60m from the western site boundary and appeared to be in good condition based on inspection of the external walls. A brick garage is located at the southern portion of the property and has an offset of about 0.50m from the western site boundary. No. 104-106 Burlington Road is on a similar elevation to the site. No basements were observed on this property.

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 south side of the road within gently (0° to 5°), south dipping topography with site levels varying from RL 23.38m at the north-western site corner to RL 24.37m at the south-eastern site corner.
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.

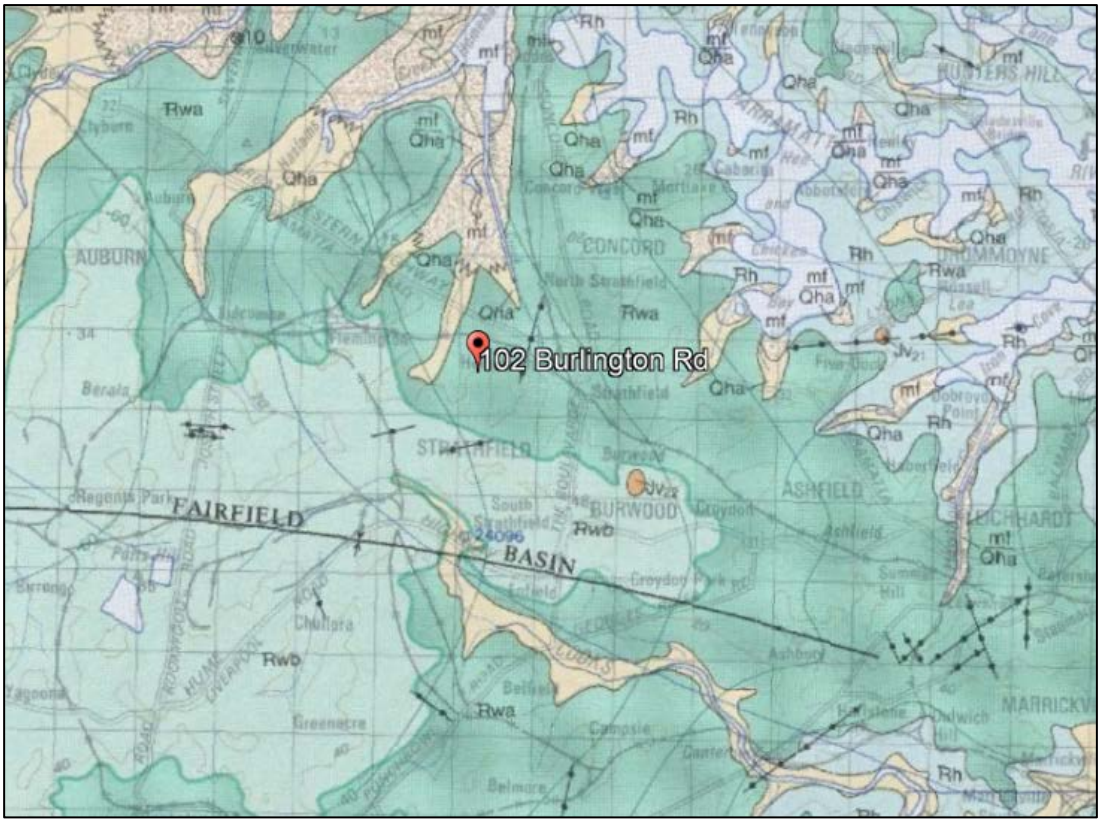


Plate 2: Excerpt of geological map showing location of site.

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**	0.00	23.90 to 24.05	0.20 to 0.30	Low plasticity silty clay fill with traces of rootlets. Fill was assessed, based on our observations during drilling to be poorly compacted;
2	**Residual Soil**	0.20 to 0.30	23.60 to 23.85	2.88 to 2.92	High plasticity, stiff to hard silty clay with trace ironstone gravels. SPT values range from 10 to refusal indicated by hammer bounce;
3	**Low Strength Shale**	3.12 to 3.18	20.72 to 20.93	2.00 to 2.88	Distinctly weathered, low strength shale;
4	**Medium Strength Shale**	5.18 to 60	18.05 to 18.72	- [3]	Fresh, medium strength shale. The shale generally contained moderately spaced, sub-horizontal bedding partings, with minimal sub-vertical joints, and fractured / extremely weathered zones.

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 BH1M 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.
- Soil aggressivity (pH, chloride and sulfate content and electrical conductivity).

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

Table 3-2 Summary of Soil Laboratory Test Results

Test/ Sample ID		BH1M 0.5-0.95m	BH2 0.5-0.95m	BH2 3.0-3.18m
Unit		2	2	2
Material Description [1]		Silty CLAY	Silty CLAY	Silty CLAY
Aggressivity	Chloride Cl (ppm)	18	-	74
	Sulfate SO_4 (ppm)	300	-	62
	pH	4.1	-	5.2
	Electrical Conductivity (µS/cm)	180	-	96
	Moisture Content (%)	22.2	25.6	11.5
Atterberg Limits	Liquid Limit (%)	-	73	-
	Plastic Limit (%)	-	26	-
	Plasticity Index (%)	-	47	-
	Linear Shrinkage (%)	-	18	-

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 very high plasticity and of high 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:

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

16 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 17 MPa.

3.4 Existing footings

The footings supporting the existing building structures were exposed at three locations. The details of the exposed footings are provided below.

3.4.1 TP1/DCP1 Location

TP1/DCP1 was located adjacent to the south-western external wall of the main dwelling where the brick wall is resting on a concrete strip footing founded at a depth of 0.20m BEGL on Unit 2 Residual Clay.

Figure 3-1 Footing exposed at TP1/DCP1 location, founded at 0.20m on stiff residual clay

3.4.2 TP2/DCP2 Location

TP2/DCP2 was located adjacent to the south-eastern external wall of the main dwelling where the brick wall is resting on a concrete strip footing founded at a depth of 0.30m BEGL on Unit 1 Fill.

Figure 3-2 Footing exposed at TP2/DCP2 location, founded at 0.30m on fill

3.4.3 TP3/DCP3 Location

TP3 was located adjacent to the north-eastern external wall of the main dwelling where the brick wall is resting on a concrete strip footing founded at a depth of 0.10m BEGL on Unit 1 Fill. DCP3 was carried out at a separate location near TP3 location outside the brick wall.

Figure 3-3 Footing exposed at TP3/DCP3 location, founded at 0.10m on fill

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;
- Reactivity of the underlying natural clays;
- Existing footings of site structures and 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 Demolition Considerations

Care should be taken during demolition, particularly the concrete pavement, to avoid damaging existing site structures which will be retained, neighbouring structures and infrastructures. Demolition of concrete slabs, pavement and floor slabs may require breaking into smaller size prior to disposal offsite. We recommend that saw cut slots be provided near adjoining buildings to reduce the risk of vibrations being transferred to nearby structures and infrastructures. If possible, the concrete slabs should be removed using hydraulic equipment rather than impact hammers.

4.4 Existing Footings

Test pits and DCPs show that the existing heritage building is founded on stiff clay with concrete strip footings of depth range from 0.10m to 0.30m BEGL. The stiff clay at founding level of concrete strip footings has been assessed to be suitable for an allowable bearing capacity of 100kPa. Since no addition is proposed on top of the existing heritage building, underpinning of existing footings is unlikely to be required.

4.5 Site Preparation

Following removal of all vegetation and trees (including their root balls), demolition of the existing sheds, slabs and pavements, all grass, topsoil, root affected soils and any deleterious fill or contaminated soil should be stripped. Based on the results of the investigation, topsoil/root affected soil should be stripped to a nominal depth. We note that it is difficult to accurately assess the depth of topsoil and root affected soils in a 100mm diameter borehole. If considered

to be an important contractual issue, we recommend that a number of shallow test pits be excavated across the site to more accurately confirm the topsoil/root affected soil stripping depth or alternatively a geotechnical inspection could be carried out after initial stripping to confirm the depth. Stripped topsoil and root affected soils should be stockpiled separately as they are considered unsuitable for reuse as engineered fill.

All existing fill will need to be stripped down to the surface of the underlying natural soils and stockpiled for reuse as engineered fill, if conform to the fill specification provided in section below.

4.6 Site Classification

The investigation results have indicated variable subsurface conditions, as well as variable soil reactivity across the site. The final site classifications will also be dependent on the following factors:

- The nature, thickness and age of proposed (site won and imported) fills;
- Level of earthworks control;
- The depth of proposed cuts;
- The depth of bedrock and groundwater if located within 1.5m depth below design surface level;
- The proximity of the proposed trees, their configuration in relation to the proposed houses, and their mature heights;
- The presence of existing mature trees, and their configuration in relation to the proposed houses;
- The control and maintenance of drainage; and
- The difference in surface levels for a particular lot between the ‘as sold’ condition and following and further preparatory earthworks (i.e. further less controlled cut and fill earthworks).

