



**SPECIALIST ADVICE TO
ST LEONARDS PROPERTY UNIT TRUST**

**ON
GEOTECHNICAL INVESTIGATION**

**FOR
PROPOSED MIXED-USE DEVELOPMENT**

**AT
46-52 NICHOLSON STREET &
59-67 CHRISTIE STREET, ST LEONARDS, NSW**

Date: 5 September 2025
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Table of Contents

1	INTRODUCTION	1
2	INVESTIGATION PROCEDURE	2
3	RESULTS OF THE INVESTIGATION	4
3.1	Site Description	4
3.2	Subsurface Conditions	6
3.3	Laboratory Test Results	9
4	PRELIMINARY COMMENTS AND RECOMMENDATIONS	9
4.1	Primary Geotechnical Issues	9
4.2	Rock Classification	12
4.3	Site Preparation	13
4.3.1	Dilapidation Surveys	13
4.3.2	Vibration Monitoring	13
4.3.3	Early Works	14
4.4	Excavation	14
4.4.1	Excavation Conditions	14
4.4.2	Potential Vibration Risks	15
4.4.3	Drainage	15
4.5	Basement Retention	16
4.5.1	Retention Systems	16
4.5.2	Retention Design Parameters	17
4.5.3	Temporary Rock Anchors	18
4.5.4	Unsupported Cuts in the Bedrock	19
4.6	Footings	20
4.6.1	Geotechnical Design	20
4.6.2	Earthquake Design Parameters	21
4.6.3	Exposure Classification	21
4.7	Basement 3 Floor Slab	21
4.8	Further Geotechnical Input	22
5	GENERAL COMMENTS	23

ATTACHMENTS

Table A: Point Load Strength Index Test Report

EnviroLab Services 'Certificate of Analysis 371536' and 'Certificate of Analysis 382739'

Borehole Logs 1, 1A, 2, 3, 3A & 4 (With Rock Core Photographs)

Figure 1: Site Location Plan

Figure 2: Borehole Location Plan

Figure 3: Section A-A Graphical Borehole Summary

Figure 4: Section B-B Graphical Borehole Summary

Figure 5: BH1/1A Groundwater Level and Daily Rainfall versus Time Plot

Figure 6: BH2 Groundwater Level and Daily Rainfall versus Time Plot

Figure 7: BH4 Groundwater Level and Daily Rainfall versus Time Plot

Vibration Emission Design Goals

Report Explanation Notes

1 INTRODUCTION

This specialist advice report presents the results of a geotechnical investigation for the proposed mixed-use development at 46-52 Nicholson Street & 59-67 Christie Street, St Leonards, NSW. The location of the site is shown in Figure 1. The investigation was initially commissioned by Jovana Bukorovic of Coronation Property Co Pty Ltd, on behalf of St Leonards Property Unit Trust, in an email dated 17 December 2024, and a subsequent email dated 9 May 2025. The current commission was based on our 'Variation 02' fee proposal, Ref. P70596AE2 Rev3 dated 14 May 2025.

We have been provided with the following relevant information:

- Architectural drawings prepared by Cox Architecture (Project No. 222141.03, Drawing Nos. A-DA-2000², A-DA-2001², A-DA-2004², A-DA-2005², A-DA-2100² to A-DA-2108², A-DA-3000² to A-DA-3003² and A-DA-4000² to A-DA-4002², dated 29 August 2025); and
- Survey plans prepared by Frank M Mason & Co Pty Ltd (Drawing Name. 33489-01, Sheets 1 to 6, dated 18 March 2021). The survey datum is the Australian Height Datum (AHD).

Based on the supplied information, we understand that the proposed development will comprise demolition of all existing structures on site, and construction of a 34-storey tower, which will overlie three basement levels. Proposed Basement 3 will be constructed at RL61.5m and will require excavation to depths between approximately 11m and 17m below existing grade. The proposed basements will extend to the north-eastern, south-eastern and south-western site boundaries, and will be set back approximately 26m from the north-western corner of the site, as shown on the attached Figure 2. The base of the proposed lift core will be constructed at RL58.5m, and will require a localised 3m deep excavation below bulk excavation level (BEL). Structural loads typical of this type of development have been assumed.

We note that at the time of preparing our 'Variation 02' fee proposal and tailoring the geotechnical investigation, the architectural drawings provided then showed a five-level basement and indicated the lowest proposed Basement 5 level was at RL57.10m, i.e. 4.4m below the current proposed Basement 3 level at RL61.5m.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions at three borehole locations and, based on the information obtained, to present our comments and recommendations on the site preparation, excavation, drainage, basement retention, footings, earthquake design parameters, exposure classification, the Basement 3 floor slab and further geotechnical input. Due to the limited site access, presence of buried services and/or changes in the basement design, three additional boreholes were completed.

This report provides specialist advice for use by the structural and civil designers in preparing their designs and no part of this report is to be considered a regulated design in accordance with the Design and Building Practitioners Act 2020.

The geotechnical investigation was carried out in conjunction with a Detailed Site Investigation (DSI) and a Groundwater Impact Assessment (GIA) by our environmental division, JK Environments (JKE). Reference should be made to the separate reports by JKE, Ref: E37264Brpt-DSI and Ref: E37264Brpt2-GIA, for the results of the DSI and GIA, respectively.

2 INVESTIGATION PROCEDURE

The investigation was carried out on 22 & 23 January 2025 and between 5 May 2025 and 11 June 2025, and comprised the drilling and testing of six boreholes (BH1, BH1A, BH2, BH3, BH3A and BH4), at the locations shown on Figure 2, to depths between 1.4m (BH3) and 33.55m (BH1A) below existing grade. The boreholes were completed using our track mounted JK205 and JK330 drill rigs, which are equipped for site investigation purposes. We note that at the time BH1 was completed, the lowest proposed basement level was at RL65.8m. BH1A was then completed to greater depth for the current basement design.

BH3 was terminated due to the presence of a buried stormwater pipe, and was then moved to BH3A, which terminated at 2.5m depth due to auger refusal on a suspected buried service. BH3A was abandoned, and BH1A was completed in its place.

Prior to the commencement of the fieldwork, a specialist sub-consultant reviewed available 'Before You Dig Australia' information and electro-magnetically scanned the borehole locations for buried services.

The borehole locations, which were limited by site access, were set out by tape measurements from existing site features. The surface RL's indicated on the attached borehole logs were interpolated between spot level heights and ground contour lines shown on the supplied survey plans, and are therefore approximate. Sheet 1 of the supplied survey plans forms the basis of Figure 2. The outlines of the proposed basements, upper ground floor level and Level 2 have been plotted onto Figure 2.

The concrete pavements at BH1, BH1A, BH2 and BH3 were cored with a diamond tipped thin-walled tube with water flush. The soil and upper weathered bedrock profiles were spiral auger drilled. The relative compaction/strength of the soil profile were assessed from the Standard Penetration Test (SPT) 'N' values, together with hand penetrometer readings on clayey soils recovered in the SPT split-spoon sampler, and by tactile examination. The strength of the underlying bedrock was initially assessed by observation of auger penetration resistance when using a twin-pronged tungsten carbide (TC) drill bit, together with examination of recovered cuttings.

At depths of 2.70m (BH1), 2.63m (BH1A), 2.50m (BH2) and 4.61m (BH4), these boreholes were extended into the bedrock to their final depths by rotary diamond coring techniques, using an HQ triple tube core barrel with water flush. The strength of the cored bedrock was assessed by examination of the recovered rock cores, together with correlations with subsequent Point Load Strength Index ($I_{s(50)}$) test results.

Groundwater observations were made in the boreholes during and on completion of drilling. Groundwater monitoring wells were installed in all boreholes, with the exception of BH3 and BH3A. The groundwater

monitoring wells comprised 50mm diameter Class 18 PVC standpipes. The machine slotted length of each standpipe (i.e. the 'response zone') was installed and sealed within the bedrock profile. Prior to their installation, BH1A, BH2 and BH4 were backfilled and sealed with bentonite to the base of the response zone. The annulus between the borehole and the slotted length of the installed standpipe was then backfilled with 2mm filter sand. Above the sand backfill, each borehole was sealed with bentonite. Cast-iron 'Gatic' covers were concreted flush with the ground surface to protect the top of the groundwater monitoring wells. The installation details are presented on the attached borehole logs.

Further details of the methods and procedures employed in the investigation are presented in the attached Report Explanation Notes.

Our geotechnical engineers and engineering geologists were present full-time during the fieldwork to set out the borehole locations, direct the borehole scanning, nominate testing and sampling, prepare the attached borehole logs, and to direct the groundwater monitoring well installations. The Report Explanation Notes define the logging terms and symbols used.

The recovered rock cores were photographed and Point Load Strength Index tested. The rock core photographs are enclosed with the borehole logs. The Point Load Strength Index test results are plotted on the borehole logs and summarised in the attached Table A. The unconfined compressive strengths (UCS), as estimated from the Point Load Strength Index test results, are also presented in Table A and summarised in Section 3.3.

Selected soil and weathered sandstone samples were returned to a NATA registered analytical laboratory, Envirolab Services Pty Ltd, for soil pH, chloride content, sulphate content and resistivity testing. The results are presented in the attached Envirolab Services 'Certificate of Analysis 371536' and 'Certificate of Analysis 382739' and summarised in Section 3.3.

Water level data loggers programmed to take readings at 10 minute intervals were installed into BH1 on 31 January 2025 and into BH2 and BH4 on 30 May 2025 for long term groundwater level monitoring. On 14 June 2025, the water level data logger was removed from BH1 and immediately reinstalled into BH1A, which had a more appropriate deeper response zone. We returned to site on 14 July 2025 to download the water level data loggers. The interim monitoring results are plotted as groundwater RL (mAHD) and daily rainfall (mm) versus time, and are presented as Figure 5 (BH1/1A), Figure 6 (BH2) and Figure 7 (BH4). The plotted rainfall data was obtained from the Bureau of Meteorology's closest monitoring station (with complete rainfall records) at Sydney Observatory Hill (Station No. 66214).

3 RESULTS OF THE INVESTIGATION

3.1 Site Description

The site is located just below the crest of a gently sloping south-west facing hillside. Nicholson Street bounds the site to the north and north-east, and extends across a shallow gully feature. The portion of Nicholson Street adjacent to the north-eastern side of the site sloped down to the north-west at about 3°, whilst the portion adjacent to the northern side sloped down to the south-east at about 1° to 2°. Christie Street bounds the site to the south-west, and grades down to the south-east at about 4°. A maximum elevation relief of about 6.5m exists between the eastern corner of the site and the lower south-western corner.

At the time of fieldwork, the site contained several two and three storey brick buildings, which appeared to be in generally good external condition based on a cursory inspection. Some of the buildings were underlain by one and two basement levels. Several planter boxes were located along the Christie Street frontage adjacent to 59, 61 and 63 Christie Street, and were between about 0.8m and 1.7m high and contained shrubs and small trees. A concrete driveway ran through the site between 48 & 50 Nicholson Street and 61 & 63 Christie Street, and sloped down to the south-west at an average grade of about 7°. There was another concrete driveway in the south-western corner of the site, which sloped down to the south-west at an average grade of about 8°.

The neighbouring six storey brick and glass clad commercial building to the south-east (63 Christie Street) abutted the south-western corner of the site, however, the northern corner of the neighbouring building was set back about 6.5m from the common boundary. The neighbouring building was underlain a split three level basement car park (Wilson), the lowest level of which we estimate to be about 12m below the adjoining portion of Christie Street. From a limited inspection inside the neighbouring basement, we consider the north-western end of the basement was set back at least 20m from the north-western side of the overlying superstructure. Along the north-western side of the neighbouring building at ground floor level was a loading dock. The provided survey plan indicates the loading dock (referred to as a “*basement*”) is at RL72.19m, which is just above the adjoining surface level of Christie Street. The north-western wall of the loading dock was of concrete block construction and retained the ground surface levels to the north-west, to an estimated maximum height of about 4m at its north-eastern corner. The neighbouring building appeared to be in good condition, based on a cursory inspection from the adjoining streets.

During our limited inspection of the neighbouring basement to the south-east, there was an opening in the one of the air vents in the concrete block wall along the south-western (Christie Street) side of the basement, about 3m above the lowest level. Inside the opening, we observed a narrow air plenum which exposed a sub-vertical sandstone cut face, as shown in Plate 1 below. No seepage was observed emanating from the rock face at this location, and the base of the plenum was ‘dry’. Based on these observations, it appears that the neighbouring basement is a ‘drained’ structure.



Plate 1: View looking north-west inside the air plenum of the neighbouring basement to the south-east

The area between the neighbouring building to the south-east and the subject site contained a gently sloping concrete pathway and a landscaped area with some shrubs and small to medium sized trees. There was a small concrete block building (noted as a substation on the supplied survey drawings) located immediately outside the north-eastern end of the south-eastern site boundary.

The Sydney Water Before You Dig (BYD) plan shows a 225mm diameter salt glazed ware (SGW) sewer pipe and a 225mm diameter vitreous clay (VC) sewer pipe below the site. A separate 225mm diameter VC sewer pipe extends along Christie Street a short distance outside the site boundary. The sewer maintenance hole within the site (63 Christie Street) is shown to have an invert depth of about 1.7m, with the two located in Christie Street adjacent to the site, shown to have invert depths of about 1.6m and 2.1m. There is also a 100mm diameter cast iron cement lined (CICL) water main below Christie Street. Below Nicholson Street are a 200mm diameter PVC water main and a 100mm diameter ductile iron cement lined (DICL) water main. The invert levels of the water mains are not shown, but we assume they would be relatively shallow. The alignments of the existing sewer mains and closest water mains have been plotted onto Figure 2.

3.2 Subsurface Conditions

The 1:100,000 geological map of Sydney (Geological Survey of NSW, Geological Series Sheet 9130) indicates the site to be underlain by Ashfield Shale of the Wianamatta Group, then Mittagong Formation bedrock, then Hawkesbury Sandstone. Generally, the boreholes encountered concrete pavements and fill, overlying residual silty clay, then weathered laminite and sandstone bedrock of the Mittagong Formation at shallow to moderate depths. Hawkesbury Sandstone was encountered below approximately 11.3m and 12.4m depth. Reference should be made to the attached borehole logs for specific details at each location. Graphical borehole summaries for Section A-A and Section B-B, as shown on Figure 2, are presented as Figure 3 and Figure 4, respectively. A summary of the encountered subsurface characteristics is provided below:

Concrete Pavements

Reinforced concrete slabs were encountered in BH1, BH1A, BH2 and BH3 and were between 110mm and 160mm thick.

Fill

Silty clay and silty sandy clay fill, and to a much lesser extent, silty sand fill, was encountered below the concrete pavements in BH2 and BH3, and from the surface of BH3A and BH4, and generally extended to depths between 0.4m (BH4) and 0.9m (BH3), however in BH3A, the fill was 2.5m deep (presumably the backfill of a buried service). Inclusions of igneous and ironstone gravel, and concrete, glass, tile and fabric fragments were found in the fill. Based on the SPT results and limited hand penetrometer readings, the deeper fill in BH3 and BH3A was assessed to be poorly compacted. BH3 was terminated on a stormwater pipe. BH3A was also terminated as the deep fill was suspected to be associated with service trench backfill.

Residual Silty Clay

Residual silty clay of predominantly medium or high plasticity and of very stiff or hard strength was encountered below the pavements in BH1 and BH1A and below the fill in BH4.

Extremely Weathered Siltstone

Extremely weathered siltstone of hard (soil) strength was encountered in BH4 below 3.4m depth and had a thickness of 1.1m. The extremely weathered siltstone contained medium to high strength iron indurated siltstone bands.



Weathered Siltstone, Sandstone and Laminite Bedrock (Mittagong Formation)

Highly weathered, moderately weathered, slightly weathered and occasionally fresh siltstone, sandstone and laminite bedrock of low, medium or high strength (with some very high strength bands) was encountered in BH1, BH1A, BH2 and BH4 below the depths and levels tabulated below:

Borehole	Approximate Surface RL (mAHD)	Depth to Weathered Bedrock Surface (m)	Approximate Weathered Bedrock Surface Level (mAHD)
BH1	75.4	2.1	73.3
BH1A	75.4	2.2	73.2
BH2	74.6	1.9	72.7
BH4	76.3	4.5	71.8

From these four boreholes, the weathered bedrock surface appears to deepen in an easterly direction. Based on the site topography, however, the bedrock surface level could increase in elevation towards the eastern corner of the site, but this has not been proven by our investigation.

The rock cored lengths, often within the upper 2m but reducing in frequency below, contained defects (i.e. inclined joints, bedding partings, clay seams, extremely weathered seams and crushed seams). 'No core' (core loss) zones were encountered in BH1 below 3.5m depth, in BH2 below 2.5m depth and in BH4 below 4.6m depth, and were 80mm, 300mm and 290mm thick, respectively; most likely weaker material washed out by the introduced drill flush water.

Sandstone Bedrock (Hawkesbury Sandstone)

Fresh sandstone bedrock of medium or high strength and containing few defects was encountered in BH1, BH1A, BH2 and BH4 below the depths and levels tabulated below:

Borehole	Approximate Surface RL (mAHD)	Depth to Fresh Sandstone Bedrock (m)	Approximate Fresh Sandstone Bedrock Level (mAHD)
BH1	75.4	11.5	63.9
BH1A	75.4	11.3	64.1
BH2	74.6	11.7	62.9
BH4	76.3	12.4	63.9

Groundwater

Apart from BH3A, all boreholes were 'dry' during and on completion of auger drilling. In BH3A, groundwater seepage was observed at 1.4m depth, with groundwater measured on completion of auger drilling at 1.4m depth. We consider the groundwater measured in BH3A is 'perched' with the assumed trench backfill. In BH2 and BH4, groundwater was measured at depths of 11.7m and 2.4m on completion of rock coring, but is not considered to be representative due to the introduced drill flush water.

The results of the interim groundwater level monitoring period between 31 January 2025 and 14 July 2025 for BH1/BH1A and between 30 May 2025 and 14 July 2025 for BH2 and BH4 are summarised in the table below. For the joint BH1/BH1A groundwater level monitoring plot, the data between 31 January 2025 and 14 June 2025 was obtained from BH1, and thereafter from BH1A.

Groundwater Monitoring Well	Approximate Surface RL / Base of Standpipe Level (mAHD)	Groundwater Level Monitoring Results (mAHD)		
		Shallowest Recorded Groundwater Level	Deepest Recorded Groundwater Level (excluding when the water was pumped out)	Approximate Average Groundwater Level
BH1A	75.4 / 53.7	63.8	60.7	61.0
BH2	74.6 / 50.2	60.0	60.0	60.0
BH4	76.3 / 51.1	61.8	61.1	61.4

From the available rainfall records, relatively heavy and prolonged rainfall events occurred in late March, late April, mid-May 2025 and in early June. Based on the investigation results and interim groundwater level monitoring results, we note the following:

- Groundwater levels are similar within the north-western portion of the site (BH1A & BH4), and only slightly deeper within the south-eastern portion (BH2), with a gently sloping hydraulic gradient;
- There was generally a negligible response in groundwater levels to rainfall, and the groundwater levels were relatively constant during the monitoring period. However, in BH1/BH1A, the two heavy rainfall events in May and June 2025 where about 75mm total rainfall occurred in two 24-hour periods, the groundwater level quickly spiked about by about 1m on each occasion, but within a few days dropped back to the average levels;
- The groundwater levels were within the bedrock profile; and
- The shallowest groundwater levels recorded during the interim monitoring period were between approximately 1.5m (BH2) below and 2.3m (BH1A) above the proposed Basement 3 floor level.

In order to satisfy the NSW Department of Planning and Environment (now DCCEEW) guidelines [DPE 2022] [Reference 1], the groundwater level monitoring will extend until at least 14 September 2025.

3.3 Laboratory Test Results

The results of the Point Load Strength Index tests carried out on the recovered rock cores correlated well with our field assessment of bedrock strength. The estimated UCS's, based on the correlation provided in AS1726:2017 'Geotechnical Site Investigations' (i.e. $UCS = 20 \times I_{s(50)}$), ranged between 10MPa and 74MPa, but with a single lower value of 4MPa in BH1 at 3.79m depth.

The soil and weathered sandstone aggression test results are summarised below:

Borehole	Sample Depth (m)	Materials	pH	Chloride Content (mg/kg)	Sulphate Content (mg/kg)	Resistivity (ohm.m)
BH1	0.5-0.95	Residual Silty Clay	4.7	66	67	110
	2.1-2.5	Weathered Sandstone	5.0	<10	33	360
BH4	0.2-0.4	Silty Sandy Clay Fill	7.4	72	59	34
	3.0-3.2	Residual Silty Clay	5.2	<10	38	260

4 PRELIMINARY COMMENTS AND RECOMMENDATIONS

At the time of preparing this report, we were in the process of analysing the results of the insitu permeability testing completed in the boreholes and part way through the minimum 3-month groundwater level monitoring program as required by DPE 2022. From the results of the permeability testing and groundwater level monitoring, we will complete a seepage analysis for the proposed basement excavation in accordance with the requirements of DPE 2022. The results will be presented, as well as our updated advice (if appropriate), in a revision of this report.

