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T&P CHIMES DEVELOPMENT PTY LTD

Additional Geotechnical Investigation

45-53 Macleay Street, Potts Point, NSW

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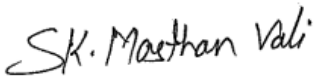	
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	Original	10 November 2025	
1	Update of SEAR's Reference Number	1 December 2025	GP
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1. Introduction

1.1 Background

At the request of Mr. Marc Yeo of T&P chimes development Pty Ltd (the Client), EI Australia (EI) has carried out an Additional Geotechnical Investigation (AGI) for the proposed development at 45-53 Macleay Street, Potts Point, NSW (the Site).

This AGI report has been prepared to provide advice and recommendations to assist in the preparation of designs and to accompany a State Significant Development Application (SSDA) for a shop top housing development. The investigation has been carried out in accordance with the agreed scope of works outlined in EI's proposal referenced P23440.1_Rev2, dated 26 June 2025.

This report has been prepared in response to the requirements contained within the secretary's Environmental Assessment Requirements (SEARs) dated 16 May 2025 issued for the SSDA SSD-83867719. Specifically this report has been prepared to address Item No.12 of the SEARs requirement.

1.2 Proposed Development

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

- Preliminary Site Investigation report prepared by JK Environments – Referenced E34303Brpt.Rev1, dated 12 February 2025;
- Geotechnical Investigation report prepared by JK Geotechnics – Referenced 34303RErpt, dated 12 September 2023;
- Architectural drawings prepared by KHA Architects, Reference No. 653, Drawing Nos. A1001 to A 1004 of Issue F and A 2002 of Issue C, last dated 10 October 2025;
- Site Survey Plan prepared by Colliers – Referenced 299-21, dated 23 March 2020. 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 and conversations with the Client, EI understands that the proposed development involves the demolition of the existing site structures and the construction of a sixteen-storey residential development with roof top level, overlying a four-level basement. The ground floor of the development will be consisted of retail premises. The Finished Floor Level (FFL) of the lowest basement level (Basement 4) varies between Reduced Levels (RLs) to Australian Height Datum (AHD) RL 14.34 m to RL 14.80 m AHD. A Bulk Excavation Level (BEL) ranging between RL 14.04 m to RL 14.50 m is assumed, which includes allowance for the construction of the basement slab. To achieve the BEL, excavation depths up to 13 m Below Existing Ground Level (BEGL) have been estimated. Locally deeper excavations may be required for footings, lift overrun pits, crane pads, pump-out pit, and service trenches. The basement extends to the northern and western site boundaries and has a setback of around 1 m and 5.3 m from the eastern and southern site boundaries respectively.

1.3 Background

A geotechnical investigation (GI) was carried out by JK Geotechnics Pty Ltd (JKG) in July to August 2023 at the Site. Findings of their investigation are presented in JKG report 34303RErpt dated 12 September 2023. The JKG GI comprised the drilling of three boreholes (BH1 to BH3) in accessible at-grade locations of the site, followed by the installation of groundwater monitoring wells within them.

The JKG GI was carried out on the basis of the original proposed development design comprising the demolition of the existing structures and the construction of a nine level mixed-

use building over two basement levels with a deep soil planter bed lining the western site boundary. Since the JKG GI report, it is understood that the design for the proposed development have changed, including the basement footprint, layout and depth.

The JKG boreholes were terminated at depths of between 11m (BH3) and 15.12m (BH1) relative to ground surface levels, or equivalent RL 16.9 m AHD (BH3) and RL 11.88 m AHD (BH1) respectively.

1.4 Objectives

The objective of the AGI was to assess site surface and subsurface conditions at two EI borehole locations to complement the available JKG borehole information for the site, and to provide geotechnical advice and recommendations to assist in the design of the proposed development.

1.5 Fieldwork Methodology

The scope of works for the AGI 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 Before You Dig Australia (BYDA) plans;
- Auger drilling of two boreholes (BH101M and BH102M) by a track-mounted drill rig using solid flight augers equipped with a 'Tungsten-Carbide' (T-C) bit. The boreholes were auger drilled to depths as shown in **Table 1-1** below:

Table 1-1 Augering and Rock Coring Depths

Borehole ID	Surface RL (m AHD)	Augering Termination		Rock Coring Termination	
		Depth (m)	RL (m AHD)	Depth (m)	RL (m AHD)
BH101M	27.25	0.7	26.55	15.10	12.15
BH102M	27.55	0.43	27.12	25.10	2.45

- 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**. Northing and easting data are presented in the detailed borehole logs in **Appendix A**.
- Continuation of BH101M and BH102M using NMLC diamond coring techniques to termination depths shown above in **Table 1-1**. The rock core photographs are presented in **Appendix A**;
- Rock core samples were sent to STS Geotechnics Pty Ltd (STS) which is a National Australian Testing Authority (NATA) accredited laboratory, for testing and storage;

- Preparation of this AGI 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.

1.6 Constraints

The AGI was limited by the intent of the investigation and the presence of existing site structures.

EI have incorporated the JKG borehole information for the development of the geotechnical ground model. However, EI cannot verify the accuracy of the information presented in the JKG report and their borehole logs.

The discussions and advice presented in this report are intended to assist in the preparation of designs for the proposed development. Further geotechnical inspections should be carried out during construction to confirm the geotechnical and groundwater models, and the 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	45-53 Macleay Street, Potts Point, NSW
Lot and Deposited Plan (DP) Identification	SP 934
Brief Site Description	At the time of our investigation, the site was occupied by a twelve-storey brick building with carpark at the ground floor and at level 1.
Site Area	The site area is approximately 1289.5m^2 (based on the provided survey plan referenced above).

Plate 1 Aerial photograph of the site (source: Sixmap, accessed on 11 Sep. 2025)

A twelve-storey apartment building of brick and concrete construction occupied the north-eastern corner of the site. The remainder of the site comprised a concrete paved car park and the ground floor parking level extended under the apartment building. The first floor of the building also comprised a parking level and the upper parking level extended south from the south-eastern side of the apartment building. The upper parking level is accessible via a concrete ramp which lined the southern site boundary. Due to the gentle grade across the site, the at-ground level car park accessed off McDonald Street is elevated above the McDonald Street and the McDonald Lane level, retained by low brick retaining wall along the McDonald Lane frontage.

2.2 Local Land Use

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

Table 2-2 Summary of Local Land Use

Direction Relative to Site	Land Use Description
North	McDonald Street, a two lane (parking lane and traffic lane) in each traffic direction, asphalt-paved divided road followed by a three-storey residential brick residential building with at least one lower ground floor/ basement.
East	Macleay Street, a two lane (parking lane and traffic lane) in each traffic direction, asphalt-paved road, followed by a multi storey residential building at 12 Macleay Street, no visible basements are observed from the street view.
South	Property at 55 Macleay Street, a three-storey residential brick building abutting the site boundary. No basements are observed from the street view.
West	McDonald Lane, a single traffic lane and parking lane asphalt-paved road with single directional traffic followed by property at 7 Mc Donald street, a three-storey residential brick residential terrace building. No visible basements observed from the street view.

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 low north west side of the road within gently (0° to 10°) dipping topography with site levels varying from R.L. 26.70 m at the north west site corner to R.L. 28.7 m at the south east site corner.
Regional Geology	Information on regional sub-surface conditions, referenced from the Department of Minerals and Energy Geological Map Sydney 1:100,000 Geological Series Sheet 9130 (DMR 1983) indicates the site to be underlain by : Hawkesbury Sandstone (Rh), which consists of medium to coarse-grained quartz sandstone with very minor shale and laminite lenses.

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 from the AGI EI boreholes and the earlier JKG boreholes has been grouped into three 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 in **Appendix A**. A summary of the depth and level of the units observed in the boreholes drilled by EI and JKG are provided in **Table 3-2**

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	0.2	27.05	0.2	Concrete pavements of 160mm to 200mm thickness, underlain by gravelly Sand fill. Fill was observed only in BH101M.
2	Class IV Sandstone	0.15 to 0.7	26.30 to 27.70	2.95 to 4.84	Fine to medium grained sandstone, low strength bedrock with widely spaced defects.
3	Class III Sandstone	3.10 to 5.30	21.70 to 24.30	- [3]	Fine to medium strength sandstone, medium strength with occasional high strength. The sandstone generally consisted of widely spaced defects, with horizontal bedding partings. Siltstone / laminite band of between 0.8m to 2m was observed sandwiched between the Class III sandstone between about RL15.6m to RL12.2m AHD.