No details on the existing fill (i.e. placement method, compaction specification, density test records, etc.) have been provided to us. Notwithstanding, based on the results of our investigation, the material is not considered to be a “structural fill” (i.e. controlled fill or engineered fill), as defined in Clause 1.2.13 of AS3798-2007 ‘Guidelines on Earthworks for Commercial and Residential Developments’. Accordingly, we consider this fill to be ‘uncontrolled’ and unsuitable as a bearing stratum under new footings and of ‘moderate risk’ (poor performance) under new pavements. The site under the existing condition is considered to be Class ‘H1’ in accordance with AS2870.

Where topsoil/fill is striped and/or replaced with engineered/controlled fill and/or natural silty clay exposed, then the site can be reclassified as Class H1. We note that abnormal moisture conditions could also exist after stripping of existing pavements, structures, and trees and vegetation resulting in a more severe Class H2. Reference should also be made to AS2870 for design, construction, performance criteria and maintenance precautions on Class H2 sites.

4.7 Excavation Methodology

4.7.1 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 understands that the proposed development will require a BEL of RL 20.90m for the basement, or an excavation depth up to 3.50m 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 (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 required for the excavation of the high strength bedrock, further advice should be sought from EI regarding vibration mitigation and monitoring. Reference should be made to **Appendix C** for a guide to acceptable limits of transmitted vibrations.

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.

4.7.2 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 a depth of 1.5m, and every 1.5m interval thereafter.
- 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.

4.8 Groundwater Considerations

Although no groundwater or significant seepage was observed during or after auger drilling of the boreholes, we note that the groundwater levels may not have become evident or stabilised in the augered boreholes within the limited observation period.

Due to the low permeability of the clayey soil and 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.

4.9 Excavation Retention

4.9.1 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 proposed basement located at the rear of the existing heritage building has a setback of about 20.30m from the eastern site boundary, 4.90m from the western site boundary, 9.00m from the southern site boundary, and the northern boundary of the basement extends below the southern end of the existing heritage building which will be restored.

Based on the above, the encountered subsurface conditions, the depth of excavation, temporary batters of no steeper than a safe angle of 1 Vertical (V) to 1 Horizontal (H) may be

feasible where space allows for the fill and residual soil profile. The above temporary batters should remain stable provided that all surcharge loads, including construction loads, are kept at a distance of at least 2h (where 'h' is the height of the batter in metres) from the crest of the batter. If steeper batters are to be used, then these must be supported by shotcrete and soil nail system designed by a suitable structural or geotechnical engineer. The stability of these batters can be assessed using computer slope stability analysis software such as Slope/W. we can complete such analysis, if commissioned to do so.

Where batters are used, the space between the batters and the permanent retaining walls will need to be carefully backfilled to reduce future settlement of the backfill. Only light compaction equipment should be used for compaction behind retaining walls so that excessive lateral pressures are not placed on the walls. This will require the backfill to be placed in thin layers, say 100mm loose thickness, appropriate to the compaction equipment being used. The compaction specification for the backfill will depend on whether paving or structures are to be supported on the fill. If the fill is to support paved areas it should be compacted to a density of at least 98% of Standard Maximum Dry Density (SMDD) for granular fill materials, but if it is only to support landscaped areas of lower compaction specification, say 95% of SMDD, may be appropriate, provided the risk of future settlement and maintenance can be accepted. An alternative for backfill would also be to use a uniform granular material, wrapped in a geofabric.

However, where space for temporary batters is not available along the proposed northern basement wall and adjacent to existing building, a suitable retention system will be required for the support of the entire depth of the excavation. For this site, we consider that an anchored and/or propped soldier pile wall with mass concrete/shotcrete in between the piles installed to below BEL to be the most suitable. Mass concrete must be installed progressively as excavation proceeds.

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.

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.

4.9.2 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**;
- Cantilevered walls, where the tops of which are restrained by the floor slabs of the permanent structure or which support movement sensitive elements, should be designed using a triangular lateral earth pressure distribution and an 'at rest' earth pressure coefficient, K_o, as shown in **Table 4-1** below;

- 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, K_o;
- 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;
- 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; and
- For piles embedded into Unit 3 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.

Table 4-1 Geotechnical Design Parameters

Material [1]		Unit 1 Fill	Unit 2 Residual Soil	Unit 3 Low Strength Shale	Unit 4 Medium Strength Shale
RL of Top of Unit (m AHD) [2]		23.90 to 24.05	23.60 to 23.85	20.72 to 20.93	18.05 to 18.72
Bulk Unit Weight (kN/m^3)		18	20	23	24
Friction Angle, ϕ' (°)		25	25	35	40
Earth Pressure Coefficients	At rest, K_o [3]	0.58	0.58	0.43	0.36
	Active, K_a [3]	0.41	0.41	0.27	-
	Passive, K_p [3]	2.46	2.46	3.69	4.60
Allowable Bearing Pressure (kPa) [5]		-	-	1000	2500
Allowable Shaft Adhesion (kPa) [4, 5]	in Compression	-	-	100	250
	in Uplift	-	-	50	125
Allowable Toe Resistance (kPa)		-	-	-	250
Allowable Bond Stress (kPa)		-	-	50	250
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).

4.10 Foundations

Following bulk excavation to RL 20.90m, we expect Unit 3 material 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 1000kPa, 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 Unit 4 medium strength shale bedrock suitable for an allowable bearing capacity of 2,500kPa.

For piles founded into weathered shale bedrock (Unit 3), these must be embedded a minimum of 0.5m into shale, and can be designed for a maximum allowable bearing pressure of 1000kPa. 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.

The aggressivity of natural soils and groundwater (if encountered) should be taken into consideration in the design.

4.11 Basement Floor Slab

Following bulk excavations for the proposed basement, shale bedrock (Unit 3) 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.

4.12 Subgrade Preparation and Engineered Fill

4.12.1 Subgrade Preparation

Earthworks recommendations provided in this report should be complemented by reference to AS3798.

1 Fill should be fully excavated down to surface of the residual soils, and stockpiled separately since these materials are not suitable for re-use as engineered fill. Such excavation may need to be carried out with the excavation sides battered at an angle of no steeper than 1 Vertical to 1 Horizontal. The new fill must be ‘keyed-in’ the sides of these batters.

2 The exposed subgrade at the base of the excavation should be proof rolled with a smooth drum roller (say 12 tonne) used in static or non-vibratory mode of operation. Caution is required when proof rolling near existing infrastructures and utilities (where present). The

purpose of the proof rolling is to detect any soft or heaving areas, and to allow for some further improvement in strength or compaction.

3 The final pass should be undertaken in the presence of an experienced geotechnician or geotechnical engineer, to detect any unstable or soft subgrade areas, and to allow for some further improvement in strength/compaction.

4 If dry conditions prevail at the time of construction then any exposed residual clay subgrade may become desiccated or have shrinkage cracks prior to pouring any concrete slabs. If this occurs, the subgrade must be watered and rolled until the cracks disappear.

5 Unstable subgrade detected during proof rolling should be locally excavated down to a sound base and replaced with engineered fill or further advice should be sought. Any fill placed to raise site levels should also be engineered fill, as per the specifications below.

If suspended floor slabs and pavement are designed, then it would be unnecessary to complete any particular subgrade preparation other than stripping of root affected soils from the footprint of the proposed building structures and replaced with surface levelling compacted fill for the floor slab formwork.

4.12.2 Engineered Fill Specifications

Any fill used to backfill unstable subgrade areas, raise surface levels or backfill service trenches should be engineered fill. Materials preferred for use as engineered fill are well-graded granular materials, such as ripped or crushed sandstone, free of deleterious substances and having a maximum particle size not exceeding 75 mm. such fill should be compacted in layers not greater than 200 mm loose thickness, to a minimum density of 98% of SMDD.

The existing clayey soils are of very high plasticity and would require laboratory testing to confirm plasticity and may require mixing with granular soils for using it as engineered fill, provided unsuitable (‘over-wet’ and ‘over-sized’) material and any deleterious material is removed.

Density tests should be regularly carried out on the fill to confirm the above specifications are achieved. The frequency of density testing should be at least one test per layer per material type per 2500 m^2 or 1 test per 500m^3 distributed reasonably evenly throughout full depth and area or 3 tests per lot, whichever requires the most tests. We recommend that at least Level 2 control of fill compaction, as defined in AS3798-2007, be adhered to on this Site. Preferably, the geotechnical testing authority (GTA) should be engaged directly on behalf of the client and not by the earthworks subcontractor.

We recommend that the engineered fill layers extend a horizontal distance of at least 1m beyond the design geometry. The roller must extend over the edge of each placed layer in order to seal the batter surface. On completion of filling, the excess under-compacted edge fill should be trimmed back to the design geometry.

The ‘tying in’ of engineered fill to temporary cut batter slopes can be achieved by locally benching the cut slopes in no greater than 0.4m high steps. This can be carried out progressively as the height of engineered fill increases.

For backfilling confined excavations such as service trenches, a similar compaction to engineered fill should be adhered to, but if light compaction equipment is used then the layer thickness should be limited to 100mm loose thickness.

During construction of the fill, platform runoff should be enhanced by providing suitable falls to reduce ponding of water on the surface of the fill. Ponding of water may lead to softening of the fill and subsequent delays in the earthworks program. A poorly drained subgrade may become un-trafficable when wet. We recommend that if soil softening occurs, the subgrade be over-excavated to below the affected soil, and then replaced with engineered fill as specified above..

