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TIME & PLACE

Geotechnical Investigation

101 North Steyne, Manly NSW

E26499.G03
3 December 2024

Document Control

Report Title: Geotechnical Investigation, 101 North Steyne, Manly NSW

Report No: E26499.G03

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Original (Saved to Digital Archives) (Z:\07 - Projects\E26499_Time and Place_Manly_PSI_GI_GME_GTA\05_Deliverables\Work in Progress\G03\E26499.G03 - Geotechnical Investigation Report.docx)	EI Australia Suite 6.01, 55 Miller Street, PYRMONT NSW 2009

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Revision	Details	Date	Amended By
	Original	3 December 2024	

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1. Introduction

1.1 Background

At the request of Time & Place (the Client), EI Australia (EI) has carried out a Geotechnical Investigation (GI) for the proposed development at 101 North Steyne, Manly NSW (the Site).

This GI report has been prepared to provide advice and recommendations to assist in the preparation of designs for the proposed development. The investigation has been carried out in accordance with the agreed scope of works outlined in EI's proposal referenced P22578.2 dated 20 August 2024, and with the Client's authorisation to proceed, dated 20 August 2024.

1.2 Proposed Development

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

- Preliminary architectural drawings prepared by Smart Design Studio – Project Reference 2408 North Steyne 101, Drawing No. DA002, DA099 to DA104, DA400, DA401 and DA450, Latest Revision E, issued for information, dated 10 September 2024;
- Detail site survey plan prepared by Beveridge Williams – Referenced Project No. 2302595, Drawing Ref. DET-001, Sheets 3 of 3, version A, dated 4 December 2023. 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; and
- Utilities investigation survey plan by Beveridge Williams – Referenced Project No. 2302595, Drawing Ref. SUI-001, Sheet 1 of 1, version A, dated 21 August 2024.

Based on the provided documents, EI understands that the proposed development involves the demolition of the existing site structures and the construction of a five-storey residential building overlying a single-level basement. The basement level is proposed to have a Finished Floor Level (FFL) of between RL2.160m in the western end of the basement footprint and RL1.37m AHD in the eastern end of the basement footprint. The car spaces along the eastern end of the basement comprise a series of four car stackers in parallel with an expected FFL of RL-1.13m AHD. A Bulk Excavation Level (BEL) ranging between RL1.8m AHD to the west and RL-2.0m AHD to the east (car stacker) is assumed, which includes allowance for the construction of the basement slab. To achieve the anticipated BEL, excavation depths of between about 4m to 8m Below Existing Ground Level (BEGL) have been estimated. Locally deeper excavations may be required for footings, lift overrun pits, crane pads, and service trenches. The basement footprint extends to all site boundaries except for minor setback from the eastern site boundary.

1.3 Objectives

The objective of the GI was to assess site surface and subsurface conditions at two borehole and one Cone Penetrometer Test (CPT) locations, and to provide preliminary geotechnical advice and recommendations to assist in the design of the proposed development.

1.4 Fieldwork Methodology

The scope of works for the GI included:

- Preparation of a Work Health and Safety Plan;
- Review of relevant geological maps for the project area;
- Site walkover inspection by a Geotechnical Engineer to assess topographical features and site conditions;

- Scanning of proposed boreholes and CPT locations for buried conductive services using a licensed service locator with reference to Before You Dig Australia (BYDA) plans;
- Auger drilling of two boreholes (BH2M and BH3M) 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 Auger drilling and CPT termination depths

Location ID	Approx. Surface RL (m AHD)	Auger Drilling Termination		CPT Termination (Refusal)	
		Depth (m)	RL (m AHD)	Depth (m)	RL (m AHD)
CPT1	5.85	-	-	14.62	-8.77
BH2M	5.8	7.5	-1.7	-	-
BH3M	6.0	10.0	-4.0	-	-

  - Measurements of groundwater seepage/levels, where possible, in the augered sections of the boreholes during and shortly after completion of auger drilling;
  - The approximate surface levels shown on the borehole logs were interpolated from spot levels shown on the supplied survey plan. Approximate boreholes and CPT locations are shown on **Figure 2**;

- One Cone Penetration Test probe (CPT1) was carried out by Stratacore Pty Ltd. Testing comprised pushing a 35 mm diameter rod with a cone sensor tip. The CPT soundings provide a continuous profile of cone and sleeve resistance, and allow interpretation of engineering properties. The CPT refusal can indicate bedrock surface, although refusal can occur on hard layers or obstruction in fill and not necessarily on bedrock. The approximate surface levels shown on the CPT plot were approximated from spot levels shown on the supplied survey plan which formed the basis of **Figure 2**. The CPT test was pushed to the refusal depth as shown in **Table 1-1**:
- Borehole BH2M and BH3M were converted into groundwater monitoring wells to allow for long-term groundwater monitoring. The screen depths of the wells are outlined in **Table 1-2**.

Table 1-2 Well Installation Depths

Borehole ID	Surface RL (m AHD)	Auger Drilling	
		Top of Screen Depth (m)	Bottom of Screen Depth (m)
BH2M	5.8	3.0	6.0
BH3M	6.0	3.0	6.0

- Soil samples were sent to STS Geotechnics Pty Ltd (STS) and SGS Australia (SGS), which are National Australian Testing Authority (NATA) accredited laboratories, for testing and storage.
- A pump-out test was carried out within monitoring well BH1M one week after installation of the monitoring well to determine the groundwater inflows of the surrounding material.
- Preparation of this GI report.

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

1.5 Constraints

The GI was limited by the intent of the investigation and the presence of existing site structures and underground assets which limited access to the subsoils in some parts of the site resulting in restricted site coverage. The discussions and advice presented in this report are preliminary and intended to assist in the preparation of initial designs for the proposed development. A supplementary geotechnical investigation is required post-demolition in the remaining areas of the site not previously investigated during the GI to complement the GI findings and updating of geotechnical advice and recommendation as appropriate. 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	101 North Steyne, Manly NSW
Lot and Deposited Plan (DP) Identification	SP4518
Brief Site Description	At the time of our investigation, the site was occupied by a three-storey brick residential flat building. Two driveways are present, one on either side of the building, and an open space at-grade car parking area in the western portion of the site. Undercover car spaces are also present at ground level under the western portion of the residential flat building. Open spaces across the site are asphalt paved.
Site Area	The site area is approximately 636.3 m^2 (based on the provided survey plan referenced above).

Plate 1 Aerial photograph of the site (source: SixMaps, accessed on 17 October 2024)

2.2 Local Land Use

The site is situated within an area of residential use. Current uses on surrounding land at the time of our presence on site are described in **Table 2-2** below. For the purpose of this report, the site boundary adjacent to North Steyne 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	102 to 104 North Steyne – a four storey residential apartment buildings with at least one basement level which is accessed via Pine lane to the west of that property. The basement structure abuts the common site boundary while the building is offset by approximately 2.0m.
East	North Steyne is an asphalt surfaced roadway, with one carriageway in each traffic direction, with parallel kerb side parking lane in the northbound traffic direction of the roadway, and a dedicated perpendicular parking bays setback from the verge in the southbound traffic direction. Further east of the roadway is Manly Beach separated by a wide grass reserve.
South	98 – 100 North Steyne – A five storey residential apartment building with at least one basement level which is accessed via Pine Lane to the west of that property. The property has some medium size trees along the common site boundary with the site. The building abuts the site boundary for most of its length.
West	Pine Lane a narrow single lane roadway with no parking in either direction. Beyond the roadway are one to two-storey residential dwellings of clad construction with detached garage that fronts onto Pine Lane.

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 near flat topography with site levels varying from RL5.6m AHD along the Pine Lane frontage to about RL5.9m AHD along the North Steyne fronatge.
Regional Geology	Information on regional sub-surface conditions, referenced from the Department of Mineral Resources Geological Map Sydney 1:100,000 Geological Series Sheet 9130 (DMR 1983) indicates the site to be underlain by modern marine and estuarine deposits consisting of coarse sand and varying shale fragments.

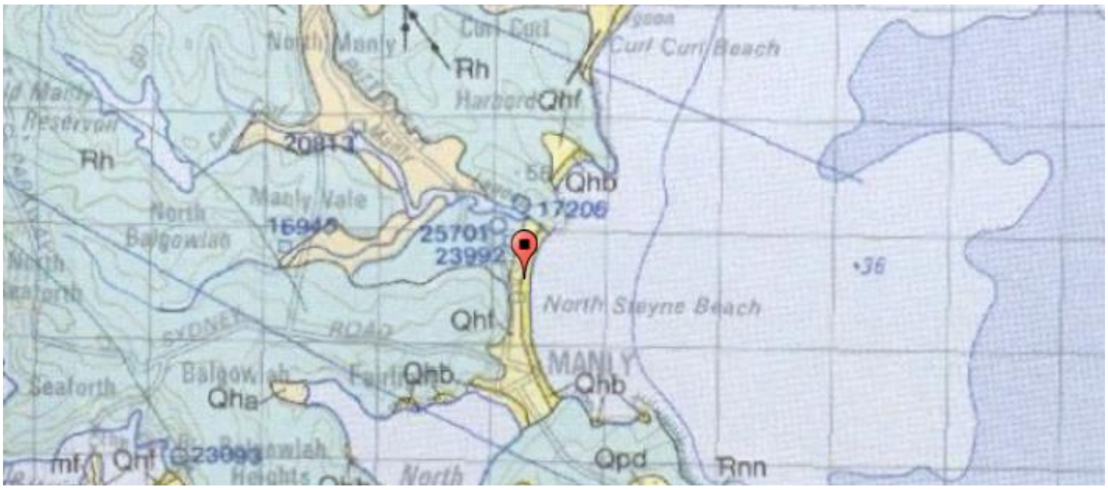


Plate 2 Excerpt of geological map showing location of site.