4.1 Primary Geotechnical Issues

Based on the results of our investigation, we consider the following to be the primary geotechnical issues for the proposed mixed-use development:

1. Inadequate Geotechnical Site Coverage: The current borehole locations were limited by site access. In order to obtain adequate site coverage, we strongly recommend that four additional cored boreholes be drilled post-demolition to a minimum depth of about 6m below BEL.
2. Existing Buried Services: There are buried services within, and immediately adjacent to the site, including Sydney Water assets, as shown on Figure 2. We strongly recommend that the details of all buried services (i.e. type, depth to invert, location, etc), including the stormwater pipe encountered in BH3, be identified and accurately plotted onto the survey plan.

Prior to the installation of the retention piles, all non-redundant buried services which run through the site, will need to be diverted away from the site.

3. Rock Excavation: The proposed basement structure will require excavation into medium, high and very high strength sandstone and laminite bedrock. Ripping by dozers, and rock hammers fitted to excavators, will be required for effective removal.

4. Basement Structure: Based on the available groundwater level monitoring results, the proposed Basement 3 excavation will extend a maximum of approximately 2.3m below the groundwater level. From our initial review of the insitu permeability test results, relatively low permeabilities were measured in BH2 and BH4, but higher values were indicated in BH1.

Following completion and confirmation by our groundwater seepage analysis, we expect that a 'drained' basement structure will likely be feasible, subject to approval by WaterNSW. For the purpose of this report, we have assumed that a 'drained' basement structure will be feasible and approved by WaterNSW.

Internal dewatering will be required during basement excavation and construction. A dewatering license will need to be obtained from the relevant authorities (i.e. Council and/or WaterNSW) to allow temporary dewatering and discharge. In order to obtain the dewatering licence, a Dewatering Management Plan (DMP) will need to be prepared.

The soil and upper weathered bedrock profiles will require support by anchored and/or internally propped soldier pile walls or contiguous pile walls, with a tentative target toe level of RL70.0m for the retention piles. This target pile toe level must be confirmed following completion of the additional cored boreholes recommended in Item 1 above. The underlying bedrock can be tentatively cut vertically, subject to progressive geotechnical inspections during excavation and stabilisation, as appropriate.

5. Groundwater Seepage Inflows into the Proposed Basement Excavation: In order to achieve BEL, the groundwater within the excavation will need to be progressively pumped out, treated and disposed of into the stormwater system once approval has been obtained. As the bedrock contains few defects, we expect at this stage that any strong seeps emanating from open joints, bedding partings, etc. can be grouted dentally (if required) by a specialist contractor (e.g. Sonic Geo, Rix Ground Engineering, Menard Oceania, etc.).
6. Inherent Movement Behind the Basement Walls: Detailed dilapidation surveys will need to be completed on all neighbouring structures that fall within the zone of influence of the basement excavation. For simplicity, this zone of influence can be taken as twice the excavation depth when measured from the basement walls. This would therefore include portions of all structures which surround the site.

Within the proposed rock excavation, lateral movements will likely occur due to relief of insitu horizontal stresses. Such insitu horizontal stresses are typically in the order of 2MPa or greater, and therefore cannot be economically restrained. Based on published data and our experience with deep basements in Sydney, typical lateral movements due to stress relief are in the order of 0.5mm to 2mm per metre depth of excavation, depending on rock quality and presence of joints and bedding partings. The maximum lateral movements are likely to occur towards mid-height of unsupported rock faces, but can occur along pre-existing bedding planes and therefore at any level and along multiple bedding planes. This lateral movement may potentially cause settlement and damage to the neighbouring building and substation to the south-east.

Once the retention design has been finalised, a detailed three-dimensional finite element analysis (3D FEA) of the basement structure will need to be carried out to assess the deformations occurring

behind the basement walls. For the adjacent Sydney Water assets, a Specialist Engineering Assessment (SEA) in accordance with SEA 2021 [Reference 2] will most likely be required to confirm that the proposed basement works will have no adverse impacts on these assets. If the SEA is required, then we recommend that it be commenced as soon as possible as Sydney Water approval may be a protracted process.

7. Existing Basement Walls: Some of the existing buildings on site contain basement levels. Safe demolition of the existing basement walls will need to be assessed by MN Builders, the structural engineer and JK Geotechnics so that the works do not undermine or remove support from the site boundaries.
8. Monitoring: Settlement and inclinometer monitoring must be carried out around the perimeter of the site (including settlement monitoring on the adjoining facade of the neighbouring building to the south-east) from the commencement of excavation and up until all basement floor slabs and the ground floor slab have been constructed. The baseline readings must be taken prior to the commencement of excavation. The monitoring must be carried out after each stage of excavation/construction to assess the magnitude of movement behind the basement walls/cut faces. All measured movements will need to be compared to the predicted movements from the 3D FEA. A detailed Geotechnical Monitoring and Contingency Action Plan (GMCAP) will therefore need to be prepared. We can complete the GMCAP if commissioned to do so.

The effects of the above geotechnical issues on design and construction are detailed in the sections which follow.

4.2 Rock Classification

An indicative engineering classification of the bedrock has been carried out in accordance with Pells et al. (2019) [Reference 3] and is tabulated below. Due to the fine grained nature of the laminites, it has been classified as 'Shale'.

Borehole	Approx. Surface RL (mAHD)	Indicative Engineering Classification of Bedrock			
		Depths (m)			
		[RL at top of Unit (mAHD)]			
		Class V	Class IV	Class III	Class II/I
BH1	75.4	2.1 – 4.1 [73.3] Sandstone	-	-	4.1 – 9.9 [71.3] Sandstone & Shale
				9.9 – 12.3 [65.5] Sandstone	12.3 – 18.3 [63.1] Sandstone
BH1A	75.4	-	2.2 – 4.0 [73.2] Sandstone	-	4.0 – 9.3 [71.4] Sandstone & Shale
				9.3 – 10.7 [66.1] Sandstone	10.7 – 33.6 [64.7] Sandstone
BH2	74.6	1.9 – 4.2 [72.7] Shale & Sandstone	-	4.2 – 8.1 [70.4] Sandstone	8.1 – 19.4 [66.5] Shale & Sandstone
				19.4 – 22.5 [55.2] Sandstone	22.5 – 27.2 [52.1] Sandstone
BH4	76.3	4.5 – 5.6 [71.8] Sandstone & Shale	-	-	5.6 – 11.3 [70.7] Sandstone & Shale
				11.3 – 13.4 [65.0] Sandstone	13.4 – 28.6 [62.9] Sandstone

The rock classifications have been plotted on Figures 3 & 4. We note that extremely weathered siltstone of hard (soil) strength is not Class V rock.

4.3 Site Preparation

4.3.1 Dilapidation Surveys

Prior to commencement of any site works, we recommend that detailed internal and external dilapidation reports be completed on all neighbouring buildings, or parts thereof, located within the zone of influence of the basement excavation; that is, within a distance equivalent to twice the excavation depth when measured from the basement walls.

For example, for a 17m deep excavation, the horizontal zone of influence is 34m. For the surrounding neighbouring buildings, only those portions within the zone of influence need to be inspected. If, however, the dilapidation reports of these portions reveal substantial existing damage, then it may be prudent to extend the surveys to include more of these buildings. Dilapidation surveys of the surrounding roadways and footpaths may also be required by Council.

Dilapidation reports provide a record of existing conditions prior to commencement of any site works. The dilapidation reports would therefore be used as a benchmark against which to set vibration limits during rock excavation, and for assessing possible future claims for damage arising from the works.

The respective owners of the neighbouring buildings/assets should be asked to confirm in writing that the dilapidation reports present a fair assessment of existing conditions. As dilapidation reports are relied upon for the assessment of potential damage claims, they must be carried out thoroughly by reputable companies with all defects rigorously described (i.e. defect type, defect location, crack width, crack length, etc.) and photographed.

The dilapidation reports should be reviewed by JK Geotechnics and the structural engineer prior to commencement of the works.

4.3.2 Vibration Monitoring

Full-time quantitative vibration monitoring should be carried out on the neighbouring buildings to the south-east and also along the road boundaries from the commencement of demolition to the end of rock excavation to check that vibrations are below acceptable limits. The vibration monitoring must include geophones affixed onto the neighbouring buildings or onto the ground, and a warning system (e.g. flashing lights, audible alarm, etc.), which is set to trigger when the permissible vibration limit has been recorded. The locations of the geophones must be assessed following review of the dilapidation survey reports, and should be jointly nominated by JK Geotechnics and the acoustic consultant. The vibrations on the neighbouring buildings to the south-east and along the road boundaries should be tentatively limited to a peak vibration velocity (PVV) of 5mm/s, subject to review of the dilapidation survey reports and confirmation by JK Geotechnics and the structural engineer.

If higher vibrations are recorded, then they should be assessed against the attached Vibration Emission Design Goals as higher vibrations may be acceptable depending on the associated vibration frequency.

Reference should be made to Section 4.4.2 below if it is confirmed that transmitted vibrations are excessive during rock excavation.

4.3.3 Early Works

The early works will include diversion of any non-redundant buried services, demolition of the existing buildings (including their basements), slabs and pavements, removal of existing trees and other vegetation, and excavation of any topsoil, root affected soils and any deleterious or contaminated fill. Reference should be made to Section 5 for guidance on the offsite disposal of excavated soil and rock.

Care must be taken during the early works and subsequent bulk excavation not to undermine or remove support from the site boundaries. Prior to any site works commencing, a meeting must be held between MN Builders, the structural engineer and JK Geotechnics to assess how the existing basements can be safely demolished without undermining or removing support from the site boundaries.

4.4 Excavation

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

4.4.1 Excavation Conditions

The proposed basement levels will require excavation to depths between approximately 11m and 17m below existing grade. The proposed lift core will require localised excavation to 3m depth below BEL. Based on the borehole information, the excavation will extend through the soil and extremely weathered siltstone profiles, but mostly into low, medium, high and occasionally very high strength sandstone and laminite bedrock.

Excavation of the soil and extremely weathered siltstone, and the low strength bedrock could be carried out using a 'digging bucket' fitted to a large hydraulic excavator (say, at least 30 tonnes), using a ripping tyre where necessary, and/or by using a dozer.

'Hard ripping' or 'hard rock' excavation conditions must be expected for the medium, high and very high strength bedrock. Through the medium and high strength bedrock, ripping may only just be possible with a Caterpillar D9 dozer and a generous allowance must be made for hydraulic rock hammer assistance, particularly when excavating through very high strength rock. Notwithstanding, rock hammers will need to be used for detailed footing, lift core and trench excavations. Prospective contractors must review the attached borehole logs, rock core photographs and Point Load Strength Index test results, and make their own assessment on what plant and equipment will be required for effective excavation.

Grid rock sawing the bedrock would facilitate excavation and would also assist in dampening the transmission of ground borne vibrations to the adjoining neighbouring structures (i.e. would reduce vibrations) provided the base of the saw slot is maintained below excavation level. Rock sawing would also improve the aesthetics

of the finished rock faces where left exposed. Dust suppression by spraying with water must be carried out whenever rock saws are being used.

4.4.2 Potential Vibration Risks

Rock excavation using hydraulic rock hammers will need to be strictly controlled as there may be direct transmission of ground vibrations to the nearby neighbouring buildings. As discussed in Section 4.3.2, we recommend that quantitative vibration monitoring be carried out during rock excavation to check that vibrations are below acceptable limits. If it is confirmed that transmitted vibrations are excessive, then it would be necessary to change to alternative rock excavation methods such as a tighter grid of rock saw slots, a smaller rock hammer and/or a rotary rock grinder.

The following procedures are recommended to reduce vibrations when using rock hammers:

- Rock saw the faces of the proposed basement excavation. This will increase the path and effectively reduce the ground borne vibrations on the nearby buildings. The base of the rock saw slot must be maintained at a lower level than the adjacent excavation at all times;
- Maintain the rock hammer oriented towards the face and enlarge the excavation by breaking small wedges off the face;
- Operate the hammer in short burst only, to reduce amplification of vibrations; and
- Use excavation contractors with appropriate experience and a competent supervisor who is aware of vibration damage risks, etc. The contractor must have all appropriate statutory and public liability insurances and must be provided with a full copy of this report and any future additional geotechnical investigation reports.

4.4.3 Drainage

During excavation, groundwater will need to be progressively pumped out as levels deepen. Groundwater is expected in the bedrock profile below RL63.0m. Notwithstanding, shallower groundwater inflows into the basement excavation are expected to occur as local seepage flows just above the soil-bedrock interface and through defects (e.g. joints, bedding partings, clay seams, extremely weathered seams, etc.) within the bedrock, particularly during and after heavy and/or prolonged rain. If there is any strong seepage emanating from the bedrock profile this may need to be rectified progressively as levels deepen such as by rock grouting.

Seepage volumes into the excavation are expected to be controllable by conventional sump and pump discharge systems. Piped discharge from the drainage system into the stormwater system can only be completed once the groundwater has been treated (in accordance with a DMP) and approvals have been obtained. The excavation must be monitored as it progresses by MN Builders, JK Geotechnics and the hydraulic engineer to confirm the drainage requirements.

In the long-term, all substructure slabs should be constructed up against the exposed vertically cut rock faces, whether they are covered with shotcrete or not, in order to intercept any localised and intermittent

groundwater seeps that may occur during and after heavy and/or prolonged rain, or due to leaking hydraulic services in the adjoining properties to the south-east, or in the road reserves. The slab edges should be sealed up against the rock faces (or shotcrete), and should incorporate either a rebate or dish drain for discharge into the stormwater system.

4.5 Basement Retention

4.5.1 Retention Systems

We recommend that the proposed vertical cuts in the soil and rock profiles be supported by cast-in-situ retention systems, which will need to be installed prior to the commencement of excavation. The proposed vertical cuts should be supported by soldier pile walls with shotcrete infill panels. Where the retention system is located in areas which are highly sensitive to lateral movement, then consideration should be given to reducing the spacing of the soldier piles.

The piled walls must be progressively shotcreted and anchored, or internally propped, as the excavation proceeds (i.e. once the restraining point has been uncovered). Subject to confirmation by the additional geotechnical investigation recommended in Section 4.1, the piles toes can be founded above BEL, with a target pile toe level at RL70.0m (i.e. between the proposed Lower Ground and Basement 1 floor slabs). The basal row of temporary rock anchors will be required a short distance above the pile toes to prevent 'kick out'. In the long-term, the proposed Lower Ground floor slab (and overlying Upper Ground floor slab) will need to be designed to provide lateral support to the piled walls, at which time the temporary rock anchors can be de-stressed.

Due to the presence of medium, high and very high strength bedrock, only high torque piling rigs should be brought to site. Conventional bored piles can be used, and must only be cleaned out when concrete is ready to be poured. Cleaning buckets should be used to remove debris and any sludge from the base of each pile hole. All piles must be tremie poured due to the expected depths of the pile holes. We recommend that the pile drilling for the basement retention be inspected by JK Geotechnics during the initial stages and then periodically during the works to confirm that a satisfactory bearing stratum and adequate base cleanliness have been achieved. The prospective contractors must be provided with a full copy of this report and any future additional geotechnical investigation reports so that appropriate plant and equipment are brought to site.

Construction of the basement walls must be of high quality. The construction sequence must be fully specified and carefully controlled to reduce potential movements.

4.5.2 Retention Design Parameters

The major consideration in the selection of earth pressures for the design of the retention system is the need to limit deformations occurring outside the excavation. The characteristic earth pressure coefficients and parameters provided below may be adopted for the design of the retention systems.

For progressively anchored or propped soldier pile walls, we recommend the use of a trapezoidal earth pressure distribution, and a lateral earth pressure of $8H$ (kPa) for the soil, extremely weathered rock and Class V/IV bedrock profiles, where H is the retained height in metres (i.e. between ground surface level and the pile toe level, which will be in Class III or better-quality bedrock). The maximum lateral earth pressure should apply over the central 50% of the height of the retention system, tapering to zero at the crest and at the pile toe level. This lateral earth pressure assumes a horizontal retained surface. The shotcrete infill panels can be designed using a lateral earth pressure of $6H$ (kPa) for the soil and weathered rock profiles.

Any surcharge affecting the walls (e.g. traffic loading, nearby footings, inclined retained surfaces, mobile crane outrigger loads, etc.) should be allowed in the design using a K_0 value of 0.55. An average bulk unit weight of 21kN/m^3 should be adopted for the soil and weathered rock profiles.

The piled walls should be designed as a drained system with measures undertaken to induce complete and permanent drainage of the ground behind the walls. A strip drain with a weep hole outlet should be provided mid-span between soldier piles behind the shotcrete. All drainage water should be piped to the stormwater system.

The proposed lift core should be designed as a tanked structure with the design head of water taken as the proposed Basement 3 BEL. In order to resist the uplift pressures, the hydrostatic lift core slab could be restrained by increasing its deadweight, tension piles (conventional bored piles), micro-piles and/or permanent rock anchors. Further advice should be sought from JK Geotechnics with respect to the design parameters once the method of resisting the uplift pressures has been finalised. Care must be taken with the tanking and waterproofing details, particularly adjacent to the proposed lift core walls.

As an alternative and more advanced approach, the piled walls can be designed using appropriate FEM software, such as PLAXIS or similar. These programs also predict the movements behind the basement walls. The more frequently used retaining wall analysis program, WALLAP, is considered inappropriate for the design of the basement retention as it can only accurately model the soil and extremely weathered rock, not the Class V/IV bedrock, and cannot predict movements behind the basement walls, only of the wall itself. Due to the numerous geotechnical engineering inputs required to drive and rationalise FEA programs, the analyses should only be carried out by engineers with a good understanding of retaining wall design, and soil and rock mechanics. We are not in favour of software which treats the soil and bedrock profiles as 'equivalent springs' as these are not geotechnical parameters.

Subject to completion of, and confirmation by, the additional geotechnical investigation recommended in Section 4.1, the possibly slightly conservative parameters tabulated below may be tentatively adopted for the FEA analyses:

Material Description	Effective Cohesion, c' (kPa)	Effective Angle of Friction, ϕ'	Elastic Modulus, E (MPa)	Poisson's Ratio	Bulk Unit Weight, γ (kN/m ³)
Residual Silty Clay of Very Stiff to Hard Strength, and Extremely Weathered Siltstone	4	26°	25	0.3	20
Class V/IV Bedrock	20	26°	100	0.3	22

We can assist the structural engineer with the FEM analyses if commissioned to do so. Alternatively, if the piled wall design is completed as part of a design and construct contract, then it must be jointly peer reviewed by JK Geotechnics and the structural engineer, and amended as appropriate.

As detailed in Section 4.1, design of the lateral support systems is a 3D problem and should be analysed as such, due to the close proximity of the neighbouring buildings to the south-east, and the fact that insitu stresses need to be incorporated. We could carry out the 3D FEA and assist the structural engineer with the analysis of the retention system, if commissioned to do so.

Once the predicted movements behind the lateral support systems have been assessed by the 3D FEA, a GMCAP, as discussed in Section 4.1, should be prepared. The purpose of the plan is to compare the actual movements of the retention system with the predicted movements, and to outline appropriate responses should the predicted movements be exceeded.

4.5.3 Temporary Rock Anchors

For temporary rock anchors, permission must be sought from the neighbouring property owners (including Council) prior to installation. Requests for permission should commence early in the design process as our experience has shown that it can take significant time for such permission to be granted. If permission is not forthcoming, then the alternative is to provide lateral support by internal bracing or propping.

Temporary rock anchors bonded at least 3m into medium strength or stronger bedrock, beyond a 45° line inclined up from the pile toe, may be tentatively designed for an allowable bond stress of 500kPa. Each anchor should have a minimum free length of 3m.

All anchors must be proof loaded to at least 1.3 times their design working load before locking off at about 85% of the working load. Lift-off tests should be carried out on at least 10% of the anchors about 4 days following locking off to confirm that the anchors are holding their load. The testing may allow an upgrading of the above bond stress. We recommend that only experienced contractors be considered for the anchor installations. We have assumed that permanent lateral support of the basement walls will be provided by the proposed structure, after which time the rock anchors can be de-stressed.

4.5.4 Unsupported Cuts in the Bedrock

Below the toes of the retention piles, the proposed basement excavation is expected to extend through competent medium, high and very high strength bedrock (i.e. Class III or better quality rock), which can be cut vertically, on condition that the exposed cut faces are progressively inspected by an experienced geotechnical engineer or engineering geologist as the excavation proceeds at no more than 1.5m depth intervals and on completion, so that any wedges that could detach are identified and appropriate support measures implemented (e.g. rock bolts, shotcrete and mesh, etc.). Our preference is that each stage of excavation progresses no closer than a 1.5m horizontal distance from the design alignment of the cut faces so that all defects can be mapped and stabilisation measures assessed prior to removal of the (1.5m wide) perimeter ‘bench’. This excavation and inspection methodology would provide early warning of a potentially unstable rock wedge.