4.12.3 Subgrade Preparation for Raft Slab

If raft slab foundations are adopted, it is recommended that they be founded on stiff residual clay or better.

Earthworks recommendations provided in this report should be complemented by reference to AS3798.

Our recommendations regarding subgrade perpetration are as follows:

- The subgrade below the basement slab or footing will need to be prepared prior to construction of the slab or footing, but the extent of the preparation, inspection and testing will depend on the footing systems adopted. A more rigorous control will be required where a raft slab is adopted.
- Following excavation, it is anticipated that predominantly residual clay will be exposed. The exposed subgrade should be proof rolled with at least a 12 tonne dead weight smooth drum vibratory roller, with the final pass of the proof rolling inspected by a geotechnical engineer or experienced earthworks foreman to detect any weak subgrade areas.
- Any weak areas detected during proof rolling should be locally excavated to a sound base and the excavated materials replaced with engineered fill. During proof rolling and any fill compaction care must be taken due to the risk of damage to the adjoining buildings from vibrations generated by such work. All vibrations should be monitored and if they are considered to be excessive they should be reduced or ceased.
- Engineered fill should preferably comprise well graded granular materials, such as ripped rock, processed demolition spoil or crushed sandstone, free of deleterious substances and having a maximum particle size not exceeding 75mm. The excavated clay may be reused as engineered fill provided it is free of deleterious material and particles greater than 75mm in size. Such fill should be compacted in horizontal layers of not greater than 200mm loose thickness, to achieve a minimum Density Index of 70%. A method specification, comprising placement of granular materials in 150mm loose layers and then compacting by not less than eight passes by a minimum 2-tonne dead weight smooth drum roller, may be adopted.
- For backfilling confined excavations such as service trenches, a similar compaction to engineered fill should be adhered to, but if light compaction equipment is used then the layer thickness should be limited to 100mm loose thickness.
- Where a raft slab is adopted, the geotechnical engineer would also need to carry out a series of Dynamic Cone Penetrometer (DCP) tests to assess the consistency of the clay. We expect that a capping layer of well graded crushed rock or recycled concrete (maximum particle size limited to 40mm) will be required to achieve adequate compaction of the upper clay. This granular layer will be required below the entire raft slab and would be of about 150mm thick.
- The performance of raft (including piled raft) slabs are also dependent on the whole of the design and construction team being familiar with the sensitivity of the situation. It is essential that any services which have to be placed in the subgrade are carefully positioned and an appropriate construction schedule/sequence is provided to the geotechnical engineer for approval at the planning stage.
- Disturbance of the subgrade must be minimised and kept outside the zone of influence of column or wall loads. A documented Inspection and Test Plan (ITP) should be prepared prior to construction with appropriate “hold” points in the Quality System.

5. Further Geotechnical Inputs

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

- Stability assessment of temporary batters using computer modelling, if required;
- Long term groundwater monitoring and seepage modelling;
- Dilapidation surveys;
- Classification of all excavated material transported off site;
- Witnessing installation of support measures (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 Patrick Elias and Urban Apartments 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 Patrick Elias and Urban Apartments 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