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.
Note 4 Observed within BH101M only.

The siltstone / laminite band was encountered in EI boreholes BH101M and BH102M, and in JKG BH1. JKG BH2 and BH3 were not sufficiently deep to intersect the siltstone / laminite band. The thickness of the siltstone / laminite band varied between about 0.8m (EI BH102M) to about 2m (JKG BH1). The RLs at which the siltstone / laminite band was intersected is as follows:

- In EI BH101M, the siltstone band is observed between RL15.61m to RL14.75m AHD;
- In EI BH102M, the siltstone band is observed between RL15.3m to RL14.49m AHD;
- The siltstone band was observed between RL14.2m to RL12.2m AHD in JKG BH1.

In addition to the generalised ground model presented in **Table 3-1**, JKG have identified:

- Class V Sandstone to be present in BH1 between RL14.8m to RL 13.8 m AHD sandwiched between the Class III Sandstone;

- Class II Sandstone to be present in BH3 below RL 20.6 m AHD to End of Hole (EOH) of RL 16.9 m AHD.

Table 3-2 Depths and RL's to Top of Units in Boreholes

Unit	Material	Depth to Top of Unit (m BEGL) *RL to Top of Unit (m AHD)*				
		BH101M (EI)	BH102M (EI)	BH1 (JKG)	BH2 (JKG)	BH3 (JKG)
1	Fill	0.2 *27.05*	- -	0.13 *26.87*	- -	- -
2	Class IV Sandstone	0.4 *26.85*	0.16 *27.39*	0.70 *26.30*	0.15 *27.25*	0.20 *27.70*
3	Class III Sandstone	4.10 *23.15*	5.0 *22.55*	5.30 *21.70*	3.10 *24.30*	3.8 *24.10*

In addition to the ground model shown in **Table 3-2**, JKG have identified:

- Class V Sandstone to be present in BH1 between RL14.8m to RL 13.8 m AHD sandwiched between the Class III Sandstone;
- Class II Sandstone to be present in BH3 below RL 20.6 m AHD to End of Hole (EOH) of RL 16.9 m AHD.

3.2 Groundwater Observations

Water circulation as is required for NMLC rock coring within the boreholes prevented observations of groundwater levels within BH101M and BH102M.

Following the completion of drilling works, groundwater monitoring wells were installed in boreholes BH101M and BH102M as shown in the well installation records presented in the borehole logs in **Appendix A.** Following installation, the wells were bailed dry with hand bailer methods. The well installation details are presented in **Table 3-3**.

Table 3-3 Well Installation Depths

Borehole ID	Surface RL (m AHD)	Well Termination Depth		Start of Screen	
		Depth (m)	RL (m AHD)	Depth (m)	RL (m AHD)
BH101M	27.25	12.3	14.95	9.3	17.95
BH102M	27.55	15.0	12.55	9.0	18.55

The groundwater levels were then measured within the monitoring wells as per **Table 3-4 Error! Reference source not found.**. EI also revisited the site on 5 September 2025, approximately 1 week after well installation, to carry out measurement of groundwater levels in the boreholes.

Table 3-4 Groundwater Measurements within the EI Monitoring Well

Borehole ID	Surface RL (m AHD)	Stick Down (m)	Groundwater Levels		
			Measurement Date	m BEGL	RL (m AHD)
BH101M	27.25	0.1	5/08/2025	5.94	21.31
BH102M	27.55	0.1	5/08/2025	8.28	19.27

3.3 Test Results

Thirty-five (35) selected rock core samples were tested by STS Geotechnics Pty Ltd to estimate the Point Load Strength Index (Is_{50}) values to assist with rock strength assessment. The results of the testing are presented in the laboratory test reports (**Appendix B**) and reproduced on the attached borehole logs (**Appendix A**). The point load strength index tests correlated reasonably well with our field assessments of the rock strength.

Two selected rock core samples were tested by STS Geotechnics Pty Ltd to determine the Unconfined Compressive Strength (UCS) values. The UCS values are shown in **Table 3-5** below.

Table 3-5 Summary of UCS Test Results

Test / Sample ID	BH102M_UCS[1]	BH102M_UCS
Depth (m BEGL)	11.0-11.26	12.0-12.25
Unit	3	3
Material Description	Class III Sandstone	Class III Sandstone
Uniaxial Compressive Strength (MPa)	4.6	20.06
Moisture Content as Tested (%)	8.9	7.2
Dry Density (t/m^3)	2.100	2.200

Based on the results of the rock strength testing and assessment of the rock cores, a summary of the rock classification to Pells et al. is presented in **Table 3-2** above.

4. Recommendations

4.1 Geotechnical Considerations

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;
- Rock excavation and vibration;
- Existing footings of neighbouring properties; and
- Foundation design for building loads.

4.2 Dilapidation Surveys

Prior to excavation and construction, we recommend that detailed dilapidation surveys be carried out on all structures and infrastructures surrounding the site that falls within the zone of influence of the excavation to allow assessment of the recommended vibration limits. 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 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 Neighbouring Footings

The neighbouring building to the south (55 Macleay Street) extends to the common site boundary. It is understood that the proposed basement is proposed to be set back from the southern common boundary by a distance of 5.1 m as shown on the architectural drawings.

Considering the shallow overburden profile as observed in JKG BH2 and BH3, where the sandstone was encountered immediately below the at-grade car park concrete hardstand slab, it is inferred that the neighbouring building would likely be founded on the shallow sandstone bedrock.

Therefore, it is not envisaged that the proposed basement excavation that is proposed to be set back 5.1 m from the southern common boundary will adversely affect the neighbouring building. Temporary excavation batters if used however, should be kept clear of the southern boundary.

4.5 Excavation Methodology

4.5.1 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.

Based on the architectural drawings referenced above, it is understood that the proposed basement footprint extends to the site boundaries along northern and western boundaries and has a setback of around 1 m and 5.1 m from the eastern and southern site boundaries respectively.

EI assumes that the proposed development will require a BEL ranging between RL 14.04 m to RL 14.50 m for the construction of the basement, or an excavation depth of around 13 m 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 all units as outlined in **Table 3-1** above. Given that the sandstone bedrock is near surface, i.e. either immediately below the at-grade carpark hardstand or up to 0.7m of overburden materials as encountered in JKG BH1, it is envisaged that a temporary retaining wall is likely not required to be installed prior to excavation. However, localised treatment / support will be required in areas where overburden soils are present. An engineered retention system may be installed after the excavation to support Units 1 and 2. Unit 3 (Class III Sandstone) may be allowed to stand vertically unsupported provided the following:

- The excavation face is absent of adversely oriented defects that may result in instability;
- Inspections be carried out by an experienced geotechnical engineer progressively during basement excavations to assess rock quality and absence of adversely oriented defects;
- Where adversely oriented defects are present which may form slip planes, wedges or unstable blocks, such areas be stabilised prior to further excavations.
- Further groundwater measurements, long term groundwater monitoring and Groundwater Seepage Assessment (GSA) be carried out to determine whether the water table is within the depth of proposed excavations and seepage flow rates which if sufficiently high would preclude unsupported cuts in Class III sandstone and the potential need for tanking of the basement.

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

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

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

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

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

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

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

4.5.2 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;
- 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.

4.6 Groundwater Considerations

Groundwater was observed in all EI monitoring wells as detailed in **Table 3-4**, all of which are above the assumed BEL ranging between RL 14.04 m to RL 14.50 m .

JKG had carried out groundwater measurements in their monitoring wells about one week following their installation. The water levels measured in the JKG monitoring wells varied between depths of 2.19m (BH1), 1.41m (BH2) and 0.99m (BH3) relative to the ground surface at the respective monitoring well locations.

EI note that based on the measured water level being within the sandstone bedrock, this is likely to be perched groundwater. Hence, we expect some minor seepage inflows into the excavation along the soil/rock interface and through any defects within the sandstone bedrock (such as jointing, and bedding planes, etc.) particularly following a period of heavy rainfall. Similarly, JKG commented that the groundwater monitoring wells are likely acting as sumps and collecting localised seepage at discrete defects in the rock mass. Due to the low permeability of the soil and bedrock profile, any groundwater inflows into the excavation should not have any adverse impact on the proposed development or on the neighbouring sites and should be manageable. 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 encountered during excavation will be able to be controlled by a conventional sump and pump system.