As a guide, the following rock face support should be expected for the bedrock:

Bedrock Classification	Indicative Rock Face Support Measures
Class III Sandstone (Mittagong Formation & Hawkesbury Sandstone)	<ul style="list-style-type: none"> • Localised pattern bolting in fractured zones; • Mesh, shotcrete and dowels <u>or</u> fibrecrete and dowels in fractured zones and weathered bands; and • Isolated rock bolting of potentially unstable rock wedges.
Class II/I Shale (Mittagong Formation)	<ul style="list-style-type: none"> • Pattern bolting, mesh and shotcrete <u>or</u> pattern bolting and fibrecrete.
Class II/I Rock (Hawkesbury Sandstone)	<ul style="list-style-type: none"> • Isolated rock bolting of potentially unstable rock wedges.

These indicative rock face support measures will need to be confirmed during our site inspections. Provision must be made in the construction program and budget for the inspections and stabilisation works.

Where dowels and rock bolts will be permanent, consideration must be given to long-term corrosion. Stainless steel bolts or hot dipped galvanised bolts with protective sheathing may be required. We forewarn that permanent dowels and rock bolts are unlikely to be permitted beyond the site boundaries. Here, shotcrete/fibrecrete facing may need to be incorporated into the stabilisation design, which in turn will need to be supported by the basement, Lower Ground and Upper Ground floor slabs in the long term.

4.6 Footings

4.6.1 Geotechnical Design

Pad and strip footings uniformly founded in the Class II/I sandstone below BEL may be tentatively designed for an allowable bearing pressure of 6MPa or an ultimate bearing pressure of 40MPa. All columns located outside the basement footprint should be supported on conventional bored piles socketed below a 45° line inclined up from BEL, and also tentatively designed for an allowable end bearing pressure of 6MPa or an ultimate end bearing pressure of 40MPa.

Conventional bored piles socketed below the 45° line and into Class II/I sandstone can be tentatively designed for allowable and ultimate shaft adhesion values of 600kPa and 1,500kPa in compression, respectively. These shaft adhesion values are on condition that the pile shaft is suitably roughened to a Roughness Class equivalent to at least R2. R2 roughness is defined as grooves of depth 1mm to 4mm, and width greater than 2mm, and at a spacing of 50mm to 200mm.

The allowable bearing pressures provided above are based upon serviceability criteria of deflections at the footing/pile base of less than 1% of the minimum footing dimension/pile diameter. We note that these footing settlements will be of an elastic nature and are expected to occur as construction proceeds.

For ultimate limit state design, settlement limitations to the structure will still need to be satisfied and can be tentatively estimated using an Elastic Modulus value of 1,200MPa and Poisson's Ratio value of 0.2 for the Class II/I sandstone. It should be noted that the ultimate bearing pressures must be used in conjunction with an appropriate '*Basic Geotechnical Strength Reduction Factor*' (ϕ_{gb}), as defined in Clause 4.3.2 of AS2159-2009 '*Piling – Design and Installation*'. Based on our assessment, which assumes that the foundation material at each footing/pile location will be inspected and tested by JK Geotechnics and that the additional investigation recommended in Section 4.1 is completed, we tentatively recommend a ϕ_{gb} value of 0.61 for a low redundancy system. Notwithstanding, the designer must make their own assessment of the ϕ_{gb} value.

The Class II/I sandstone bedrock is potentially suitable for a higher allowable bearing pressure and ultimate bearing pressure in the order of 10MPa and 80MPa, respectively, subject to the findings of the additional investigation recommended in Section 4.1 and the proximity of the Class III sandstone below BEL, as encountered in BH2. Notwithstanding, significant additional rock proving, in excess of the four additional cored boreholes, would be required to justify the higher design bearing pressures.

Inclined joints, which could potentially isolate unstable rock wedges, were encountered in the cored boreholes. As such, pad and strip footings located in close proximity to the proposed lift core excavation must be tentatively founded below a 45° line inclined up from the base. However, if flatter, problematic joints are observed during our inspection of the lift core excavation, then the nearby pad and strip footings may need to be appropriately deepened, or replaced as bored piles.

All pad and strip footing excavations must be cleaned out and then inspected by a geotechnical engineer prior to the installation of the reinforcement cages. As part of the inspection process, at least one third of

all pad and strip footing excavations will need to be spoon tested. Spoon testing includes drilling a minimum 50mm diameter hole from the base of the footing excavation to a depth of at least 1.5 times the minimum footing width, but to a depth of no less than 1.5m. The hole is then flushed clean and filled with clear water (to soften any seams) and then scraped with a grooving tool to assess the location and thickness of any defects below the base of the footing excavation. Following inspection and approval, all footings should be poured without delay to reduce deterioration due to weathering.

All piles located outside the basement footprint must be cleaned out, inspected and poured on the same day as drilling. We also recommend that the roughness of the rock socket be inspected using a downhole camera system. All pile holes should be cleaned out using a cleaning bucket (for all pile diameters) for effective removal of sludge and loose material. Due to the expected groundwater seepage, the piles should only be cleaned out when concrete is ready to be tremie poured. All piles must be inspected by a geotechnical engineer, and compared to the borehole information to confirm that a satisfactory bearing stratum has been achieved.

4.6.2 Earthquake Design Parameters

The following parameters should be adopted for earthquake design in accordance with AS1170.4:2024 'Structural Design Actions, Part 4: Earthquake Actions in Australia':

- Hazard Factor (Z) = 0.08
- Site Subsoil Class (Retention Design) = Class C_e
- Site Subsoil Class (Footing Design) = Class B_e

4.6.3 Exposure Classification

The soil and weathered sandstone aggression test results have indicated moderately acidic to alkaline conditions, low chloride and sulfate contents, and high (favourable) resistivity values. In accordance with Table 4.8.1 of AS3600:2018 'Concrete Structures' and Table 6.4.2(C) of AS2159-2009, the exposure classification to concrete footings and concrete piles is 'A2' and 'mild', respectively.

4.7 Basement 3 Floor Slab

Subject to approval from Council or WaterNSW, a drained Basement 3 floor slab should be constructed independent of the building footings and walls (i.e. designed as a 'floating' slab). Slab joints should be designed to resist shear forces but not bending moments by providing dowelled or keyed joints. The 'floating' slab should be provided with at least a 100mm thick sub-base of good quality, durable, single size, crushed rock (free of fines) such as 'Blue Metal' gravel or crushed concrete aggregate (free of fines), which will also act as underfloor drainage.

The underfloor drainage should include a sump and pump dewatering system. Groundwater seepage monitoring should be carried out during basement excavation prior to finalising the design of the pump out

facility. In order to avoid flooding of proposed Basement 3, appropriately sized sumps each with an automatic level control pump will be required to intermittently discharge the seepage water to the stormwater system. Outlets into the stormwater system will require approval from the relevant authorities.

4.8 Further Geotechnical Input

The following is a summary of the further geotechnical input which is required and which has been detailed in the preceding sections of this report:

- Review and update of this report on completion of the 3-month groundwater level monitoring and seepage analysis, as required by WaterNSW;
- Assessment on how the existing basements can be safely demolished without undermining or removing support from the site boundaries;
- Additional geotechnical investigation post-demolition;
- Additional advice once the method of resisting the uplift pressures of lift core has been finalised;
- Preparation of a Dewatering Management Plan (DMP);
- FEM analysis of the basement retention system and excavation;
- Preparation of a Geotechnical Monitoring and Contingency Action Plan (GMCAP);
- Dilapidation survey reports on all neighbouring structures, or parts thereof, located within the zone of influence of the basement excavation;
- Review of the dilapidation survey reports;
- Quantitative vibration monitoring on the neighbouring buildings to the south-east and along the road boundaries;
- Inspection of perimeter pile wall drilling;
- Proof testing and lift-off testing of temporary rock anchors for the basement walls;
- Progressive rock face inspections (below toes of the piled walls) as the excavation proceeds;
- Groundwater monitoring of seepage volumes;
- Internal footing inspections, including spoon testing;
- Pile inspections for columns located outside the basement footprint.



5 GENERAL COMMENTS

The preliminary recommendations presented in this report include specific issues to be addressed during the design and construction phases of the project. In the event that any of the advice presented in this report is not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

Occasionally, the subsurface conditions between the completed boreholes may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

This report provides preliminary advice on geotechnical aspects for the proposed civil and structural design. Contract Documents and Specifications should not be prepared based on this report.

A waste classification is required for any soil and/or bedrock excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), Excavated Natural Material (ENM), General Solid, Restricted Solid or Hazardous Waste. Analysis can take up to seven to ten working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) could be expected. We strongly recommend that this requirement is addressed prior to the commencement of excavation on site.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

References

1. Minimum Requirements for Building Site Groundwater Investigations and Reporting (2022). NSW Department of Planning and Environment. Ref.PUB20/940, Version V02.2210, October 2022.
2. Sydney Water Specialist Engineering Assessment (SEA) Procedure (2021). Sydney Water, Doc No. D0001870, Version 1, 19 February 2021.
3. Pells, P. J. N., Mostyn, G., Bertuzzi, R. and Wong, P. K. (2019) Classification of Sandstones and Shales in the Sydney Region: A Forty Year Review. Australian Geomechanics, Volume 54, No. 2, June 2019.

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT



Client: ST LEONARDS PROPERTY UNIT TRUST **Ref No:** 37263AE
Project: PROPOSED MIXED USE DEVELOPMENT **Report:** A
Location: 46-52 NICHOLSON STREET & 59-67 CHRISTIE STREET, ST LEONARDS, NSW **Report Date:** 28/07/25

Page 1 of 8

BOREHOLE NUMBER	DEPTH (m)	$I_{s(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
1	2.83 - 2.87	2.1	42	A
	3.20 - 3.22	2.8	56	A
	3.59 - 3.62	0.6	12	A
	3.85 - 3.89	0.8	16	A
	4.25 - 4.29	2.1	42	A
	4.77 - 4.82	1.3	26	A
	5.16 - 5.20	1.7	34	A
	5.75 - 5.79	1.2	24	A
	6.27 - 6.30	1.6	32	A
	6.72 - 6.75	1.6	32	A
	7.24 - 7.27	1.7	34	A
	7.82 - 7.85	1.7	34	A
	8.16 - 8.19	2.5	50	A
	8.91 - 8.94	3.7	74	A
	9.18 - 9.22	3.5	70	A
	9.75 - 9.80	3.3	66	A
	10.22 - 10.25	2.3	46	A
	10.71 - 10.74	1.7	34	A
	11.07 - 11.10	1.3	26	A
	11.66 - 11.70	1	20	A
	12.21 - 12.25	1.1	22	A
	12.82 - 12.86	1.5	30	A
	13.27 - 13.32	1.4	28	A
	14.34 - 14.38	1.5	30	A
	14.83 - 14.86	1.2	24	A

NOTE: SEE PAGE 8

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Page 2 of 8

BOREHOLE NUMBER	DEPTH (m)	$I_{s(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
1	15.22 - 15.24	1.3	26	A
	15.74 - 15.78	1.8	36	A
	16.17 - 16.21	1.6	32	A
	16.82 - 16.85	1	20	A
	17.14 - 17.17	1.7	34	A
	17.76 - 17.81	1.2	24	A
	18.11 - 18.15	1.2	24	A
1A	2.69 - 2.74	2.7	54	A
	2.83 - 2.87	2.1	42	A
	3.12 - 3.16	1.8	36	A
	3.79 - 3.81	0.2	4	A
	3.83 - 3.87	0.5	10	A
	4.17 - 4.21	1.6	32	A
	4.86 - 4.90	1.3	26	A
	5.77 - 5.81	1.4	28	A
	6.21 - 6.24	1.3	26	A
	6.72 - 6.76	1.6	32	A
	7.06 - 7.09	1.6	32	A
	7.78 - 7.81	2.1	42	A
	8.12 - 8.16	3.6	72	A
	8.78 - 8.80	2.2	44	A
	9.18 - 9.21	2	40	A
9.69 - 9.74	2.4	48	A	
10.22 - 10.26	1.5	30	A	
10.82 - 10.86	0.9	18	A	

NOTE: SEE PAGE 8

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Client: ST LEONARDS PROPERTY UNIT TRUST **Ref No:** 37263AE
Project: PROPOSED MIXED USE DEVELOPMENT **Report:** A
Location: 46-52 NICHOLSON STREET & 59-67 CHRISTIE STREET, ST LEONARDS, NSW **Report Date:** 28/07/25

Page 3 of 8

BOREHOLE NUMBER	DEPTH (m)	$I_{s(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
1A	11.09 - 11.13	1.4	28	A
	11.86 - 11.89	1.3	26	A
	12.10 - 12.14	0.8	16	A
	12.71 - 12.74	0.9	18	A
	13.12 - 13.15	1	20	A
	13.81 - 13.85	1.2	24	A
	14.12 - 14.15	1	20	A
	14.75 - 14.80	1.3	26	A
	15.13 - 15.17	1.3	26	A
	15.63 - 15.68	1.7	34	A
	16.83 - 16.87	1.4	28	A
	17.14 - 17.18	1.3	26	A
	17.82 - 17.86	1.4	28	A
	18.11 - 18.16	1.3	26	A
	18.81 - 18.85	1.5	30	A
	19.12 - 19.15	1.3	26	A
	19.83 - 19.86	1.7	34	A
	20.17 - 20.22	1.3	26	A
	20.87 - 20.91	1.6	32	A
	21.11 - 21.14	1.5	30	A
	21.70 - 21.73	1.3	26	A
	22.13 - 22.17	1.4	28	A
	22.83 - 22.87	1.4	28	A
	23.08 - 23.13	1.8	36	A
	23.81 - 23.86	1.8	36	A

NOTE: SEE PAGE 8

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT



Client: ST LEONARDS PROPERTY UNIT TRUST **Ref No:** 37263AE

Project: PROPOSED MIXED USE DEVELOPMENT **Report:** A

Location: 46-52 NICHOLSON STREET & 59-67 CHRISTIE STREET, ST LEONARDS, NSW **Report Date:** 28/07/25

Page 4 of 8

BOREHOLE NUMBER	DEPTH (m)	$I_{s(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
1A	24.11 - 24.15	1.2	24	A
	24.80 - 24.84	1.5	30	A
	25.11 - 25.16	1.5	30	A
	25.78 - 25.82	1.9	38	A
	26.06 - 26.11	1.6	32	A
	26.76 - 26.80	1.2	24	A
	27.20 - 27.24	1.8	36	A
	27.83 - 27.86	1.7	34	A
	28.11 - 28.15	1.5	30	A
	28.76 - 28.80	1.8	36	A
	29.16 - 29.19	2.3	46	A
	29.86 - 29.89	1.5	30	A
	30.30 - 30.34	1.4	28	A
	30.73 - 30.77	1.8	36	A
	31.03 - 31.07	1	20	A
2	31.90 - 31.93	1.3	26	A
	32.50 - 32.53	1.7	34	A
	32.66 - 32.69	1	20	A
	3.62 - 3.64	1	20	A
	4.29 - 4.31	0.7	14	A
	4.87 - 4.90	1.3	26	A
	5.23 - 5.27	0.9	18	A
5.79 - 5.82	0.8	16	A	
6.11 - 6.14	2.3	46	A	
6.61 - 6.63	1	20	A	

NOTE: SEE PAGE 8

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT



Client: ST LEONARDS PROPERTY UNIT TRUST **Ref No:** 37263AE

Project: PROPOSED MIXED USE DEVELOPMENT **Report:** A

Location: 46-52 NICHOLSON STREET & 59-67 CHRISTIE STREET, ST LEONARDS, NSW **Report Date:** 28/07/25

Page 5 of 8

BOREHOLE NUMBER	DEPTH (m)	$I_{s(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
2	7.10 - 7.13	0.9	18	A
	7.78 - 7.81	2.4	48	A
	8.21 - 8.24	3.5	70	A
	8.86 - 8.88	1.9	38	A
	9.34 - 9.36	3	60	A
	9.84 - 9.86	2.3	46	A
	10.11 - 10.14	0.5	10	A
	10.69 - 10.72	1.3	26	A
	11.25 - 11.28	1.1	22	A
	11.85 - 11.88	1	20	A
	12.07 - 12.09	1	20	A
	12.73 - 12.75	0.7	14	A
	13.17 - 13.20	1	20	A
	13.83 - 13.86	1	20	A
	14.16 - 14.19	1	20	A
	14.73 - 14.75	1.2	24	A
	15.21 - 15.24	1.3	26	A
	15.80 - 15.83	1.3	26	A
	16.27 - 16.30	1	20	A
	16.87 - 16.90	1.9	38	A
	17.23 - 17.27	2	40	A
	17.73 - 17.77	1.8	36	A
	18.35 - 18.39	1	20	A
	18.89 - 18.93	1.6	32	A
	19.13 - 19.17	1.3	26	A

NOTE: SEE PAGE 8

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT



Client: ST LEONARDS PROPERTY UNIT TRUST **Ref No:** 37263AE

Project: PROPOSED MIXED USE DEVELOPMENT **Report:** A

Location: 46-52 NICHOLSON STREET & 59-67 CHRISTIE STREET, ST LEONARDS, NSW **Report Date:** 28/07/25

Page 6 of 8

BOREHOLE NUMBER	DEPTH (m)	$I_{s(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
2	19.67 - 19.71	1.1	22	A
	20.08 - 20.11	0.9	18	A
	20.22 - 20.25	0.8	16	A
	21.27 - 21.30	1.7	34	A
	21.70 - 21.73	1.2	24	A
	21.87 - 21.90	1.8	36	A
	22.18 - 22.21	1.9	38	A
	22.55 - 22.58	1.8	36	A
	23.07 - 23.09	1.4	28	A
	23.71 - 23.75	1	20	A
	24.27 - 24.30	1.2	24	A
	24.82 - 24.86	1.7	34	A
	25.19 - 25.23	0.9	18	A
	25.70 - 25.75	2	40	A
	26.10 - 26.13	1.1	22	A
26.62 - 26.66	1.3	26	A	
27.03 - 27.06	0.9	18	A	
27.18 - 27.21	1.7	34	A	
4	5.33 - 5.36	1.3	26	A
	5.80 - 5.83	1.8	36	A
	6.13 - 6.16	1.5	30	A
	6.68 - 6.71	3	60	A
	7.20 - 7.22	2.4	48	A
	7.76 - 7.79	1.2	24	A
	8.49 - 8.51	3	60	A

NOTE: SEE PAGE 8

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT



Client: ST LEONARDS PROPERTY UNIT TRUST **Ref No:** 37263AE

Project: PROPOSED MIXED USE DEVELOPMENT **Report:** A

Location: 46-52 NICHOLSON STREET & 59-67 CHRISTIE STREET, ST LEONARDS, NSW **Report Date:** 28/07/25

Page 7 of 8

BOREHOLE NUMBER	DEPTH (m)	$I_{s(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
4	8.76 - 8.80	2	40	A
	9.08 - 9.11	2.9	58	A
	9.59 - 9.62	2.1	42	A
	10.17 - 10.19	3.7	74	A
	10.79 - 10.82	1.2	24	A
	11.25 - 11.27	2.3	46	A
	11.74 - 11.78	1.4	28	A
	12.60 - 12.62	0.8	16	A
	12.68 - 12.70	0.8	16	A
	13.17 - 13.19	1.1	22	A
	13.87 - 13.89	1.1	22	A
	14.26 - 14.28	1.1	22	A
	14.78 - 14.81	0.9	18	A
	15.20 - 15.23	1.1	22	A
	15.88 - 15.90	1	20	A
	16.43 - 16.46	1.7	34	A
	16.77 - 16.80	1.1	22	A
	17.08 - 17.12	1.2	24	A
	17.66 - 17.70	1.8	36	A
	18.21 - 18.24	1.8	36	A
	18.86 - 18.90	1.7	34	A
	19.13 - 19.17	1.6	32	A
	19.70 - 19.74	1.1	22	A
	20.21 - 20.25	1.3	26	A
	20.75 - 20.78	1.3	26	A

NOTE: SEE PAGE 8

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT



Client: ST LEONARDS PROPERTY UNIT TRUST **Ref No:** 37263AE

Project: PROPOSED MIXED USE DEVELOPMENT **Report:** A

Location: 46-52 NICHOLSON STREET & 59-67 CHRISTIE STREET, ST LEONARDS, NSW **Report Date:** 28/07/25

Page 8 of 8

BOREHOLE NUMBER	DEPTH (m)	$I_{s(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
4	21.17 - 21.21	1.3	26	A
	21.80 - 21.84	1.2	24	A
	22.09 - 22.13	1.2	24	A
	22.75 - 22.79	1.1	22	A
	23.18 - 23.22	1.5	30	A
	23.61 - 23.64	2	40	A
	24.18 - 24.21	2	40	A
	24.73 - 24.76	1.8	36	A
	25.31 - 25.34	1.3	26	A
	25.77 - 25.80	1.3	26	A
	26.15 - 26.18	1.9	38	A
	26.81 - 26.86	1.8	36	A
	27.27 - 27.30	1.6	32	A
	27.84 - 27.88	1.9	38	A
28.50 - 28.52	1.8	36	A	

NOTES

1. In the above table, testing was completed in test direction A for the axial direction, D for the diametral direction, B for the block test and L for the lump test.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RMS T223.
4. For reporting purposes, the $I_{s(50)}$ has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa.
5. The estimated Unconfined Compressive Strength was calculated from the Point Load Strength Index based on the correlation provided in AS1726:2017 'Geotechnical Site Investigations' and rounded off to the nearest whole number: U.C.S. = 20 $I_{s(50)}$.