3. Investigation Results

3.1 Stratigraphy

For the development of a site-specific geotechnical model, the stratigraphy observed in the GI has been grouped into five 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 and in the CPT plot for CPT1 as 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 during this investigation is provided in **Table 3-1.**

Table 3-1 Summary of Subsurface Conditions

Unit	Material[2]	Depth to Top of Unit (m BEGL)[1]	RL of Top of Unit (m AHD)[1]	Observed Thickness (m)	Comments
1	**Fill**	0.00	5.8 to 6.0	0.5 to 0.7	Fill was generally observed within BH2M and BH3M beneath a 200mm thick asphalt concrete surfacing. Fill comprised generally silty sand. A layer of sandstone gravels was observed within the fill stratum in BH3M only. EI note due to the nature of CPT, the thickness of the fill stratum cannot be observed though we expect similar conditions as observed in BH2M and BH3M.
2	**Variable Very Loose to Medium Dense Sand**	0.5 to 0.7	5.3	3.3 to 4.0	Fine to medium grained, variable loose to very loose conditions was observed within BH2M and BH3M only, with SPT N values within this material ranging between 4 and 6. Medium dense sand was observed within this layer inCPT1
3	**Medium Dense to Very Dense Sand**	3.5 to 4.5	1.3 to 2.35	3.5 to 4.0	Interbedded medium dense and very dense sands were observed within CPT1 as materials with a friction ratio of less than 1% and a cone resistance of between a minimum 5MPa. Fine to medium grained, medium dense sand was observed within BH3M, with SPT N values within this material ranging between 16 and 28. Dense sand was encountered in BH2M as determined by SPT at depth of 4.5m BEGL.
4	**Very Loose to Loose Sand**	7.5	-1.5 to -1.65	3 (CPT1)	Loose sands were observed within BH3M with recorded SPT N values of 9. CPT1 recorded cone resistance of less than 2.5MPa
5	**Medium Dense Sand (CPT1 only)**	7.50	-1.70	2.8	Medium dense sands were observed within the CPT location as materials with a friction ratio of less than 1% and a cone resistance of between 5MPa and 12MPa.
6	**Very Dense Sand (CPT1 only)**	13.50	-7.70	-[3]	Very Dense Sand was observed within the CPT location as materials with a friction ratio of less than 1% and a cone resistance of greater than 20MPa.

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 CPT1.

3.2 Groundwater Observations

Groundwater seepage was observed during auger drilling of BH2M and BH3M. The depth of groundwater seepage during augering is recorded on the borehole logs in **Appendix A**.

Following completion of auger drilling, boreholes BH2M and BH3M were left open and groundwater levels measured and are summarised in **Table 3-2** below. The groundwater level within CPT1 has been inferred from the pore water pressure collected during the testing.

Table 3-2 Groundwater Measurements Within the Boreholes and CPT1

Borehole ID	Depth to Groundwater Seepage Level During Augering and Probing
CPT1	5.0m (RL 0.8m AHD)
BH2	5.1m (RL 0.7m AHD)
BH3	4.9m (RL 1.1m AHD)

3.3 Test Results

Five soil samples collected during the fieldwork were selected for laboratory testing to assess the following:

- Soil aggressivity (pH, chloride and sulfate content and electrical conductivity).
- Particle Size Distribution (PSD)

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

Table 3-3 Summary of Soil Laboratory Test Results

Test / Sample ID	CPT1/BH1M_1.4-1.5[2]	BH2M_0.5-0.95	BH2M_4.5-4.95	BH3M_1.5-1.95	BH3M_6.0-6.45
Unit	2	2	3	2	3
Material[1]	Marine Soil	Marine Soil	Marine Soil	Marine Soil	Marine Soil
USCS Description	SAND	SAND	SAND with Silt	SAND with Silt	SAND
Aggressivity					
Chloride Cl (ppm)	2.6	33	-	-	5.5
Sulfate SO_4 (ppm)	<5	49	-	-	5.6
pH	9.1	6.8	-	-	9.3
Electrical Conductivity (µS/cm)	52	160	-	-	73
% Moisture	5.4	2.1	-	-	18.8
Particle Size Distribution					
Gravel (%)	-	-	0	0	-
Sand (%)	-	-	93	93	-
Clay & Silt	-	-	7	7	-

Test / Sample ID	CPT1/BH1M_1.4-1.5[2]	BH2M_0.5-0.95	BH2M_4.5-4.95	BH3M_1.5-1.95	BH3M_6.0-6.45
(%)					

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

Note 2 Sample collected adjacent to CPT1 in BH1M, which was undertaken as a part of the Preliminary Site Investigation E26499.E01, 30 October 2024.

The results of the particle size distribution test on a sample collected from borehole BH2M and BH3M between the depths of 4.5m to 4.95m (BH2M) and 1.5m to 1.95m (BH3M) respectively indicated the material constitutes poorly graded sand with trace silt.

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

- ‘Mild’ for buried concrete structural elements; and
- ‘Severe’ for buried steel structural elements.

4. Recommendations

4.1 Geotechnical Considerations

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

- Presence of very loose to loose sands;
- Basement excavation and retention;
- Groundwater within the depth of the excavation; 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 Footings and Structures

Prior to any excavation, we recommend that the as-built drawings to be obtained for the neighbouring building to the north (102 to 104 North Steyne) and the neighbouring building to the south (98-100 North Steyne) to understand their basement footprint and depth.

Due to the limited setback distance of the neighbouring building(s) to the common site boundary, EI notes that the footings of the neighbouring building(s) should be investigated prior excavations. Should the adjacent building footings be located within the zone of influence of excavations for the subject development, underpinning of the neighbouring properties building(s) may be required prior to the installation of the shoring system.

4.5 Excavation Methodology

4.5.1 Excavation Assessment

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

The basement level is proposed to have a Finished Floor Level (FFL) of between RL2.160m in the western end of the basement footprint and RL1.37m AHD in the eastern end of the basement footprint. The car spaces along the eastern end of the basement comprise a series of four car stackers in parallel with an expected FFL of RL-1.13m AHD. A Bulk Excavation Level (BEL) ranging between RL1.8m AHD to the west and RL-2.0m AHD to the east (car stacker) is assumed, which includes allowance for the construction of the basement slab. To achieve the anticipated BEL, excavation depths of between about 4m to 8m Below Existing Ground Level (BEGL) have been estimated.

Based on the borehole logs, the proposed basement excavations will therefore extend through material units 1, 2 and possible 3 in the western and central portion of the site where the BEL is assumed to be varying from about RL1.8m to RL1.0m AHD as outlined in **Table 3-1**. The car stacker excavation at the eastern end of the site is expected to encounter material units 1 through to 4. As such, an engineered retention system must be installed prior to excavation commencing to support the full height of the excavation.

All units could be excavated using buckets of large earthmoving Hydraulic Excavators.

The primary issues associated with the excavation will be controlling the groundwater that may be encountered towards the bulk excavation level broadly across the basement footprint, and particularly in the eastern end of the site where the car stacker excavation will extend below the water table, and provide adequate support to adjoining structures/infrastructures. To allow for the construction of the basement slab, lift pits, service trenches, and the car stacker in ‘dry’ condition, temporary dewatering will be required. In this regards, it is anticipated that the groundwater table will be maintained at a depth of about 1m below the bulk excavation level and deeper around lift pits and car stacker. Placement of a course of recycled hard wearing aggregates above the excavation floor may assist trafficability during basement earthworks.

Dewatering has the potential to cause some drawdown and ground settlement below the adjoining sites. The extent of the drawdown depends upon the depth to which the cut-off system is installed and the pumping operations. A dewatering assessment is required to inform design of the cut-off system and evaluation of dewatering induced ground settlements beyond the excavation boundary.

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;
- After excavation to the first row of supports or anchors, but prior to installation of these supports or anchors;
- After excavation to any subsequent rows of supports or anchors, but prior to installation of these supports or anchors;
- After installation of the retaining structures, but before commencement of excavation;
- After excavation to the base of the excavation;
- After de-stressing and removal of any rows of supports or anchors; and
- One month after completion of the permanent retaining structure or after three consecutive measurements not less than a week apart showing no further movements, whichever is the latter.

4.6 Groundwater Considerations

Groundwater was observed in the two monitoring wells as detailed in **Table 3-2**, which is at a level below the assumed BEL of RL1.8m AHD in the western end of the basement footprint, but will be in close proximity of the assumed BEL of RL1.0m AHD in the central and eastern portion of the basement footprint. Car stacker excavations in the eastern end of the basement footprint will be below the groundwater table.

The volume of groundwater entering the basement excavation (which will require dewatering) decreases as the depth of embedment of the perimeter shoring system increases.

Council and the NSW Department of Climate Change, Energy, the Environment and Water (DCCEEW), formerly Department of Planning and Environment (DPE) do not allow permanent dewatering; therefore, the basement must be designed as a tanked structure. Temporary dewatering for construction purposes is normally allowed provided it is properly designed and managed to ensure that the likely drawdown will have no adverse impact on adjoining structures/infrastructures. A dewatering licence may also be required. Groundwater quality testing, particularly with regard to acidity generated as a result of acid sulfate soils, will be required to permit discharge into the stormwater.

Spear points will need to be installed internally around the perimeter of the retention system, as well as possibly internally for the lift overrun pits. The spear points should be connected with a header pipe to allow the pumped water to discharge into the stormwater system, or to the recharge wells/infiltration trenches.

Dewatering has the potential to cause some drawdown and ground settlement below adjoining sites; the extent of the drawdown depends upon the depth to which the cut-off system is installed and the pumping operations. Settlements would affect any adjoining buildings supported on shallow footing systems and if records of the footing systems of the adjoining buildings are available, these should be reviewed to assess the risk from dewatering.

A critical factor relating to dewatering of the site is maintenance of the depressed groundwater levels until such a time as the building has significant weight to prevent movement should the pump system fail and the groundwater level rise.

A detailed monitoring program should be implemented to identify the risks and trigger levels decided for when the contingency measures need to be taken.

4.6.1 Drawdown and Settlement

The groundwater level within the basement outline should be lowered to at least 1.0m below the basement level, prior to commencement of the excavation, to allow for construction within dry conditions. Groundwater modelling provides an estimate of the pumping rate required to lower groundwater level below the base of proposed excavation.

We would suggest that advice on dewatering is sought from a specialised contractor. However, it should be noted that lowering groundwater levels outside the site perimeters could affect (settlement) the foundations of nearby structures and roads etc. Therefore, dewatering of the site without cut-off walls might not be acceptable in a built up area, including the proposed development site.