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.

Council and WaterNSW normally do not allow permanent dewatering and the basement may be required to be designed as a tanked structure. Should a drained basement be desired, additional investigations, monitoring and analysis will be required including seepage analysis, the installation of additional monitoring wells, long-term groundwater monitoring, and laboratory testing. Reference should be made to Department of Planning and Environment (DPE) guidelines “Minimum requirements for building site groundwater investigation and reporting”, dated October 2022. EI should be contacted for further advice for the approval process for a drained basement.

4.7 Excavation Retention

4.7.1 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.

Basing on the architectural drawings referenced above, it is understood that, the proposed basement footprint extends to the northern and western site boundaries, and has a setback of around 1 m and 5.1 m from the eastern and southern site boundaries respectively. Based on the encountered subsurface conditions, temporary excavation batters in Unit 1 materials (where present) may be possible on the southern and eastern excavation faces.

Based on the basement setbacks, and the shallow depth of overburden soils, temporary batters of no steeper than a profile of 1 Vertical (V) to 1.75 Horizontal (H) may be feasible where space allows for the fill and residual soil profile (where encountered). 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.

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.

As the basement extends to the northern and western site boundaries, temporary batters are not feasible, and a suitable retention system will be required to support the Unit 1 materials. The retention system should be installed along the northern and western sides immediately after excavation of the Unit 1 materials. Considering the relatively thin profile of the overburden soils stratum, there may be a potential for a reinforced shotcrete retaining wall with rock bolts / anchors to provide support to the Units 1 materials, or alternatively a block wall dowelled into the bedrock to provide lateral restraint. However, excavation to enable construction of the reinforced shotcrete / dowelled block retaining wall would need to be carried out in short sections given the sandy nature of the subsoils, for example 1m to 1.5m width sections/panels. Some spalling of the sandy overburden soils can be expected. Sequencing of the excavation for these sections/panels should be carried out at every fourth panel, i.e. form 1 and skip 2 in between. The width and sequencing of these section/panel excavation should be assessed progressively during excavation and varied as appropriate to suit the site conditions. Appropriate subsurface drainage should also be incorporated to mitigate hydrostatic pressure buildup behind the retaining wall. The retention system should be designed to adequately resist lateral soil loads and surcharge loads including pedestrian, construction plant and for vehicular loads. It is recommended that the reinforced shotcrete or dowelled block retaining wall be recessed into the excavation such that a ledge is created on top of the vertically cut sandstone to support the underside of the reinforced shotcrete or dowelled block retaining wall and for formation of a spoon drain as appropriate to capture seepage from the strip drains behind the wall.

Unit 2 and Unit 3 materials can be cut vertically without prior construction of temporary support. For vertical cuts, the excavations must be inspected by a geotechnical engineer at regular intervals to check for any inclined joints or weak seams that require stabilisation. Such geotechnical inspections should be carried out at depth intervals of no more than 1.5m. If adverse defects are encountered, the stabilisation measures may comprise rock bolts, shotcrete and mesh or dental treatment of thin weak seams using non-shrink grout, and this should be allowed for.

In the permanent case, considering that the Unit 2 (Class IV sandstone) can extend to depths of 5.3m (BH1) relative to ground surface levels, it is recommended that permanent support be provided to Class IV sandstone. Suitable permanent retention of Class IV sandstone may be in the form of reinforced shotcrete wall with rock bolts/dowels. Appropriate subsurface drainage should be installed behind the retaining wall to mitigate against the build-up of hydrostatic pressures behind the retaining wall. Unit 3 (Class III sandstone) is anticipated to stand vertically with no permanent support required. It must be noted however, that localised treatment may be required where defects and adversely oriented joints/fractures are present, and especially where weathered siltstone bands are present.

The existence of significant horizontal in-situ stresses in bedrock, particularly in the Sydney basin, is well established. The release of such stresses during the basement excavation may cause adverse impact on the stability of the excavation faces and thus increase the movements. Monitoring of several deep excavations within sandstone and shale in the Sydney region indicates that the lateral displacement at the top of the excavation is generally between 0.5mm to 2mm per meter depth of excavation. As the maximum depth of excavation into sandstone is of about 13 m, a lateral deflection at the crest of the excavation between 7 mm to 26 mm can be expected which will reduce in a stepped fashion to zero at the bulk excavation level. Monitoring of the lateral movement as the excavation progresses is recommended. An assessment of such movements and their impact can be carried out using finite element software such as PLAXIS.

4.7.2 Retaining Wall Design Parameters

The parameters presented in **Table 4-1** may be used for static design of temporary and permanent retaining walls at the subject site. EI note that the below parameters, particularly with determining lateral earth pressures, are for preliminary planning purposes. We recommend that detailed analysis such as the use of finite element analysis software be used to design retaining walls.

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_0.

The retaining walls should be designed as drained and measures are to be taken to provide complete and permanent drainage behind the walls. For reinforced shotcrete retaining walls with rock bolts / anchors / dowels, weep holes through the wall and strip drains behind the wall should be incorporated in design.

If rock bolts / dowels / anchors extend beyond the site boundaries, then permission from the neighbouring properties would need to be obtained prior to installation. Also, the presence of neighbouring basements and/or services and their levels must be confirmed prior to finalising anchor design.

Anchors should have their bond length within Unit 2 (Class IV or better sandstone). For the design of anchors bonded into Unit 2 or better materials, the allowable bond stress value outlined in **Table 4-1** below may be used, subject to the following conditions:

- Anchor bond lengths of at least 3m behind the 'active' zone of the excavation (taken as a 45 degree zone above the base of the excavation) is provided;
- Overall stability, including anchor group interaction, is satisfied;
- All anchors should be proof loaded to at least 1.25 times the design working load before locked off at working load. Such proof loading is to be witnessed by and engineer independent of the anchoring contractor. We recommend that only experienced contractors be considered for anchor installation with appropriate insurances;
- If permanent anchors are to be used, these must have appropriate corrosion provisions for longevity.

Table 4-1 Geotechnical Design Parameters

Material [1]	Unit 1 Fill	Unit 2 Class IV Sandstone	Unit 3 Class III Sandstone
RL of Top of Unit (m AHD) [2]	26.87 to 27.05	26.30 to 27.70	21.70 to 24.30
Bulk Unit Weight (kN/m^3)	18	22	23
Friction Angle, ϕ' (°)	26	38	40
Earth Pressure at rest, K_o [3]	0.56		
Active Earth Pressure, K_a [3]	0.39		
Passive Earth Pressure, K_p [3]	-		
Young's Modulus, E (MPa)	-	200	500
Cohesion, c' (kPa)		50	200
Poisson's Ratio		0.25	0.25
Allowable Bearing Pressure (kPa) [5]	-	1000	3500
Ultimate Shaft Adhesion in Compression (kPa)	-	300	800
Ultimate Shaft Adhesion in Uplift (kPa)	-	150	400
Allowable Toe Resistance (kPa)			700
Design Bond Stress (kPa)			300
Earthquake Site Risk Classification	AS 1170.4:2007 indicates earthquake subsoil Class B_e.(Shallow Rock) AS 1170.4:2007 indicates the hazard factor (z) for Sydney is 0.8.		

Note 1 More detailed descriptions of subsurface conditions are available on the borehole logs in Appendix A.

Note 2 Approximate levels of top of unit at the time of our investigation. Levels may vary across the site.

Note 3 Earth pressures are provided on the assumption that the ground behind the retaining walls is horizontal.

Note 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.

Note 5 To adopt these parameters we have assumed that:

a. Footings have a nominal socket of at least 0.3m, into the relevant founding material;
b. For piles, there is intimate contact between the pile and foundation material (a clean socket roughness category of R2 or better);
c. Potential soil and groundwater aggressivity will be considered in the design of piles and footings;
d. 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;
e. 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;
f. The concrete is poured on the same day as drilling, inspection and cleaning.
g. The piles are founded in competent sandstone not affected by lower strength siltstone/shale bands

Note 6 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).