CERTIFICATE OF ANALYSIS 371536

Client Details

Client	JK Geotechnics
Attention	Salvatore Wedde
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	37263AE, 46-52 Nicholson St & 59-67 Christie St
Number of Samples	2 Soil
Date samples received	29/01/2025
Date completed instructions received	29/01/2025

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.
 Samples were analysed as received from the client. Results relate specifically to the samples as received.
 Results are reported on a dry weight basis for solids and on an as received basis for other matrices.
Please refer to the last page of this report for any comments relating to the results.

Report Details

Date results requested by	05/02/2025
Date of Issue	05/02/2025
NATA Accreditation Number 2901. This document shall not be reproduced except in full.	
Accredited for compliance with ISO/IEC 17025 - Testing. Tests not covered by NATA are denoted with *	

Results Approved By

Diego Bigolin, Inorganics Supervisor

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil			
Our Reference		371536-1	371536-2
Your Reference	UNITS	1	1
Depth		0.5-0.95	2.1-2.5
Date Sampled		22/01/2025	22/01/2025
Type of sample		Soil	Soil
Date prepared	-	30/01/2025	30/01/2025
Date analysed	-	30/01/2025	30/01/2025
pH 1:5 soil:water	pH Units	4.7	5.0
Chloride, Cl 1:5 soil:water	mg/kg	66	<10
Sulphate, SO4 1:5 soil:water	mg/kg	67	33
Resistivity in soil*	ohm m	110	360

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity (non NATA). Resistivity (calculated) may not correlate with results otherwise obtained using Resistivity-Current method, depending on the nature of the soil being analysed.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

Client Reference: 37263AE, 46-52 Nicholson St & 59-67 Christie St

QUALITY CONTROL: Misc Inorg - Soil					Duplicate			Spike Recovery %		
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			30/01/2025	[NT]	[NT]	[NT]	[NT]	30/01/2025	[NT]
Date analysed	-			30/01/2025	[NT]	[NT]	[NT]	[NT]	30/01/2025	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	101	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	107	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	114	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]

Result Definitions

NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.
Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.	
The recommended maximums for analytes in urine are taken from "2018 TLVs and BEIs", as published by ACGIH (where available). Limit provided for Nickel is a precautionary guideline as per Position Paper prepared by AIOH Exposure Standards Committee, 2016.	
Guideline limits for Rinse Water Quality reported as per analytical requirements and specifications of AS 4187, Amdt 2 2019, Table 7.2	

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.

Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals (not SPOCAS); 60-140% for organics/SPOCAS (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Where matrix spike recoveries fall below the lower limit of the acceptance criteria (e.g. for non-labile or standard Organics <60%), positive result(s) in the parent sample will subsequently have a higher than typical estimated uncertainty (MU estimates supplied on request) and in these circumstances the sample result is likely biased significantly low.

Measurement Uncertainty estimates are available for most tests upon request.

Analysis of aqueous samples typically involves the extraction/digestion and/or analysis of the liquid phase only (i.e. NOT any settled sediment phase but inclusive of suspended particles if present), unless stipulated on the Envirolab COC and/or by correspondence. Notable exceptions include certain Physical Tests (pH/EC/BOD/COD/Apparent Colour etc.), Solids testing, total recoverable metals and PFAS where solids are included by default.

Samples for Microbiological analysis (not Amoeba forms) received outside of the 2-8°C temperature range do not meet the ideal cooling conditions as stated in AS2031-2012.

Report Comments

MISC_INORG_DRY: pH

Samples were out of the recommended holding time for this analysis.

CERTIFICATE OF ANALYSIS 382739

Client Details

Client	JK Geotechnics
Attention	Jacob Feng
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	37263AE, Proposed Mixed Use Development
Number of Samples	2 Soil
Date samples received	06/06/2025
Date completed instructions received	06/06/2025

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.
 Samples were analysed as received from the client. Results relate specifically to the samples as received.
 Results are reported on a dry weight basis for solids and on an as received basis for other matrices.
Please refer to the last page of this report for any comments relating to the results.

Report Details

Date results requested by	16/06/2025
Date of Issue	29/07/2025
Reissue Details	Revised COA R01 has been re-issued as client requested sample depth amendment.
NATA Accreditation Number 2901. This document shall not be reproduced except in full.	
Accredited for compliance with ISO/IEC 17025 - Testing. Tests not covered by NATA are denoted with *	

Results Approved By

Diego Bigolin, Inorganics Supervisor

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil			
Our Reference		382739-1	382739-2
Your Reference	UNITS	4	4
Depth		0.2-0.4	3.0-3.2
Date Sampled		30/05/2025	30/05/2025
Type of sample		Soil	Soil
Date prepared	-	16/06/2025	16/06/2025
Date analysed	-	16/06/2025	16/06/2025
pH 1:5 soil:water	pH Units	7.4	5.2
Chloride, Cl 1:5 soil:water	mg/kg	72	<10
Sulphate, SO4 1:5 soil:water	mg/kg	59	38
Resistivity in soil*	ohm m	34	260

Client Reference: 37263AE, Proposed Mixed Use Development

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity (non NATA). Resistivity (calculated) may not correlate with results otherwise obtained using Resistivity-Current method, depending on the nature of the soil being analysed.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

Client Reference: 37263AE, Proposed Mixed Use Development

QUALITY CONTROL: Misc Inorg - Soil				Duplicate				Spike Recovery %		
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			13/06/2025	[NT]	[NT]	[NT]	[NT]	13/06/2025	[NT]
Date analysed	-			13/06/2025	[NT]	[NT]	[NT]	[NT]	13/06/2025	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	99	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	91	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	97	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]

Result Definitions

NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.

Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

Air volumes are typically provided by customers (often as flow rate(s) and sampling time(s) and/or simply volumes) sampled or exposure times (determines 'volume' passive badges are exposed to)). Hence in such circumstances the volume measurement is inevitably not covered by Envirolab's NATA accreditation. An exception may occur where Envirolab Newcastle does the sampling where accreditation exists for certain types of sampling and hence volume determination(s). Note air volumes are often used to determine concentrations for dust and/or analyses on filters, sorbents and in impingers. For canister sampling, the air volume is covered by Envirolab's NATA accreditation.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals (not SPOCAS); 60-140% for organics/SPOCAS (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Measurement Uncertainty estimates are available for most tests upon request.

Analysis of aqueous samples typically involves the extraction/digestion and/or analysis of the liquid phase only (i.e. NOT any settled sediment phase but inclusive of suspended particles if present), unless stipulated on the Envirolab COC and/or by correspondence. Notable exceptions include certain Physical Tests (pH/EC/BOD/COD/Apparent Colour etc.), Solids testing, total recoverable metals and PFAS where solids are included by default.

For Dust Deposit Gauge (DDG) analysis the sampling, sampling period and funnel exposure area do not fall under Envirolab's NATA accreditation (unless the Newcastle laboratory where responsible for the sampling), hence the annotation on the DDG units of reporting.

Urine Analysis - The BEI values listed are taken from the 2022 edition of "TLVs and BEIs Threshold Limits" by ACGIH.

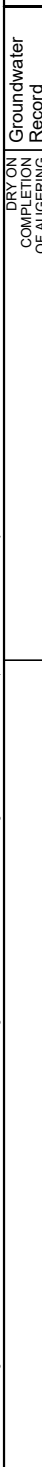
Report Comments

Samples were out of the recommended holding time for this analysis pH.

BOREHOLE LOG

Client: ST LEONARDS PROPERTY UNIT TRUST
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 46-52 NICHOLSON STREET & 59-67 CHRISTIE STREET, ST LEONARDS, NSW

Job No.: 37263AE **Method:** SPIRAL AUGER **R.L. Surface:** ~75.4 m
Date: 22/1/25 TO 23/1/25 **Datum:** AHD
Plant Type: JK330 **Logged/Checked By:** S.W./A.J.H.

Groundwater Record	SAMPLES				Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
DRY ON COMPLETION OF AUGERING 						75		CH	CONCRETE: 150mm.t Silty CLAY: high plasticity, red brown, trace of fine to medium grained ironstone gravel.	w>PL	VSt	210 220 230	6mm DIA. REINFORCEMENT, 60mm TOP COVER RESIDUAL	
					N = 14 4,7,7	1			as above, but light grey mottled red brown.	w-PL	VSt - Hd	>600 >600 >600	SCREEN: 4.45kg(<10L), 0.5-1.0m, NO FCF	
					N = 20 5,9,11	2		CI	Silty CLAY: medium plasticity, light grey and red brown, with iron indurated sandstone bands.	w<PL	Hd	>600 >600 >600	SCREEN: 1.6kg(<10L), 1.5-2.0m, NO FCF	
						2		-	SANDSTONE: fine to medium grained, grey and brown, with extremely weathered seams.	HW	L		MITTAGONG FORMATION LOW 'TC' BIT RESISTANCE	
						3			REFER TO CORED BOREHOLE LOG	MW	M - H		MODERATE TO HIGH RESISTANCE GROUNDWATER MONITORING WELL INSTALLED TO 18.29m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 9.29m TO 18.29m. CASING 0.15m TO 9.29m. 2mm SAND FILTER PACK 8.15m TO 18.29m. BENTONITE SEAL 3.01m TO 8.15m. BACKFILLED WITH SAND AND CUTTINGS TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.	
						4								
						5								
						6								
						69								

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 37263AE STLEONARDS.GPJ <-DrawingFile>> 25/08/2025 12:25 10.01.00.01 Dajjal Lab and In Situ Tool - DGD Lib JK 9.02.4.2019.05.31 P.fj JK 9.01.0.2019.02.20

CORED BOREHOLE LOG

Client:	ST LEONARDS PROPERTY UNIT TRUST
Project:	PROPOSED MIXED USE DEVELOPMENT
Location:	46-52 NICHOLSON STREET & 59-67 CHRISTIE STREET, ST LEONARDS, NSW

Job No.: 37263AE	Core Size: HQ	R.L. Surface: ~75.4 m
Date: 22/1/25 TO 23/1/25	Inclination: VERTICAL	Datum: AHD
Plant Type: JK330	Bearing: N/A	Logged/Checked By: S.W./A.J.H.

Water Loss/Level Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_p(50)$	SPACING (mm)	DEFECT DETAILS		Formation
									Specific	General	
	73			START CORING AT 2.70m							
	72	3		SANDSTONE: fine to medium grained, orange brown and grey, bedded at 0-15°.	MW	H	2.1		(2.74m) CS, 5°, 18 mm.t (2.81m) Be, 5°, P, R, Cn		Mittagong Formation
				NO CORE 0.08m					(2.98m) XWS, 5°, 30 mm.t (3.07m) CS, 5°, 17 mm.t (3.14m) Be, 10°, P, R, Fe Ct (3.16m) Be, 10°, P, R, Clay Ct (3.25m) Be, 5°, Un, S, Cn (3.32m) XWS, 0°, 12 mm.t (3.42m) Be, 5°, P, S, Fe Sn (3.44m) Be, 10°, C, S, Clay Ct		Mittagong Formation
	71	4		SANDSTONE: fine to medium grained, orange brown and grey, bedded at 0-15°.	MW	M	0.60		(3.70m) Be, 0°, P, R, Fe Sn (3.74m) Cr, 20°, P, R, Clay Ct		
				SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 10-20°.		H	0.80		(4.03m) Be, 0°, P, R, Fe Sn		
				as above, but bedded at 0-10°.	FR		2.1		(4.37m) Be, 10°, P, R, Cn		
	70	5					1.3				
							1.7				
	69	6					1.2				
							1.6			(6.37m) Be, 5°, P, R, Cn	Mittagong Formation
							1.5				
	68	7		LAMINITE: Sandstone, fine grained, light grey, and Siltstone, dark grey, bedded at 0-10°.			1.7			(6.96m) Be, 0°, P, R, Cn	
							1.8				
	67	8					2.5				
							3.7				

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER: 37263AE ST LEONARDS.GPJ <DrawingFiles> 25/08/2025 12:25 10.01.00.01 D:\gel Lab and In Situ Tool - DGD Lib - JK 9.02.4 2019-05-31 Proj - JK 9.01.0 2019-03-20

CORED BOREHOLE LOG

Client: ST LEONARDS PROPERTY UNIT TRUST
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 46-52 NICHOLSON STREET & 59-67 CHRISTIE STREET, ST LEONARDS, NSW

Job No.: 37263AE **Core Size:** HQ **R.L. Surface:** ~75.4 m
Date: 22/1/25 TO 23/1/25 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK330 **Bearing:** N/A **Logged/Checked By:** S.W./A.J.H.

Water Loss/Level Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_p(50)$	DEFECT DETAILS		Formation	
								SPACING (mm)	DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness		
								Specific	General		
	66			LAMINITE: Sandstone, fine grained, light grey, and Siltstone, dark grey, bedded at 0-10°. (continued)	FR	VH	3.5				Mittagong Formation
	10			SANDSTONE: fine grained, grey, with occasional dark grey laminae, bedded at 0-10°.		H	3.3		(9.89m) Be, 10°, P, R, Cn		
	65						2.3		(10.26m) Be, 0°, P, R, Cn (10.45m) Be, 10°, Un, E, Cn		
	11						1.7		(10.68m) Be, 10°, C, R, Cb Ct (10.84m) Be, 15°, C, R, Cn		
	64			SANDSTONE: fine to coarse grained, light grey and light brown, with occasional grey laminae, bedded at 0-10°.			1.3				Hawkesbury Sandstone
	12						1.0		(11.49m) Be, 0°, P, R, Clay FILLED, 5 mm.t (12.00m) J, 35°, Ir, S, Cn		
	63			SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 0-20°.			1.1		(12.30m) Be, 10°, P, R, Clay Ct		
	13						1.5				
	62						1.4				Hawkesbury Sandstone
	14						1.4				
	61						1.5		(14.16m) Be, 5°, P, R, Clay Ct		
	15						1.2				
	60						1.3				
	0%						1.8				

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 37263AE ST LEONARDS.GPJ <DrawingFile> 25/08/2025 12:25 10.01.00.01 D:\geol Lab and In Situ Tool - DGD [Lib] JK 9.02.4 2019-05-31 P1: JK 9.01.0 2019-03-20

CORED BOREHOLE LOG

Client: ST LEONARDS PROPERTY UNIT TRUST
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 46-52 NICHOLSON STREET & 59-67 CHRISTIE STREET, ST LEONARDS, NSW

Job No.: 37263AE **Core Size:** HQ **R.L. Surface:** ~75.4 m
Date: 22/1/25 TO 23/1/25 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK330 **Bearing:** N/A **Logged/Checked By:** S.W./A.J.H.

Water Loss/Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_p(50)$	DEFECT DETAILS		Formation				
									SPACING (mm)	DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness					
								600	200	60	20	Specific	General		
	0% RETURN	59	17		SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 0-20°. (continued)	FR	H	1.6							Hawkesbury Sandstone
		58	18		as above, but bedded at 0-10°.			1.0 1.7 1.2 1.2					(17.71m) Be, 0°, Un, R, Clay Ct		
		57	19		END OF BOREHOLE AT 18.34 m										
		56	20												
		55	21												
		54	22												
		53													

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER: 37263AE STLEONARDS.GPJ <DrawingFile> 25/08/2025 12:25 10.01.00.01 D:\gel Lab and In Situ Tool - DCD\Lib JK 9.02.4 2019-05-31 Proj JK 9.01.0 2018-03-20

PROJECT: PROPOSED MIXED USE DEVELOPMENT

PROJECT No: 37263AE

DATE: 23/01/25

LOCATION ID: BH1

DEPTH: 2.70m - 9.00m



37263AE BH1 START CORING AT: 2.70m



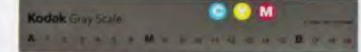
PROJECT: PROPOSED MIXED USE DEVELOPMENT

PROJECT No: 37263AE

DATE: 23/01/25

LOCATION ID: BH1 cont'd

DEPTH: 9.00m - 17.00m



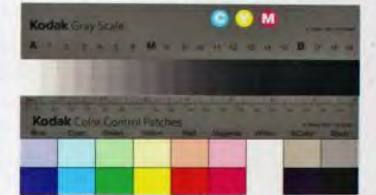
PROJECT: PROPOSED MIXED USE DEVELOPMENT

PROJECT No: 37263AE

DATE: 23/01/25

LOCATION ID: BH1 cont'd

DEPTH: 17.00m - 18.34m



BOREHOLE LOG

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Location: 46-52 NICHOLSON STREET & 59-67 CHRISTIE STREET, ST LEONARDS, NSW

Job No.: 37263AE **Method:** SPIRAL AUGER **R.L. Surface:** ~75.4 m
Date: 9/6/25 TO 11/6/25 **Datum:** AHD
Plant Type: JK330 **Logged/Checked By:** K.R./A.J.H.

Groundwater Record	SAMPLES				Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
DRY ON COMPLETION OF AUGERING <small>JK 9.02.4 LIB.GLB Log_JK AUGERHOLE - MASTER 37263AE STLEONARDS.GPJ <-DrawingFile>> 25/08/2025 12:25 10.01.00.01 Dajjal Lab and In Situ Tool - DCD Lib_JK 9.02.4.2019.05.31 P.jj JK 9.01.0.2019.02.20</small>						75		CH	CONCRETE: 160mm.t Silty CLAY: high plasticity, red brown, trace of fine to medium grained ironstone gravel.	w>PL	VSt		6mm DIA. REINFORCEMENT, 60mm TOP COVER RESIDUAL	
					N = 19 10,9,10		1			as above, but red brown mottled light grey.			270 290 310	HAND AUGER TO 0.5m DEPTH
						74		CL-CI	Silty CLAY: low to medium plasticity, light grey, with ironstone gravel bands.	w-PL	VSt - Hd		380 430 410	BANDS OF LOW 'TC' BIT RESISTANCE
					N = 23 7,12,11		2							
						73		-	SANDSTONE: fine to medium grained, brown and light grey.	MW	L - M			MITTAGONG FORMATION LOW TO MODERATE RESISTANCE
							3			REFER TO CORED BOREHOLE LOG				BOREHOLE BACKFILLED WITH BENTONITE 21.72m TO 33.55m
						4							GROUNDWATER MONITORING WELL INSTALLED TO 21.72m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 15.72m TO 21.72m. CASING 0.04m TO 15.72m. 2mm SAND FILTER PACK 15.00m TO 21.72m. BENTONITE SEAL 12.00m TO 15.00m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETE GATIC COVER.	
						5								
						6								
						69								

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Job No.: 37263AE **Core Size:** HQ **R.L. Surface:** ~75.4 m
Date: 9/6/25 TO 11/6/25 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK330 **Bearing:** N/A **Logged/Checked By:** K.R./A.J.H.