Appropriate cut-off walls for the proposed excavation may consist of installation of a watertight permanent retaining wall, such as a secant grout injected pile wall, bentonite diaphragm wall etc, as outlined in **Section 4.7** below. Groundwater modelling provides an estimate of desired cut-off wall embedment (below base of excavation) to ensure that the groundwater inflow into the excavation and drawdown is tolerable.

Trafficability problems could arise locally during wet weather, or if water is allowed to pond on these materials.

It is recommended that groundwater levels outside the excavation in the vicinity of the adjacent properties be monitored and kept to less than 1m below the normal groundwater levels. The following general procedure is recommended to monitor groundwater drawdown levels.

- Install standpipes in accessible areas on the adjacent properties to monitor groundwater drawdown levels during dewatering;
- Measure ground levels on a weekly basis for three week prior to the operation of the dewatering system to establish baseline pre-development levels;
- Measure groundwater levels twice a day during the first two days of dewatering, then daily during the first week of dewatering and then twice weekly until decommissioning of the dewatering pumps, or until a lesser frequency is advised by the geotechnical engineer;
- The measured values are to be provided to the geotechnical engineer on the day of measurement for review; and,
- Where drawdown levels exceed 1m (trigger level) below pre-development groundwater levels, the change in groundwater level should be investigated and measures put in place to rectify the exceedance. These measures could include reducing of pumping rates or the suspension of dewatering.

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.

Based on the provided architectural plans, the proposed basement outline extends to all site boundaries except for minor setback from the eastern site boundary.

Based on the above, the close proximity of the surrounding buildings, the encountered subsurface conditions, presence of the groundwater table that will be intersected during basement bulk excavations, and the required excavation depth, temporary batters are not recommended for this site. Unsupported vertical cuts in sand will not be achievable and will most certainly slump below the groundwater table. Collapse of the material may result in injury to personnel and/or damage to nearby structures/infrastructures and equipment.

Due to the sandy nature of the soils and the groundwater table within the proposed excavation depth, suitable retention/cut-off system may comprise of the following:

- Due to the good interlock between adjoining panels and the generally good finish that is achieved with CSM walls, the may be preferred for this site, provided the long term integrity of this form of construction is addressed.
- Secant Pile Walls: Alternate piles are first drilled and concreted at a close spacing. The intermediate piles are then installed by drilling out the soil between each pair and part of the already installed piles. Should the second ‘hard’ piles disengage from the first ‘soft’ piles, then remedial works would be required to rectify any seepage inflows. Any gaps between the piles may result in loss of material and water inflow from behind the wall which may lead to settlements adjacent to the wall and may result in damage to neighbouring structures and services. The resulting out of position piles may also affect internal layout/clearances.
- Steel Sheet Piles: sheet piles may be considered provided the sheets together with supplementary support system such as anchors and/or bracing are sufficiently stiff, and provided vibration issues during installation can be addressed. Vibration monitoring must be carried out and strict adherence to vibration limits. Should excessive vibration and/or ground movements be detected during installation, the installation must be aborted immediately and seek further advice from the designers.

The retention system will need to be installed to depths which satisfy stability, piping, founding and groundwater cut-off considerations. Anchors/props and shotcrete must be installed progressively as excavation proceeds. Considering that the neighbouring basements is likely to extend to meet the common northern and southern site boundaries, anchoring of the subject basement shoring wall along these elevations will be not possible and therefore would be limited to internal propping / bracing.

Only double-rotary encased grout injected CFA piles should be used for this site should a secant pile retention/cut-off system be considered. Due to the collapsible nature of the sandy soils and the presence of groundwater, bored piers are not recommended for this site. The proposed pile locations should take into account the presence of the neighbouring anchors and/or the presence of buried services. Further advice should be sought from prospective piling contractors who should be provided with a copy of this report.

4.7.2 Retaining Wall Design Parameters

The following parameters 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.

- For progressively anchored or propped walls where minor movements can be tolerated (provided there are no buried movement sensitive services), we recommend the use of a rectangular earth pressure distribution of 6H kPa for the soil profile, where H is the retained height in meters;
- For progressively anchored or propped walls which support areas which are highly sensitive to movement (such as areas where movement sensitive structures or infrastructures or buried services are located in close proximity), we recommend the use of a rectangular earth pressure distribution of 8H kPa for the soil profile, where ‘H’ is the retained height in meters;
- 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.

- Full hydrostatic pressures should be taken into consideration in the design of the retaining walls, assuming an external water level, say at least 1.0m, above the highest groundwater level measured to date. The hydrostatic pressure should extend to the base of the perimeter cut-off.
- The lateral toe resistance of the retention system can be achieved by sufficient embedment below the ground in front of the wall. For embedment depth design, a triangular lateral earth pressure distribution should be adopted, with a ‘passive’ earth pressure coefficient, K_p, as shown in **Table 4-1**, for sands, assuming horizontal ground in front of the wall. The effect of the groundwater must also be taken into account. We note that significant deflection is required in order to mobilise the full ‘passive’ pressure of a soil, and therefore a factor of safety of at least 2 should be adopted. The ‘passive’ pressure due to the upper 0.5 m below bulk excavation level should be ignored in the analysis to take excavation tolerances into account. All localised excavations in front of the wall (such as pile rig working platform, buried services, footings, lift overrun pits, etc.) should also be taken into account in the wall design.
- If temporary anchors extend beyond the site boundaries, then permission from the neighbouring properties would need to be obtained prior to installation. Also, the presence of neighbouring basements and/or services and their levels must be confirmed prior to finalising anchor design**.**
- Anchors should have their bond length within medium dense or better sand. For the design of anchors within sand, the drained friction angle 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 locking off at about 80% of their working load. Such proof loading is to be witnessed by and engineer independent of the anchoring contractor. Lift-off tests should be carried out on at least 10% of the anchors 24 to 48 hours following locking off to confirm that the anchors are holding their load. Usually anchors are commissioned on design and construct basis so that failure of anchors to hold their load does not then become a contractual issue. We recommend that only experienced contractors be considered for anchor design, specification and installation with appropriate insurances;
  - If permanent anchors are to be used, these must have appropriate corrosion provisions for longevity;
  - **Uncased anchor holes within the sands will almost certainly collapse and temporary casing of these holes will be required. It is good practice for anchors to be a “design and construct” sub-contract to avoid disputes should anchors fail to hold their test load.**

Table 4-1 Geotechnical Design Parameters

		Unit 1	Unit 2	Unit 3	Unit 4	Unit 5	Unit 6
Material [1]		Fill	Variable Very Loose to Medium Dense Sand	Medium Dense to Very Dense Sand	Very Loose to Loose Sand	Medium Dense Sand (CPT1 only)	Very Dense Sand (CPT1 only)
RL of Top of Unit (m AHD) [2]		5.8 to 6.0	5.3	1.3 to 2.35	-1.5 to -1.65	-1.7	-7.7
Bulk Unit Weight (kN/m^3)		16	16	19	15	18	20
Friction Angle, ϕ' (°)		28	28	35	28	30	37
Earth Pressure Coefficients	At rest, K_o [3]	0.53	0.53	0.43	0.53	0.5	0.4
	Active, K_a [3]	0.36	0.36	0.27	0.36	0.33	0.25
	Passive, K_p [3]			3.69	2.77	3	4
Young's Modulus, E (MPa)		7	10	50	7	50	100
Poisson's Ratio		0.35	0.35	0.3	0.35	0.3	0.25
Ultimate Bond Stress (kPa)[4]		-	-	150	-	100	200
Earthquake Site Risk Classification		AS 1170.4:2007 indicates earthquake subsoil Class D_e.(Deep Soil Site) AS 1170.4:2007 indicates the hazard factor (z) for Sydney is 0.08.					

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 Ultimate bond stress should be reduced by 50% for bond zone below the groundwater table.

4.8 Foundations

Raft slabs are well suited to uniform slab conditions and building loads. Further detailed evaluation of expected performance including the evaluation of allowable bearing pressures and settlements would be required once design loads, founding level, and column layout are better known.

In the case of a piled stiffened raft slab, the piles are designed to their ultimate capacity and act as settlement reducers to the stiffened raft slab.

The subgrade preparation below any raft slabs will be important in the final performance of the raft. Detailed analysis of a piled raft would be required to estimate the settlements and the contact pressures below the raft. Further discussion regarding sub-grade preparation is provided in **Section 4.10** below.

For sand subgrades, a 150mm thick layer of good quality granular material such as recycled concrete or crushed rock should be placed and compacted over the prepared surface, particularly at heavily loaded areas. This layer helps confine the sandy soils from disturbances and improve the compacted and density of the surface soils. Alternatively, a concrete blinding layer of 100mm thickness may be used.

Alternatively, the proposed development may be supported using piled footings founded in sands of at least medium dense or better. Based on the encountered subsurface conditions, double rotary grout injected cased CFA piles are recommended for this site. Due to the collapsible nature of the sands and the presence of groundwater, bored piles are not recommended for this site.

Parameters including end bearing and skin friction can only be properly assessed once pile loading, dimensions and layouts are better defined, and are dependent on soil densities, groundwater levels, and the presence of any weaker/softer layers beneath. Pile design should be completed by an experienced pile design engineer. Further detailed assessment of the expected performance of such designs would be required once design loads are finalised. As a guide, concrete CFA piles of 0.5m diameter founded in medium dense sand below the very loose to loose sand layer that extends to a depth of 10.5m BEGL would have an allowable end bearing capacity of about 800kPa. Refined bearing pressures can be determined following further geotechnical investigation comprising additional CPT probing across the site.

All piles must be designed in accordance with the Australian Standard AS2159-2009 Piling – Design and installation.

4.9 Basement Floor Slab

Following bulk excavations for the proposed basement, Unit 2 very loose to loose sands above the groundwater are expected to be exposed at BEL in the western portion of the site. The central and eastern portion of the site is expected to expose medium dense sand (Unit 3) in near vicinity of the groundwater table. Loose sand can be expected at BEL within the footprint of the proposed car stacker footprint. Considering that the groundwater level is likely to be tidal, we therefore recommend that the basement floor slab should be designed fully tanked and the design is likely to be controlled by the hydrostatic uplift pressures. A pile rig working platform is likely to be required at BEL should foundation piles be constructed from that level.