4.8 Foundations

Following bulk excavations ranging between RL 14.04 m to RL 14.50 m, it is expected that a siltstone band within Unit 3 sandstone will likely to be exposed at the BEL. The exposed siltstone band is of very low strength improving to medium strength with depth, with very closely spaced defects. Considering the scale of the proposed development, it is our opinion that footings should be founded below the siltstone band on the underlying sandstone bedrock of at least medium to high strength. Based on the three boreholes (EI BH101M and EI BH102M, and JKG BH1) where the siltstone / laminite band was intersected, it is envisages that the competent sandstone below the siltstone band would be encountered below about RL14.75 m (BH101M) to RL 12.2m AHD (JKG BH1).

From a practicality perspective, excavations of not more than 1m deep below the BEL may be achievable for deep spread footings. However, where the depth to the competent sandstone is in excess of about 1m below the BEL, bucket piers or shallow bored piles may need to be considered.

It is recommended that all footings be founded within Unit 3 Class III sandstone of medium strength or better to provide uniform support and reduce the potential for differential settlement.

Pad or strip footings founded in Unit 3 Class III sandstone may be preliminarily designed for an allowable bearing capacity of 3,500 kPa based on serviceability.

Geotechnical inspections of all foundation excavations are recommended due to the presence of the siltstone band at the BEL and to confirm that the required bearing capacity is achieved at all the footing locations. Inspections will also identify any variations between borehole records and actual site conditions.

4.9 Basement Floor Slab

Following bulk excavations for the proposed basement, siltstone / laminite band 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 Council and WaterNSW for any permanent discharge of seepage into the drainage system. Given the subsurface conditions, we expect that seepage volumes would be low and within acceptable limits manageable by drainage systems. However, if permission for discharge is not obtained, the basement may need to be designed as a tanked basement.

5. Conclusion

Based on the subsurface conditions encountered, geotechnical model and experience on similar projects, the proposed development is considered feasible from a geotechnical perspective.

6. Further Geotechnical Inputs

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

- Long term groundwater monitoring and seepage modelling;
- Stability assessment of temporary batters using computer modelling, if required;
- Dilapidation surveys;
- If directed by Sydney Water or any other asset owners, carry out specialist engineering assessment, further geotechnical investigations, monitoring equipment installs and/or subsequent monitoring;
- Classification of all excavated material transported off site;
- Geotechnical inspections of rock faces during excavation by experience geotechnical professional at depth of no greater than 1.5m within medium to high strength bedrock, if vertical cut are adopted;
- Geotechnical inspections of all new footings 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.

7. Statement of Limitations

This report has been prepared for the exclusive use of Mr. Marc Yeo and T&P chimes development 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 Mr. Marc Yeo and T&P chimes development 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

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

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

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Abbreviations

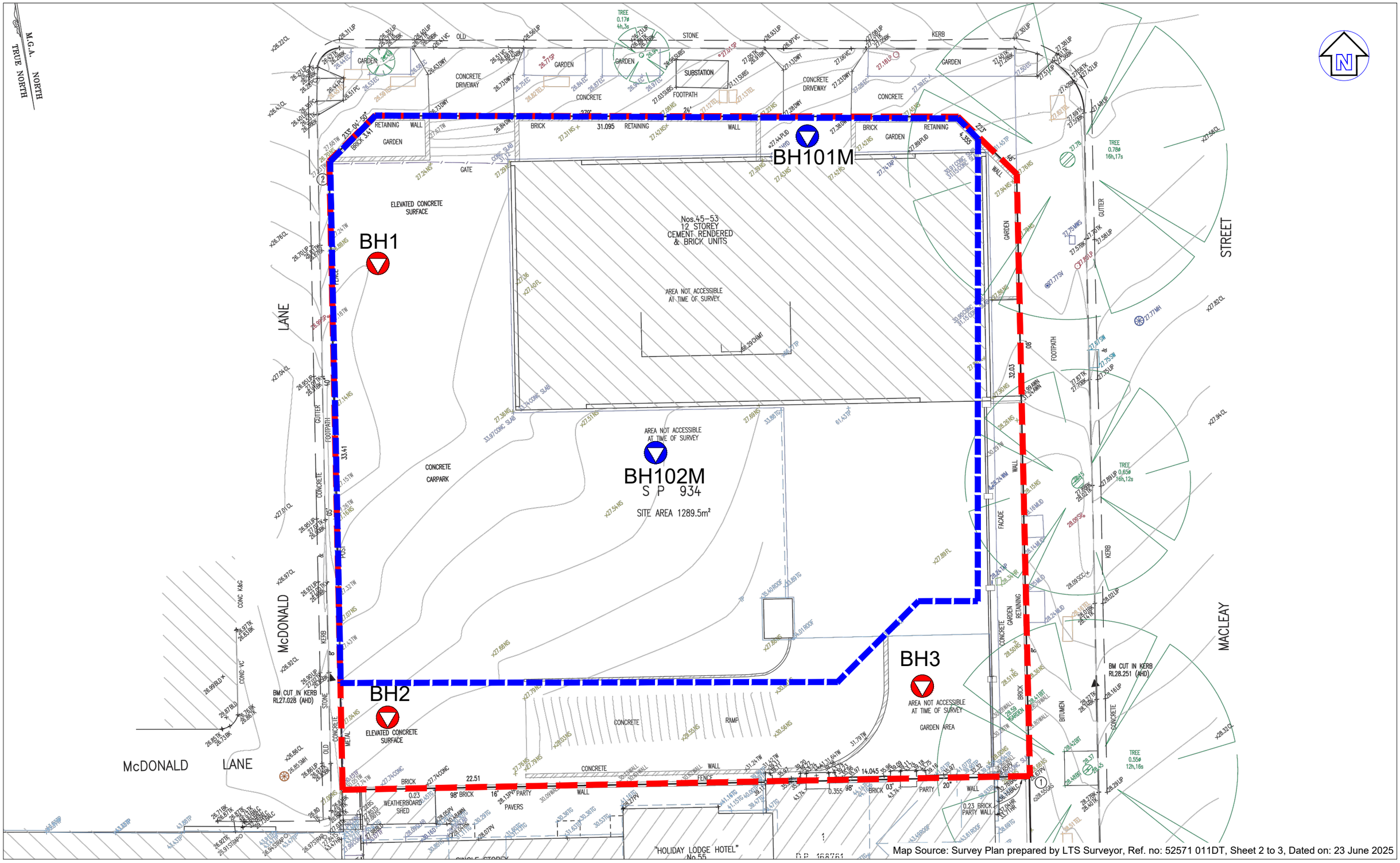
AGI	Additional Geotechnical Investigation
AHD	Australian Height Datum
AS	Australian Standard
BEL	Bulk Excavation Level
BEGL	Below Existing Ground Level
BH	Borehole
DBYD	Dial Before You Dig
DP	Deposited Plan
EI	EI Australia
NATA	National Association of Testing Authorities, Australia
RL	Reduced Level
SPT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

Figures

eiaustralia Contamination | Remediation | Geotechnical Suite 6.01, 55 Miller Street, PYRMONT 2009 Ph (02) 9516 0722 Fax (02) 9518 5088	Drawn:	M.S.	**T&P Chimes Development Pty Ltd** Additional Geotechnical Investigation 45-53 Macleay Street, Potts Point NSW Site Locality Plan
	Approved:	M.L.	
	Date:	29-01-26	
	Scale:	Not To Scale	

Figure: 1

Project: E26863.G04



LEGEND **(Note: All locations shown are approximate)**

- Site boundary
- Basement boundary
- Borehole/monitoring well location (JKGeotechnics,2023)
- Borehole/monitoring well location (EI Australia, 2025)

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Drawn:	M.S.
Approved:	M.L.
Date:	29-01-26

T&P Chimes Development Pty Ltd
Additional Geotechnical Investigation
45-53 Macleay Street, Potts Point NSW
Borehole Location Plan