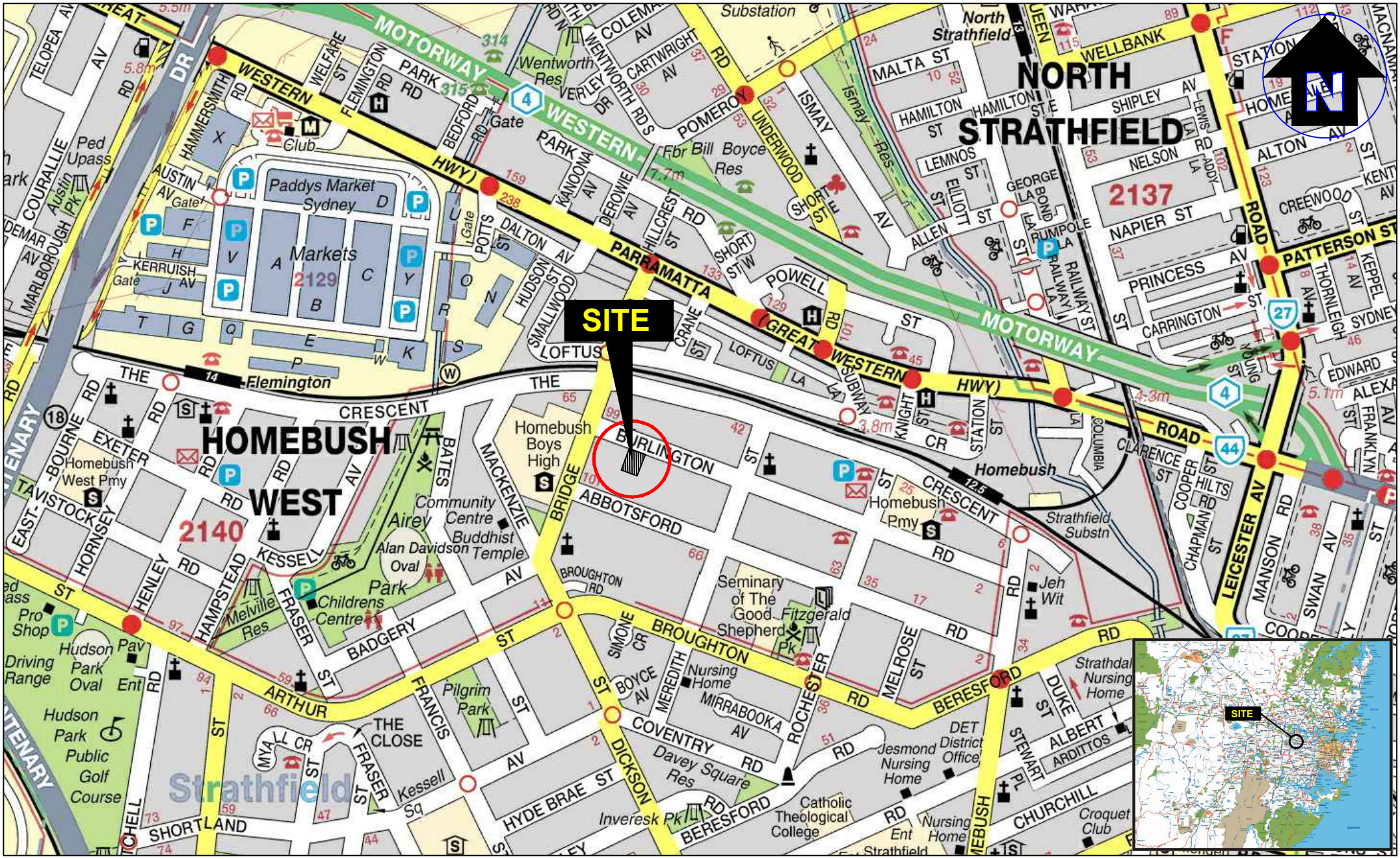
Figure 1 Site Locality Plan

Figure 2 Borehole, Test Pit and DCP Location Plan

Figure 3 Test Pit 1 (TP1) Cross-section

Figure 4 Test Pit 2 (TP2) Cross-section

Figure 5 Test Pit 3 (TP3) Cross-section



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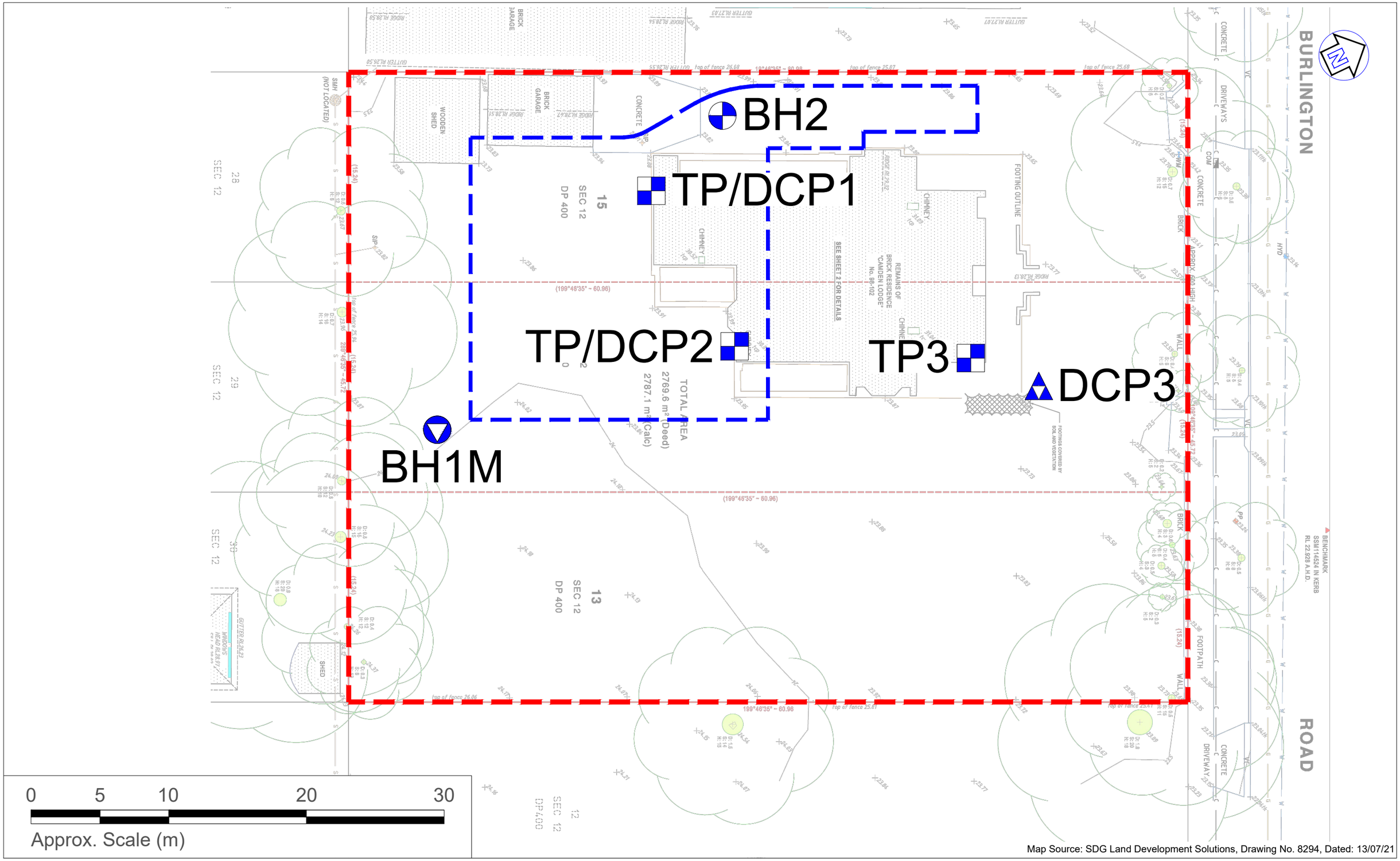
Date:	8-10-21
Scale:	Not To Scale

Urban Apartments Pty Ltd
Geotechnical Investigation
102 Burlington Road, Homebush NSW

Site Locality Plan

Figure: 1

Project: E25324.G03_Rev1



Map Source: SDG Land Development Solutions, Drawing No. 8294, Dated: 13/07/21

LEGEND **(All locations are approximate)**

- Site boundary
- Proposed basement boundary
- Borehole location
- Monitoring well location
- DCP location
- Test pit and DCP location

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Sampling Location Plan

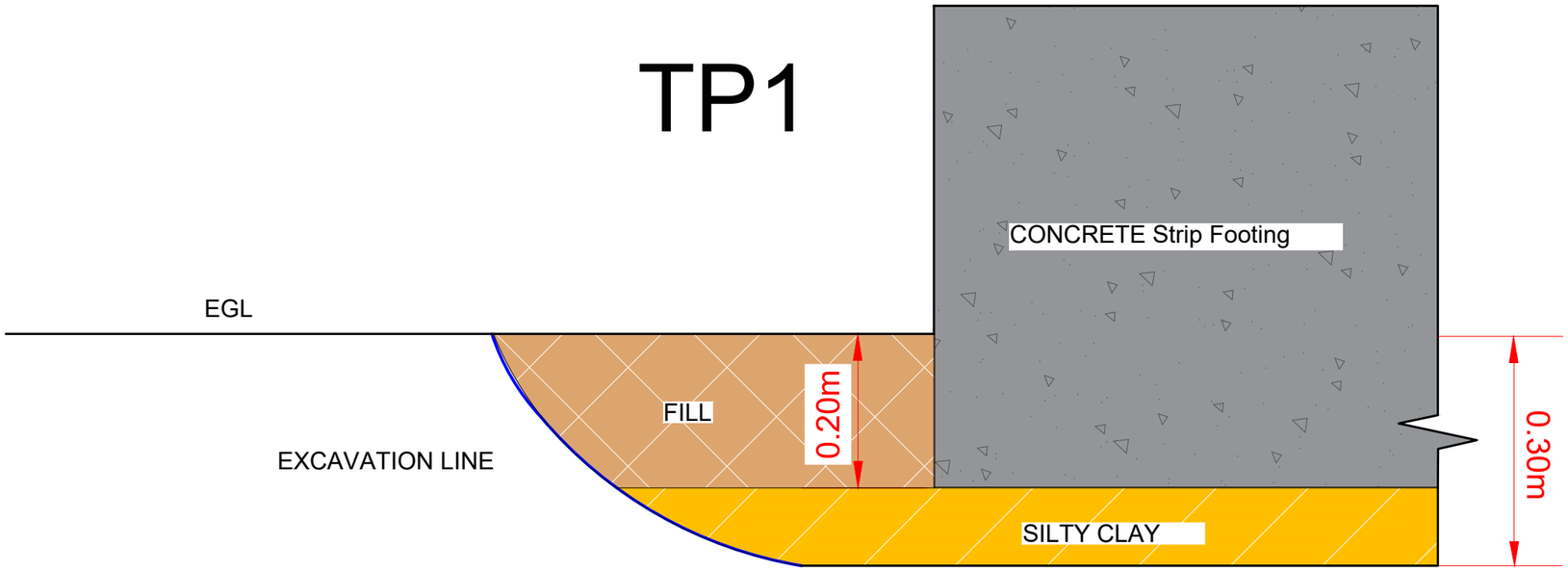
Figure: 2

Project: E25324.G03_Rev1

South

North

TP1



LEGEND

- CONCRETE Wall
- FILL
- SILTY CLAY

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Test Pit Cross-Section (TP1)

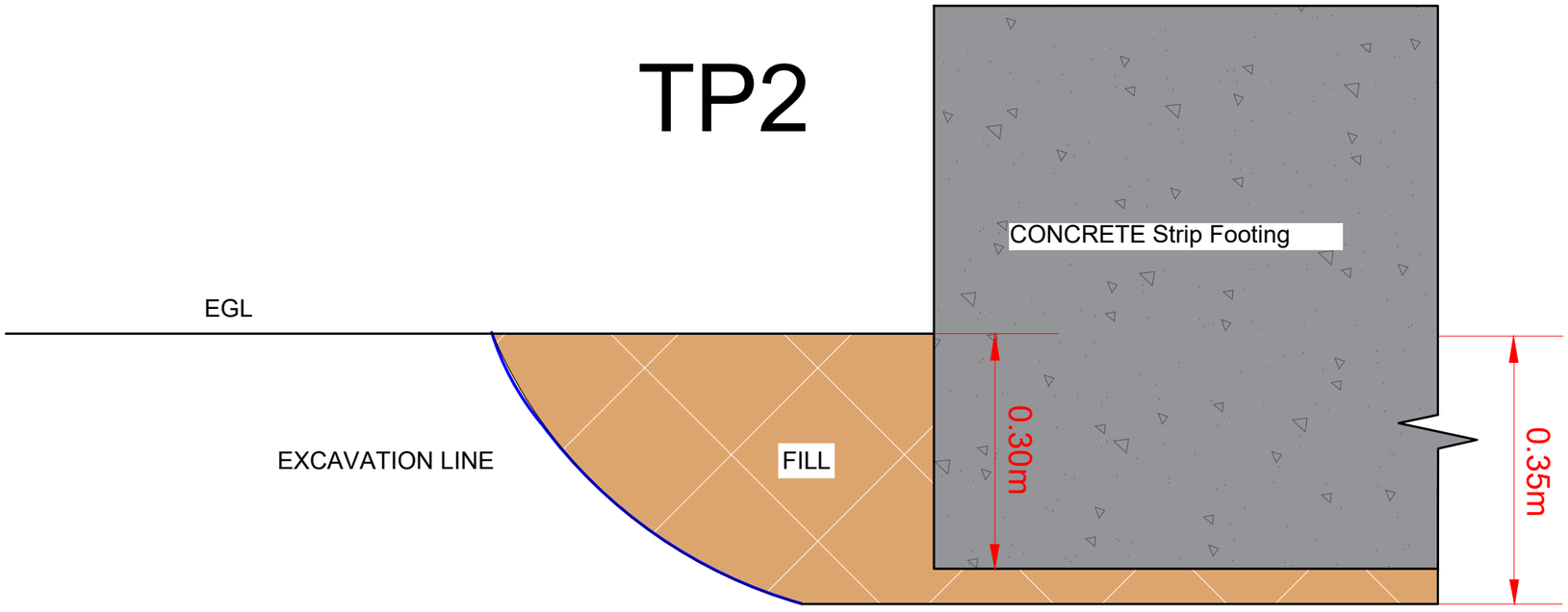
Figure: 3

Project: E25324.G03_Rev1

South

North

TP2



LEGEND

- CONCRETE Wall
- FILL

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Urban Apartments Pty Ltd
Geotechnical Investigation
102 Burlington Road, Homebush, NSW
Test Pit Cross-Section (TP1)