4.10 Subgrade Preparation and Engineered Fill

4.10.1 Subgrade Preparation for Raft Slab

If raft slab foundations are adopted, it is recommended that they be founded on medium dense sands or better.

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

Our recommendations regarding subgrade preparation are as follows:

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

5. Further Geotechnical Inputs

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

- Additional Geotechnical Investigation in the form of three additional CPTs to enable development of a more refined ground model and to confirm the ground conditions across the site;
- Obtain and review as-built drawings of the neighbouring building
- Groundwater measurements, long term groundwater monitoring and seepage modelling. Where required, additional groundwater monitoring wells may need to be installed.
- Dilapidation surveys;
- Design of working platforms (if required) for construction plant by an experienced and qualified geotechnical engineer;
- Numerical analysis for the design of retaining walls and pile rafts;
- Classification of all excavated material transported off site;
- Witnessing installation of support measures and proof-testing of anchors (if required).
- Geotechnical inspections of all new footings/piles by an experienced geotechnical professional before concrete or steel are placed to verify their bearing capacity and the in-situ nature of the founding strata; and
- Ongoing monitoring of groundwater inflows into the bulk excavation;

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

6. Statement of Limitations

This report has been prepared for the exclusive use of Kim Zoljalali and Time & Place 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 Kim Zoljalali and Time & Place

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

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

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

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

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

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

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

References

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

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

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

AS3600:2018, *Concrete Structures,* Standards Australia

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

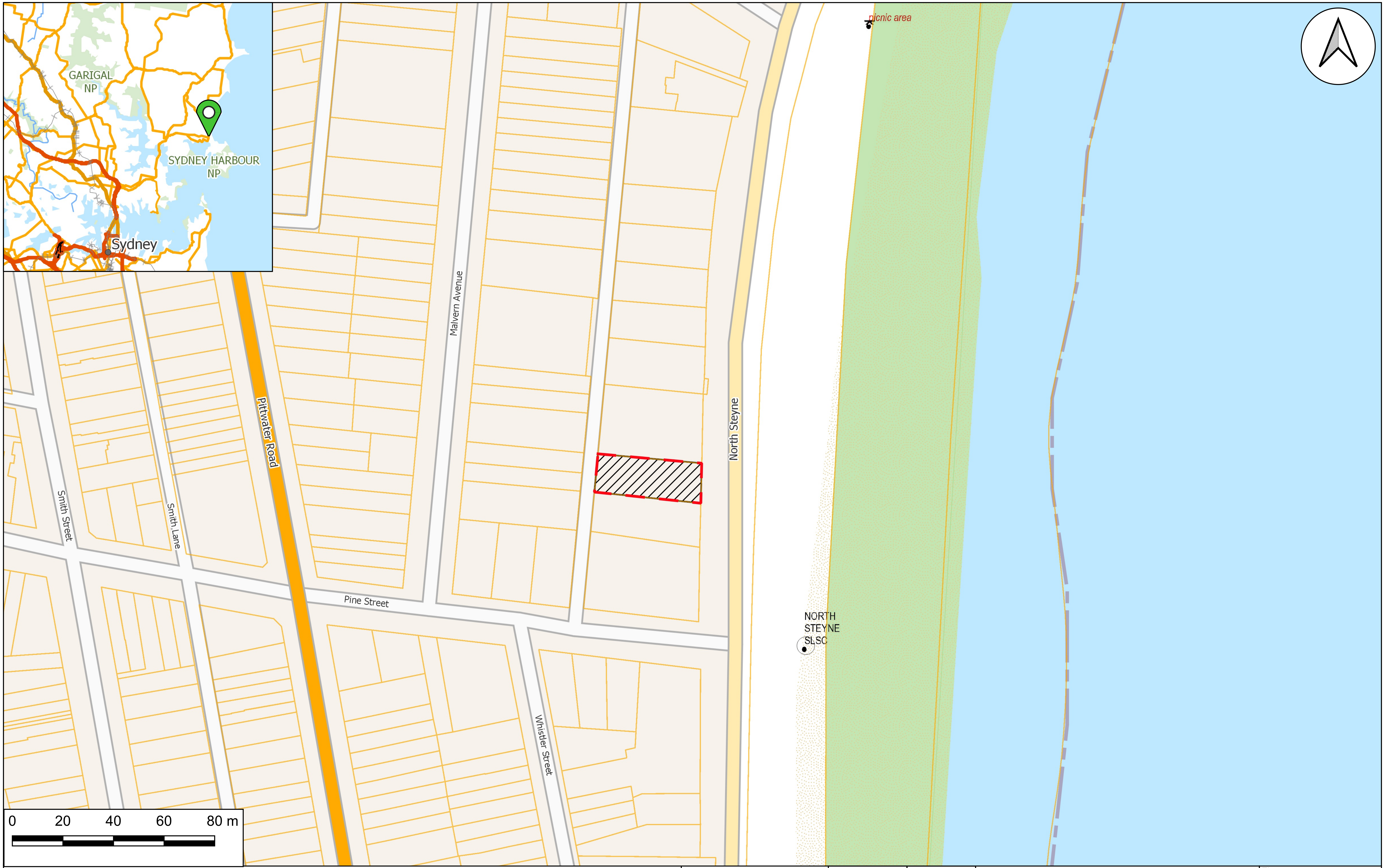
NSW Department of Finance and Service, Spatial Information Viewer, maps.six.nsw.gov.au.

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

Abbreviations

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

Figures



LEGEND. Note: All locations are approximate

▨ Site Boundary

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

Drawn:	A.C.
Approved:	K.X.
Date:	5/11/2024

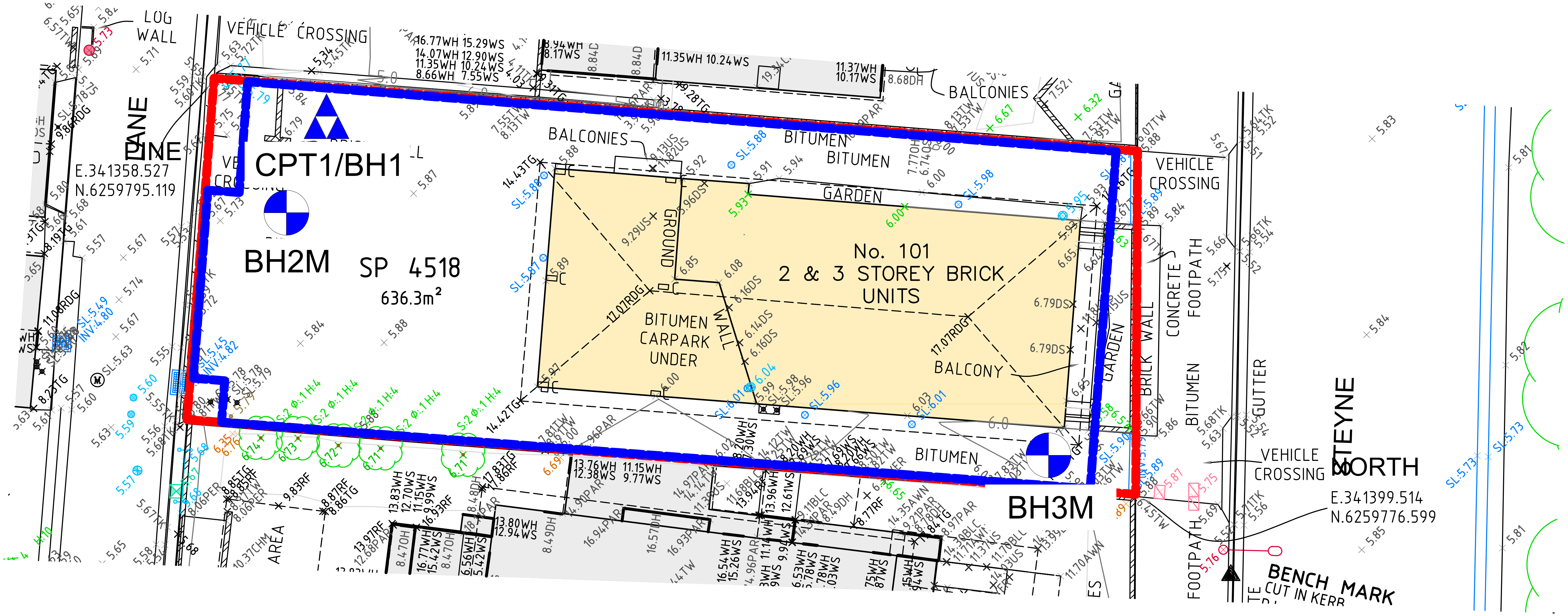
Time & Place

101 North Steyne, Manly NSW

NSW Site Locality Plan

Figure: 1

Project: E26499.G03



0 5 10

Approx. Scale (m)

Map Source:

LEGEND (All Locations are Approximate)

◐	Borehole/DCP location
▲	CPT location
- - - -	Site Boundary
- - - -	Basement Boundary

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Approved:	X.X.
Date:	1710/2024

Time and Place Pty Ltd
New Developement
101 North Steyne Street, Manly, NSW
Test Location Plan