Figure: 2

Project: E26863.G04

Appendix A Borehole Logs And Explanatory Notes

BOREHOLE LOG

BH ID: BH101M

Location	45-53 Macleay Steet, Potts Point NSW			**Started**	29 July 2025
Client	T&P Chimes Development Pty Ltd			**Completed**	29 July 2025
Job No.	E26863.G04			**Logged By**	MS **Date** 29 July 2025
Sheets	1 of 3			**Review By**	ML **Date** 29 September 2025
Drilling Contractor	Geosense Drilling Engineers	**Surface RL**	≈27.25 m (AHD)	**Northing**	6251032.8 (MGA 2020 Zone 56)
Plant	Comacchio Geo 205	**Inclination**	90°	**Easting**	335869.2 (MGA 2020 Zone 56)

METHOD	GROUND WATER LEVELS	SAMPLES & FIELD TESTS	SAMPLE RECOVERY	DEPTH (m)	GRAPHIC LOG	RL (m AHD)	MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY / REL. DENSITY	MATERIAL ORIGIN & OBSERVATIONS
AD/T	GWNE			0.00		27.25	CONCRETE: 200mm thick	-	-	CONCRETE
				0.20		27.05	FILL: Gravelly SAND: medium to coarse grained, brown, with medium to cobbles sub-angular to angular gravels, trace brick	W		FILL
				0.40		26.85	SANDSTONE: fine to medium grained, brown, very low strength rock	-		BEDROCK
				0.70		26.55	*Log continued on next page.*			
				1						
				2						
				3						
				4						
				5						
				6						
				7						
				8						
				9						
				10						

BOREHOLE CORE LOG

BH ID: BH101M

Location	45-53 Macleay Steet, Potts Point NSW			**Started**	29 July 2025
Client	T&P Chimes Development Pty Ltd			**Completed**	29 July 2025
Job No.	E26863.G04			**Logged By**	MS **Date** 29 July 2025
Sheets	2 of 3			**Review By**	ML **Date** 29 September 2025
Drilling Contractor	Geosense Drilling Engineers	**Surface RL**	≈27.25 m (AHD)	**Northing**	6251032.8 (MGA 2020 Zone 56)
Plant	Comacchio Geo 205	**Inclination**	90°	**Easting**	335869.2 (MGA 2020 Zone 56)

METHOD	Flush Return	TCR %	RQD %	DEPTH (m)	RL (m AHD)	MATERIAL DESCRIPTION	WEATHERING	ESTIMATED STRENGTH Is(50) ▼ - Axial ▽ - Diametral	DISCONTINUITIES & ADDITIONAL DATA	FRACTURE SPACING
				0		*Log continued from previous page.*				
NMLC	100%	100	98	0.55		SANDSTONE: fine to medium grained, brown, pale grey, thinly bedded			0.72: DB 0.75: DB 1.52: HB 2.48-2.52: XWS 2.66: BP SM CN 2.81: BP SM Clay VN 3.13: HB 3.35: JT 10° PR SM CN	
		100	97	3.55	23.70	From 3.55m, pale grey, thinly to medium bedded			5.17: BP SM CN	
				5.35	21.90	From 5.35m, dark grey, grey with shale clay bands	FR		5.36-5.46: XWS	
				5.47	21.78	From 5.47m, pale grey, thinly to thickly bedded				
		96	96	7.75	19.50	From 7.75m, massive			6.87: BP SM Clay VN 7.26-7.29: XWS 7.60: BP SM CN	
				10						

BOREHOLE CORE LOG

BH ID: BH101M

Location	45-53 Macleay Steet, Potts Point NSW			**Started**	29 July 2025
Client	T&P Chimes Development Pty Ltd			**Completed**	29 July 2025
Job No.	E26863.G04			**Logged By**	MS **Date** 29 July 2025
Sheets	3 of 3			**Review By**	ML **Date** 29 September 2025
Drilling Contractor	Geosense Drilling Engineers	**Surface RL**	≈27.25 m (AHD)	**Northing**	6251032.8 (MGA 2020 Zone 56)
Plant	Comacchio Geo 205	**Inclination**	90°	**Easting**	335869.2 (MGA 2020 Zone 56)

METHOD	Flush Return	TCR %	RQD %	DEPTH (m)	GRAPHIC LOG	RL (m AHD)	MATERIAL DESCRIPTION	WEATHERING	ESTIMATED STRENGTH Is(50) ▼ - Axial ▽ - Diametral (VL 0-1, L 0-3, M 1, H 3, VH 10, EH)	DISCONTINUITIES & ADDITIONAL DATA	FRACTURE SPACING (30, 100, 300, 1000, 3000)
NMLC	100%	95	95	10.50		16.75	From 7.75m, massive				
				11			From 10.50m, medium to thickly bedded, pale grey with orange brown			10.52: BP SM CN 10.86: BP SM CN 11.30: BP SM CN	
				11.64		15.61	SILTSTONE: dark grey, thinly bedded with sandstone laminae			11.64: BP SM CN 11.80-11.90: JT 50° CU SM CN	
				12						12.30: HB 12.40: JT 10° PR SM CN 12.44-12.50: XWS	
				12.50		14.75	SANDSTONE: medium to coarse grained, pale grey, thickly bedded	FR			
		95	95	13						13.74: BP SM CN	
				14							
				14.37		12.88	From 14.37m to 14.44m, with siltstone band			14.38: BP SM CN	
				14.71		12.54	From 14.71m to 14.80m, with siltstone laminae			14.72: JT 10° CU SM CN	
				15		12.15	Terminated at 15.10m. Target Depth Reached.				
				16							
				17							
				18							
				19							
				20							

MONITORING WELL LOG

BH ID: BH101M

Location	45-53 Macleay Steet, Potts Point NSW			**Started**	29 July 2025
Client	T&P Chimes Development Pty Ltd			**Completed**	29 July 2025
Job No.	E26863.G04			**Logged By**	MS **Date** 29 July 2025
Sheets	1 of 2			**Review By**	ML **Date** 29 September 2025
Drilling Contractor	Geosense Drilling Engineers	**Surface RL**	≈27.25 m (AHD)	**Northing**	6251032.8 (MGA 2020 Zone 56)
Plant	Comacchio Geo 205	**Inclination**	90°	**Easting**	335869.2 (MGA 2020 Zone 56)

WATER	SAMPLES & FIELD TESTS	DEPTH (m)	RL (m AHD)	MATERIAL DESCRIPTION	MOISTURE CONDITION	BACKFILL DETAILS	STANDPIPE DETAILS
		0.00	27.25	CONCRETE: 200mm thick	-		Well Stickup =0.0m (RL 27.25m)
		0.20	27.05	FILL: Gravelly SAND: medium to coarse grained, brown, with medium to cobbles sub-angular to angular gravels, trace brick	W		
		0.40	26.85	SANDSTONE: fine to medium grained, brown, very low strength rock	-		
		0.70	26.55	SANDSTONE: fine to medium grained, brown, pale grey, thinly bedded			
		3.55	23.70	From 3.55m, pale grey, thinly to medium bedded		Sand 0.00m - 8.00m	0.0m - 9.30m PVC casing (50mm Ø)
		5.35	21.90	From 5.35m, dark grey, grey with shale clay bands			
		5.47	21.78	From 5.47m, pale grey, thinly to thickly bedded			
		7.75	19.50	From 7.75m, massive		Bentonite 8.00m - 8.50m	

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

MONITORING WELL LOG

BH ID: BH101M

Location	45-53 Macleay Steet, Potts Point NSW			**Started**	29 July 2025
Client	T&P Chimes Development Pty Ltd			**Completed**	29 July 2025
Job No.	E26863.G04			**Logged By**	MS **Date** 29 July 2025
Sheets	2 of 2			**Review By**	ML **Date** 29 September 2025
Drilling Contractor	Geosense Drilling Engineers	**Surface RL**	≈27.25 m (AHD)	**Northing**	6251032.8 (MGA 2020 Zone 56)
Plant	Comacchio Geo 205	**Inclination**	90°	**Easting**	335869.2 (MGA 2020 Zone 56)

WATER	SAMPLES & FIELD TESTS	DEPTH (m)	RL (m AHD)	MATERIAL DESCRIPTION	MOISTURE CONDITION	BACKFILL DETAILS	STANDPIPE DETAILS
		10.50	16.75	From 7.75m, massive		Sand 8.50m - 12.30m	9.30m - 12.30m PVC screen (50mm Ø)
				From 10.50m, medium to thickly bedded, pale grey with orange brown			
		11.64	15.61	SILTSTONE: dark grey, thinly bedded with sandstone laminae			
		12.50	14.75	SANDSTONE: medium to coarse grained, pale grey, thickly bedded		Bentonite 12.30m - 13.80m	
		14.37	12.88	From 14.37m to 14.44m, with siltstone band		Sand 13.80m - 15.10m	
		14.71	12.54	From 14.71m to 14.80m, with siltstone laminae			
		15	12.15	Terminated at 15.10m. Target Depth Reached.			