Water Loss/Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_p(50)$	DEFECT DETAILS			Formation
									SPACING (mm)	DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness	Specific	
		73			START CORING AT 2.63m				600			
			3		SANDSTONE: fine to medium grained, orange brown and grey, with dark grey laminae, bedded at 0-15°.	MW	H	2.7 2.1	600		(2.67m) CS, 0°, 5 mm.t (2.80m) Be, 5°, Ir, R, Cn (2.84m) CS, 0°, 45 mm.t	Mitigong Formation
		72					1.8	200		(3.16m) Be, 10°, P, S, Clay Vn (3.23m) XWS, 5°, 20 mm.t (3.35m) XWS, 0°, 25 mm.t		
			4		SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 10-20°.	FR	L - M	0.20 0.50	60		(3.66m) Be, 0°, P, S, Cn (3.81m) Be, 5°, P, S, Cn (3.92m) Be, 0°, P, S, Cn	
		71			as above, but bedded at 0-5°.		H	1.6	60		(4.68m) Be, 10°, Ir, R, Cn	
			5					1.3	20			
		70						1.4				
			6					1.3				
		69						1.6				
			7		LAMINITE: Sandstone, fine grained, grey, and Siltstone, dark grey, bedded at 0-10°.			1.6			(6.87m) Be, 0°, P, S, Cn	
		68						2.1				
			8				VH	3.6				
		67					H	2.2				

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 37263AE STLEONARDS.GPJ <DrawingFile> 25/08/2025 12:25 10.01.00.01 D:\geol\lab\and\in\slu\tool - DGD [Lib: JK 9.02.4 2019-05-31 Proj: JK 9.01.0 2019-03-20]

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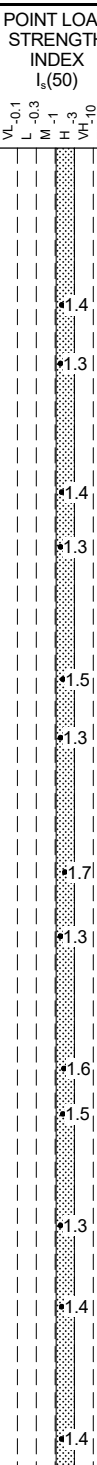
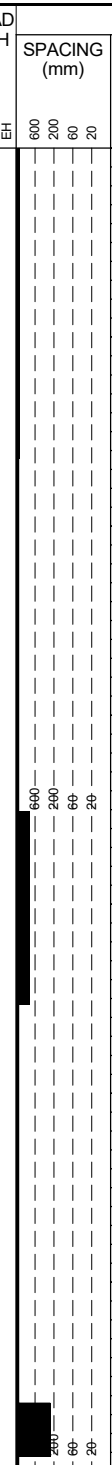
Water Loss/Level Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_p(50)$	SPACING (mm)	DEFECT DETAILS		Formation
									Specific	General	
	66			LAMINITE: as above	FR	H	2.0			(9.32m) Be, 0°, P, S, Cn	Mittagong Formation
	65	10		SANDSTONE: fine grained, grey, with occasional dark grey laminae, bedded at 0-15°.			2.4			(10.06m) Be, 20°, C, R, Cn	
50-75% RETURN	64	11		SANDSTONE: fine to coarse grained, light grey, with dark grey laminae, bedded at 0-10°.		M - H	1.5			(10.34m) Be, 10°, C, R, Cb FILLED (10.36m) Be, 10°, C, R, Cb FILLED (10.56m) Be, 5°, C, R, Cn (10.67m) Be, 10°, C, R, Cn	
	63	12		as above, but fine to medium grained, bedded at 0-20°.			0.90			(11.33m) Be, 0°, P, S, Clay Vn	
0% RETURN	62	13				H	1.4				Hawkesbury Sandstone
	61	14		SANDSTONE: fine to medium grained, light grey, with dark grey laminae and occasional siltstone clasts, bedded at 0-10°.			1.3				
	60	15					1.0				
							1.2				
							1.0				
							1.3				
							1.3				
							1.7			(15.37m) Be, 0°, P, R, Clay Vn	
										(15.87m) Be, 5°, P, R, Cn	

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER: 37263AE ST LEONARDS.GPJ <-DrawingFile> 25/08/2025 12:25 10.01.00.01 D:\gel Lab and In Situ Tool - DGD [Lib: JK 9.02.4 2019-05-31 Proj: JK 9.01.0 2018-03-20]

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Date: 9/6/25 TO 11/6/25 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK330 **Bearing:** N/A **Logged/Checked By:** K.R./A.J.H.

Water Loss/Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_p(50)$	SPACING (mm)	DEFECT DETAILS		Formation					
										Specific	General						
		59			SANDSTONE: fine to medium grained, light grey, with dark grey laminae and occasional siltstone clasts, bedded at 0-10°. (continued)	FR	H					Hawkesbury Sandstone					
		17															
		58															
		18															
		57															
		19															
		56															
		20															
		55															
		21															
		54															
		22															
		53															

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 37263AE STLEONARDS.GPJ <DrawingFile> 25/08/2025 12:25 10.01.00.01 D:\gel Lab and In Situ Tool - DGD [Lib.] JK 9.02.4 2019-05-31 Proj. JK 9.01.0 2019-03-20

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Plant Type: JK330 **Bearing:** N/A **Logged/Checked By:** K.R./A.J.H.

Water Loss Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_p(50)$	DEFECT DETAILS		Formation				
									SPACING (mm)	DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness					
								600	200	60	20	Specific	General		
		52			SANDSTONE: fine to medium grained, light grey, with dark grey laminae and occasional siltstone clasts, bedded at 0-10°. <i>(continued)</i>	FR	H	1.8							
		24						1.8							
		51			as above, but with occasional siltstone clasts.			1.2							
		25						1.5					(24.89m) Be, 0°, P, S, Clay Ct		
		50						1.5					(25.48m) Be, 0°, P, S, Clay Ct		
		26						1.9							
		49						1.6					(26.38m) Be, 0°, P, R, Cb Ct		
		27						1.2					(27.09m) Be, 5°, P, S, Cn		
		48						1.8							
		28						1.7							
		47						1.5							
		29			SANDSTONE: fine to medium grained, light grey, with dark grey laminae, and occasional siltstone clasts, bedded at 10-20°.			1.8					(28.74m) Be, 0°, Ir, R, Cn		
		46						2.3							
								1.5					(29.90m) Be, 10°, P, S, Clay Ct		

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER: 37263AE ST LEONARDS.GPJ <DrawingFile> 25/08/2025 12:25 10.01.00.01 D:\gel Lab and In Situ Tool - DGD [Lib: JK 9.02.4 2019-05-31 Proj: JK 9.01.0 2018-03-20]

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Date: 9/6/25 TO 11/6/25 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK330 **Bearing:** N/A **Logged/Checked By:** K.R./A.J.H.

Water Loss/Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_p(50)$	DEFECT DETAILS		Formation	
									SPACING (mm)	DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness		
								600 200 60 20	Specific	General		
		45			SANDSTONE: fine to medium grained, light grey, with dark grey laminae, and occasional siltstone clasts, bedded at 10-20°. <i>(continued)</i>	FR	H	1.4				Hawkesbury Sandstone
		31						1.8				
		44						1.0				
		32						1.3				
		43						1.7				
		33						1.0				
		42										
					END OF BOREHOLE AT 33.55 m							
		34										
		41										
		35										
		40										
		36										
		39										

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 37263AE STLEONARDS.GPJ <DrawingFile> 25/08/2025 12:25 10.01.00.01 D:\gel Lab and In Situ Tool - DGD [Lib JK 9.02.4 2019-05-31 Proj JK 9.01.0 2019-03-20] 0% RETURN

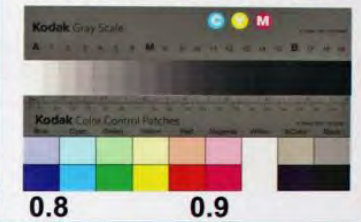
PROJECT: PROPOSED MIXED USE DEVELOPMENT

PROJECT No: 37263AE

DATE: 17/06/25

LOCATION ID: BH1A

DEPTH: 2.63m - 9.00m



37263AE

START CORING AT 2.63m



PROJECT PROPOSED MIXED USE DEVELOPMENT

PROJECT No: 37263AE

DATE: 17/06/25

LOCATION ID: BH1A (cont'd) DEPTH: 9.00m - 17.00m



PROJECT PROPOSED MIXED USE DEVELOPMENT

PROJECT No: 37263AE

DATE: 17/06/25

LOCATION ID: BH1A (cont'd) DEPTH: 17.00m - 25.00m



PROJECT: PROPOSED MIXED USE DEVELOPMENT

PROJECT No: 37263AE

DATE: 17/06/25

LOCATION ID: BH1A (cont'd) DEPTH: 25.00m-33.00m



0.1

0.2

0.3

0.4

0.5

0.6

0.7

0.8

0.9

25

26

27

28

29

30

31

32

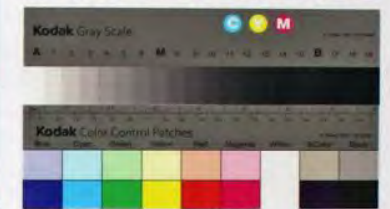


PROJECT: PROPOSED MIXED USE DEVELOPMENT

PROJECT No: 37263AE

DATE: 23/01/25

LOCATION ID: BH1A (cont'd) **DEPTH:** 33.00m - 33.55m



0.1 0.2 0.3 0.4 0.5 0.6 0.7

0.8 0.9

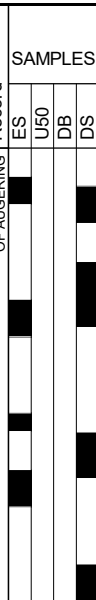
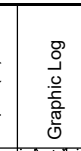
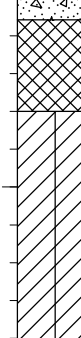
33

END OF BOREHOLE AT 33.55m

BOREHOLE LOG

Client: ST LEONARDS PROPERTY UNIT TRUST
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 46-52 NICHOLSON STREET & 59-67 CHRISTIE STREET, ST LEONARDS, NSW

Job No.: 37263AE **Method:** SPIRAL AUGER **R.L. Surface:** ~74.6 m
Date: 5/5/25 TO 6/5/25 **Datum:** AHD
Plant Type: JK330 **Logged/Checked By:** J.F./A.J.H.

Groundwater Record	SAMPLES				Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
DRY ON COMPLETION OF AUGERING 					N = 12 4,5,7	74		-	CONCRETE: 115mm.t FILL: Silty clay, high plasticity, dark grey, trace of fine to medium grained ironstone gravel.	w>PL			6mm DIA. REINFORCEMENT, 90mm TOP COVER SCREEN: 1.60kg (<10L) 0.15-0.5m, NO FCF	
						73			CH	Silty CLAY: high plasticity, red brown and light grey, trace of fine to medium grained ironstone gravel. as above, but light grey mottled red brown, with extremely weathered seams.	w>PL w<PL	Hd	420 470 510	RESIDUAL
					N > 11 14,11/ 100mm REFUSAL		2		-	SILTSTONE: dark grey and red brown, with extremely weathered seams.	HW	L		MITTAGONG FORMATION VERY LOW 'TC' BIT RESISTANCE
						72			REFER TO CORED BOREHOLE LOG				BOREHOLE BACKFILLED WITH BENTONITE 24.40m TO 27.24m GROUNDWATER MONITORING WELL INSTALLED TO 24.40m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 15.40m TO 24.40m. CASING 0.10m TO 15.4m. 2mm SAND FILTER PACK 14.55m TO 24.40m. BENTONITE SEAL 8.60m TO 14.55m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETE GATIC COVER.	

JK 3.02.4 LIB.GLB Log_JK AUGERHOLE - MASTER 37263AE STLEONARDS.GPJ <-DrawingFile>> 25/08/2025 12:25 10.01.00.01 Dajjal Lab and In Situ Tool - DGD Lib_JK 0.02.4.2019.05.31 Pij JK 0.01.0.2019.02.20

CORED BOREHOLE LOG

Client: ST LEONARDS PROPERTY UNIT TRUST
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 46-52 NICHOLSON STREET & 59-67 CHRISTIE STREET, ST LEONARDS, NSW

Job No.: 37263AE **Core Size:** HQ **R.L. Surface:** ~74.6 m
Date: 5/5/25 TO 6/5/25 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK330 **Bearing:** N/A **Logged/Checked By:** J.F./A.J.H.

Water Loss/Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_p(50)$	SPACING (mm)	DEFECT DETAILS		Formation
										Specific	General	
					START CORING AT 2.50m							
		72			NO CORE 0.30m							
			3		Extremely Weathered sandstone: sandy silty CLAY, low to medium plasticity, light grey, with iron indurated bands.	XW	Hd					
					SANDSTONE: fine to medium grained, light grey and red brown, bedded at 0-10°.	HW	L					
						MW	M - H					
		71						11.0				
			4		SANDSTONE: fine to medium grained, light grey and light brown, with grey and dark grey laminae, bedded at 0-15°.	SW		10.70				
		70										
			5					1.3				
								0.90				
		69						0.80				
			6		as above, but light grey, with siltstone seams.	FR		2.3				
		68						1.0				
			7					0.90				
		67						2.4				
			8		as above, but with siltstone clasts.			3.5				
					LAMINITE: Sandstone, fine grained, light grey, and Siltstone, dark grey, bedded at 0-5°.			1.9				
		66										

JK 9.02.4 LIB.GLB Log_JK_CORED BOREHOLE - MASTER_37263AE ST LEONARDS.GPJ <DrawingFile> 25/08/2025 12:28 10.01.00.01 D:\gel Lab and In Situ Tool - DGD [Lib: JK 9.02.4 2019-05-31 Proj: JK 9.01.0 2019-03-20]

CORED BOREHOLE LOG

Client: ST LEONARDS PROPERTY UNIT TRUST
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 46-52 NICHOLSON STREET & 59-67 CHRISTIE STREET, ST LEONARDS, NSW

Job No.: 37263AE **Core Size:** HQ **R.L. Surface:** ~74.6 m
Date: 5/5/25 TO 6/5/25 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK330 **Bearing:** N/A **Logged/Checked By:** J.F./A.J.H.

Water Loss/Level Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_p(50)$	DEFECT DETAILS		Formation
								SPACING (mm)	DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness	
								Specific	General	
	65	10		LAMINITE: Sandstone, fine grained, light grey, and Siltstone, dark grey, bedded at 0-5°. <i>(continued)</i>	FR	H - VH	+3.0	600	(9.49m) Be, 0°, P, S, Clay FILLED, 3 mm.t	Mittagong Formation
	64	11				M - H	+2.3	200	(9.56m) Be, 0°, P, S, Clay FILLED, 7 mm.t	
	63	12		SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 0-10°.		H	+0.50	60	(9.73m) Be, 5°, P, S, Clay Ct	Hawkesbury Sandstone
	62	13					+1.3	20	(9.98m) Be, 0°, P, S, Clay Ct	
	61	14					+1.1			
	60	15					+1.0		(12.23m) XWS, 0°, 10 mm.t	
	59						+1.0			
							+1.0			
							+1.0			
							+1.0			
							+1.2		(15.00m) Be, 5°, P, S, Clay Ct	
							+1.3			
							+1.3			

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER: 37263AE ST LEONARDS.GPJ <-DrawingFile>> 25/08/2025 12:28 10.01.00.01 D:\gel Lab and In Situ Tool - DGD Lib JK 9.02.4 2019-05-31 Proj JK 9.01.0 2019-03-20

CORED BOREHOLE LOG

Client: ST LEONARDS PROPERTY UNIT TRUST
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 46-52 NICHOLSON STREET & 59-67 CHRISTIE STREET, ST LEONARDS, NSW

Job No.: 37263AE **Core Size:** HQ **R.L. Surface:** ~74.6 m
Date: 5/5/25 TO 6/5/25 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK330 **Bearing:** N/A **Logged/Checked By:** J.F./A.J.H.

Water Loss/Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_p(50)$	SPACING (mm)	DEFECT DETAILS		Formation
										Specific	General	
			58		SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 0-10°. <i>(continued)</i>	FR	H	1.0	600		(16.50m) Be, 0°, P, S, Clay Ct	Hawkesbury Sandstone
			17					1.9	200			
			57		2.0	60						
			18		1.8	60						
			56		1.0	60				(18.67m) Be, 5°, P, S, Clay Ct		
			19		1.6	60						
			55		1.3	60						
			20		1.1	60				(19.88m) Be, 0°, P, S, Clay Ct		
			54		0.90	60				(20.00m) J, 85°, P, R, Cn		
			21		0.80	60				(20.70m) J, 75°, P, R, Cn (20.36-21.17m) Crushed core (drilling induced)		
			53		1.7	60				(21.00m) J, 70°, P, R, Cn		
			22		1.2	60				(21.54m) Be, 0°, P, R, Cb Ct		
			52		1.8	60				(21.81m) Be, 0°, P, S, Clay Ct		
					1.9	60						
					1.8	60				(22.46m) Be, 10°, P, S, Cb Ct (22.48m) XWS, 0°, 10 mm.t		

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER - 37263AE ST LEONARDS.GPJ <DrawingFile> 25/08/2025 12:28 10.01.00.01 D:\gel Lab and In Situ Tool - DGD [Lib JK 9.02.4 2019-05-31 Proj JK 9.01.0 2018-03-20]

CORED BOREHOLE LOG

Client: ST LEONARDS PROPERTY UNIT TRUST
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 46-52 NICHOLSON STREET & 59-67 CHRISTIE STREET, ST LEONARDS, NSW

Job No.: 37263AE **Core Size:** HQ **R.L. Surface:** ~74.6 m
Date: 5/5/25 TO 6/5/25 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK330 **Bearing:** N/A **Logged/Checked By:** J.F./A.J.H.

Water Loss/Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_p(50)$	DEFECT DETAILS		Formation							
									SPACING (mm)	DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness								
									Specific	General								
		51			SANDSTONE: fine to medium grained, light grey, with dark grey laminae bedded at 0-10°.	FR	H		600 200 60 20		Hawkesbury Sandstone							
		24																
		50																
		25											M - H					
		49															(23.84m) XWS, 0°, 10 mm.t	
		26															(25.63m) Be, 0°, P, S, Clay Ct	
		48															(25.83m) J, 35°, P, S, Cn	
		27																
		47										END OF BOREHOLE AT 27.24 m						
		28																
		46																
		29																
		45																

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 37263AE STLEONARDS.GPJ <DrawingFile> 25/08/2025 12:28 10.01.00.01 D:\gel Lab and In Situ Tool - DGD \Lib JK 9.02.4 2019-05-31 Proj JK 9.01.0 2018-03-20

PROJECT: PROPOSED MIXED USE DEVELOPMENT

PROJECT No: 37263AE

DATE: 06/05/25

LOCATION ID: BH2

DEPTH: 2.50m - 9.00M



0.1

0.2

0.3

0.4

0.5

0.6

0.7

0.8

0.9

37263AE BH2 START CORING AT 250 m

2

→ NO CORE: 0.30 m.t

3

4

5

6

7

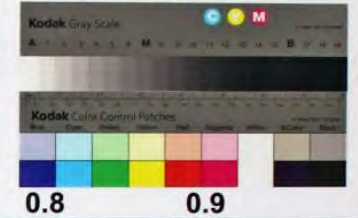
8

PROJECT: PROPOSED MIXED USE DEVELOPMENT

PROJECT No: 37263AE

DATE: 06/05/25

LOCATION ID: BH2 (cont'd) DEPTH: 9.00m - 17.00m

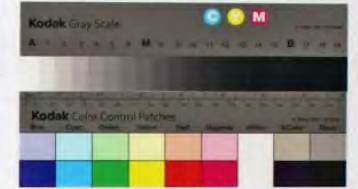


PROJECT: PROPOSED MIXED USE DEVELOPMENT

PROJECT No: 37263AE

DATE: 13/05/25

LOCATION ID: BH2 (cont'd) DEPTH: 17.00m-25.00m



0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9



PROJECT: PROPOSED MIXED USE DEVELOPMENT

PROJECT No: 37263AE

DATE: 13/05/25

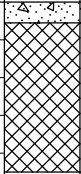
LOCATION ID: BH2 (cont'd) DEPTH: 25.00m-27.24m



BOREHOLE LOG

Client: ST LEONARDS PROPERTY UNIT TRUST
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 46-52 NICHOLSON STREET & 59-67 CHRISTIE STREET, ST LEONARDS, NSW

Job No.: 37263AE **Method:** SPIRAL AUGER **R.L. Surface:** ~72.6 m
Date: 7/5/25 **Datum:** AHD
Plant Type: JK330 **Logged/Checked By:** J.F./A.J.H.

Groundwater Record	SAMPLES				Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	ES	U50	DB	DS										
DRY ON COMPLETION					N > 3 4.2, 1/ 100mm REFUSAL	72			-	CONCRETE: 110mm.t FILL: Silty sand, fine to medium grained, dark grey, with fine to medium grained igneous gravel. FILL: Silty clay, medium to high plasticity, dark brown and dark grey, with concrete and tile fragments.	M w>PL			7mm DIA. REINFORCEMENT, 70mm TOP COVER APPEARS POORLY COMPACTED
							1			-	VOID: >0.5m.t			
						71				END OF BOREHOLE AT 1.40 m				BOREHOLE TERMINATED DUE TO THE PRESENCE OF A STORMWATER PIPE
						2								
						70								
						3								
						69								
						4								
						68								
						67								
						66								

JK 0.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 37263AE STLEONARDS.GPJ <-DrawingFile>> 25/08/2025 12:28 10.01.00.01 Dajjal Lab and In Situ Tool - DCD Lib JK 0.02.4 2019.05.31 P.jj JK 0.01.0 2018.02.20

BOREHOLE LOG

Client: ST LEONARDS PROPERTY UNIT TRUST
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 46-52 NICHOLSON STREET & 59-67 CHRISTIE STREET, ST LEONARDS, NSW

Job No.: 37263AE **Method:** SPIRAL AUGER **R.L. Surface:** ~76.3 m
Date: 30/4/25 TO 2/5/25 **Datum:** AHD
Plant Type: JK330 **Logged/Checked By:** J.F./A.J.H.