Figure:
2

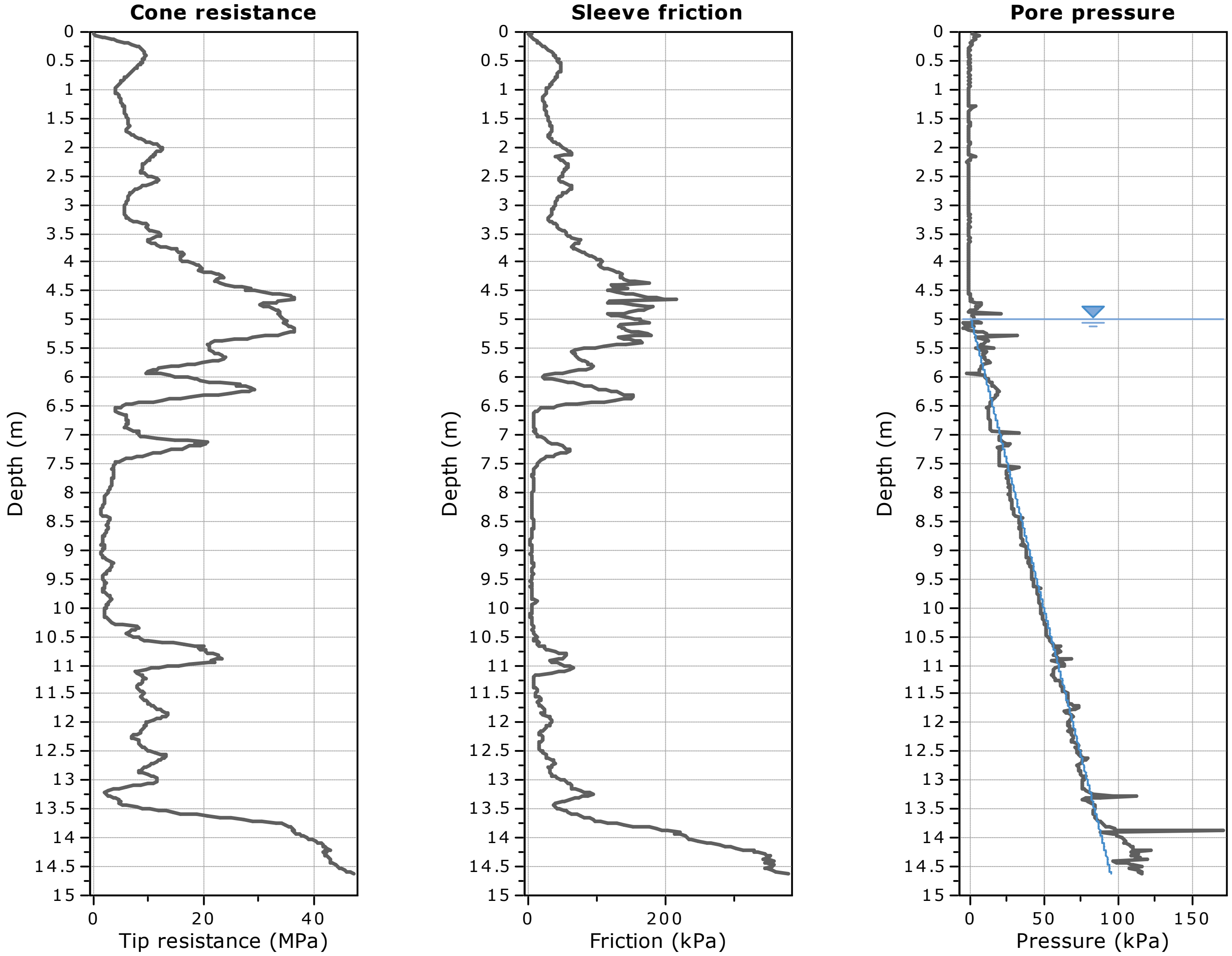
Project: E26499.G03

Appendix A Borehole Logs And Explanatory Notes

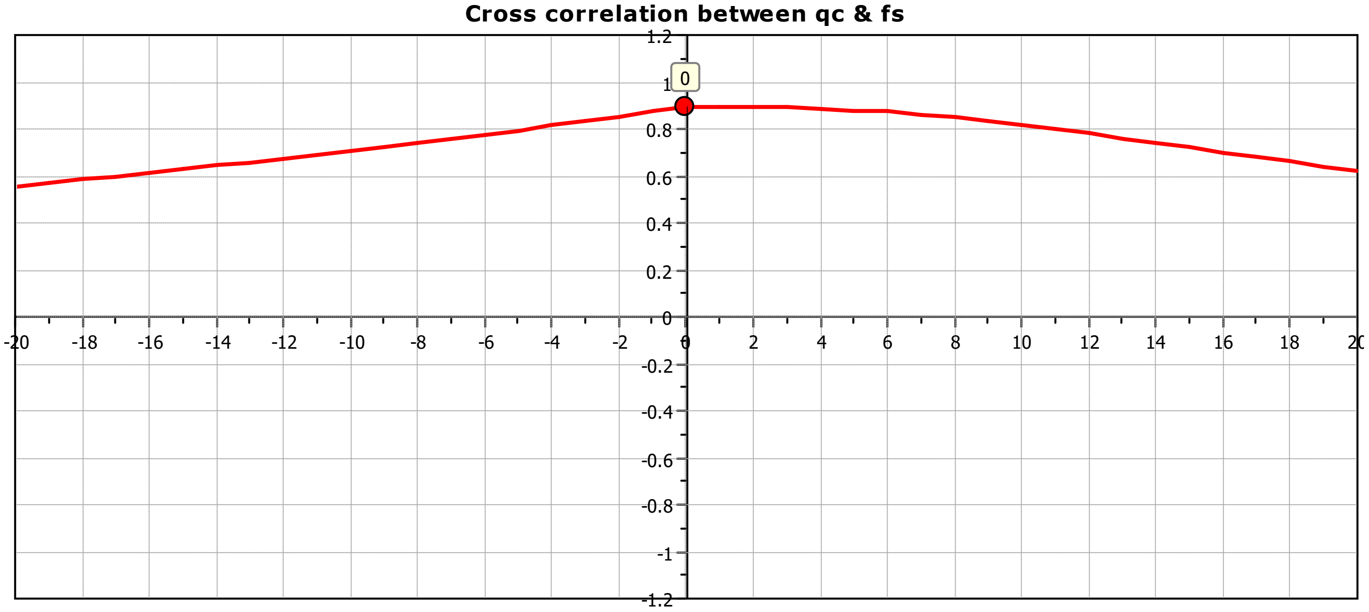
EI Australia
Suite 6.01, 55 Miller Street,
PYRMONT NSW 2009
www.eiaustralia.com.au

Project: **E26499**
Location: **Manly**

CPT: CPT01_mod
Total depth: 14.62 m, Date: 26/9/2024
Surface Elevation: 5.85m
Cone Type: Piezocone
Cone Operator: Stratacore



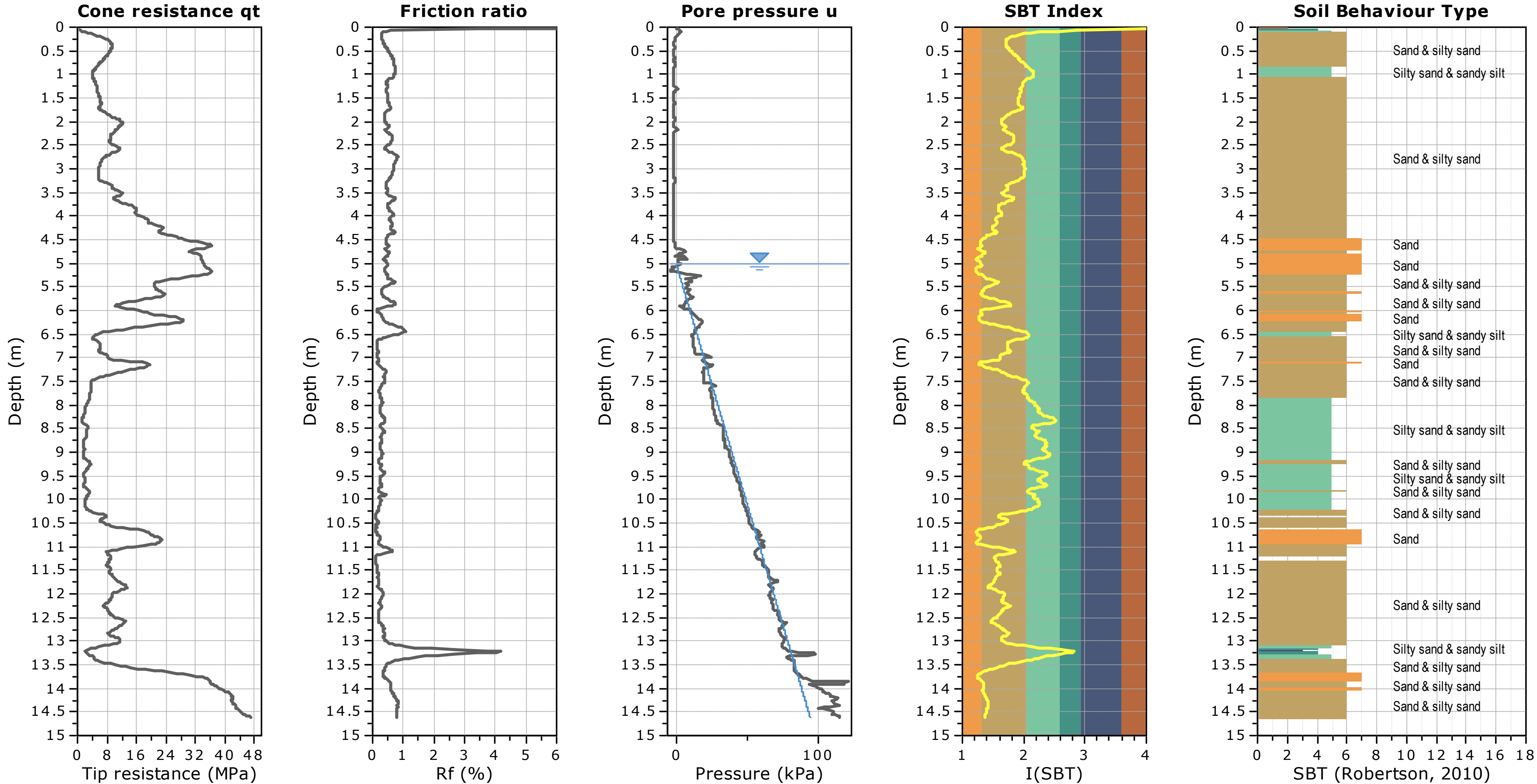
The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



EI Australia
Suite 6.01, 55 Miller Street,
PYRMONT NSW 2009
www.eiaustralia.com.au