CORE PHOTOGRAPH OF BOREHOLE: BH101M

Project	Proposed Development	**East**	335869.2	**Depth Range**	0.7m to 15.10m		
Location	45-53 Macleay Street, Potts Point NSW	**North**	6251032.8	**Contractor**	Geosense Drilling Engineers Pty Ltd		
Position	See Figure 2	**Surface RL**	27.25m	**Drill Rig**	Hanjin D&B 8D		
Job No.	E26863.G04	**Inclination**	90°	**Logged**	MS	**Date**	29/07/2025
Client	T&P Chimes Development Pty Ltd	**Box**	1-2 of 4	**Checked**	ML	**Date**	29/01/2026

CORE PHOTOGRAPH OF BOREHOLE: BH101M

Project	Proposed Development	**East**	335869.2	**Depth Range**	0.7m to 15.10m		
Location	45-53 Macleay Street, Potts Point NSW	**North**	6251032.8	**Contractor**	Geosense Drilling Engineers Pty Ltd		
Position	See Figure 2	**Surface RL**	27.25m	**Drill Rig**	Hanjin D&B 8D		
Job No.	E26863.G04	**Inclination**	90°	**Logged**	MS	**Date**	29/07/2025
Client	T&P Chimes Development Pty Ltd	**Box**	3-4 of 4	**Checked**	ML	**Date**	29/01/2026

BOREHOLE LOG

BH ID: BH102M

Location	45-53 Macleay Steet, Potts Point NSW			**Started**	28 July 2025
Client	T&P Chimes Development Pty Ltd			**Completed**	28 July 2025
Job No.	E26863.G04			**Logged By**	MS **Date** 28 July 2025
Sheets	1 of 4			**Review By**	ML **Date** 29 September 2025
Drilling Contractor	Geosense Drilling Engineers	**Surface RL**	≈27.55 m (AHD)	**Northing**	6251016.4 (MGA 2020 Zone 56)
Plant	Comacchio Geo 205	**Inclination**	90°	**Easting**	335859.5 (MGA 2020 Zone 56)

METHOD	GROUND WATER LEVELS	SAMPLES & FIELD TESTS	SAMPLE RECOVERY	DEPTH (m)	GRAPHIC LOG	RL (m AHD)	MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY / REL. DENSITY	MATERIAL ORIGIN & OBSERVATIONS
AD/T	GWNE			0.00		27.55	CONCRETE: 160mm thick	-	-	CONCRETE
				0.16		27.39	SANDSTONE: fine to medium grained, pale grey, orange, very low to low strength rock			BEDROCK
				0.43		27.12	*Log continued on next page.*			

BOREHOLE CORE LOG

BH ID: BH102M

Location	45-53 Macleay Steet, Potts Point NSW			**Started**	28 July 2025
Client	T&P Chimes Development Pty Ltd			**Completed**	28 July 2025
Job No.	E26863.G04			**Logged By**	MS **Date** 28 July 2025
Sheets	2 of 4			**Review By**	ML **Date** 29 September 2025
Drilling Contractor	Geosense Drilling Engineers	**Surface RL**	≈27.55 m (AHD)	**Northing**	6251016.4 (MGA 2020 Zone 56)
Plant	Comacchio Geo 205	**Inclination**	90°	**Easting**	335859.5 (MGA 2020 Zone 56)

METHOD	Flush Return	TCR %	RQD %	DEPTH (m)	GRAPHIC LOG	RL (m AHD)	MATERIAL DESCRIPTION	WEATHERING	ESTIMATED STRENGTH Is(50) ▼ - Axial ▽ - Diametral (VL 0-1, L 0-3, M 1, H 3, VH 10, EH)	DISCONTINUITIES & ADDITIONAL DATA	FRACTURE SPACING (30, 100, 300, 1000, 3000)
				0			*Log continued from previous page.*				
NMLC	100%	100	100	1			SANDSTONE: fine to medium grained, grey, mottled orange, thinly to medium bedded			1.40: BP SM Clay VN 1.54: HB 1.73: BP SM CN	
				2							
		97	97	2.94 3		24.61	From 2.94m, grey			3.09: HB 3.65: HB 3.98-4.00: XWS 4.59-4.63: XWS 4.67-4.69: XWS 4.91: BP SM Clay VN	
				4							
				5				FR			
				6						6.37: BP SM Clay VN	
				7						7.08-7.12: XWS	
		96	96	8							
				9							
				10							

BOREHOLE CORE LOG

BH ID: BH102M

Location	45-53 Macleay Steet, Potts Point NSW			**Started**	28 July 2025
Client	T&P Chimes Development Pty Ltd			**Completed**	28 July 2025
Job No.	E26863.G04			**Logged By**	MS **Date** 28 July 2025
Sheets	3 of 4			**Review By**	ML **Date** 29 September 2025
Drilling Contractor	Geosense Drilling Engineers	**Surface RL**	≈27.55 m (AHD)	**Northing**	6251016.4 (MGA 2020 Zone 56)
Plant	Comacchio Geo 205	**Inclination**	90°	**Easting**	335859.5 (MGA 2020 Zone 56)

METHOD	Flush Return	TCR %	RQD %	DEPTH (m)	GRAPHIC LOG	RL (m AHD)	MATERIAL DESCRIPTION	WEATHERING	ESTIMATED STRENGTH Is(50) ▼ - Axial ▽ - Diametral (VL 0-1, L 0-3, M 1, H 3, VH 10, EH)	DISCONTINUITIES & ADDITIONAL DATA	FRACTURE SPACING (30, 100, 300, 1000, 3000)
NMLC	100%	100	100	11			From 2.94m, grey	FR	▼ M	10.82: JT 40° CU SM CN	
										11.25: JT 15° CU SM CN	
										11.35: JT 10° CU SM CN	
				12						11.80: JT 15° CU SM CN	
										11.95-12.00: HB	
										12.13: BP SM CN	
				12.25		15.30	SILTSTONE: dark grey, thinly bedded with sandstone laminae	DW	▼ VL	12.26: DB	
										12.43: BP SM CN	
										12.63: BP SM CN	
				13					▼ M	12.90: BP SM CN	
				13.06		14.49	SANDSTONE: fine to medium grained, grey			13.07: BP SM CN	
		97	94	14							
										14.54: DB	
										14.72: BP SM CN	
				15							
										15.32: DB	
				15.43		12.12	From 15.43m, thinly to medium bedded			15.51: BP SM CN	
		98	98	16				FR		16.55: BP SM CN	
										16.72: HB	
				17							
										17.53: BP SM Clay VN	
				18							
										18.45: JT 15° CU SM CN	
	90%			19							
										19.57: JT 15° CU SM CN	
				20							

BOREHOLE CORE LOG

BH ID: BH102M

Location	45-53 Macleay Steet, Potts Point NSW			**Started**	28 July 2025
Client	T&P Chimes Development Pty Ltd			**Completed**	28 July 2025
Job No.	E26863.G04			**Logged By**	MS **Date** 28 July 2025
Sheets	4 of 4			**Review By**	ML **Date** 29 September 2025
Drilling Contractor	Geosense Drilling Engineers	**Surface RL**	≈27.55 m (AHD)	**Northing**	6251016.4 (MGA 2020 Zone 56)
Plant	Comacchio Geo 205	**Inclination**	90°	**Easting**	335859.5 (MGA 2020 Zone 56)

METHOD	Flush Return	TCR %	RQD %	DEPTH (m)	GRAPHIC LOG	RL (m AHD)	MATERIAL DESCRIPTION	WEATHERING	ESTIMATED STRENGTH Is(50) ▼ - Axial ▽ - Diametral (VL 0-1, L 0-3, M 1, H 3, VH 10, EH)	DISCONTINUITIES & ADDITIONAL DATA	FRACTURE SPACING (30, 100, 300, 1000, 3000)
NMLC	90%	99	99	21			From 15.43m, thinly to medium bedded	FR	▼ M 1	20.20: JT 10° CU SM CN 20.45: JT 10° CU SM CN 21.45: BP SM CN	
				21.70		5.85	From 21.70m, thinly bedded				
		100	100	22							
				23							
				24							
		100	100	25		2.45	Terminated at 25.10m. Target Depth Reached.		▼ H 3	24.82: BP SM CN	
				26							
				27							
				28							
				29							
				30							