Groundwater Record	SAMPLES	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING ON COMPLETION OF CORING				76			FILL: Silty sandy clay, low plasticity, dark grey, fine to medium grained sand, trace of fine to medium grained igneous gravel, and root fibres.	w>PL			GRASS COVER HAND AUGER TO 0.45m DEPTH
			N = 10 3,4,6		1	CH	Silty CLAY: high plasticity, light grey mottled dark grey. as above, but dark grey, trace of ash.	w>PL	VSt - Hd	360 420 440	SCREEN: 11.40kg 0-0.2m, NO FCF RESIDUAL
			N = 8 3,3,5		2	CI-CH	Silty CLAY: medium to high plasticity, brown, trace of fine to medium grained ironstone gravel, and ash.		VSt	370 350 410	
			N > 8 11,8/ 50mm REFUSAL		3		Silty CLAY: medium to high plasticity, red brown, trace of fine to medium grained ironstone gravel. as above, but with extremely weathered sandstone seams.	w<PL	Hd	210 220 230 300 310 330 410 600 >600	HP TESTING ON REMOULDED SAMPLE TOO FRIABLE FOR HP TESTING
					4	-	Extremely Weathered siltstone: silty CLAY, low plasticity, light grey and dark grey, with medium to high strength iron indurated bands.	XW	Hd		MITTAGONG FORMATION VERY LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
				5		SANDSTONE: fine to medium grained, grey. REFER TO CORED BOREHOLE LOG	MW	L - M		LOW TO MODERATE RESISTANCE 'TC' BIT REFUSAL BOREHOLE BACKFILLED WITH BENTONITE 25.22m TO 28.63m	
				6							GROUNDWATER MONITORING WELL INSTALLED TO 25.22m. CLASS 18 MACHINE SLOTTED 50mm DIA, PVC STANDPIPE 17.22m TO 25.22m. CASING 0.11m TO 17.22m. 2mm SAND FILTER PACK 16.40m TO 25.22m. BENTONITE SEAL 3.40m TO 16.40m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.

JK 3.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 37263AE STLEONARDS.GPJ <-DrawingFile>> 25/08/2025 12:28 10.01.00.01 D:\glb\Lab and In Situ Tool - DCD Lib JK 3.02.4 2019-05-31 P1j JK 3.01.0 2018-02-20

CORED BOREHOLE LOG

Client: ST LEONARDS PROPERTY UNIT TRUST
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 46-52 NICHOLSON STREET & 59-67 CHRISTIE STREET, ST LEONARDS, NSW

Job No.: 37263AE **Core Size:** HQ **R.L. Surface:** ~76.3 m
Date: 30/4/25 TO 2/5/25 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK330 **Bearing:** N/A **Logged/Checked By:** J.F./A.J.H.

Water Loss/Level Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX I _s (50)	SPACING (mm)	DEFECT DETAILS		Formation
									Specific	General	
	72			START CORING AT 4.61m							
				NO CORE 0.29m							
	71	5		SANDSTONE: fine to medium grained, light brown and light grey, bedded at 0-5°.	MW	H				(4.93m) XWS, 0°, 30 mm.t (4.95m) Cr, 0°, 25 mm.t (5.06m) XWS, 0°, 20 mm.t (5.11m) XWS, 0°, 10 mm.t	Mittagong Formation
				LAMINITE: Sandstone, fine grained, light grey, and Siltstone, dark grey, bedded at 0-5°.	FR		1.3		(5.28m) XWS, 0°, 10 mm.t (5.40m) CS, 0°, 10 mm.t (5.56m) CS, 5°, 15 mm.t		
		6		SANDSTONE: fine grained, light grey, with dark grey laminae, bedded at 0-10°.			1.8				
							1.5				
		70					3.0				
				LAMINITE: Sandstone, fine grained, light grey, and Siltstone, dark grey, bedded at 0-10°.			2.4		(7.39m) Be, 10°, P, S, Clay FILLED, 5 mm.t		
		69					1.2		(7.73m) Be, 5°, P, S, Fe Ct (7.84m) Be, 0°, P, S, Clay Vn		
							2.9		(8.28m) Be, 5°, P, S, Clay Vn		
		68					3.0				
							2.0				
		67				H - VH	2.1		(9.51m) CS, 0°, 8 mm.t		
							3.7				
		66				H	1.2				

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER - 37263AE ST LEONARDS.GPJ <<DrawingFile>> 25/08/2025 12:28 10.01.00.01 D:\gel Lab and In Situ Tool - DGD [Lib: JK 9.02.4 2019-05-31 Proj: JK 9.01.0 2018-03-20] 05% RETURN

CORED BOREHOLE LOG

Client: ST LEONARDS PROPERTY UNIT TRUST
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 46-52 NICHOLSON STREET & 59-67 CHRISTIE STREET, ST LEONARDS, NSW

Job No.: 37263AE **Core Size:** HQ **R.L. Surface:** ~76.3 m
Date: 30/4/25 TO 2/5/25 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK330 **Bearing:** N/A **Logged/Checked By:** J.F./A.J.H.

Water Loss Level Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_p(50)$	SPACING (mm)	DEFECT DETAILS		Formation
									Specific	General	
	65			LAMINITE: as above	FR	H	2.3		(11.08m) Be, 0°, P, S, Clay FILLED, 5 mm.t		Mittagong Formation
	12			SANDSTONE: fine grained, light grey, with dark grey laminae and siltstone bands, bedded at 0-5°.		M - H	1.4		(11.84m) XWS, 0°, 15 mm.t		
	64			SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 0-10°.		H	0.80		(12.38m) Be, 0°, P, S, Clay Vn		Hawkesbury Sandstone
	13						0.80				
	63						1.1		(13.35m) XWS, 0°, 15 mm.t		
	14						1.1				
	62						1.1				
	15						0.90				
	61						1.1		(15.36m) Be, 5°, P, S, Clay Ct		
	16						1.0		(16.19m) Be, 0°, P, S, Clay Vn		
	60						1.7				
	17						1.1				
	59						1.2		(17.30m) Be, 0°, P, S, Clay FILLED, 3 mm.t		
							1.8				

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 37263AE STLEONARDS.GPJ <DrawingFile>> 25/08/2025 12:28 10.01.00.01 D:\gel Lab and In Situ Tool - DGD [Lib] JK 9.02.4 2019-05-31 Proj JK 9.01.0 2019-03-20

CORED BOREHOLE LOG

Client: ST LEONARDS PROPERTY UNIT TRUST
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 46-52 NICHOLSON STREET & 59-67 CHRISTIE STREET, ST LEONARDS, NSW

Job No.: 37263AE **Core Size:** HQ **R.L. Surface:** ~76.3 m
Date: 30/4/25 TO 2/5/25 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK330 **Bearing:** N/A **Logged/Checked By:** J.F./A.J.H.

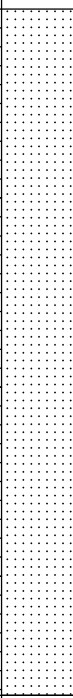
Water Loss Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_p(50)$	SPACING (mm)	DEFECT DETAILS		Formation
										Specific	General	
		58			SANDSTONE: fine to medium grained, light grey, with dark grey laminae, bedded at 0-10°. <i>(continued)</i>	FR	H	1.8	600		(18.34m) Be, 0°, P, S, Clay Ct	Hawkesbury Sandstone
		19						1.7	200			
		57						1.6	60			
		20						1.1	20			
		56						1.3	60			
		21						1.3	20		(21.11m) Be, 5°, P, R, Cb Ct	
		55						1.3	60			
		22						1.2	20		(21.73m) Be, 5°, P, R, Cb Ct	
		54						1.2	60			
		23						1.1	20		(22.23m) Be, 0°, P, R, Clay Ct	
		53			1.5	600						
		24			2.0	200		(23.81m) Be, 0°, P, R, Cb Ct				
		52			SANDSTONE: fine to medium grained, light grey, with grey laminae and occasional siltstone bands, bedded at 0-15°.			2.0	600			
								1.8	200		(24.28m) Be, 0°, P, S, Clay FILLED, 5 mm.t	
									600		(24.95m) Be, 0°, P, S, Clay Ct	

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER - 37263AE ST LEONARDS.GPJ <DrawingFile> 25/08/2025 12:28 10.01.00.01 D:\gel Lab and In Situ Tool - DGD [Lib: JK 9.02.4 2019-05-31 Proj: JK 9.01.0 2019-03-20]

CORED BOREHOLE LOG

Client: ST LEONARDS PROPERTY UNIT TRUST
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 46-52 NICHOLSON STREET & 59-67 CHRISTIE STREET, ST LEONARDS, NSW

Job No.: 37263AE **Core Size:** HQ **R.L. Surface:** ~76.3 m
Date: 30/4/25 TO 2/5/25 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK330 **Bearing:** N/A **Logged/Checked By:** J.F./A.J.H.

Water Loss Level Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_p(50)$	DEFECT DETAILS		Formation
								SPACING (mm)	DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness	
							600 200 60 20	Specific	General	
95% RETURN	51			SANDSTONE: fine to medium grained, light grey, with grey laminae and occasional siltstone bands, bedded at 0-15°. <i>(continued)</i>	FR	H	1.3			Hawkesbury Sandstone
	26	50		as above, but without siltstone bands.			1.3			
	27	49					1.9			
	28	48					1.8			
	29	47		END OF BOREHOLE AT 28.63 m			1.6			
	30	46					1.9			
	31	45					1.8			

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 37263AE STLEONARDS.GPJ <DrawingFile> 25/08/2025 12:28 10.01.00.01 D:\gel Lab and In Situ Tool - DGD \Lib JK 9.02.4 2019-05-31 Proj JK 9.01.0 2018-03-20

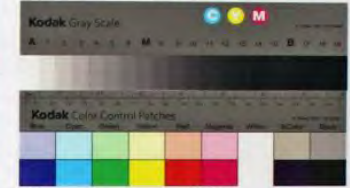
PROJECT: PROPOSED MIXED USE DEVELOPMENT

PROJECT No: 37263AE

DATE: 05/05/25

LOCATION ID: BH4

DEPTH: 4.61m - 11.00m



0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9



37263AE BH4 START CORING AT 4.61 m

4

→ NO CORE: 0.29m.t

5

6

7

8

9

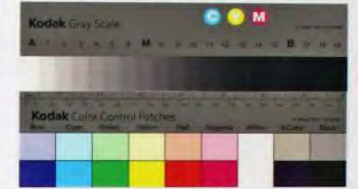
10

PROJECT: PROPOSED MIXED USE DEVELOPMENT

PROJECT No: 37263AE

DATE: 05/05/25

LOCATION ID: BH4 (cont'd) DEPTH: 11.00m-19.00m



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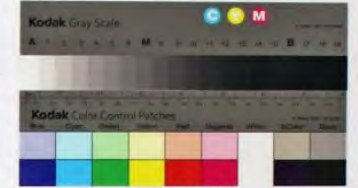


PROJECT: PROPOSED MIXED USE DEVELOPMENT

PROJECT No: 37263AE

DATE: 05/05/25

LOCATION ID: BH4 (cont'd) DEPTH: 19.00m-27.00m



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PROJECT: PROPOSED MIXED USE DEVELOPMENT

PROJECT No: 37263AE

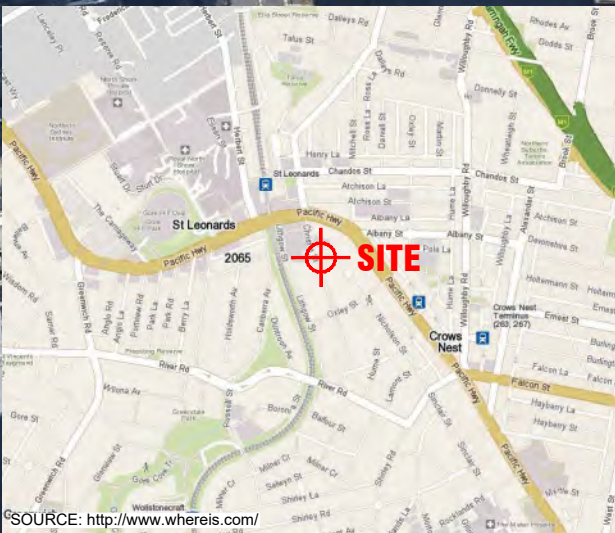
DATE: 06/05/25

LOCATION ID: BH4 (cont'd) DEPTH: 27.00m-28.63m



0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9





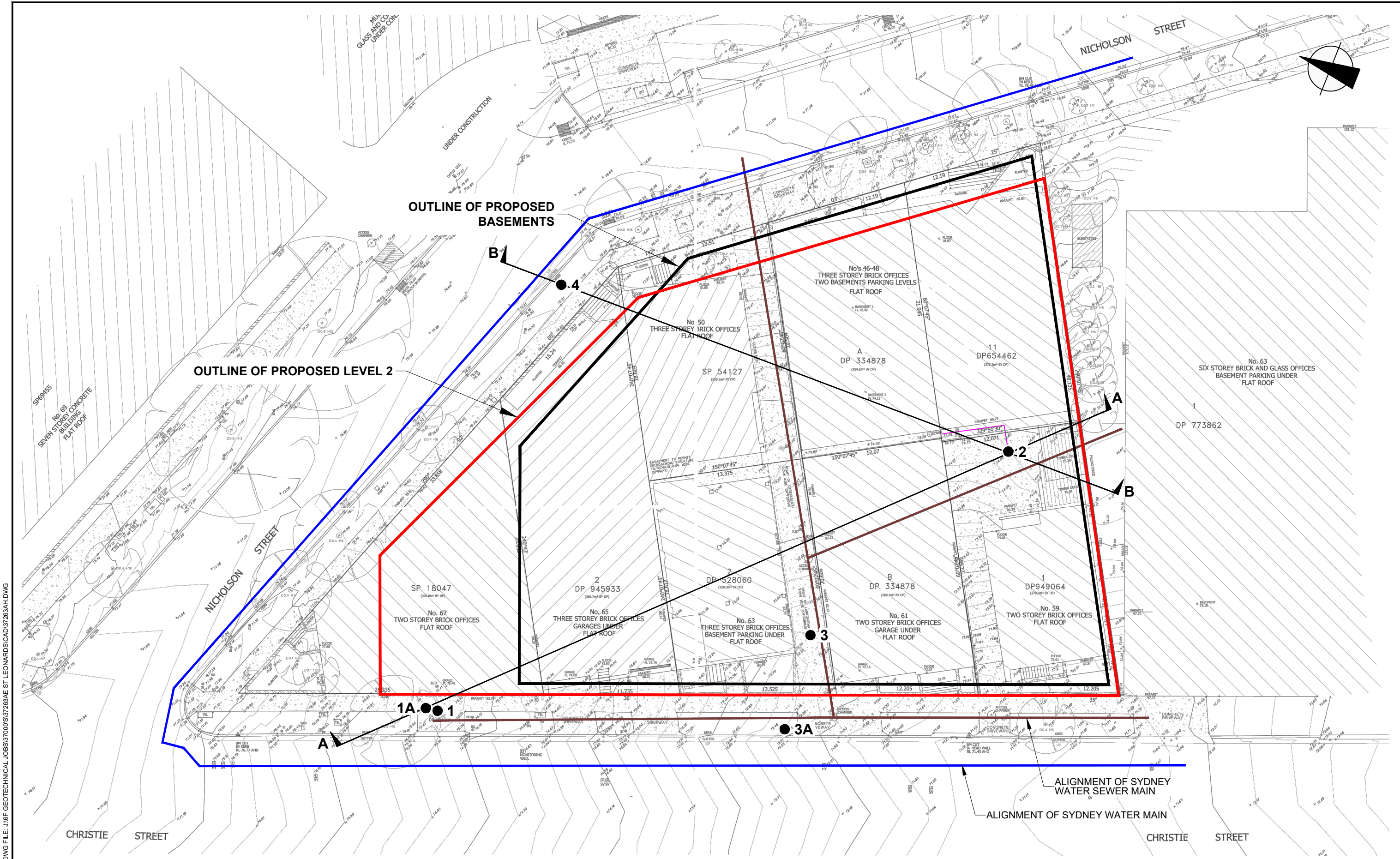
AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

Title:		SITE LOCATION PLAN
Location:		46-50 NICHOLSON STREET AND 59-67 CHRISTIE STREET, ST LEONARDS, NSW
Report No:	37263AH	Figure No: 1

This plan should be read in conjunction with the JK Geotechnics report.

JKGeotechnics



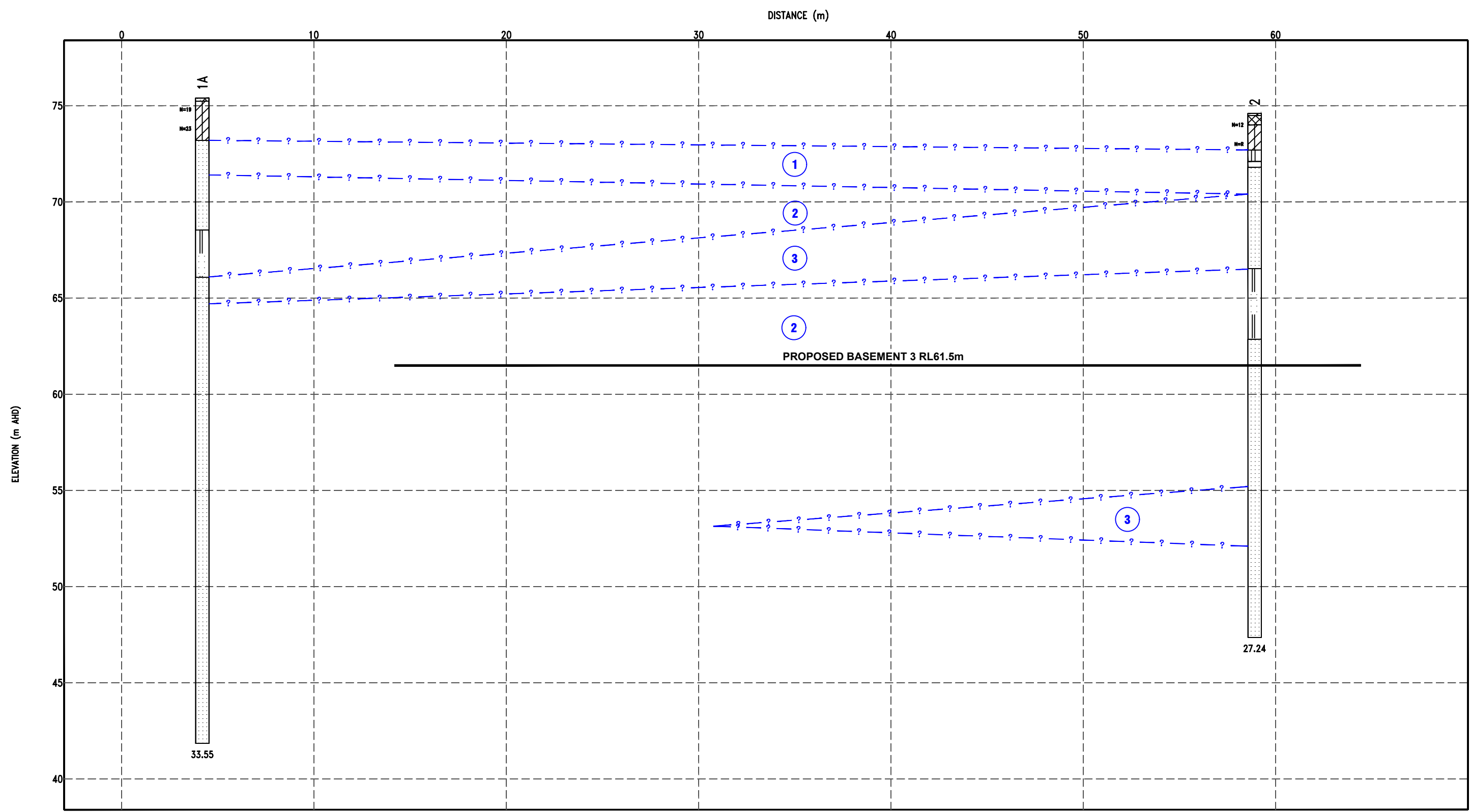


NOTES:
 1. REFER TO FIGURE 3 FOR SECTION A-A GRAPHICAL BOREHOLE SUMMARY.
 2. REFER TO FIGURE 4 FOR SECTION B-B GRAPHICAL BOREHOLE SUMMARY.