Project: **E26499**
Location: **Manly**

CPT: CPT01_mod
Total depth: 14.62 m, Date: 17/10/2024
Surface Elevation: 5.85m
Cone Type: Piezocone
Cone Operator: Stratacore



SBT legend

🟥 1. Sensitive fine grained	4. Clayey silt to silty clay	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to clayey sand
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

BOREHOLE LOG

BH ID: BH2M

Location 101 North Steyne, Manly, NSW
Client Time & PlacePty Ltd
Job No. E26499.G03
Sheets 1 of 1

Started 08 October 2024
Completed 08 October 2024
Logged By LL **Date** 08 October 2024
Review By ML **Date** 22 November 2024

Drilling Contractor Geosense Drilling Engineers
Plant Comacchio Geo 205
Surface RL ≈5.80 m (AHD)
Inclination 90°
Latitude -
Longitude -

METHOD	GROUND WATER LEVELS	SAMPLES & FIELD TESTS	DEPTH (m)	RL (m AHD)	MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY / REL. DENSITY	MATERIAL ORIGIN & OBSERVATIONS
AD/T			0.00	5.80	ASPHALT: 200mm thick	-	-	ASPHALT
		BH2M_0.30-0.40	0.20	5.60	FILL: Silty SAND: fine to medium grained, grey brown	M	-	FILL
		BH2M_0.50-0.95 SPT 0.50-0.95 2,2,2 N=4	0.50	5.30	SAND: fine to medium grained, brown		VL	MARINE SOIL
		BH2M_1.50-1.95 SPT 1.50-1.95 2,2,3 N=5	1.50	4.30	From 1.50m to 1.50m, pale brown		L	
		BH2M_2.50-2.60						
		BH2M_3.00-3.45 SPT 3.00-3.45 2,2,3 N=5						
		BH2M_4.00-4.10						
		BH2M_4.50-4.95 SPT 4.50-4.95 4,13,22 N=35	4.50	1.30	From 4.50m, pale brown / brown		D	
	▷					W	MD	
			7	-1.20	Terminated at 7.00m. Target Depth Reached. Terminated at 7.00m. Target Depth Reached.			

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

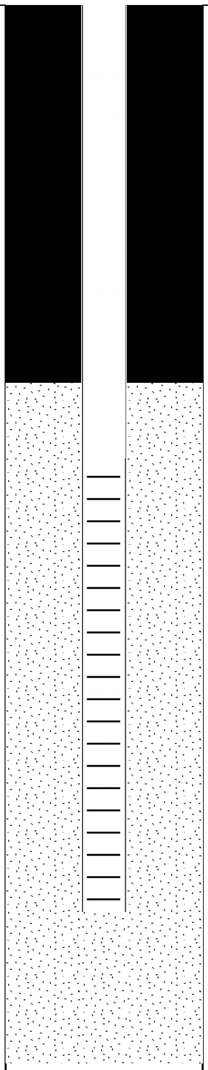
MONITORING WELL LOG

BH ID: BH2M

Location	101 North Steyne, Manly, NSW	**Started**	08 October 2024		
Client	Time & PlacePty Ltd	**Completed**	08 October 2024		
Job No.	E26499.G03	**Logged By**	LL	**Date**	08 October 2024
Sheets	1 of 1	**Review By**	ML	**Date**	22 November 2024

Drilling Contractor	Geosense Drilling Engineers	**Surface RL**	≈5.80 m (AHD)	**Latitude**	-
Plant	Comacchio Geo 205	**Inclination**	90°	**Longitude**	-

WATER	SAMPLES & FIELD TESTS	DEPTH (m)	RL (m AHD)	MATERIAL DESCRIPTION	MOISTURE CONDITION	BACKFILL DETAILS	STANDPIPE DETAILS
		0.00	5.80	ASPHALT: 200mm thick	-	Bentonite 0.00m - 2.50m	Well Stickup =0.0m (RL 5.80m)
	BH2M_0.30-0.40	0.20	5.60	FILL: Silty SAND: fine to medium grained, grey brown	M		0.0m - 3.0m PVC casing (50mm Ø)
	BH2M_0.50-0.95 SPT 0.50-0.95 2,2,2 N=4	0.50	5.30	SAND: fine to medium grained, brown			
	BH2M_1.50-1.95 SPT 1.50-1.95 2,2,3 N=5	1.50	4.30	From 1.50m to 1.50m, pale brown			
	BH2M_2.50-2.60					Sand 2.50m - 7.00m	
	BH2M_3.00-3.45 SPT 3.00-3.45 2,2,3 N=5						3.0m - 6.0m PVC screen (50mm Ø)
	BH2M_4.00-4.10						
	BH2M_4.50-4.95 SPT 4.50-4.95 4,13,22 N=35	4.50	1.30	From 4.50m, pale brown / brown			
▽					W		
		7	-1.20	Terminated at 7.00m. Target Depth Reached. Terminated at 7.00m. Target Depth Reached.			



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

BOREHOLE LOG

BH ID: BH3M

Location	101 North Steyne, Manly, NSW	**Started**	08 October 2024		
Client	Time & PlacePty Ltd	**Completed**	08 October 2024		
Job No.	E26499.G03	**Logged By**	LL	**Date**	08 October 2024
Sheets	1 of 2	**Review By**	ML	**Date**	22 November 2024

Drilling Contractor	Geosense Drilling Engineers	**Surface RL**	≈6.00 m (AHD)	**Latitude**	-
Plant	Comacchio Geo 205	**Inclination**	90°	**Longitude**	-

METHOD	GROUND WATER LEVELS	SAMPLES & FIELD TESTS	DEPTH (m)	RL (m AHD)	MATERIAL DESCRIPTION	MOISTURE CONDITION	CONSISTENCY / REL. DENSITY	MATERIAL ORIGIN & OBSERVATIONS
AD/T		BH3M_0.10-0.20	0.00	6.00	ASPHALT: 200mm thick	-	-	ASPHALT
			0.20	5.80	FILL: Silty SAND: fine to medium grained, grey, with fine grained, sub sub-angular gravels			FILL
		BH3M_0.50-0.60	0.30	5.70	FILL: SANDSTONE: pale grey, distinctly weathered to fresh, medium strength			
		BH3M_0.90-1.00 BH3M_1.50-1.95 SPT 1.50-1.95 2,3,3 N=6 BH3M_2.40-2.50	0.70	5.30	SAND: fine to medium grained, brown	M	VL - L	MARINE SOIL
		BH3M_3.00-3.45 SPT 3.00-3.45 4,6,6 N=12 BH3M_4.00-4.10 BH3M_4.50-4.95 SPT 4.50-4.95 6,12,16 N=28	3				L - MD	
	▷	BH3M_5.50-5.60 BH3M_6.00-6.45 SPT 6.00-6.45 1,5,11 N=16	5			W		
		BH3M_7.50-7.95 SPT 7.50-7.95 3,4,5 N=9 BH3M_9.50-9.95 SPT 9.50-9.95 2,4,5 N=9	7.00	-1.00	From 7.00m, medium grained, with shellfish debris		L	

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

BOREHOLE LOG

BH ID: BH3M

Location	101 North Steyne, Manly, NSW	**Started**	08 October 2024		
Client	Time & PlacePty Ltd	**Completed**	08 October 2024		
Job No.	E26499.G03	**Logged By**	LL	**Date**	08 October 2024
Sheets	2 of 2	**Review By**	ML	**Date**	22 November 2024

Drilling Contractor	Geosense Drilling Engineers	**Surface RL**	≈6.00 m (AHD)	**Latitude**	-
Plant	Comacchio Geo 205	**Inclination**	90°	**Longitude**	-

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
						-4.00	Terminated at 10.00m. Target Depth Reached. Terminated at 10.00m. Target Depth Reached.			
				11						
				12						
				13						
				14						
				15						
				16						
				17						
				18						
				19						
				20						

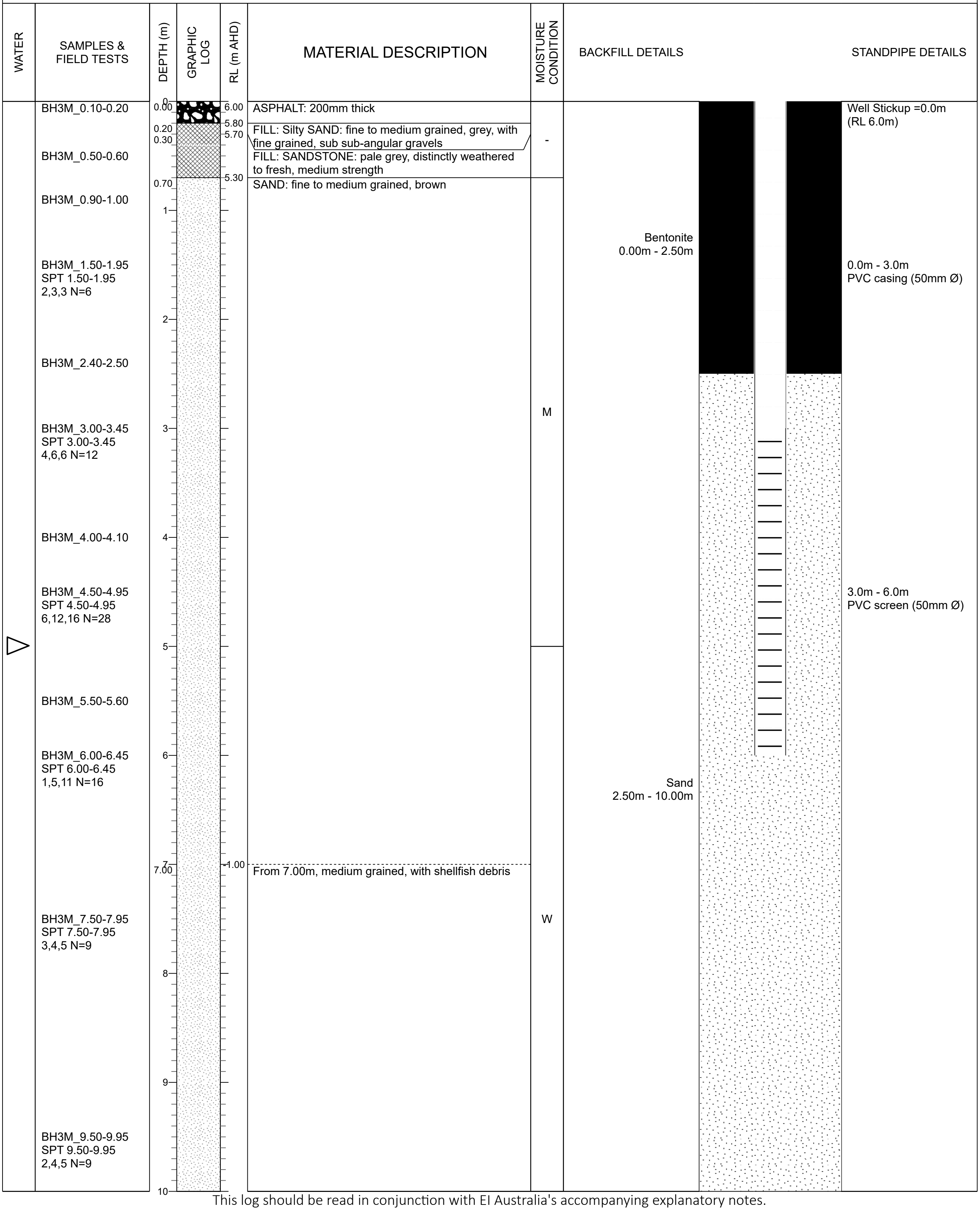
This log should be read in conjunction with EI Australia's accompanying explanatory notes.