MONITORING WELL LOG

BH ID: BH102M

Location	45-53 Macleay Steet, Potts Point NSW			**Started**	28 July 2025
Client	T&P Chimes Development Pty Ltd			**Completed**	28 July 2025
Job No.	E26863.G04			**Logged By**	MS **Date** 28 July 2025
Sheets	1 of 3			**Review By**	ML **Date** 29 September 2025
Drilling Contractor	Geosense Drilling Engineers	**Surface RL**	≈27.55 m (AHD)	**Northing**	6251016.4 (MGA 2020 Zone 56)
Plant	Comacchio Geo 205	**Inclination**	90°	**Easting**	335859.5 (MGA 2020 Zone 56)

WATER	SAMPLES & FIELD TESTS	DEPTH (m)	GRAPHIC LOG	RL (m AHD)	MATERIAL DESCRIPTION	MOISTURE CONDITION	BACKFILL DETAILS	STANDPIPE DETAILS
		0.00		27.55	CONCRETE: 160mm thick			Well Stickup =0.0m (RL 27.55m)
		0.16		27.39	SANDSTONE: fine to medium grained, pale grey, orange, very low to low strength rock	-		
		0.43		27.12	SANDSTONE: fine to medium grained, grey, mottled orange, thinly to medium bedded			
		2.94		24.61	From 2.94m, grey		Sand 0.00m - 8.00m	0.0m - 9.0m PVC casing (50mm Ø)
							Bentonite 8.00m - 8.50m	

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

MONITORING WELL LOG

BH ID: BH102M

Location	45-53 Macleay Steet, Potts Point NSW			**Started**	28 July 2025
Client	T&P Chimes Development Pty Ltd			**Completed**	28 July 2025
Job No.	E26863.G04			**Logged By**	MS **Date** 28 July 2025
Sheets	2 of 3			**Review By**	ML **Date** 29 September 2025
Drilling Contractor	Geosense Drilling Engineers	**Surface RL**	≈27.55 m (AHD)	**Northing**	6251016.4 (MGA 2020 Zone 56)
Plant	Comacchio Geo 205	**Inclination**	90°	**Easting**	335859.5 (MGA 2020 Zone 56)

WATER	SAMPLES & FIELD TESTS	DEPTH (m)	GRAPHIC LOG	RL (m AHD)	MATERIAL DESCRIPTION	MOISTURE CONDITION	BACKFILL DETAILS	STANDPIPE DETAILS
29/07/2025 02:48 PM		11			From 2.94m, grey		Sand 8.50m - 15.00m	9.0m - 15.0m PVC screen (50mm Ø)
		12						
		12.25		15.30	SILTSTONE: dark grey, thinly bedded with sandstone laminae			
		13						
		13.06		14.49	SANDSTONE: fine to medium grained, grey			
		14						
		15						
		15.43		12.12	From 15.43m, thinly to medium bedded		Bentonite 15.00m - 16.80m	
		16						
		17						
		18						
		19						
		20						

MONITORING WELL LOG

BH ID: BH102M

Location	45-53 Macleay Steet, Potts Point NSW			**Started**	28 July 2025
Client	T&P Chimes Development Pty Ltd			**Completed**	28 July 2025
Job No.	E26863.G04			**Logged By**	MS **Date** 28 July 2025
Sheets	3 of 3			**Review By**	ML **Date** 29 September 2025
Drilling Contractor	Geosense Drilling Engineers	**Surface RL**	≈27.55 m (AHD)	**Northing**	6251016.4 (MGA 2020 Zone 56)
Plant	Comacchio Geo 205	**Inclination**	90°	**Easting**	335859.5 (MGA 2020 Zone 56)

WATER	SAMPLES & FIELD TESTS	DEPTH (m)	GRAPHIC LOG	RL (m AHD)	MATERIAL DESCRIPTION	MOISTURE CONDITION	BACKFILL DETAILS	STANDPIPE DETAILS
		21			From 15.43m, thinly to medium bedded		Sand 16.80m - 25.10m	
		21.70		5.85	From 21.70m, thinly bedded			
		22						
		23						
		24						
		25		2.45	Terminated at 25.10m. Target Depth Reached.			
		26						
		27						
		28						
		29						
		30						

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

CORE PHOTOGRAPH OF BOREHOLE: BH102M

Project	Proposed Development	**East**	335859.5	**Depth Range**	0.43m to 25.10m		
Location	45-53 Macleay Street, Potts Point NSW	**North**	6251016.4	**Contractor**	Geosense Drilling Engineers Pty Ltd		
Position	See Figure 2	**Surface RL**	27.55m	**Drill Rig**	Hanjin D&B 8D		
Job No.	E26863.G04	**Inclination**	90°	**Logged**	MS	**Date**	28/07/2025
Client	T&P Chimes Development Pty Ltd	**Box**	1-2 of 6	**Checked**	ML	**Date**	29/01/2026

CORE PHOTOGRAPH OF BOREHOLE: BH102M

Project	Proposed Development	**East**	335869.2	**Depth Range**	0.7m to 15.10m		
Location	45-53 Macleay Street, Potts Point NSW	**North**	6251032.8	**Contractor**	Geosense Drilling Engineers Pty Ltd		
Position	See Figure 2	**Surface RL**	27.25m	**Drill Rig**	Hanjin D&B 8D		
Job No.	E26863.G04	**Inclination**	90°	**Logged**	MS	**Date**	28/07/2025
Client	T&P Chimes Development Pty Ltd	**Box**	3-4 of 6	**Checked**	ML	**Date**	29/01/2026

CORE PHOTOGRAPH OF BOREHOLE: BH102M

Project	Proposed Development	**East**	335869.2	**Depth Range**	0.7m to 15.10m		
Location	45-53 Macleay Street, Potts Point NSW	**North**	6251032.8	**Contractor**	Geosense Drilling Engineers Pty Ltd		
Position	See Figure 2	**Surface RL**	27.25m	**Drill Rig**	Hanjin D&B 8D		
Job No.	E26863.G04	**Inclination**	90°	**Logged**	MS	**Date**	28/07/2025
Client	T&P Chimes Development Pty Ltd	**Box**	5-6 of 6	**Checked**	ML	**Date**	29/01/2026

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 (%)

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

RQD = Rock Quality Designation (%)

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

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			

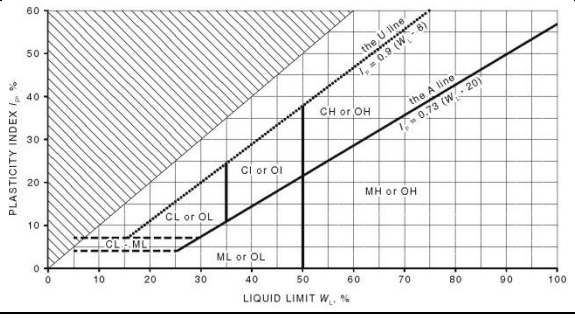
CLASSIFICATION AND INFERRED STRATIGRAPHY

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

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

Point Load Strength Index Report

Project: E26863.G04 45-53 MACLEAY STREET, POTTS POINT NSW
Client: EI AUSTRALIA PTY LTD
Address: SUITE 6.01, 55 MILLER STREET, PYRMONT NSW 2009
Test Method: AS 4133.4.1
Sampling Procedure: Samples Supplied By Client (Not covered under NATA Scope of Accreditation)

Project No.: 31380/1097E-L
Report No.: 25/2006
Report Date: 08/08/2025
Page: 1 0F 2

Borehole / Sample No.	Depth (m)	Date Sampled	Date Tested	Test Type	Is (MPa)	$Is_{(50)}$ (MPa)	Rock Type	Failure Type	Moisture
BH101M	0.87	29/07/2025	05/08/2025	A	**0.15**	**0.15**	SS	3	M
BH101M	1.54	29/07/2025	05/08/2025	A	**0.16**	**0.16**	SS	3	M
BH101M	2.69	29/07/2025	05/08/2025	A	**0.29**	**0.29**	SS	3	M
BH101M	4.38	29/07/2025	05/08/2025	A	**0.47**	**0.47**	SS	3	M
BH101M	5.92	29/07/2025	05/08/2025	A	**0.70**	**0.71**	SS	3	M
BH101M	8.47	29/07/2025	05/08/2025	A	**0.73**	**0.75**	SS	3	M
BH101M	9.83	29/07/2025	05/08/2025	A	**1.00**	**1.00**	SS	3	M
BH101M	10.55	29/07/2025	05/08/2025	A	**0.70**	**0.71**	SS	3	M
BH101M	11.19	29/07/2025	05/08/2025	A	**1.10**	**1.10**	SS	3	M
BH101M	11.76	29/07/2025	05/08/2025	A	**0.05**	**0.05**	SS	3	M
BH101M	12.13	29/07/2025	05/08/2025	A	**0.09**	**0.09**	SS	3	M
BH101M	12.83	29/07/2025	05/08/2025	A	**0.46**	**0.46**	SS	3	M
BH101M	13.37	29/07/2025	05/08/2025	A	**0.8**	**0.81**	SS	3	M
BH101M	14.70	29/07/2025	05/08/2025	A	**1**	**1**	SS	3	M
BH102M	0.54	28/07/2025	05/08/2025	A	**0.2**	**0.2**	SS	3	M
BH102M	2.73	28/07/2025	05/08/2025	A	**0.29**	**0.29**	SS	3	M
BH102M	3.41	28/07/2025	05/08/2025	A	**0.28**	**0.29**	SS	3	M
BH102M	4.80	28/07/2025	05/08/2025	A	**0.29**	**0.3**	SS	3	M
BH102M	5.29	28/07/2025	05/08/2025	A	**0.83**	**0.84**	SS	3	M
BH102M	7.81	28/07/2025	05/08/2025	A	**0.89**	**0.9**	SS	3	M
BH102M	9.25	28/07/2025	05/08/2025	A	**0.53**	**0.53**	SS	3	M
BH102M	10.65	28/07/2025	05/08/2025	A	**0.66**	**0.67**	SS	3	M
BH102M	10.90	28/07/2025	05/08/2025	A	**0.77**	**0.78**	SS	3	M
BH102M	11.28	28/07/2025	05/08/2025	A	**1**	**1**	SS	3	M
BH102M	11.78	28/07/2025	05/08/2025	A	**0.83**	**0.84**	SS	3	M
BH102M	12.37	28/07/2025	05/08/2025	A	**0.045**	**0.045**	ST/YS	3	M
BH102M	12.89	28/07/2025	05/08/2025	A	**0.7**	**0.71**	SS	3	M
BH102M	13.12	28/07/2025	05/08/2025	A	**1.2**	**1.2**	SS	3	M
BH102M	13.75	28/07/2025	05/08/2025	A	**1**	**1**	SS	3	M

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 = Diametrial
I = Irregular
C = Cube

Moisure Condition
W = Wet
M = Moist
D = Dry

Rock Type
SS = Sandstone
ST = Siltstone
SH = Shale
YS = Claystone
IG = Igneous

Remarks:

Approved Signatory.....

Technician: NL

Mrigesh Tamang - Manager

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

Point Load Strength Index Report

Project: E26863.G04 45-53 MACLEAY STREET, POTTS POINT NSW
Client: EI AUSTRALIA PTY LTD
Address: SUITE 6.01, 55 MILLER STREET, PYRMONT NSW 2009
Test Method: AS 4133.4.1
Sampling Procedure: Samples Supplied By Client (Not covered under NATA Scope of Accreditation)

Project No.: 31380/1097E-L
Report No.: 25/2006
Report Date: 08/08/2025
Page: 2 0F 2

Borehole / Sample No.	Depth (m)	Date Sampled	Date Tested	Test Type	Is (MPa)	$Is_{(50)}$ (MPa)	Rock Type	Failure Type	Moisture
BH102M	15.35	28/07/2025	05/08/2025	A	**0.88**	**0.89**	SS	3	M
BH102M	16.59	28/07/2025	05/08/2025	A	**1.20**	**1.20**	SS	3	M
BH102M	18.68	28/07/2025	05/08/2025	A	**1.20**	**1.20**	SS	3	M
BH102M	19.48	28/07/2025	05/08/2025	A	**2.00**	**2.10**	SS	3	M
BH102M	21.50	28/07/2025	05/08/2025	A	**1.10**	**1.20**	SS	3	M
BH102M	23.72	28/07/2025	05/08/2025	A	**2.40**	**2.50**	SS	3	M
BH102M	24.84	28/07/2025	05/08/2025	A	**2.00**	**2.00**	SS	3	M

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 = Diametrial
I = Irregular
C = Cube

Moisure Condition
W = Wet
M = Moist
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Rock Type
SS = Sandstone
ST = Siltstone
SH = Shale
YS = Claystone
IG = Igneous

Remarks:

Approved Signatory.....

Technician: NL

Mrigesh Tamang - Manager

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Uniaxial Compressive Strength of Rock Core

Project: E26863.G04 45-53 MACLEAY STREET, POTTS POINT NSW
Client: EI AUSTRALIA PTY LTD
Address: SUITE 6.01, 55 MILLER STREET, PYRMONT NSW 2009
Test Method: AS4133.1.1, 4.2.2

Project No.: 31380/1097E-L
Report No.: 25/2047
Report Date: 08/08/25
Page: 1 of 3

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

Sample No.	1	2				
Location (BH)	BH102M	BH102M				
Depth (m)	11.0- 11.26	12.0- 12.25				
Sample Description	Sandstone	Sandstone				
Date Cored	28-29/07/2025	28-29/07/2025				
Date Tested	6/08/2025	6/08/2025				
Testing Machine	MAN 8-700026	MAN 8-700026				
Sample Diameter (mm)	51.9	51.9				
Sample Height (mm)	119.3	144.0				
L/D Ratio	2.3	2.8				
Sample Conditioning	Unsoaked	Unsoaked				
Test Duration (min:sec)	9	10				
Failure Description	Single Shear	Single Shear				
Uniaxial Compressive Strength (MPa)	**4.6**	**20.06**				
Moisture Content (%)	**8.9**	**7.2**				
Dry Density (t/m^3)	**2.100**	**2.200**				
Storage Conditions	Plastic Wrapped	Plastic Wrapped				
Other Comments	-	-				

Remarks:

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

Technician: KM

Mrigesh Tamang - Manager

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Unconfined Compressive Strength of Rock Cores

Project:	E26863.G04 45-53 MACLEAY STREET, POTTS POINT NSW	Project No.:	31380/1097E-L
Client:	**EI AUSTRALIA PTY LTD**	Report No.:	25/2047
Address:	SUITE 6.01, 55 MILLER STREET, PYRMONT NSW 2009	Report Date:	8/08/2025
		Page:	2 of 3

BH102M (11.0- 11.26)m

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Unconfined Compressive Strength of Rock Cores

Project:	E26863.G04 45-53 MACLEAY STREET, POTTS POINT NSW	Project No.:	31380/1097E-L
Client:	**EI AUSTRALIA PTY LTD**	Report No.:	25/2047
Address:	SUITE 6.01, 55 MILLER STREET, PYRMONT NSW 2009	Report Date:	8/08/2025
		Page:	3 of 3

BH102M (12.0- 12.25)m

Appendix C Vibration Limits

German Standard DIN 4150 – Part 3: 2016-12 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally considered to be conservative.

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

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

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

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

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

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

Appendix D Important Information

Important Information

SCOPE OF SERVICES

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

RELIANCE ON DATA

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

GEOTECHNICAL ENGINEERING

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

LIMITATIONS OF SITE INVESTIGATION

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

SUBSURFACE CONDITIONS ARE TIME DEPENDENT

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

VERIFICATION OF SITE CONDITIONS

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

REPRODUCTION OF REPORTS

 Where information from the accompanying report is to be included in contract documents or engineering specification for the project, the entire report should be included in order to minimize the likelihood of misinterpretation from logs.

REPORT FOR BENEFIT OF CLIENT

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

OTHER LIMITATIONS

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