0 3 6 9 12 15 SCALE 1:300 @A3 METRES		Title: BOREHOLE LOCATION PLAN	
This plan should be read in conjunction with the JK Geotechnics report.		Location: 46-50 NICHOLSON STREET AND 59-67 CHRISTIE STREET, ST LEONARDS, NSW	
Report No: 37263AH		Figure No: 2	
JKGeotechnics			

PLOT DATE: 4/09/2025 8:18:16 AM DWG FILE: JKGF GEOTECHNICAL JOBS\37000\SI\37263AE ST LEONARDS\CAD\37263AH.DWG

PLOT DATE: 4/09/2025 8:20:04 AM DWG FILE: J:\GF GEOTECHNICAL\JOBS\37000\SI\37263AE ST LEONARDS\CAD\37263AH.DWG



MATERIAL GRAPHIC

- | | | | | | |
|--|-------------------------|--|---------------------------------|--|-----------|
| | NO CORE | | FILL | | SILTSTONE |
| | SILTY CLAY (CL, CI, CH) | | LAMINITE (SILTSTONE, SANDSTONE) | | |
| | CONCRETE | | SANDSTONE | | |

LEGEND

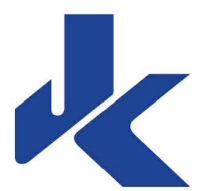
- ① CLASS V/IV SANDSTONE & SHALE
- ② CLASS III/I SANDSTONE & SHALE
- ③ CLASS III SANDSTONE

0 2 4 6 8 10

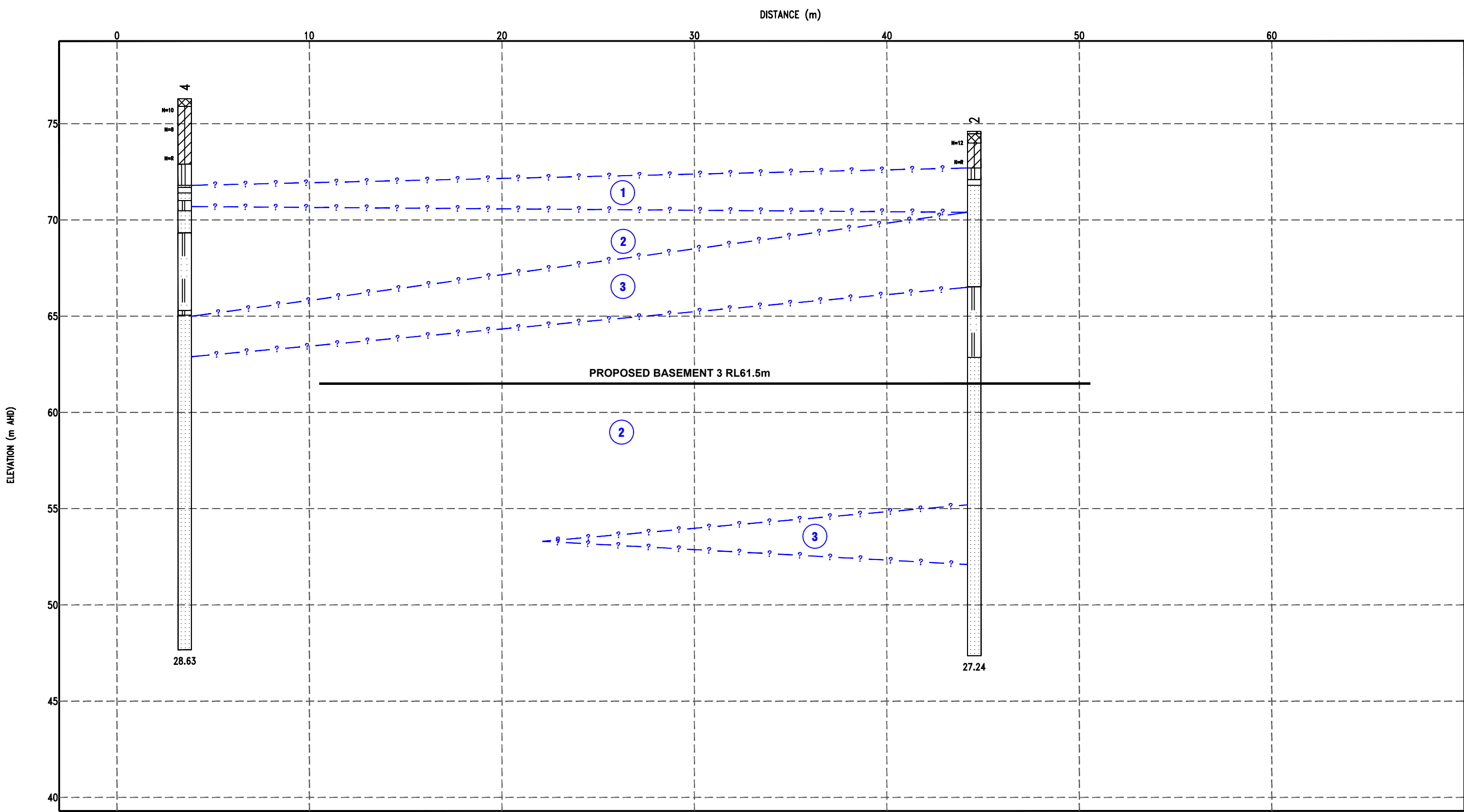
 SCALE 1:200 @A3 METRES

This plan should be read in conjunction with the JK Geotechnics report.

Title: SECTION A-A	
GRAPHICAL BOREHOLE SUMMARY	
Location: 46-50 NICHOLSON STREET AND 59-67 CHRISTIE STREET, ST LEONARDS, NSW	
Report No: 37263AH	Figure No: 3
JKGeotechnics	



PLOT DATE: 4/09/2025 8:20:15 AM DWG FILE: J:\GF GEOTECHNICAL\JOBS\37000\37263AE ST LEONARDS\CAD\37263AH.DWG

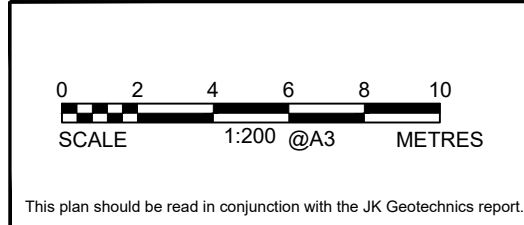


MATERIAL GRAPHIC

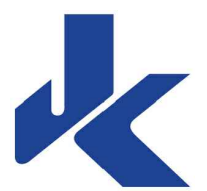
- | | | |
|-------------------------|---------------------------------|-----------|
| NO CORE | FILL | SILTSTONE |
| SILTY CLAY (CL, CI, CH) | LAMINITE (SILTSTONE, SANDSTONE) | |
| CONCRETE | SANDSTONE | |

LEGEND

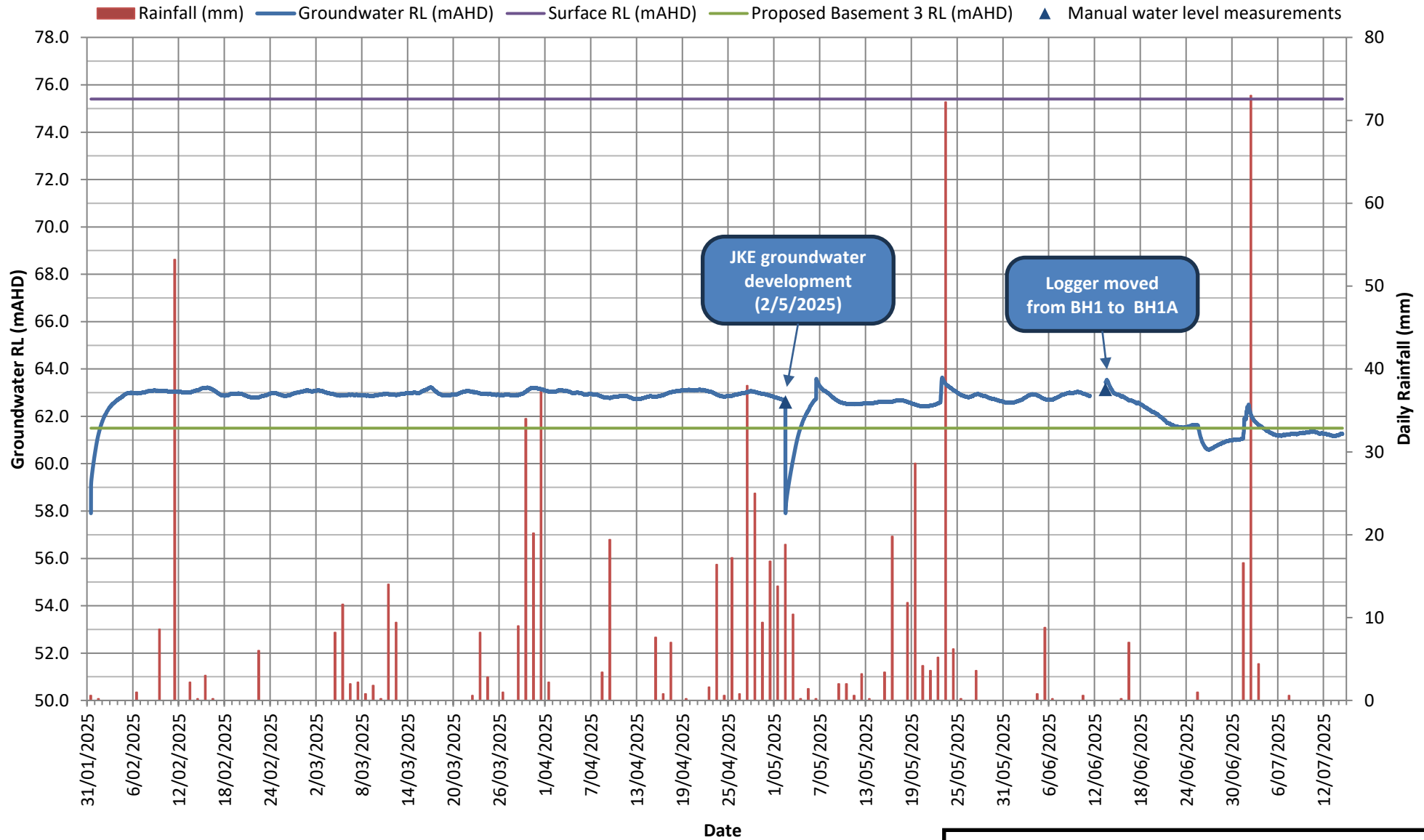
- ① CLASS V/IV SANDSTONE & SHALE
- ② CLASS III/I SANDSTONE & SHALE
- ③ CLASS III SANDSTONE



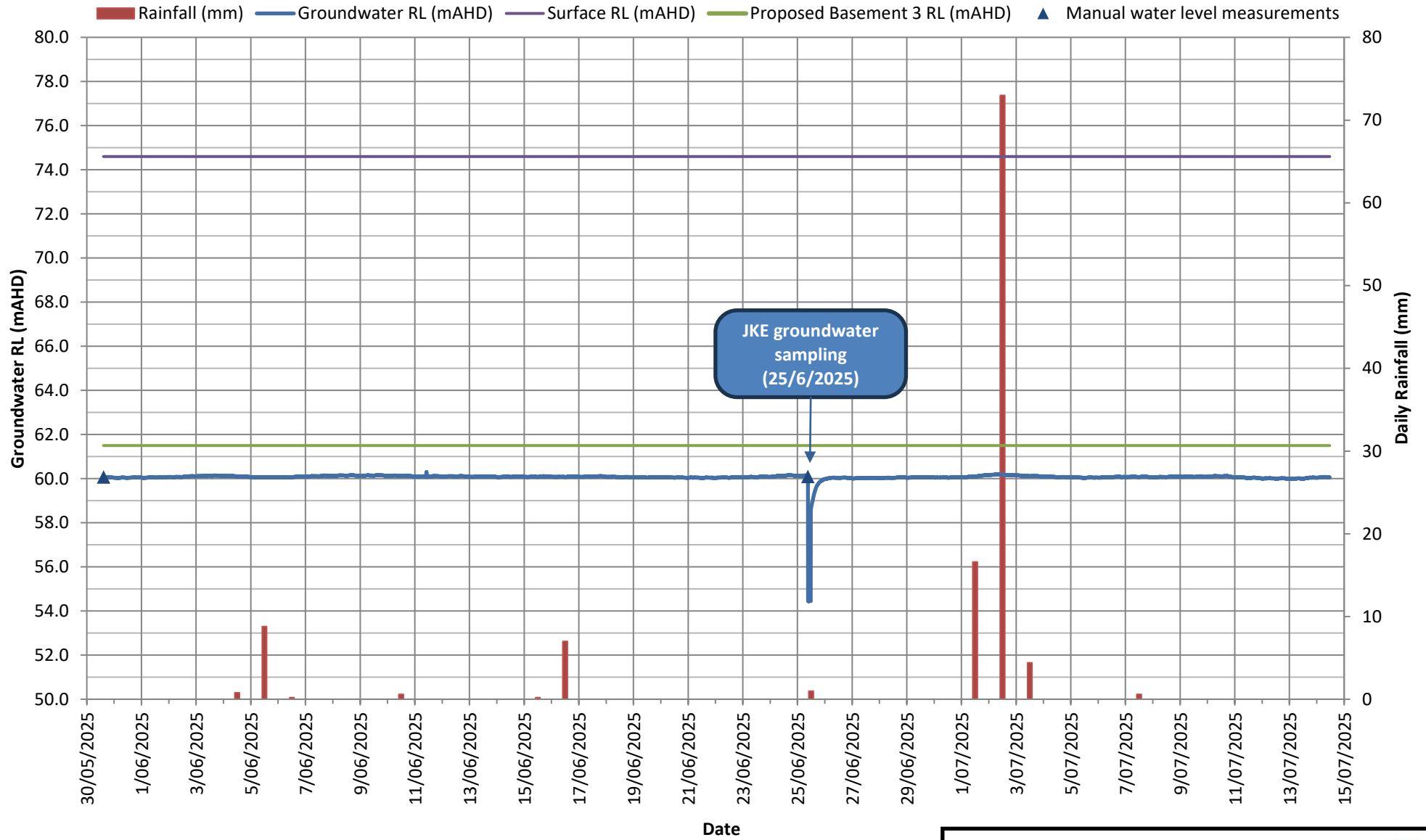
Title: SECTION B-B	
GRAPHICAL BOREHOLE SUMMARY	
Location: 46-50 NICHOLSON STREET AND 59-67 CHRISTIE STREET, ST LEONARDS, NSW	
Report No: 37263AH	Figure No: 4
JKGeotechnics	



Groundwater Level and Daily Rainfall -v- Time Plot BH1/1A

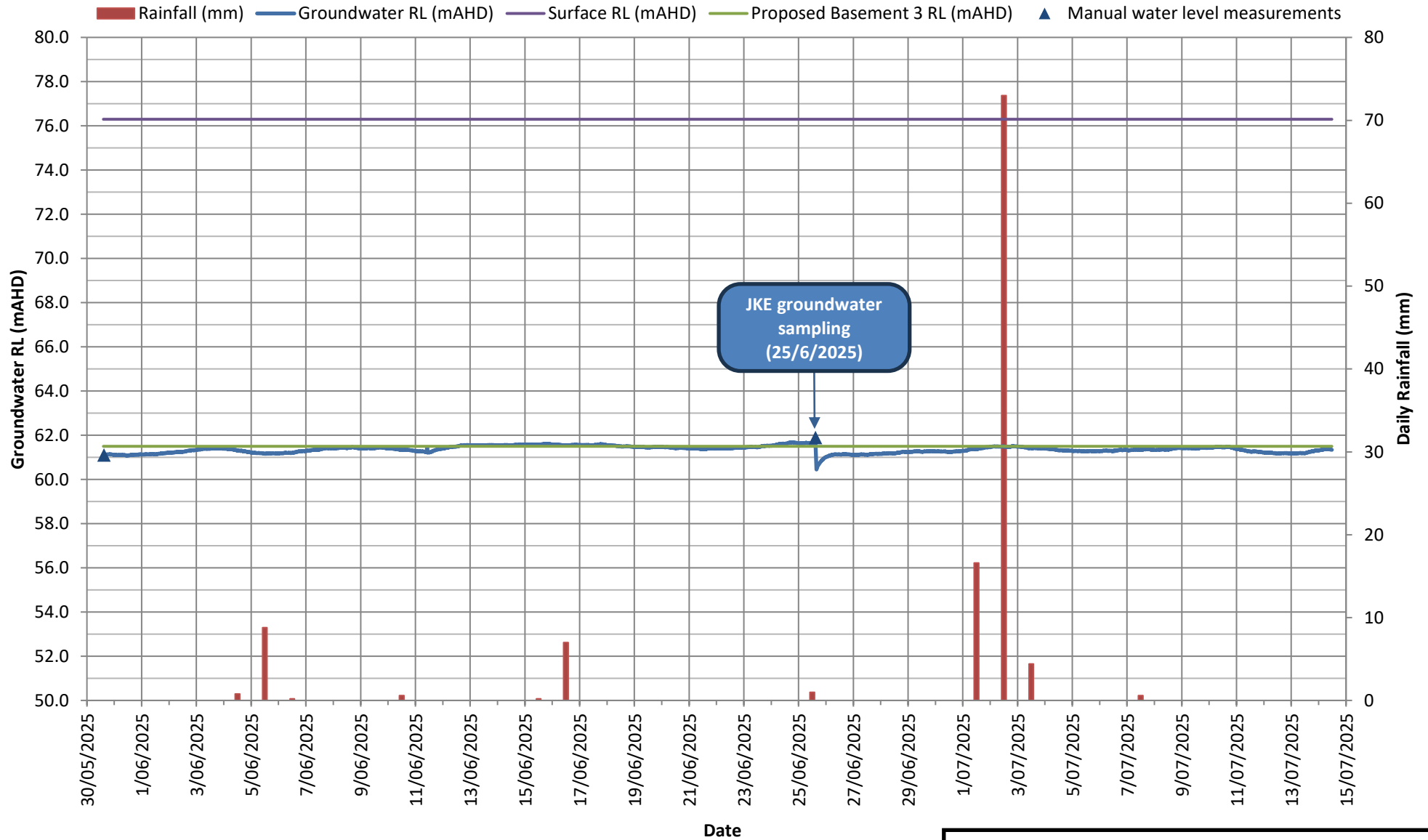


Groundwater Level and Daily Rainfall -v- Time Plot BH2



Rainfall data from: Sydney (Observatory Hill), Station No. 66214

Groundwater Level and Daily Rainfall -v- Time Plot BH4



Date

Rainfall data from: Sydney (Observatory Hill), Station No. 66214

JKGeotechnics
Report No. 37263AE Figure No. 7





VIBRATION EMISSION DESIGN GOALS

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 recognised to be conservative.

The DIN 4150 values (maximum levels measured in any direction at the foundation, OR, maximum levels measured in (x) or (y) horizontal directions, in the plane of the uppermost floor), are summarised in Table 1 below.

It should be noted that peak vibration velocities higher than the minimum figures in Table 1 for low frequencies may be quite ‘safe’, depending on the frequency content of the vibration and the actual condition of the structure.

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 effects such as superficial 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 1: DIN 4150 – Structural Damage – Safe Limits for Building Vibration

Group	Type of Structure	Peak Vibration Velocity in mm/s			
		At Foundation Level at a Frequency of:			Plane of Floor of Uppermost Storey
		Less than 10Hz	10Hz to 50Hz	50Hz to 100Hz	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 (eg. buildings that are under a preservation order).	3	3 to 8	8 to 10	8

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

REPORT EXPLANATION NOTES

INTRODUCTION

These notes have been provided to amplify the geotechnical report in regard to classification methods, field procedures and certain matters relating to the Comments and Recommendations section. Not all notes are necessarily relevant to all reports.

The ground is a product of continuing natural and man-made processes and therefore exhibits a variety of characteristics and properties which vary from place to place and can change with time. Geotechnical engineering involves gathering and assimilating limited facts about these characteristics and properties in order to understand or predict the behaviour of the ground on a particular site under certain conditions. This report may contain such facts obtained by inspection, excavation, probing, sampling, testing or other means of investigation. If so, they are directly relevant only to the ground at the place where and time when the investigation was carried out.

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726:2017 'Geotechnical Site Investigations'. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the predominating particle size and behaviour as set out in the attached soil classification table qualified by the grading of other particles present (eg. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	< 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2.36mm
Gravel	2.36 to 63mm
Cobbles	63 to 200mm
Boulders	> 200mm

Non-cohesive soils are classified on the basis of relative density, generally from the results of Standard Penetration Test (SPT) as below:

Relative Density	SPT 'N' Value (blows/300mm)
Very loose (VL)	< 4
Loose (L)	4 to 10
Medium dense (MD)	10 to 30
Dense (D)	30 to 50
Very Dense (VD)	> 50

Cohesive soils are classified on the basis of strength (consistency) either by use of a hand penetrometer, vane shear, laboratory testing and/or tactile engineering examination. The strength terms are defined as follows.

Classification	Unconfined Compressive Strength (kPa)	Indicative Undrained Shear Strength (kPa)
Very Soft (VS)	≤ 25	≤ 12
Soft (S)	> 25 and ≤ 50	> 12 and ≤ 25
Firm (F)	> 50 and ≤ 100	> 25 and ≤ 50
Stiff (St)	> 100 and ≤ 200	> 50 and ≤ 100
Very Stiff (VSt)	> 200 and ≤ 400	> 100 and ≤ 200
Hard (Hd)	> 400	> 200
Friable (Fr)	Strength not attainable – soil crumbles	

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'shale' is used to describe fissile mudstone, with a weakness parallel to bedding. Rocks with alternating inter-laminations of different grain size (eg. siltstone/claystone and siltstone/fine grained sandstone) is referred to as 'laminite'.