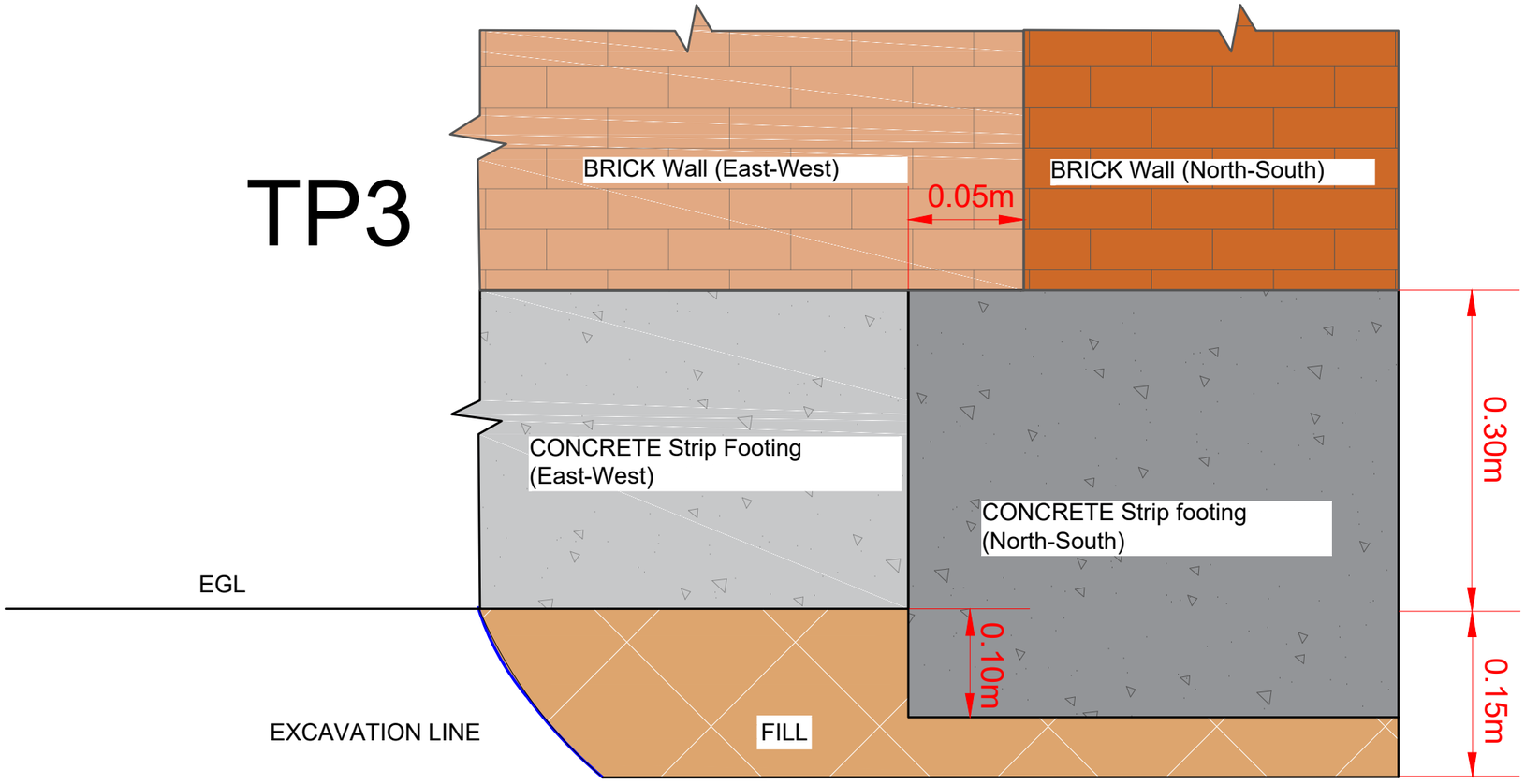
Figure: 3

Project: E25324.G03_Rev1

West

East

TP3



LEGEND

- CONCRETE Wall
- BRICK Wall
- FILL

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Urban Apartments Pty Ltd
Geotechnical Investigation
102 Burlington Road, Homebush, NSW
Test Pit Cross-Section (TP1)

Figure: 3

Project: E25324.G03_Rev1

Appendix A – Borehole, DCP Logs, and Explanatory Notes

BOREHOLE LOG

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BH NO. BH1M

Project	Residential Development	**Sheet**	1 of 2
Location	102 Burlington Road, Homebush NSW	**Date Started**	16/09/2021
Position	Refer to Figure 2	**Date Completed**	16/09/2021
Job No.	E25324.G03	**Logged By** KX	**Date** 16/09/2021
Client	Urban Apartments Pty Ltd	**Reviewed By** SR	**Date** 08/10/2021

Drilling Contactor	Geosense	**Surface RL**	≈24.05 m AHD
Drill Rig	Comacchio 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
AD/TC	-	GWNE	0	24.05 *0.20*				-	FILL: Silty CLAY; low plasticity, dark brown, trace rootlets.	M<PL	-	FILL
				23.85				CH	Silty CLAY: high plasticity, orange brown.			RESIDUAL SOIL
			1	*1.00* 23.05	SPT 0.50-0.95 m 3,5,6 N=11				From 1.0 m, changes to pale brown and pale grey, trace sub-angular ironstone gravels.		St	
				1.50 22.55	SPT 1.50-1.95 m 13,19,27 N=46				From 1.5 m, grading into extremely weathered material.	M<PL	H	
			2									
			3	*3.12*	SPT 3.00-3.12 m 9/ 120 mm HB				Continued as Cored Borehole			
			4									
			5									
			6									
			7									
			8									
			9									
			10									

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

EIA 2.00.3 LIB.GLB Log EIA NON-CORED BOREHOLE 1 E25324.G03.GPJ <<DrawingFile>> 08/10/2021 09:23 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: EIA 2.00.3 2017-11-21 Prj: EIA 2.00.1 2017-09-26

CORED BOREHOLE LOG

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BH NO. BH1M

Project	Residential Development	**Sheet**	2 OF 2
Location	102 Burlington Road, Homebush NSW	**Date Started**	16/09/2021
Position	Refer to Figure 2	**Date Completed**	16/09/2021
Job No.	E25324.G03	**Logged By** KX	**Date** 16/09/2021
Client	Urban Apartments Pty Ltd	**Reviewed By** SR	**Date** 08/10/2021

Drilling Contactor	Geosense	**Surface RL**	≈24.05 m AHD
Drill Rig	Comacchio 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)
				0							
				1							
				2							
				3	*3.12*		Continuation from non-cored borehole				
NMLC	80-90% RETURN	100	75	4	20.93		SHALE: grey and brown, with dark grey laminations, thinly bedded.	DW		3.41-3.47: CS, 60mm	
				5						4.76: JT, 40 - 50°, CN, PL, RF, OP	
		103	94	6	*6.00* 18.05		From 6.0 m, changes to dark grey, with pale grey laminations.	FR		5.58: JT, 30 - 40°, CN, PL, RF, OP	
		86	57	7							
				8	*8.00* 16.05		Hole Terminated at 8.00 m Target Depth Reached.				
				9							
				10							

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

EIA 2.00.3 LIB.GLB Log EIA CORED BOREHOLE 1 E25324.G03.GPJ <<DrawingFile>> 08/10/2021 09:24 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: EIA 2.00.3 2017-11-21 Prj: EIA 2.00.1 2017-09-26

MONITORING WELL LOG

MW NO. BH1M

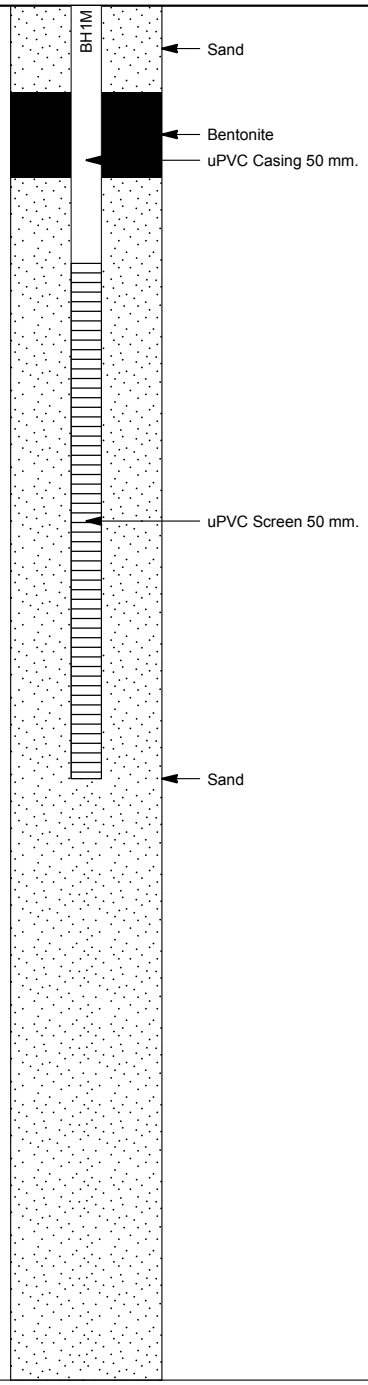
Project	Residential Development	**Sheet**	1 of 2
Location	102 Burlington Road, Homebush NSW	**Date Started**	16/09/2021
Position	Refer to Figure 2	**Date Completed**	16/09/2021
Job No.	E25324.G03	**Logged By** KX	**Date** 16/09/2021
Client	Urban Apartments Pty Ltd	**Reviewed By** SR	**Date** 08/10/2021

Drilling Contactor	Geosense	**Surface RL**	≈24.05 m AHD
Drill Rig	Comacchio 205	**Inclination**	-90°

METHOD	WATER	DEPTH (m)	RL (m AHD)	GRAPHIC LOG	SOIL/ROCK MATERIAL DESCRIPTION
AD/TC	GWNE	0	24		FILL: Silty CLAY; low plasticity, dark brown, trace rootlets.
					Silty CLAY: high plasticity, orange brown.
		1	23		From 1.0 m, changes to pale brown and pale grey, trace sub-angular ironstone gravels.
					From 1.5 m, grading into extremely weathered material.
		2	22		
		3	21		SHALE: grey and brown, with dark grey laminations, thinly bedded.
NMLC	80-90% RETURN	4	20		
		5	19		
		6	18		From 6.0 m, changes to dark grey, with pale grey laminations.
		7	17		
		8	16		Hole Terminated at 8.00 m Target Depth Reached.

PIEZOMETER CONSTRUCTION DETAILS

ID	Type	Stick Up & RL	Tip Depth & RL	Installation Date	Static Water Level
BH1M	Standpipe	1.50 m 22.55 m	4.50 m 19.55 m		



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

EIA 2.00.3 LIB.GLB Log EIA PIEZOMETER INSTALLATION LOG E25324.G03.GPJ <<DrawingFile>> 08/10/2021 09:25 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: EIA 2.00.3 2017-11-21 Prj: EIA 2.00.1 2017-09-26

CORE PHOTOGRAPH OF BOREHOLE: BH1M

Project	Proposed Residential Development			**Depth Range**	3.12m to 8.0m BEGL		
Location	102 Burlington Road, Homebush NSW			**Contractor**	Geosense Drilling Engineers Pty Ltd		
Position	See Figure 2	**Surface RL**	≈ 24.05m	**Drill Rig**	Comacchio 205		
Job No.	E25324.G03	**Inclination**	-90°	**Logged**	KX	**Date**	16 / 9 / 2021
Client	Urban Apartments Pty Ltd	**Box**	1 of 1	**Checked**	SR	**Date**	8 / 10 / 2021

BOREHOLE LOG

BH NO. BH2

Project	Residential Development	**Sheet**	1 of 2
Location	102 Burlington Road, Homebush NSW	**Date Started**	16/09/2021
Position	Refer to Figure 2	**Date Completed**	16/09/2021
Job No.	E25324.G03	**Logged By** EM	**Date** 16/09/2021
Client	Urban Apartments Pty Ltd	**Reviewed By** SR	**Date** 08/10/2021

Drilling Contactor	Geosense	**Surface RL**	≈23.90 m AHD
Drill Rig	Comacchio 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
AD/TC	-	GWNE	0	23.90				-	FILL: Silty CLAY; low plasticity, dark brown.	M<PL	-	FILL
				0.30								
				23.60				CH	Silty CLAY: high plasticity, orange brown.			RESIDUAL SOIL
					SPT 0.50-0.95 m							
					3,4,6							
				0.80	N=10							
				23.10					From 0.8 m, changes to pale brown to pale grey.		St	
			1									
				1.60	SPT 1.50-1.67 m							
				22.30	21,4/ 20 mm HB				From 1.6 m, grading to extremely weathered material.	M<PL		
			2								H	
				3.18	SPT 3.00-3.18 m							
					9,6/ 30 mm HB							
									Continued as Cored Borehole			
			3									
			4									
			5									
			6									
			7									
			8									
			9									
			10									

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

EIA 2.00.3 LIB.GLB Log EIA NON-CORED BOREHOLE 1 E25324.G03.GPJ <<DrawingFile>> 08/10/2021 09:23 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: EIA 2.00.3 2017-11-21 Prj: EIA 2.00.1 2017-09-26

CORED BOREHOLE LOG

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BH NO. BH2

Project	Residential Development	**Sheet**	2 OF 2
Location	102 Burlington Road, Homebush NSW	**Date Started**	16/09/2021
Position	Refer to Figure 2	**Date Completed**	16/09/2021
Job No.	E25324.G03	**Logged By** EM	**Date** 16/09/2021
Client	Urban Apartments Pty Ltd	**Reviewed By** SR	**Date** 08/10/2021

Drilling Contactor	Geosense	**Surface RL**	≈23.90 m AHD
Drill Rig	Comacchio 205	**Inclination**	-90°

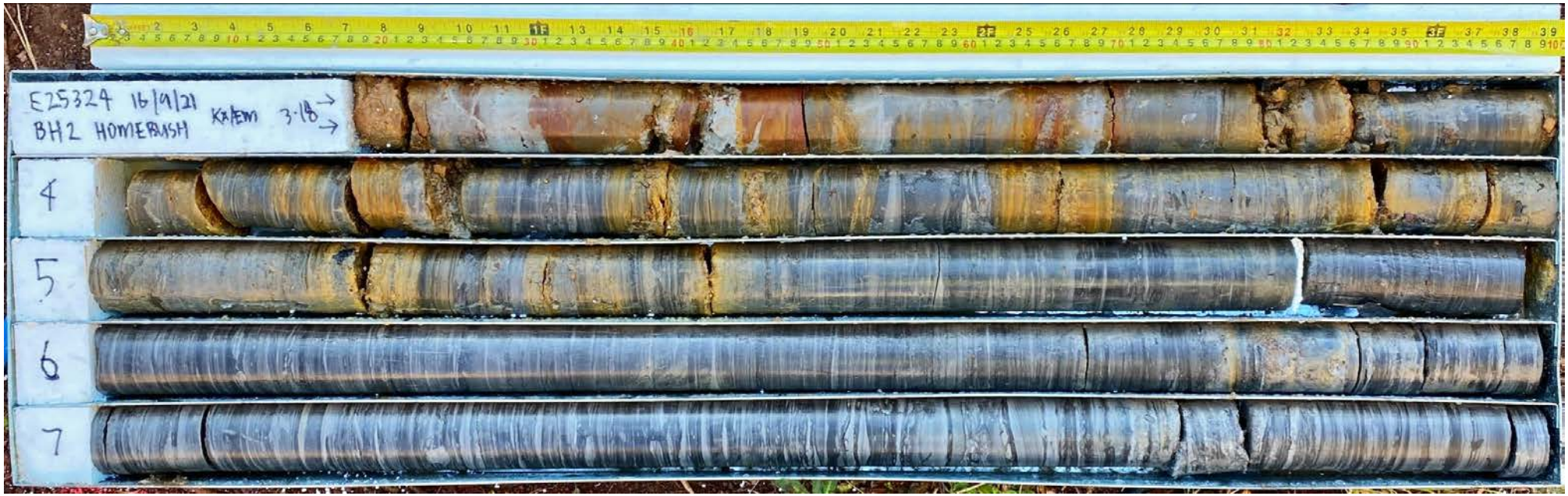
Drilling						Field Material Description				Defect Information	
METHOD	WATER	TCR	RQD (SCR)	DEPTH (metres)	*DEPTH* RL	GRAPHIC LOG	ROCK / SOIL MATERIAL DESCRIPTION	WEATHERING	INFERRED STRENGTH $Is_{(50)}$ MPa (VL 0.1, L 0.3, M 1, H 3, VH 10, EH)	DEFECT DESCRIPTION & Additional Observations	Average Defect Spacing (mm) (30, 100, 300, 1000, 3000)
				0							
				1							
				2							
				3	*3.18*		Continuation from non-cored borehole				
NMLC	90-95% RETURN	100	78	4	20.72		SHALE: dark grey to brown, with pale grey laminations, thinly bedded.	DW		3.38-3.41: XWS, 30mm	
					3.87 20.03		From 3.87 m, changes to dark grey to brown with pale grey laminations.			3.81-3.82: XWS, 10mm 4.07: JT, 30°, CN, PL, RF, OP 4.23-4.24: CS, 10mm	
				5	*5.42* 18.48		From 5.42 m, dark grey, medium bedded.	FR			
	70% RETURN	100	100	6							
				7							
				8	*8.00* 15.90		Hole Terminated at 8.00 m Target Depth Reached.				
				9							
				10							

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

EIA 2.00.3 LIB.GLB Log EIA CORED BOREHOLE 1 E25324.G03.GPJ <<DrawingFile>> 08/10/2021 09:24 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: EIA 2.00.3 2017-11-21 Prj: EIA 2.00.1 2017-09-26

CORE PHOTOGRAPH OF BOREHOLE: BH2

Project	Proposed Residential Development			**Depth Range**	3.18m to 8.0m BEGL		
Location	102 Burlington Road, Homebush NSW			**Contractor**	Geosense Drilling Engineers Pty Ltd		
Position	See Figure 2	**Surface RL**	≈ 23.9m	**Drill Rig**	Comacchio 205		
Job No.	E25324.G03	**Inclination**	-90°	**Logged**	EM	**Date**	16 / 9 / 2021
Client	Urban Apartments Pty Ltd	**Box**	1 of 1	**Checked**	SR	**Date**	8 / 10 / 2021



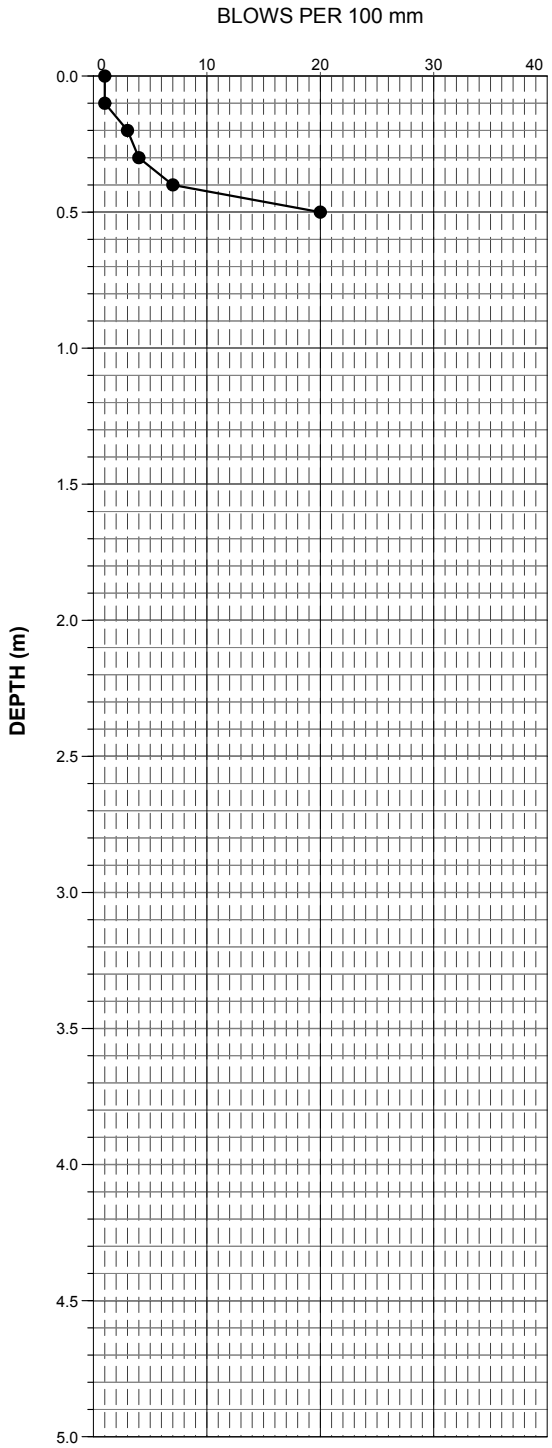
DYNAMIC CONE PENETROMETER TEST RESULTS

DCP NO. TP1

Project	Residential Development	**Sheet**	1 of 1
Location	102 Burlington Road, Homebush NSW	**Date**	16/09/2021
Position	Refer to Figure 2	**Logged By** KX	**Date** 16/09/2021
Job No.	E25324.G03	**Reviewed By** SR	**Date** 08/10/2021
Client	Urban Apartments Pty Ltd		

Surface RL ≈23.88 m AHD
Inclination -90°

DEPTH (m)	NO OF BLOWS PER 100 mm
0.00-0.1	1
0.10-0.2	1
0.20-0.3	3
0.30-0.4	4
0.40-0.5	7
0.50-0.56	20 / 60 mm HB.



Termination Remark
Target Depth Reached.

Final Depth (m)	0.56

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

EIA 2.00.3 LIB.GLB Log EIA DCP WITH TEXT AND PLOT E25324.G03.GPJ <<DrawingFile>> 08/10/2021 09:26 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: EIA 2.00.3 2017-11-21 Prj: EIA 2.00.1 2017-09-26

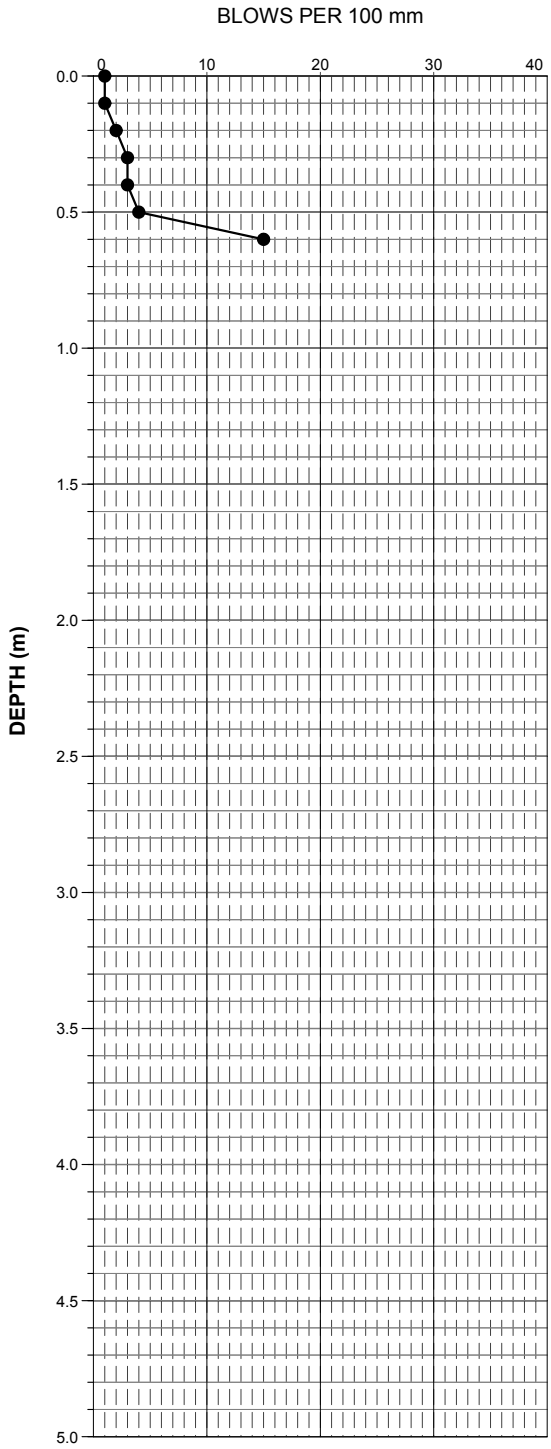
DYNAMIC CONE PENETROMETER TEST RESULTS

DCP NO. TP2

Project	Residential Development	**Sheet**	1 of 1
Location	102 Burlington Road, Homebush NSW	**Date**	16/09/2021
Position	Refer to Figure 2	**Logged By** KX	**Date** 16/09/2021
Job No.	E25324.G03	**Reviewed By** SR	**Date** 08/10/2021
Client	Urban Apartments Pty Ltd		

Surface RL ≈23.98 m AHD
Inclination -90°

DEPTH (m)	NO OF BLOWS PER 100 mm
0.00-0.1	1
0.10-0.2	1
0.20-0.3	2
0.30-0.4	3
0.40-0.5	3
0.50-0.6	4
0.60-0.7	15 / HB



Termination Remark
Target Depth Reached.

Final Depth (m)	0.70

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

EIA 2.00.3 LIB.GLB Log EIA DCP WITH TEXT AND PLOT E25324.G03.GPJ <<DrawingFile>> 08/10/2021 09:26 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: EIA 2.00.3 2017-11-21 Prj: EIA 2.00.1 2017-09-26

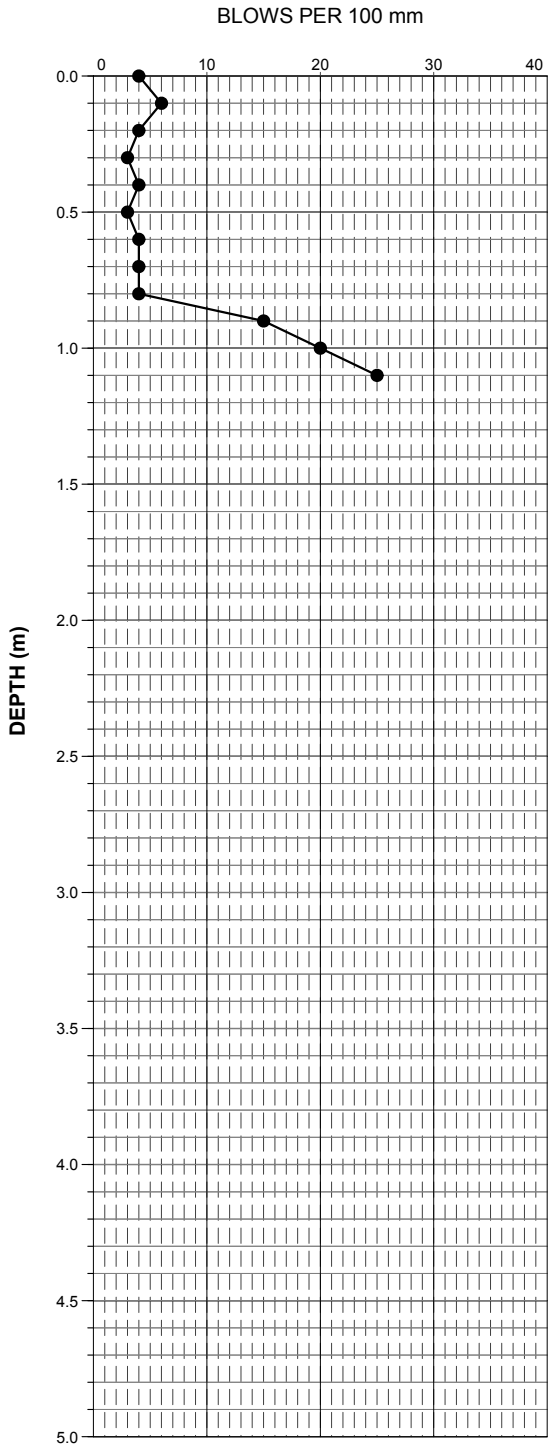
DYNAMIC CONE PENETROMETER TEST RESULTS

DCP NO. DCP 3

Project	Residential Development	**Sheet**	1 of 1
Location	102 Burlington Road, Homebush NSW	**Date Started**	16/09/2021
Position	Refer to Figure 2	**Date Completed**	16/09/2021
Job No.	E25324.G03	**Logged By** KX	**Date** 16/09/2021
Client	Urban Apartments Pty Ltd	**Reviewed By** SR	**Date** 08/10/2021

Surface RL ≈23.60 m AHD
Inclination -90°

DEPTH (m)	NO OF BLOWS PER 100 mm
0.00-0.1	4
0.10-0.2	6
0.20-0.3	4
0.30-0.4	3
0.40-0.5	4
0.50-0.6	3
0.60-0.7	4
0.70-0.8	4
0.80-0.9	4
0.90-1	15
1.00-1.1	20
1.10-1.16	25 / 60 mm HB



Termination Remark
Practical Refusal

Final Depth (m)	1.16

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

EIA 2.00.3 LIB.GLB Log EIA DCP WITH TEXT AND PLOT E25324.G03.GPJ <<DrawingFile>> 08/10/2021 09:26 10.0.000 Datgel Lab and In Situ Tool - DGD | Lib: EIA 2.00.3 2017-11-21 Prj: EIA 2.00.1 2017-09-26

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 (%)

$$= \frac{Length\ of\ core\ recovered}{Length\ of\ core\ run} \times 100$$

RQD = Rock Quality Designation (%)

$$= \frac{\sum Axial\ lengths\ of\ core > 100mm}{Length\ of\ core\ run} \times 100$$

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			

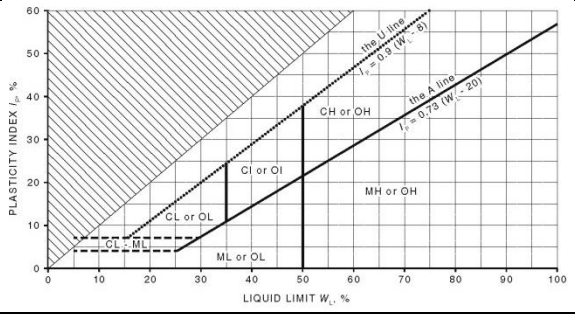
CLASSIFICATION AND INFERRED STRATIGRAPHY

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	Edward Marks
Client	EI AUSTRALIA
Address	SUITE 6.01 55 MILLER STREET PYRMONT NSW 2009
Telephone	61 2 95160722
Facsimile	(Not specified)
Email	Edward.Marks@eiaustralia.com.au
Project	**E25234.G03 102 Burlington Rd, Homebush**
Order Number	**E25234.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	**SE223748 R0**
Date Received	17/9/2021
Date Reported	24/9/2021

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: 21/9/2021

PARAMETER	UOM	LOR	BH1M_0.5-0.95 SOIL - 16/9/2021 SE223748.001	BH2_3.0-3.18 SOIL - 16/9/2021 SE223748.002
Chloride	mg/kg	0.25	**18**	**74**
Sulfate	mg/kg	5	**300**	**62**

pH in soil (1:5) [AN101] Tested: 21/9/2021

PARAMETER	UOM	LOR	BH1M_0.5-0.95 SOIL - 16/9/2021 SE223748.001	BH2_3.0-3.18 SOIL - 16/9/2021 SE223748.002
pH	pH Units	0.1	**4.1**	**5.2**

Conductivity and TDS by Calculation - Soil [AN106] Tested: 21/9/2021

PARAMETER	UOM	LOR	BH1M_0.5-0.95 SOIL - 16/9/2021 SE223748.001	BH2_3.0-3.18 SOIL - 16/9/2021 SE223748.002
Conductivity of Extract (1:5 dry sample basis)	µS/cm	1	180	96

Moisture Content [AN002] Tested: 23/9/2021

PARAMETER	UOM	LOR	BH1M_0.5-0.95 SOIL - 16/9/2021 SE223748.001	BH2_3.0-3.18 SOIL - 16/9/2021 SE223748.002
% Moisture	%w/w	1	**22.2**	**11.5**

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.
**	Indicative data, theoretical holding time exceeded.	NVL	Not validated.	LOR	Limit of Reporting.
***	Indicates that both * and ** apply.	IS	Insufficient sample for analysis.	↑↓	Raised/lowered Limit of Reporting.
		LNR	Sample listed, but not received.		

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.

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STS 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: 102 BURLINGTON ROAD, HOMEBUSH NSW - E25324 — Project No.: 31380

Client: EI Australia Pty Ltd — Report No.: 21/2734

Address: Suite 6.01, 55 Miller Street, Pyrmont NSW 2009 — Report Date: 23/09/2021

Test Method: AS1289.3.1.2, .3.2.1, .3.1.1, .3.4.1 — Page: 1 OF 2

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

STS / Sample No.	5587D-L/1					
Sample Location	Borehole 2					
Material Description	Silty Clay, brown and grey some of gravel					
Depth (m)	0.5 - 0.95					
Sample Date	16/09/2021					
Sample History	Oven Dried					
Method of Preparation	Dry Sieved					
Liquid Limit (%)	**73**					
Plastic Limit (%)	**26**					
Plasticity Index	**47**					
Linear Shrinkage (%)	**18.0**					
Mould Size (mm)	127					
Crumbing	N					
Curling	Y					

Remarks:

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

Technician: ZW — Orlando Mendoza - Laboratory Manager

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

Moisture Content of Soil and Aggregate Samples

Project: 102 BURLINGTON ROAD, HOMEBUSH NSW - E25324 — Project No.: 31380

Client: EI Australia Pty Ltd — Report No.: 21/2734

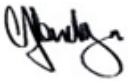
Address: Suite 6.01, 55 Miller Street, Pyrmont NSW 2009 — Report Date: 23/09/2021

Test Method: AS1289.2.1.1 — Page: 2 OF 2

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

STS / Sample No.	5587D-L/1					
Sample Location	Borehole 2					
Material Description	Silty Clay, brown and grey some of gravel					
Depth (mm)	0.5 - 0.95					
Sample Date	16/09/2021					
Moisture Content (%)	**25.6**					

Remarks:



Approved Signatory.....

Technician: ZW

Orlando Mendoza - Laboratory Manager

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

Point Load Strength Index Report

Project: 102 BURLINGTON ROAD, HOMEBUSH NSW - E25324
Client: EI Australia Pty Ltd
Address: Suite 6.01, 55 Miller Street, Pyrmont NSW 2009
Test Method: AS4133.4.1

Project No.: 31380/5587D-L
Report No.: 21/2692
Report Date: 21/09/2021
Page: 1 OF 1

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

Date Samples Drilled / Taken: 16/09/2021

Borehole No. 1M

Depth	Test Type	Is(50) (Mpa)	Rock Type	Failure Type	Moisture
3.19	A	**0.240**	TS	3	D
3.60	A	**0.200**	TS	3	D
4.17	A	**0.160**	TS	3	M
4.83	A	**0.072**	SH	3/1	M
5.20	A	**0.150**	SH	3/1	M
5.80	A	**0.040**	SH	3	M
6.48	A	**0.510**	SH	3	D
7.52	A	**0.760**	SH	3	D

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

Date Samples Drilled / Taken: 16/09/2021

Borehole No. 2

Depth	Test Type	Is(50) (Mpa)	Rock Type	Failure Type	Moisture
3.30	A	**0.370**	TS	3	D
3.70	A	**0.160**	SH	3	M
4.40	A	**0.073**	SH	3	M
4.84	A	**0.220**	SH	3/1	W
5.27	A	**0.120**	SH	3/1	W
5.70	A	**0.480**	SH	3	D
6.45	A	**0.400**	SH	3	D
7.43	A	**0.870**	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.

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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.

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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.