MONITORING WELL LOG

BH ID: BH3M

Location	101 North Steyne, Manly, NSW	**Started**	08 October 2024		
Client	Time & PlacePty Ltd	**Completed**	08 October 2024		
Job No.	E26499.G03	**Logged By**	LL	**Date**	08 October 2024
Sheets	1 of 2	**Review By**	ML	**Date**	22 November 2024

Drilling Contractor	Geosense Drilling Engineers	**Surface RL**	≈6.00 m (AHD)	**Latitude**	-
Plant	Comacchio Geo 205	**Inclination**	90°	**Longitude**	-



WATER	SAMPLES & FIELD TESTS	DEPTH (m)	RL (m AHD)	MATERIAL DESCRIPTION	MOISTURE CONDITION
	BH3M_0.10-0.20	0.00	6.00	ASPHALT: 200mm thick	
		0.20	5.80	FILL: Silty SAND: fine to medium grained, grey, with fine grained, sub sub-angular gravels	-
	BH3M_0.50-0.60	0.30	5.70	FILL: SANDSTONE: pale grey, distinctly weathered to fresh, medium strength	
	BH3M_0.90-1.00	0.70	5.30	SAND: fine to medium grained, brown	M
	BH3M_1.50-1.95 SPT 1.50-1.95 2,3,3 N=6				
	BH3M_2.40-2.50				
	BH3M_3.00-3.45 SPT 3.00-3.45 4,6,6 N=12				
	BH3M_4.00-4.10				
	BH3M_4.50-4.95 SPT 4.50-4.95 6,12,16 N=28				
▷		5			
	BH3M_5.50-5.60				W
	BH3M_6.00-6.45 SPT 6.00-6.45 1,5,11 N=16				
		7.00	1.00	From 7.00m, medium grained, with shellfish debris	
	BH3M_7.50-7.95 SPT 7.50-7.95 3,4,5 N=9				
	BH3M_9.50-9.95 SPT 9.50-9.95 2,4,5 N=9				

BACKFILL DETAILS	STANDPIPE DETAILS
	Well Stickup =0.0m (RL 6.0m)
Bentonite 0.00m - 2.50m	0.0m - 3.0m PVC casing (50mm Ø)
Sand 2.50m - 10.00m	3.0m - 6.0m PVC screen (50mm Ø)

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

MONITORING WELL LOG

BH ID: BH3M

Location 101 North Steyne, Manly, NSW
Client Time & PlacePty Ltd
Job No. E26499.G03
Sheets 2 of 2

Started 08 October 2024
Completed 08 October 2024
Logged By LL **Date** 08 October 2024
Review By ML **Date** 22 November 2024

Drilling Contractor Geosense Drilling Engineers
Plant Comacchio Geo 205
Surface RL ≈6.00 m (AHD)
Inclination 90°
Latitude -
Longitude -

WATER	SAMPLES & FIELD TESTS	DEPTH (m)	GRAPHIC LOG	RL (m AHD)	MATERIAL DESCRIPTION	MOISTURE CONDITION	BACKFILL DETAILS	STANDPIPE DETAILS
				-4.00	Terminated at 10.00m. Target Depth Reached. Terminated at 10.00m. Target Depth Reached.			
		11						
		12						
		13						
		14						
		15						
		16						
		17						
		18						
		19						
		20						

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

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

DRILLING/EXCAVATION METHOD

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

PENETRATION RESISTANCE

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

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

WATER

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

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

SAMPLING AND TESTING

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

GEOLOGICAL BOUNDARIES

——————— = Observed Boundary (position known)

– – – – – – – – – – – = Observed Boundary (position approximate)

– –?– –?– –?– – = Boundary (interpreted or inferred)

ROCK CORE RECOVERY

TCR=Total Core Recovery (%)

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

RQD = Rock Quality Designation (%)

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

METHOD OF SOIL DESCRIPTION USED ON BOREHOLE AND TEST PIT LOGS

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

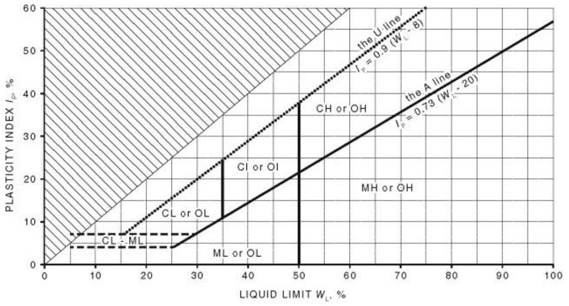
CLASSIFICATION AND INFERRED STRATIGRAPHY

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

PARTICLE SIZE CHARACTERISTICS

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

PLASTICITY PROPERTIES



GROUP SYMBOLS

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

MOISTURE CONDITION

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

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

CONSISTENCY

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

DENSITY

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

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

MINOR COMPONENTS

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

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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

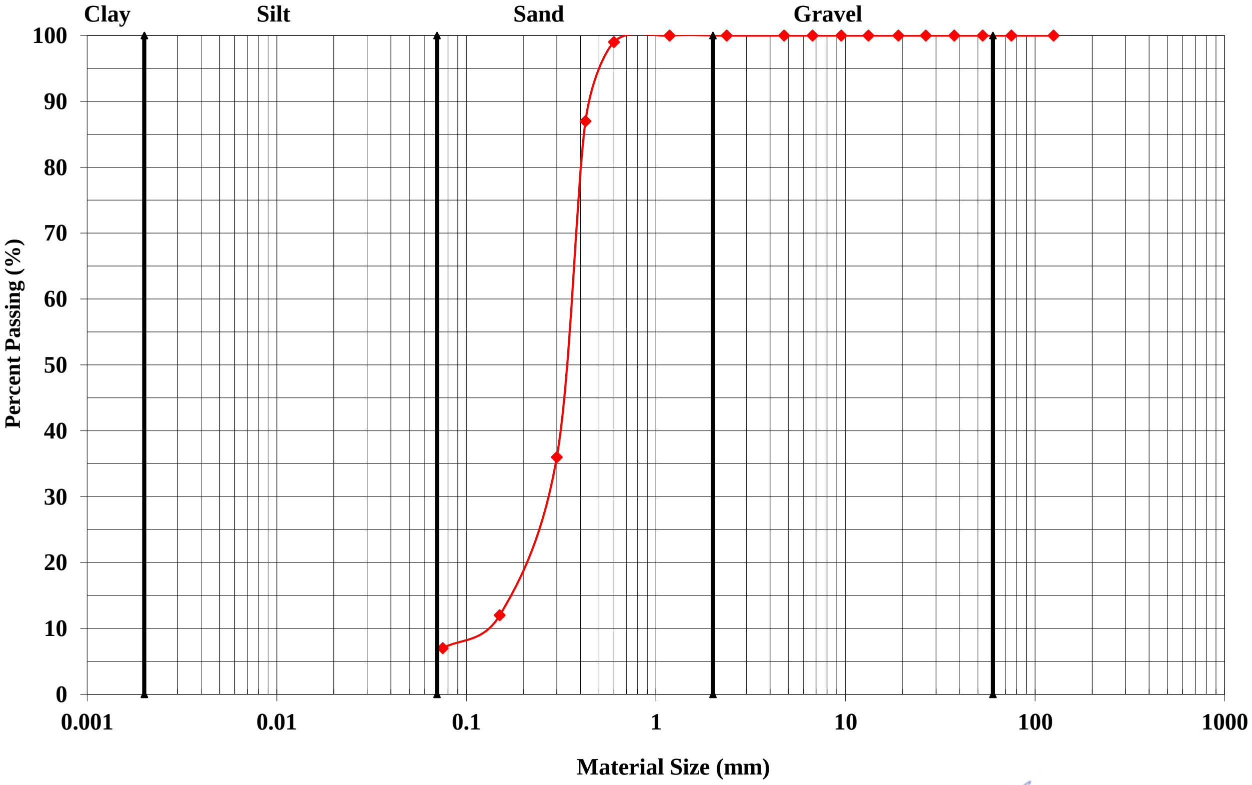
NATA
Accredited for Compliance with ISO/IEC 17025 - Testing
No. 2750

Particle Size Distribution

Project: E26499.G03: 101 North Steyne, Manly
Client: EI AUSTRALIA PTY LTD
Address: Suite 6.01, 55 Miller Street, Pyrmont NSW 2009
Test Method: AS1289.3.6.1
Sampling Procedure: Samples Supplied By Client (Not covered under NATA Scope of Accreditation)
Material Description: SAND,white brown, trace Silt and Clay

STS / Sample No.: 9232D-L/1
Sample Location: BH2M
Depth (m): 4.5-4.95
Date Sampled: 08/10/2024

Project No.: 31380
Report No.: 24/3236
Report Date: 21/10/2024
Page: 1 of 2
Client Project No: 0



Sieve Size (mm)	Percent Passing (%)
125.0	100.0
75.0	100.0
53.0	100.0
37.5	100.0
26.5	100.0
19.0	100.0
13.2	100.0
9.5	100.0
6.7	100.0
4.75	100.0
2.36	100.0
1.18	100.0
0.60	99.0
0.425	87.0
0.30	36.0
0.15	12.0
0.075	7.0

Remarks:

Technician: BV

Approved Signatory.......
Mrigesh Tamang - Manager

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

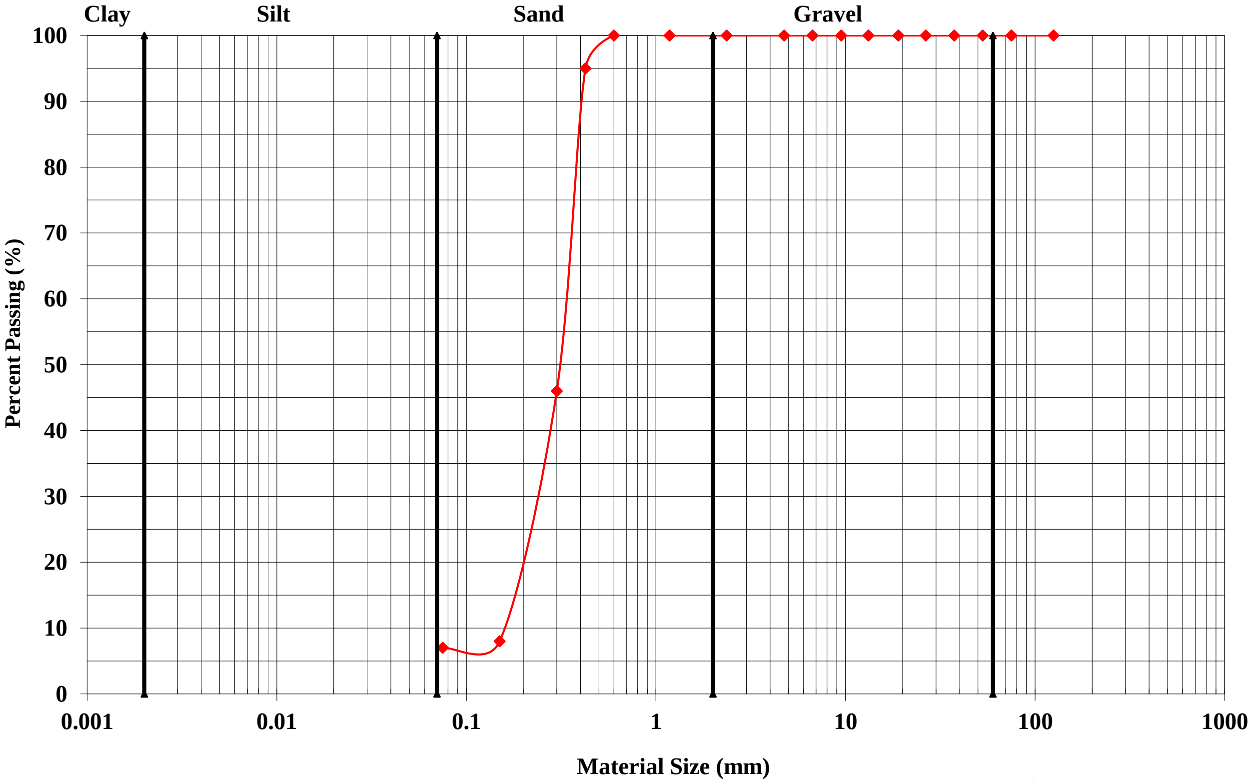
Accredited for Compliance with ISO/IEC 17025 - Testing No. 2750

Particle Size Distribution

Project: E26499.G03: 101 North Steyne, Manly
Client: EI AUSTRALIA PTY LTD
Address: Suite 6.01, 55 Miller Street, Pyrmont NSW 2009
Test Method: AS1289.3.6.1
Sampling Procedure: Samples Supplied By Client (Not covered under NATA Scope of Accreditation)
Material Description: SAND,white brown, trace Silt and Clay

STS / Sample No.: 9232D-L/2
Sample Location: BH3M
Depth (m): 1.5 - 1.95
Date Sampled: 08/10/2024

Project No.: 31380
Report No.: 24/3236
Report Date: 21/10/2024
Page: 2 of 2
Client Project No: 0



Sieve Size (mm)	Percent Passing (%)
125.0	100.0
75.0	100.0
53.0	100.0
37.5	100.0
26.5	100.0
19.0	100.0
13.2	100.0
9.5	100.0
6.7	100.0
4.75	100.0
2.36	100.0
1.18	100.0
0.60	100.0
0.425	95.0
0.30	46.0
0.15	8.0
0.075	7.0

Remarks:

Technician: BV

Approved Signatory.......
Mrigesh Tamang - Manager

ANALYTICAL REPORT

Accreditation No. 2562

CLIENT DETAILS

Contact	Lawrence Li
Client	EI AUSTRALIA
Address	SUITE 6.01 55 MILLER STREET PYRMONT NSW 2009
Telephone	61 2 95160722
Facsimile	(Not specified)
Email	Lawrence.Li@eiaustralia.com.au
Project	**E26499.G03 101 North Steyne, Marily, NSW**
Order Number	**E26499.G03**
Samples	3

LABORATORY DETAILS

Manager	Shane McDermott
Laboratory	SGS Alexandria Environmental
Address	Unit 16, 33 Maddox St Alexandria NSW 2015
Telephone	+61 2 8594 0400
Facsimile	+61 2 8594 0499
Email	au.environmental.sydney@sgs.com
SGS Reference	**SE272607 R2**
Date Received	16/10/2024
Date Reported	31/10/2024

COMMENTS

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

This report cancels and supersedes the report No.SE2722607 R1. dated 30/10/2024 due to Client request - amended sampling dates.

SIGNATORIES

Shane

Shane MCDERMOTT
Laboratory Manager

Ying Ying ZHANG
Laboratory Technician

Soluble Anions (1:5) in Soil/Solids by Ion Chromatography [AN245] Tested: 17/10/2024

			CPT1_1.4-1.5 SOIL - 26/9/2024	BH2M_0.5-0.95 SOIL - 8/10/2024	BH3M_6.0-6.45 SOIL - 8/10/2024
PARAMETER	**UOM**	**LOR**	**SE272607.001**	**SE272607.002**	**SE272607.003**
Chloride	mg/kg	0.25	2.6	33	5.5
Sulfate	mg/kg	5	<5.0	49	5.6

Moisture Content [AN002] Tested: 17/10/2024

PARAMETER	UOM	LOR	CPT1_1.4-1.5 SOIL - 26/9/2024 SE272607.001	BH2M_0.5-0.95 SOIL - 8/10/2024 SE272607.002	BH3M_6.0-6.45 SOIL - 8/10/2024 SE272607.003
% Moisture	%w/w	1	5.4	2.1	18.8

pH in soil (1:5) [AN101] Tested: 17/10/2024

			CPT1_1.4-1.5	BH2M_0.5-0.95	BH3M_6.0-6.45
			SOIL	SOIL	SOIL
			-	-	-
			26/9/2024	8/10/2024	8/10/2024
PARAMETER	UOM	LOR	SE272607.001	SE272607.002	SE272607.003
pH	pH Units	0.1	9.1	6.8	9.3

Conductivity and TDS by Calculation - Soil [AN106] Tested: 17/10/2024

			CPT1_1.4-1.5 SOIL - 26/9/2024	BH2M_0.5-0.95 SOIL - 8/10/2024	BH3M_6.0-6.45 SOIL - 8/10/2024
PARAMETER	**UOM**	**LOR**	**SE272607.001**	**SE272607.002**	**SE272607.003**
Conductivity of Extract (1:5 dry sample basis)	µS/cm	1	52	160	73

METHOD	METHODOLOGY SUMMARY
AN002	The test is carried out by drying (at either 40°C or 105°C) a known mass of sample in a weighed evaporating basin. After fully dry the sample is re-weighed. Samples such as sludge and sediment having high percentages of moisture will take some time in a drying oven for complete removal of water.
AN101	pH in Soil Sludge Sediment and Water: pH is measured electrometrically using a combination electrode and is calibrated against 3 buffers purchased commercially. For soils, sediments and sludges, an extract with water (or 0.01M $CaCl_2$) is made at a ratio of 1:5 and the pH determined and reported on the extract. Reference APHA 4500-H+.
AN106	Conductivity and TDS by Calculation: Conductivity is measured by meter with temperature compensation and is calibrated against a standard solution of potassium chloride. Conductivity is generally reported as µmhos /cm or µS/cm @ 25°C. For soils, an extract of as received sample with water is made at a ratio of 1:5 and the EC determined and reported on the extract, or calculated back to the as-received sample. Salinity can be estimated from conductivity using a conversion factor, which for natural waters, is in the range 0.55 to 0.75. Reference APHA 2510 B.
AN245	Anions by Ion Chromatography: A water sample is injected into an eluent stream that passes through the ion chromatographic system where the anions of interest ie Br, Cl, NO_2, NO_3 and SO_4 are separated on their relative affinities for the active sites on the column packing material. Changes to the conductivity and the UV-visible absorbance of the eluent enable identification and quantitation of the anions based on their retention time and peak height or area. APHA 4110 B

FOOTNOTES

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

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

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

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

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

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

a. 1 Bq is equivalent to 27 pCi
b. 37 MBq is equivalent to 1 mCi

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

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

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Appendix C Vibration Limits

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

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

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

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

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

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

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

Appendix D Important Information

Important Information

SCOPE OF SERVICES

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

RELIANCE ON DATA

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

GEOTECHNICAL ENGINEERING

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

LIMITATIONS OF SITE INVESTIGATION

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

SUBSURFACE CONDITIONS ARE TIME DEPENDENT

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

VERIFICATION OF SITE CONDITIONS

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

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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

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