SAMPLING

Sampling is carried out during drilling or from other excavations to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure. Bulk samples are similar but of greater volume required for some test procedures.

Undisturbed samples are taken by pushing a thin-walled sample tube, usually 50mm diameter (known as a U50), into the soil and withdrawing it with a sample of the soil contained in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shrink-swell behaviour, strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling used are given on the attached logs.

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by the Company and some comments on their use and application. All methods except test pits, hand auger drilling and portable Dynamic Cone Penetrometers require the use of a mechanical rig which is commonly mounted on a truck chassis or track base.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils and 'weaker' bedrock if it is safe to descend into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for a large excavator. Limitations of test pits are the problems associated with disturbance and difficulty of reinstatement and the consequent effects on close-by structures. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction or to design and construct the structure so as not to be adversely affected by poorly compacted backfill at the test pit location.

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Refusal of the hand auger can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Continuous Spiral Flight Augers: The borehole is advanced using 75mm to 115mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling and insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface by the flights or may be collected after withdrawal of the auger flights, but they can be very disturbed and layers may become mixed. Information from the auger sampling (as distinct from specific sampling by SPTs or undisturbed samples) is of limited reliability due to mixing or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table is of even lesser reliability than augering above the water table.

Rock Augering: Use can be made of a Tungsten Carbide (TC) bit for auger drilling into rock to indicate rock quality and continuity by variation in drilling resistance and from examination of recovered rock cuttings. This method of investigation is quick and relatively inexpensive but provides only an indication of the likely rock strength and predicted values may be in error by a strength order. Where rock strengths may have a significant impact on construction feasibility or costs, then further investigation by means of cored boreholes may be warranted.

Wash Boring: The borehole is usually advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be assessed from the cuttings, together with some information from "feel" and rate of penetration.

Mud Stabilised Drilling: Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilise the borehole. The term 'mud' encompasses a range of products ranging from bentonite to polymers. The mud tends to mask the cuttings and reliable identification is only possible from intermittent intact sampling (eg. from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, NMLC or HQ triple tube core barrels, which give a core of about 50mm and 61mm diameter, respectively, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as NO CORE. The location of NO CORE recovery is determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the bottom of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils, as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289.6.3.1–2004 (R2016) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – Standard Penetration Test (SPT)'*.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63.5kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

- In the case where full penetration is obtained with successive blow counts for each 150mm of, say, 4, 6 and 7 blows, as

N = 13
4, 6, 7

- In a case where the test is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm, as

N > 30
15, 30/40mm

The results of the test can be related empirically to the engineering properties of the soil.

A modification to the SPT is where the same driving system is used with a solid 60° tipped steel cone of the same diameter as the SPT hollow sampler. The solid cone can be continuously driven for some distance in soft clays or loose sands, or may be used where damage would otherwise occur to the SPT. The results of this Solid Cone Penetration Test (SCPT) are shown as 'N_c' on the borehole logs, together with the number of blows per 150mm penetration.

Cone Penetrometer Testing (CPT) and Interpretation:

The cone penetrometer is sometimes referred to as a Dutch Cone. The test is described in Australian Standard 1289.6.5.1–1999 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Static Cone Penetration Resistance of a Soil – Field Test using a Mechanical and Electrical Cone or Friction-Cone Penetrometer'*.

In the tests, a 35mm or 44mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck. The CPT does not provide soil sample recovery.

As penetration occurs (at a rate of approximately 20mm per second), the information is output as incremental digital records every 10mm. The results given in this report have been plotted from the digital data.

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa. There are two scales presented for the cone resistance. The lower scale has a range of 0 to 5MPa and the main scale has a range of 0 to 50MPa. For cone resistance values less than 5MPa, the plot will appear on both scales.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed as a percentage.

The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and occasionally very soft clays, rising to 4% to 10% in stiff clays and peats. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

Correlations between CPT and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of CPT values can be made to empirically derive modulus or compressibility values to allow calculation of foundation settlements.

Stratification can be inferred from the cone and friction traces and from experience and information from nearby boreholes etc. Where shown, this information is presented for general guidance, but must be regarded as interpretive. The test method provides a continuous profile of engineering properties but, where precise information on soil classification is required, direct drilling and sampling may be preferable.

There are limitations when using the CPT in that it may not penetrate obstructions within any fill, thick layers of hard clay and very dense sand, gravel and weathered bedrock. Normally a 'dummy' cone is pushed through fill to protect the equipment. No information is recorded by the 'dummy' probe.

Flat Dilatometer Test: The flat dilatometer (DMT), also known as the Marchetti Dilometer comprises a stainless steel blade having a flat, circular steel membrane mounted flush on one side.

The blade is connected to a control unit at ground surface by a pneumatic-electrical tube running through the insertion rods. A gas tank, connected to the control unit by a pneumatic cable, supplies the gas pressure required to expand the membrane. The control unit is equipped with a pressure regulator, pressure gauges, an audio-visual signal and vent valves.

The blade is advanced into the ground using our CPT rig or one of our drilling rigs, and can be driven into the ground using an SPT hammer. As soon as the blade is in place, the membrane is inflated, and the pressure required to lift the membrane (approximately 0.1mm) is recorded. The pressure then required to lift the centre of the membrane by an additional 1mm is recorded. The membrane is then deflated before pushing to the next depth increment, usually 200mm down. The pressure readings are corrected for membrane stiffness.

The DMT is used to measure material index (I_b), horizontal stress index (K_0), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_0), over-consolidation ratio (OCR), undrained shear strength (C_u), friction angle (ϕ), coefficient of consolidation (C_h), coefficient of permeability (K_h), unit weight (γ), and vertical drained constrained modulus (M).

The seismic dilatometer (SDMT) is the combination of the DMT with an add-on seismic module for the measurement of shear wave velocity (V_s). Using established correlations, the SDMT results can also be used to assess the small strain modulus (G_0).

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a 16mm diameter rod with a 20mm diameter cone end with a 9kg hammer dropping 510mm. The test is described in Australian Standard 1289.6.3.2–1997 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – 9kg Dynamic Cone Penetrometer Test'*.

The results are used to assess the relative compaction of fill, the relative density of granular soils, and the strength of cohesive soils. Using established correlations, the DCP test results can also be used to assess California Bearing Ratio (CBR).

Refusal of the DCP can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Vane Shear Test: The vane shear test is used to measure the undrained shear strength (C_u) of typically very soft to firm fine grained cohesive soils. The vane shear is normally performed in the bottom of a borehole, but can be completed from surface level, the bottom and sides of test pits, and on recovered undisturbed tube samples (when using a hand vane).

The vane comprises four rectangular blades arranged in the form of a cross on the end of a thin rod, which is coupled to the bottom of a drill rod string when used in a borehole. The size of the vane is dependent on the strength of the fine grained cohesive soils; that is, larger vanes are normally used for very low strength soils. For borehole testing, the size of the vane can be limited by the size of the casing that is used.

For testing inside a borehole, a device is used at the top of the casing, which suspends the vane and rods so that they do not sink under self-weight into the 'soft' soils beyond the depth at which the test is to be carried out. A calibrated torque head is used to rotate the rods and vane and to measure the resistance of the vane to rotation.

With the vane in position, torque is applied to cause rotation of the vane at a constant rate. A rate of 6° per minute is the common rotation rate. Rotation is continued until the soil is sheared and the maximum torque has been recorded. This value is then used to calculate the undrained shear strength. The vane is then rotated rapidly a number of times and the operation repeated until a constant torque reading is obtained. This torque value is used to calculate the remoulded shear strength. Where appropriate, friction on the vane rods is measured and taken into account in the shear strength calculation.

LOGS

The borehole or test pit logs presented herein are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on the frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or core drilling will enable the most reliable assessment, but is not always practicable or possible to justify on economic grounds. In any case, the boreholes or test pits represent only a very small sample of the total subsurface conditions.

The terms and symbols used in preparation of the logs are defined in the following pages.

Interpretation of the information shown on the logs, and its application to design and construction, should therefore take into account the spacing of boreholes or test pits, the method of drilling or excavation, the frequency of sampling and testing and the possibility of other than 'straight line' variations between the boreholes or test pits. Subsurface conditions between boreholes or test pits may vary significantly from conditions encountered at the borehole or test pit locations.

GROUNDWATER

Where groundwater levels are measured in boreholes, there are several potential problems:

- Although groundwater may be present, in low permeability soils it may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes and may not be the same at the time of construction.
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must be washed out of the hole or 'reverted' chemically if reliable water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after the groundwater level has stabilised at intervals ranging from several days to perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from perched water tables or surface water.

FILL

The presence of fill materials can often be determined only by the inclusion of foreign objects (eg. bricks, steel, etc) or by distinctly unusual colour, texture or fabric. Identification of the extent of fill materials will also depend on investigation methods and frequency. Where natural soils similar to those at the site are used for fill, it may be difficult with limited testing and sampling to reliably assess the extent of the fill.

The presence of fill materials is usually regarded with caution as the possible variation in density, strength and material type is much greater than with natural soil deposits. Consequently, there is an increased risk of adverse engineering characteristics or behaviour. If the volume and quality of fill is of importance to a project, then frequent test pit excavations are preferable to boreholes.

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soils for Engineering Purposes'* or appropriate NSW Government Roads & Maritime Services (RMS) test methods. Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building) the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

Reasonable care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.
- Details of the development that the Company could not reasonably be expected to anticipate.

If these occur, the Company will be pleased to assist with investigation or advice to resolve any problems occurring.

SITE ANOMALIES

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The Company would

be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Copyright in all documents (such as drawings, borehole or test pit logs, reports and specifications) provided by the Company shall remain the property of Jeffery and Katauskas Pty Ltd. Subject to the payment of all fees due, the Client alone shall have a licence to use the documents provided for the sole purpose of completing the project to which they relate. Licence to use the documents may be revoked without notice if the Client is in breach of any obligation to make a payment to us.

REVIEW OF DESIGN

Where major civil or structural developments are proposed or where only a limited investigation has been completed or where the geotechnical conditions/constraints are quite complex, it is prudent to have a joint design review which involves an experienced geotechnical engineer/engineering geologist.

SITE INSPECTION

The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related.

Requirements could range from:

- i) a site visit to confirm that conditions exposed are no worse than those interpreted, to
- ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types and appropriate footing or pile founding depths, or
- iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL



FILL



TOPSOIL



CLAY (CL, CI, CH)



SILT (ML, MH)



SAND (SP, SW)



GRAVEL (GP, GW)



SANDY CLAY (CL, CI, CH)



SILTY CLAY (CL, CI, CH)



CLAYEY SAND (SC)



SILTY SAND (SM)



GRAVELLY CLAY (CL, CI, CH)



CLAYEY GRAVEL (GC)



SANDY SILT (ML, MH)



PEAT AND HIGHLY ORGANIC SOILS (Pt)

ROCK



CONGLOMERATE



SANDSTONE



SHALE/MUDSTONE



SILTSTONE



CLAYSTONE



COAL



LAMINITE



LIMESTONE



PHYLLITE, SCHIST



TUFF



GRANITE, GABBRO



DOLERITE, DIORITE



BASALT, ANDESITE



QUARTZITE

OTHER MATERIALS



BRICKS OR PAVERS



CONCRETE



ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 68% of soil excluding oversize fraction is greater than 0.075mm)	GRAVEL (more than half of coarse fraction is larger than 2.36mm)	GW	Gravel and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 4$ $1 < C_c < 3$
		GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		GM	Gravel-silt mixtures and gravel-sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	Fines behave as silt
		GC	Gravel-clay mixtures and gravel-sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	Fines behave as clay
	SAND (more than half of coarse fraction is smaller than 2.36mm)	SW	Sand and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 6$ $1 < C_c < 3$
		SP	Sand and gravel-sand mixtures, little or no fines	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		SM	Sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	N/A
		SC	Sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	

Laboratory Classification Criteria

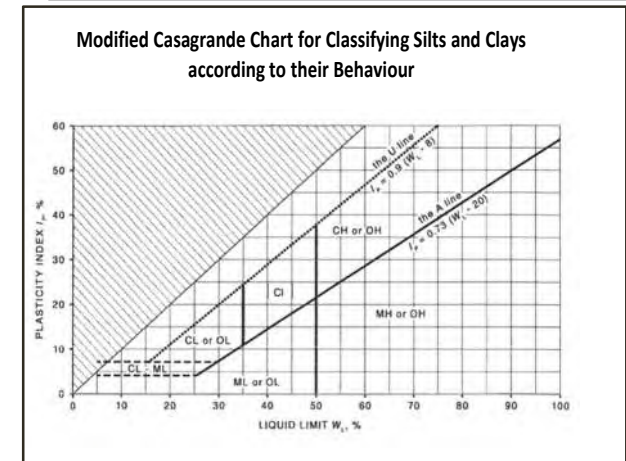
A well graded coarse grained soil is one for which the coefficient of uniformity $C_u > 4$ and the coefficient of curvature $1 < C_c < 3$. Otherwise, the soil is poorly graded. These coefficients are given by:

$$C_u = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_c = \frac{(D_{30})^2}{D_{10} D_{60}}$$




Where D_{10} , D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

- NOTES:**
- For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.
 - Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.
 - Clay soils with liquid limits $> 35\%$ and $\leq 50\%$ may be classified as being of medium plasticity.
 - The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	
fine grained soils (more than 35% of soil excluding oversize fraction is less than 0.075mm)	SILT and CLAY (low to medium plasticity)	ML	Inorganic silt and very fine sand, rock flour, silty or clayey fine sand or silt with low plasticity	None to low	Slow to rapid	Low	Below A line
		CL, CI	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay	Medium to high	None to slow	Medium	Above A line
		OL	Organic silt	Low to medium	Slow	Low	Below A line
	SILT and CLAY (high plasticity)	MH	Inorganic silt	Low to medium	None to slow	Low to medium	Below A line
		CH	Inorganic clay of high plasticity	High to very high	None	High	Above A line
		OH	Organic clay of medium to high plasticity, organic silt	Medium to high	None to very slow	Low to medium	Below A line
	Highly organic soil	Pt	Peat, highly organic soil	–	–	–	–



LOG SYMBOLS

Log Column	Symbol	Definition																											
Groundwater Record		Standing water level. Time delay following completion of drilling/excavation may be shown.																											
		Extent of borehole/test pit collapse shortly after drilling/excavation.																											
		Groundwater seepage into borehole or test pit noted during drilling or excavation.																											
Samples	ES	Sample taken over depth indicated, for environmental analysis.																											
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.																											
	DB	Bulk disturbed sample taken over depth indicated.																											
	DS	Small disturbed bag sample taken over depth indicated.																											
	ASB	Soil sample taken over depth indicated, for asbestos analysis.																											
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.																											
	SAL	Soil sample taken over depth indicated, for salinity analysis.																											
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.																											
	N _c =	<table border="1" style="display: inline-table; vertical-align: middle;"> <tr> <td>5</td> </tr> <tr> <td>7</td> </tr> <tr> <td>3R</td> </tr> </table>	5	7	3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60° solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.																							
	5																												
	7																												
3R																													
VNS = 25	Vane shear reading in kPa of undrained shear strength.																												
PID = 100	Photoionisation detector reading in ppm (soil sample headspace test).																												
Moisture Condition (Fine Grained Soils) (Coarse Grained Soils)	w > PL	Moisture content estimated to be greater than plastic limit.																											
	w ≈ PL	Moisture content estimated to be approximately equal to plastic limit.																											
	w < PL	Moisture content estimated to be less than plastic limit.																											
	w ≈ LL	Moisture content estimated to be near liquid limit.																											
	w > LL	Moisture content estimated to be wet of liquid limit.																											
	D	DRY – runs freely through fingers.																											
	M	MOIST – does not run freely but no free water visible on soil surface.																											
	W	WET – free water visible on soil surface.																											
	Strength (Consistency) Cohesive Soils	VS	VERY SOFT – unconfined compressive strength ≤ 25kPa.																										
		S	SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.																										
F		FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.																											
St		STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.																											
VSt		VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.																											
Hd		HARD – unconfined compressive strength > 400kPa.																											
Fr		FRIABLE – strength not attainable, soil crumbles.																											
()		Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.																											
Density Index/ Relative Density (Cohesionless Soils)		<table border="1" style="margin-left: auto; margin-right: auto;"> <thead> <tr> <th></th> <th>Density Index (I_D) Range (%)</th> <th>SPT 'N' Value Range (Blows/300mm)</th> </tr> </thead> <tbody> <tr> <td>VL</td> <td>VERY LOOSE</td> <td>≤ 15</td> <td>0 – 4</td> </tr> <tr> <td>L</td> <td>LOOSE</td> <td>> 15 and ≤ 35</td> <td>4 – 10</td> </tr> <tr> <td>MD</td> <td>MEDIUM DENSE</td> <td>> 35 and ≤ 65</td> <td>10 – 30</td> </tr> <tr> <td>D</td> <td>DENSE</td> <td>> 65 and ≤ 85</td> <td>30 – 50</td> </tr> <tr> <td>VD</td> <td>VERY DENSE</td> <td>> 85</td> <td>> 50</td> </tr> <tr> <td>()</td> <td colspan="3">Bracketed symbol indicates estimated density based on ease of drilling or other assessment.</td> </tr> </tbody> </table>		Density Index (I _D) Range (%)	SPT 'N' Value Range (Blows/300mm)	VL	VERY LOOSE	≤ 15	0 – 4	L	LOOSE	> 15 and ≤ 35	4 – 10	MD	MEDIUM DENSE	> 35 and ≤ 65	10 – 30	D	DENSE	> 65 and ≤ 85	30 – 50	VD	VERY DENSE	> 85	> 50	()	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.		
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Hand Penetrometer Readings	300	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.																											
	250																												

Log Column	Symbol	Definition	
Remarks	'V' bit	Hardened steel 'V' shaped bit.	
	'TC' bit	Twin pronged tungsten carbide bit.	
	T ₆₀	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.	
	Soil Origin	The geological origin of the soil can generally be described as:	
		RESIDUAL	– soil formed directly from insitu weathering of the underlying rock. No visible structure or fabric of the parent rock.
		EXTREMELY WEATHERED	– soil formed directly from insitu weathering of the underlying rock. Material is of soil strength but retains the structure and/or fabric of the parent rock.
		ALLUVIAL	– soil deposited by creeks and rivers.
		ESTUARINE	– soil deposited in coastal estuaries, including sediments caused by inflowing creeks and rivers, and tidal currents.
MARINE		– soil deposited in a marine environment.	
AEOLIAN		– soil carried and deposited by wind.	
COLLUVIAL	– soil and rock debris transported downslope by gravity, with or without the assistance of flowing water. Colluvium is usually a thick deposit formed from a landslide. The description 'slopewash' is used for thinner surficial deposits.		
LITTORAL	– beach deposited soil.		

Classification of Material Weathering

Term	Abbreviation	Definition
Residual Soil	RS	Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.
Extremely Weathered	XW	Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.
Highly Weathered	Distinctly Weathered (Note 1)	The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Moderately Weathered		
Slightly Weathered	SW	Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.
Fresh	FR	Rock shows no sign of decomposition of individual minerals or colour changes.

NOTE 1: The term 'Distinctly Weathered' is used where it is not practicable to distinguish between 'Highly Weathered' and 'Moderately Weathered' rock. 'Distinctly Weathered' is defined as follows: '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'. There is some change in rock strength.

Rock Material Strength Classification

Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Guide to Strength	
			Point Load Strength Index $I_{s(50)}$ (MPa)	Field Assessment
Very Low Strength	VL	0.6 to 2	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 30mm thick can be broken by finger pressure.
Low Strength	L	2 to 6	0.1 to 0.3	Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium Strength	M	6 to 20	0.3 to 1	Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High Strength	H	20 to 60	1 to 3	A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very High Strength	VH	60 to 200	3 to 10	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
Extremely High Strength	EH	> 200	> 10	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

Abbreviations Used in Defect Description

Cored Borehole Log Column	Symbol Abbreviation	Description	
Point Load Strength Index	• 0.6	Axial point load strength index test result (MPa)	
	x 0.6	Diametral point load strength index test result (MPa)	
Defect Details	– Type	Be	Parting – bedding or cleavage
		CS	Clay seam
		Cr	Crushed/sheared seam or zone
		J	Joint
		Jh	Healed joint
		Ji	Incipient joint
		XWS	Extremely weathered seam
	– Orientation	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
	– Shape	P	Planar
		C	Curved
		Un	Undulating
		St	Stepped
		Ir	Irregular
	– Roughness	Vr	Very rough
		R	Rough
		S	Smooth
		Po	Polished
		Sl	Slickensided
	– Infill Material	Ca	Calcite
		Cb	Carbonaceous
		Clay	Clay
		Fe	Iron
		Qz	Quartz
		Py	Pyrite
	– Coatings	Cn	Clean
		Sn	Stained – no visible coating, surface is discoloured
		Vn	Veneer – visible, too thin to measure, may be patchy
		Ct	Coating ≤ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres