SPECIALIST ADVICE TO

MONTEFIORE

ON

GEOTECHNICAL INVESTIGATIONS

FOR

PROPOSED AGED CARE AND RESIDENTIAL LIVING DEVELOPMENT

AT

MASADA COLLEGE, COLLEGE ROAD, ST IVES, NSW

Date: 3 June 2026
Ref: 37393PN2rpt rev2

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1 EXECUTIVE SUMMARY AND INTRODUCTION

This geotechnical specialist advice report has been prepared by JK Geotechnics Pty Ltd to accompany a State Significant Development Application (**SSDA**) for the development of seniors housing accommodation at 19 - 25 College Crescent, St Ives, NSW, in the Ku-ring-gai Local Government Area (**LGA**). The site is legally described as Lot 2 and Lot 3 in DP616326.

This report has been prepared to address the Secretary's Environmental Assessment Requirements (**SEARs**) (SSD-98363228) issued by the Department of Planning, Housing and Infrastructure on 10th November 2025. This report concludes that the proposed seniors housing is suitable and warrants approval subject to the implementation of the mitigation measures detailed in Sections 5 and 6 of this report.

Following the implementation of the recommended mitigation measures, the remaining impacts are considered appropriate.

The proposed development will include:

- Site preparation and demolition of existing structures.
- Construction of four buildings ranging in heights from five to eight storeys comprising:
    - approximately one hundred and twenty-six (126) Independent Living Units
    - approximately forty-eight (48) residential care beds.
    - a range of resident amenities and facilities.
    - basement car parking containing approximately 260 spaces.
- Vehicular and pedestrian access from Link Road and College Crescent.
- Associated landscaping works.

This report has been prepared to address the Secretary's Environmental Assessment Requirements (**SEARs**) and accompanying cover letter issued for the project (reference SSD-98363228) dated 10th November 2025.

Specifically, this report has been prepared to respond to geotechnical aspects of the SEARs requirement detailed below.

Item	**Description of requirement**	**Section reference**
12. Ground and Groundwater Conditions	Geotechnical Assessment: Assess potential impacts on soil resources and related infrastructure and riparian lands on and near the site and including erosion.	Section 4 of this report

The primary area of the site is located at 19-25 College Crescent, St Ives, within the Ku-ring-gai Local Government Area and is approximately 10 kilometres north of Chatswood and 19 kilometres north of Sydney CBD. This primary site is legally described as Lot 3 in DP 616326.

A small part of site to be developed is the adjoining lot upon which the Masada College is located at 9-15 Link Road, St Ives (legally described as part of Lot 2 in DP 616326). This part of the development site will specifically be used for access purposes and provide a direct connection from the proposal to the Link Road frontage.

On this basis, the site (for the purpose of this SSDA) comprises:

- the (primary) residual land, being 14,480 square metres (accommodating the proposed buildings and associated spaces); and
- part of the adjoining land, being 1,060 square metres (accommodating the proposed access driveway and pedestrian link from the proposed buildings to the Link Road frontage).

The site is zoned SP2 Infrastructure under the Ku-ring-gai LEP 2015.

From the provided Architectural Drawings (detailed below, all Revision 1) prepared by Jackson Teece, the lowest basement level will be at RL148.05m, and the upper basement levels will be at RL151.25m. Excavation to depths between about 5m and 8m below existing levels is expected to be required. From a provided TTW structural brief, maximum working column loads in the order of 7.5MN are anticipated.

DRAWING SCHEDULE

DRAWING LIST

SHEET NO	SHEET NAME		
DA-000	COVER SHEET	DA-300	SECTIONS - 01
DA-011	SITE CONTEXT	DA-301	SECTIONS - 02
DA-012	SITE ANALYSIS	DA-302	SECTIONS - 03
DA-020	DEMOLITION PLANS	DA-303	SECTIONS - 04
DA-030	SITE PLAN	DA-304	SECTIONS - 05
DA-108-1	FLOOR PLAN - BASEMENT 02 - NORTH	DA-500	TYPICAL ILU PLANS 1 BED
DA-108-2	FLOOR PLAN - BASEMENT 02 - SOUTH	DA-501	TYPICAL ILU PLANS 2 BED
DA-109-1	FLOOR PLAN - BASEMENT 01 - NORTH	DA-502	TYPICAL ILU PLANS 2 BED + STUDY AND 3 BED ILU
DA-109-2	FLOOR PLAN - BASEMENT 01 - SOUTH	DA-503	TYPICAL ILU PLANS 3 BED
DA-110-1	FLOOR PLAN - GROUND FLOOR - NORTH	DA-505	TYPICAL ILU PLANS 2 BED - BUILDING A
DA-110-2	FLOOR PLAN - GROUND FLOOR - SOUTH	DA-506	TYPICAL ILU PLAN 3 BED - BUILDING A
DA-110-3	FLOOR PLAN - GROUND FLOOR - LINK ROAD ACCESS	DA-510	SMOKE COMPARTMENT PLAN - RESIDENTIAL CARE FACILITY
DA-111-1	FLOOR PLAN - LEVEL 01 - NORTH	DA-600	GFA PLANS
DA-111-2	FLOOR PLAN - LEVEL 01 - SOUTH	DA-602	ADG TOTAL COMMUNAL OPEN SPACE
DA-112-1	FLOOR PLAN - LEVEL 02 - NORTH	DA-603	SEPP (HOUSING) 2021 RCF LANDSCAPED AREA
DA-112-2	FLOOR PLAN - LEVEL 02 - SOUTH	DA-700	ADG CROSS VENTILATION
DA-113-1	FLOOR PLAN - LEVEL 03 - NORTH	DA-701	ADG AND SEPP SOLAR ACCESS
DA-113-2	FLOOR PLAN - LEVEL 03 - SOUTH	DA-702	ADG COMPLIANCE MATRIX
DA-114-1	FLOOR PLAN - LEVEL 04 - NORTH	DA-750	DEVELOPMENT SUMMARY
DA-114-2	FLOOR PLAN - LEVEL 04 - SOUTH	DA-800	SHADOW DIAGRAMS 21 JUNE 9-10AM
DA-115-1	FLOOR PLAN - LEVEL 05 - NORTH	DA-801	SHADOW DIAGRAMS 21 JUNE 11AM-12PM
DA-115-2	FLOOR PLAN - LEVEL 05 - SOUTH	DA-802	SHADOW DIAGRAMS 21 JUNE 1-2PM
DA-116-1	FLOOR PLAN - LEVEL 06 - NORTH	DA-803	SHADOW DIAGRAMS 21 JUNE 3PM
DA-116-2	FLOOR PLAN - LEVEL 06 - SOUTH	DA-804	ELEVATIONAL SHADOW DIAGRAMS - COLLEGE CRESCENT HOUSES 21 JUNE
DA-117-1	FLOOR PLAN - LEVEL 07 - NORTH	DA-805	ELEVATIONAL SHADOW DIAGRAMS - CHABAD BUILDING 21 JUNE
DA-117-2	FLOOR PLAN - LEVEL 07 - SOUTH	DA-806	SHADOW DIAGRAMS 21 MARCH 9AM, 12PM
DA-120-1	FLOOR PLAN - ROOF - NORTH	DA-807	SHADOW DIAGRAMS 21 MARCH 3PM
DA-120-2	FLOOR PLAN - ROOF - SOUTH	DA-808	SHADOW DIAGRAMS 21 SEPTEMBER 9AM, 12PM
DA-201	ELEVATIONS - 01	DA-809	SHADOW DIAGRAMS 21 SEPTEMBER 3PM
DA-202	ELEVATIONS - 02	DA-850	DIRECT SUN ANALYSIS DIAGRAM 21 JUN 9AM-12PM
DA-203	ELEVATIONS - 03	DA-851	DIRECT SUN ANALYSIS DIAGRAM 21 JUN 1PM-3PM
DA-204	ELEVATIONS - 04	DA-860	ADG PRINCIPLE COMMUNAL OPEN SPACE SOLAR ANALYSIS - VILLAGE GREEN
DA-205	ELEVATIONS - 05	DA-861	ADG PRINCIPLE COMMUNAL OPEN SPACE SOLAR ANALYSIS - VILLAGE HEART
DA-206	ELEVATIONS - 06	DA-862	ADG PRINCIPLE COMMUNAL OPEN SPACE SOLAR ANALYSIS - LEVEL 01 TERRRACE
		DA-863	ADG PRINCIPLE COMMUNAL OPEN SPACE SOLAR ANALYSIS - TOTAL

The purpose of the investigations was to obtain geotechnical information on the subsurface conditions, and to use this as a basis to provide comments and recommendations on excavation conditions, hydrogeological conditions, shoring options, retaining wall design parameters, footing design, on-grade floor slabs, external pavement design parameters, and additional geotechnical input required. Additionally, the investigation was also carried out to address Items 12 and 13 of the provided Government Planning Secretary's Environmental Assessment Requirements (SEARs), WaterNSW/DPE/NRAR considerations, and SSDA requirements.

The supplementary geotechnical investigation was carried out in conjunction with a Detailed Site Investigation (DSI) by our environmental division, JK Environments (JKE). Reference should be made to the report prepared by JKE, Ref: E37395PHrpt2, for the results of the DSI.

2 INITIAL INVESTIGATION PROCEDURE

Eleven boreholes, BH1 to BH11, were drilled to depths between 1.4m (BH9) and 7.7m (BH5) using spiral augering techniques with our track mounted JK305 drill rig. With the exception of BH1 and BH2 which had a target depth of 1.95m, all of the boreholes refused in the bedrock prior to achieving their target depth of 12m. The compaction of the fill and strength of the residual soils were assessed from the results of Standard Penetration Tests (SPTs) completed in the boreholes and hand penetrometer tests on recovered cohesive soil samples. The strength of the bedrock was assessed from observation of the drilling resistance of a tungsten carbide (TC) drill bit attached to the augers, tactile examination of recovered rock chips, and correlation with the results of subsequent laboratory moisture content tests. The assessment of rock strength in this manner is approximate, and variation by about one order of strength would not be unexpected.

Groundwater observations were made during and on completion of drilling each borehole. Groundwater monitoring wells were subsequently installed in BH4, BH6 and BH11 to allow for ongoing groundwater monitoring, and we revisited site about 2 weeks following installation to measure standing water levels in the wells.

The borehole locations, as shown on the attached Borehole Location Plan (Figure 2) were measured using a differential GPS, the order of accuracy of which is expected to be about 50mm in all directions. The existing surface levels at the borehole locations, as shown on the borehole logs, were also measured using the GPS. The datum of the levels is Australian Height Datum (AHD).

Our geotechnical engineer, Jacob Feng, was on site full time during the fieldwork and set out the borehole locations, nominated the sampling and testing, directed the well installation, and prepared the attached borehole logs. For details of the investigation techniques adopted, and a glossary of logging terms and symbols used, reference should be the attached Report Explanation Notes.

Selected soil and rock chip samples were returned to Soil Test Services Pty Ltd (STS), a NATA accredited laboratory, for moisture content, Atterberg Limits, linear shrinkage, Standard compaction, and 4 day soaked CBR testing, the results of which are presented in the attached STS Tables A and B. Selected soil samples were also returned to Envirolab Services Pty Ltd, a NATA accredited laboratory, for pH, chloride content, sulphate content, and resistivity testing, the results of which are presented in attached Envirolab 'Certificate of Analysis' 374352.

3 SUPPLEMENTARY INVESTIGATION PROCEDURE

The fieldwork for the supplementary investigation was carried out between 15 and 20 January 2026 and comprised the drilling and testing of seven boreholes, BH101 to BH107. Groundwater monitoring wells were installed into three of the boreholes, BH102, BH104 and BH107 in accordance with DPE 2022. The borehole locations are also shown on the attached Figure 2. The details of the investigation are discussed below.

BH101 to BH107 were drilled and tested to depths ranging between 9.0m (BH102, BH104 and BH105) and 15.0m (BH103) below existing surface levels. The boreholes were completed using our track mounted JK305 drill rig, which are equipped for site investigation purposes. The soil and upper weathered bedrock profiles were spiral auger drilled. The relative compaction of the fill and the strength of the natural soil profile were assessed from the SPT 'N' values, together with hand penetrometer readings on clayey soils recovered in the SPT split-spoon sampler and on remoulded auger samples, and by tactile examination. The strength of the underlying bedrock was initially assessed by observation of auger penetration resistance when using a twin-pronged TC drill bit. From depths between 4.16m (BH102) and 6.06m (BH107), the boreholes were extended into the bedrock to their final depths by rotary diamond coring techniques, using NMLC triple tube core barrels with water flush. The strength of the cored bedrock was assessed by examination of the recovered rock cores and the results of subsequent laboratory Point Load Strength Index ($I_{S(50)}$) test results.

Groundwater observations were made during and on completion of auger drilling. Groundwater monitoring wells were installed in BH102, BH104 and BH107. The groundwater monitoring well in each borehole comprised a 50mm diameter Class 18 PVC standpipe. The machine slotted (screened) length of each standpipe (i.e. the 'response zone') was installed within the bedrock and residual profile. The annulus between the borehole and the slotted length was 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. Groundwater level data loggers were subsequently installed into BH102, BH104 and BH107 to allow for longer term groundwater level monitoring, and we completed preliminary measurements of groundwater levels on 30 January 2026 and 20 February 2026. Groundwater level monitoring was completed on 6 May 2026, and the results of the monitoring are presented on the attached Figures 5, 6 and 7. Pump out tests were also completed in the installed wells to assess the permeability of the soils and bedrock.

The borehole locations were set out by tape measurements from existing site features. The surface RL's shown on the attached borehole logs were interpolated between spot heights and ground contour lines indicated on provided survey plans prepared by LTS (Ref No. 52005 001ST, Sheets 1 to 12, dated 27 April 2023), and are therefore approximate.

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

Our geotechnical engineer was 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 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 summarised in the attached Table C.

4 RESULTS OF INVESTIGATION

4.1 Site Description

The surrounding area is characterised by residential flat buildings, aged care facilities, educational uses and commercial/retail uses within St Ives Town Centre, which is located approximately 400 metres west of the site on Mona Vale Road.

The site is located on a moderately sloping, south-east facing hillside. The site itself sloped down to the south-east at an average of about 5°. Link Road and College Crescent bound the site to the south-west and east, respectively. For the purpose of this report, the site has been taken to be the area outlined in red on Figure 1, which is wholly within the existing Masada College property.

At the time of fieldwork, the majority of the site comprised a near-level grassed field, with some trees and garden beds along the perimeter. In the south-eastern corner of the site were south and east facing batters below the level area (i.e. this portion of the site appeared to have been filled) which graded at between about 15° and 25°, and were a maximum of about 4m high. The north-western corner of the site contained tennis courts which were obscured by dense vegetation and could not be accessed or viewed. The north-eastern corner of the site, adjacent to the tennis courts, was about 1m higher than the playing fields. A concrete footpath extended between the tennis courts/higher area, and the playing fields.

The western portion of the site, where the new driveway to Link Road is proposed, was occupied by an asphaltic concrete surfaced carpark, garden areas, a tennis court, and demountable buildings.

To the north of the site is 132-138 Killeaton Street, which comprises a residential development with six residential flat buildings.

To the east of this development (at 142 Killeaton Street) comprises town houses and further east (at 144-146 Killeaton Street on the corner of Yarrabung Road) exists an aged care facility (Estia Healthcare). The site is bounded to the east by College Crescent, with tall street trees providing partial screening of the site. On the eastern side of College Crescent are detached houses, set back from the road and generally below street level, with significant street-front planting.

To the south of the site is Chabad North Shore. Further south are large, detached houses along Stanley Street and Lancaster Avenue, and residential flat buildings to the south-west along Stanley Street

To the west of the site is Masada College, which includes early learning, junior and secondary schools. An easement will be created on the edge of the campus to provide vehicle access from Link Road for the development. Link Road, a four-lane regional road, runs west of the school campus and connects to Mona Vale Road. West of Link Road are residential buildings along Newhaven Place, and further west are medium and higher-density housing and commercial premises in St Ives Town Centre. This includes five-six -storey buildings along Porters Lane, St Ives Shopping Centre and St Ives Village Green. The shopping centre precinct is expected to be subject to future major redevelopment.

Surface levels across the boundaries were generally similar.

A map of the regional context of the site and an aerial of the site is shown in Plates 1 and 2 below 1 and Figure 2.

Plate 1 Regional Context

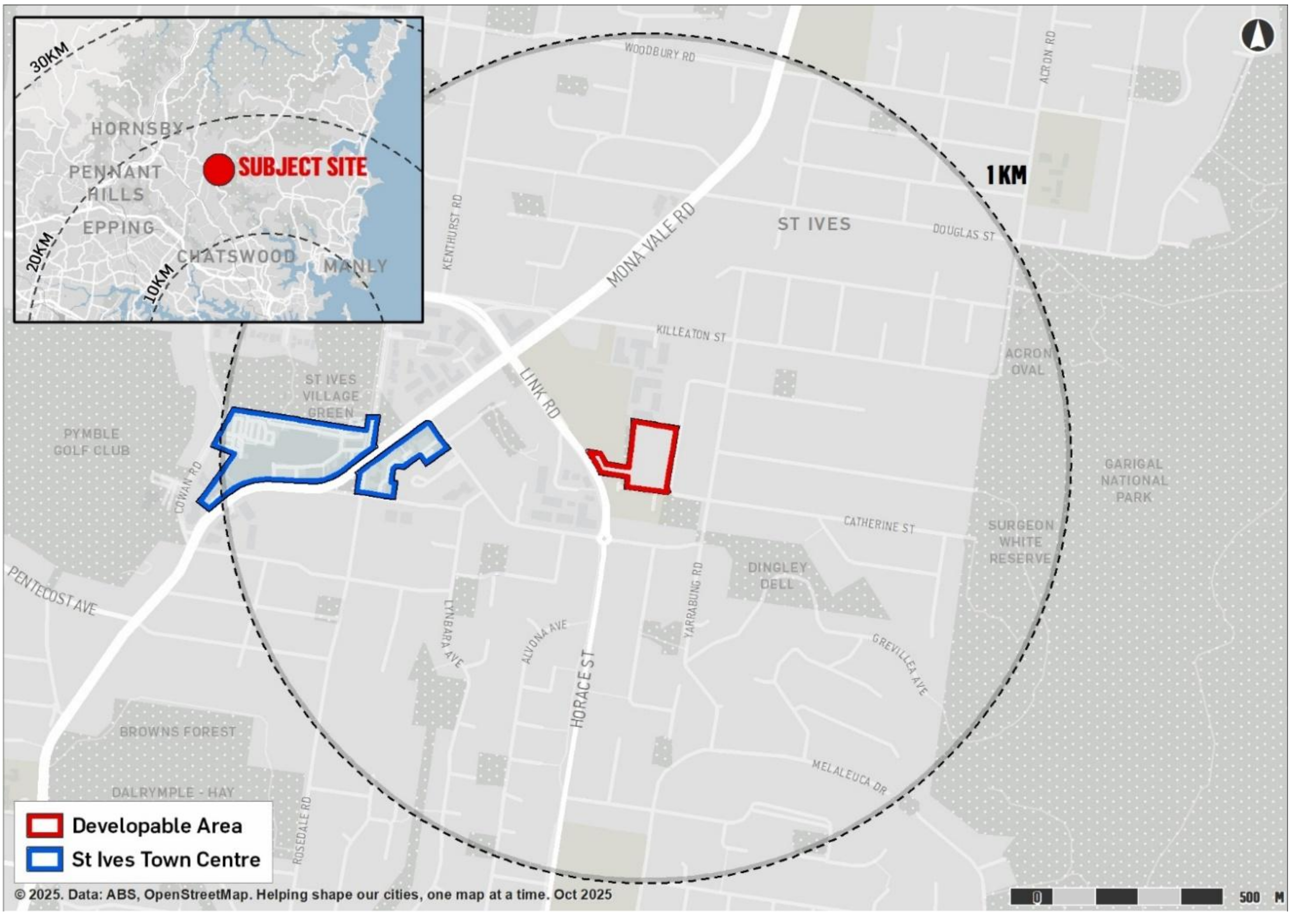

Plate 2 Aerial Photograph

Source: Urbis

4.2 Subsurface Conditions

The 1:100,000 geological map of Sydney indicates the site is underlain by Ashfield Shale of the Wianamatta Group. The current boreholes encountered a generalised subsurface profile comprising a variable, but generally shallow depth of fill overlying residual clays, with predominantly siltstone (shale) bedrock, to a lesser extent sandstone bedrock, from shallow to moderate depth. Groundwater was encountered at moderate depth.

For details of the encountered conditions, reference should be made to the attached borehole logs. A table summarising the encountered conditions is presented below.

Borehole ID	Approximate Surface RL (mAHD)	Depth to Base of Fill (m)	Depth to Top of Extremely Weathered or Very Low Strength Bedrock (m) / Reduced Level [mAHD]	Depth to Top of Bedrock of at least Low Strength (m) / Reduced Level [mAHD]
1	157.0	0.3	-	-
2	157.5	0.3	-	-
3	155.6	0.5	1.05 / [154.55]	1.05 / [154.55]

Borehole ID	Approximate Surface RL (mAHD)	Depth to Base of Fill (m)	Depth to Top of Extremely Weathered or Very Low Strength Bedrock (m) / Reduced Level [mAHD]	Depth to Top of Bedrock of at least Low Strength (m) / Reduced Level [mAHD]
4	155.0	2.7	5.2 / [149.8]	5.9 / [149.1]
5	155.6	1.4	3.7 / [151.9]	6.4 / [149.2]
6	156.2	0.3	1.5 / [154.7]	4.3 / [151.9]
7	154.1	0.5	1.8 / [152.3]	5.3 / [148.8]
8	155.8	0.3	2.1 / [153.7]	6.1 / [149.7]
9	156.5	0.5	0.5 / [156]	0.8 / [155.7]
10	157.8	0.4	2.1 / [155.7]	3.6 / [154.2]
11	157.1	0.2	1.6 / [155.5]	4.7 / [152.4]
101*	155.2	0.2	2.2 / [153]	8.5 / [146.7]
102	152.2	0.5	2.7 / [149.5]	3.8 / [148.4]
103	155.4	1.4	4.2 / [151.2]	5.5 / [149.9]
104*	156.0	0.3	1.0 / [155.0]	>9.0 / [>147.0]
105	155.4	0.2	2.0 / [153.4]	3.5 / [151.9]
106	156.6	0.2	0.8 / [155.8]	7.0 / [149.6]
107*	157.6	0.2	2.4 / [155.2]	9.2 / [148.4]

Extremely weathered or very low strength material between 1.0m and 3.5m thick was encountered within the better-quality bedrock

Fill

Fill comprising clayey soils of variable plasticity, was encountered from the surface in all the boreholes, and in most locations extended to less than 1m depth; in BH4, BH5 and BH103m the fill extended to respective depths of 2.7m, 1.4m and 1.4m. Based on the SPT results and hand penetrometer readings, the fill was assessed to be moderately or well compacted.

Residual Soils

Residual silty clay of high plasticity and very stiff or hard strength was encountered beneath the fill in all of the boreholes.

Bedrock

Extremely weathered (XW) siltstone of hard (soil) strength and siltstone bedrock of very low strength, occasionally containing stronger (i.e. low, medium and high strength) bands, were encountered in all the boreholes from generally shallow depth, and this strata was of varying thicknesses, as shown in table above.

The bedrock then mostly improved to be of predominantly low and greater strength, at the depths detailed in the table above, and in most of the cored boreholes, the basal portion of the bedrock was sandstone. However, in BH104, after initially improving to be of between very low and medium strength below 2.0m depth, the siltstone bedrock over almost the entire cored portion of the borehole was extremely weathered and of hard (soil) strength, and with BH104 terminated in this material.

Groundwater

Groundwater seepage was encountered during drilling in the boreholes except in BH102, at depths between 1.0m (BH101) and 4.0m (BH105). These levels are within XW or bedrock profile except for BH101 and BH103, where the groundwater seepage levels were measured within the residual profile. On completion of drilling, standing water was measured in BH106 at 5.0m depth.

The standing groundwater levels measured between January and May 2026 are summarised in the table below.

Borehole ID	Surface RLs (mAHD)	Highest Recorded Groundwater Reduced Level [mAHD]	Approximate Average Recorded Groundwater Reduced Level [mAHD]
102	152.2	151.4	151.1
104	156.0	153.3	153.1
107	157.6	153.6	153.3

4.3 Laboratory Test Results

The results of the Atterberg Limits tests confirmed the residual clays to be of high plasticity, and the results of the linear shrinkage tests indicated the natural soils to be highly reactive to moisture content change. The results of the moisture content tests correlated well with our field assessment of the bedrock strength. The four day soaked CBR tests returned CBR values of 3.5% or 6% for the residual clays, when compacted to 98% of their respective Standard Maximum Dry Density (SMDD) at close to their respective Standard Optimum Moisture Content (SOMC). The in-situ moisture content of the clays was between about 1% 'dry' and 5% 'wet' of the respective SOMCs. The CBR samples swelled by between 0.5% and 2% during soaking, which also indicates the clays are reactive to moisture content change.

The results of the Point Load Strength Index tests (JKG Table A) carried out on the recovered rock cores from BH101 to BH107 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 x $I_{S(50)}$), ranged from <1MPa and 64MPa, with a median of 22MPa. The estimated UCS's are predominantly between 10MPa and 30MPa.

The results of the Envirolab testing indicated the soils to be moderately acidic, with very low chloride and sulphate content, and high (favourable) resistivity.

4.4 Permeability Testing

Rising head (pump out) permeability tests were also completed in all the installed wells. The recorded water level vs time data recorded in the wells following pump out tests was analysed using published permeability formulae. The results of the permeability testing, all of which are for the weathered bedrock, are shown in the table below.

Borehole	Horizontal Coefficient of Permeability (m/s)
BH102	1.6×10^{-8}
BH104	8.8×10^{-8}
BH107	7.5×10^{-7}

The range of values tabulated above are consistent with published typical values for ‘siltstone bedrock with defects.

5 COMMENTS AND RECOMMENDATIONS

5.1 Dilapidation Surveys

Prior to any demolition or excavation commencing, we recommend that detailed dilapidation reports be prepared for any adjoining structures/properties within 20m of the proposed works. The dilapidation surveys should comprise a detailed inspection both externally and internally, with all defects rigorously described, e.g. defect location, defect type, crack width, crack length, etc. The respective property owners should be provided with a copy of the dilapidation reports and be asked to confirm that they present a fair representation of the existing conditions. We note that Council/TfNSW may also require that dilapidation reports be prepared for their adjoining assets.

Such reports can be used as a baseline against which to assess possible future claims for damage arising from the works.

5.2 Hydrogeology

Based on our investigations groundwater is present in the extremely weathered / very low strength bedrock between about RL151m and RL154m, with the groundwater level appearing to fall to the south-east . The measurements indicated that the groundwater levels are above the bulk excavation levels.

Notwithstanding, given the expected generally low permeability of the soil and bedrock, we consider that construction of a drained basement would be feasible and appropriate from a geotechnical standpoint. Groundwater seepage into the basement excavation would be generally expected to reduce as the excavation progresses, and the surrounding profile is drained of water. Locally higher inflows could be expected to occur through open joints or bedding planes during and following heavy rainfall events.

Long term groundwater flows would be expected to be of limited volume and would be able to be controlled by draining them to a sump for periodic pumped disposal to the stormwater system, or, where possible, for beneficial reuse such as for irrigation.

Groundwater seepage into the excavation must be monitored by the site foreman and geotechnical engineer as excavation progresses to confirm that seepage volumes are within the range anticipated.

Overall, it is considered that the construction of a drained basement will not be adversely affected by groundwater provided engineer designed drainage systems are constructed. Similarly, it is not expected that the development will have an adverse effect on the surrounding structures or improvements, or on regional groundwater flows.

We note that from extensive experience, WaterNSW (and Council) will likely require detailed groundwater seepage analysis be completed for the proposed basement to assess groundwater inflows and groundwater drawdown around the basements. In order to discharge water to stormwater in either the temporary or permanent case, groundwater quality will also need to be assessed. To inform the seepage analysis, groundwater monitoring and permeability testing of the subsurface profile will be required. We note that a tanked basements may be conditioned by either WaterNSW or Council, and would also be feasible.

In order to allow for a drained basement in the permanent case, it is also likely that Council and WaterNSW will prefer that any groundwater seepage be reused on site, e.g. for irrigation. This would be addressed once relevant authority conditions have been finalised.

5.3 Excavation

Excavations for the proposed basement will extend through the soil profile and into the underlying siltstone and sandstone bedrock.

Following installation of appropriate shoring, excavation of soils and bedrock of up to very low strength is expected to be readily achievable using conventional techniques such as the buckets of large hydraulic excavators.

Excavation through siltstone bedrock of low and greater strength will be expected to be slower, and we recommend grid sawing and hammering with smaller excavators and/or ripping using a large excavator (at least 30 tonne in size) in combination with sawing. Given the anticipated plan dimensions of the basement, ripping with a heavy dozer (equivalent or larger) would be feasible.

We recommend that considerable caution be taken during rock excavation on the site as there will likely be direct transmission of ground vibrations to existing nearby buildings. Further, there will also be transmission of vibrations to nearby services.

The dilapidation reports and the excavation procedures should be carefully reviewed prior to the commencement of excavation, so that appropriate equipment is used.

Excavation using hydraulic rock hammers should commence away from likely critical areas (i.e. away from existing structures and services). We recommend that continuous vibration monitoring be carried out during all demolition and excavation works where there are structures within about 20m of the excavation. Vibrations, measured as Peak Particle Velocity (PPV), must be limited to no higher than 5mm/sec for nearby buildings and services, subject to confirmation by the project structural engineer and/or a specialist vibration consultant that these vibration levels can be tolerated by those structures. This vibration limit must also be reviewed following completion of the dilapidation reports on the nearby buildings. If higher vibrations are recorded, they should be assessed against the attached Vibration Emission Design Goals as higher vibrations may be tolerable depending on the associated vibration frequency. If it is confirmed that transmitted vibrations are excessive, then it would be necessary to use smaller plant or alternative techniques, e.g. grid sawing in conjunction with ripping.

The use of grid sawing in conjunction with ripping presents an alternative low vibration excavation technique, however, productivity is likely to be slower and cost greater. When using a rock saw, the resulting dust must be suppressed by spraying with water.

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

- Maintain rock hammers orientated towards the face and enlarge excavation by breaking small wedges off the face.
- Use rock hammers in short bursts only to reduce amplification of vibrations.
- Maintain a sharp moil on the hammer.

We recommend use of excavation contractors with experience in such work with a competent supervisor who is aware of vibration damage risks. The contractor should be provided with a full copy of this and future reports and have all appropriate statutory and public liability insurances.

5.4 Shoring and Retaining Wall Design

5.4.1 Basement Shoring Options

Where space permits, temporary batter slopes up to about 2m in height within the soil and weathered bedrock of 1 Vertical (V) to 1 Horizontal (H) could be adopted in the short term, provided that no surcharge loads, including construction loads and existing footing loads, are close to the crests of the batters. In the south-east corner, we note that the first several metres of excavation will comprise the removal of the existing fill batter. Where higher batters are contemplated, specific geotechnical advice should be obtained, but benching will likely be required. Whilst the basement appears to be well offset from the site boundaries, such that batters could be accommodated, consideration must be given to the inherent difficulties and time consuming nature associated with subsequent backfilling (with engineered fill) of walls constructed within battered excavations. Where deeper excavation is expected, we consider that adoption of engineer designed shoring system installed prior to excavation commencing will be a more practical solution. Further, should locally high seepage flows occur, batter slopes will be less stable, and localised slumping and erosion may occur.

The shoring system may be incorporated into the permanent basement retention system. The effect of ground movements on any structures and services that lie within the influence zone of the excavation must also be taken into account. The influence zone of the excavation may be defined as a horizontal distance of 2H (where 'H' is the depth of the excavation in metres) behind the wall, but also depends on surcharge loads. Suitable retention systems, given the subsoil conditions encountered, would include a soldier pile wall with reinforced shotcrete infill panels, or a contiguous pile wall. Conventional bored piles are considered to be suitable for the expected subsurface conditions.

To reduce excavation induced movements behind the shoring walls, the shoring system(s) must be anchored or braced as the excavation progresses. Approval from neighbouring land owners would be required prior to the installation of anchors into their property (if required).

Whilst siltstone bedrock of medium and high strength was encountered above bulk excavation level in many of the boreholes, the quality of the siltstone (and sandstone) is variable, and the bedrock is not considered suitable to stand unsupported even in the temporary case. As such the piles should be installed to sufficient depth below bulk excavation level, including below footing and service excavations, to satisfy stability and founding considerations.

Drilling of rock sockets will be difficult through medium or greater strength bedrock, requiring the use of large drilling rigs equipped with rock augers and coring buckets (for bored piles).

5.4.2 Basement Shoring Design Parameters

The major consideration in the selection of earth pressures for the design of basement shoring walls is the need to limit deformations occurring outside the excavation. The following characteristic earth pressure coefficients and subsoil parameters may be adopted for the preliminary design of temporary or permanent retention systems.

For anchored or propped soldier pile or contiguous pile walls where minor movements can be tolerated, e.g. landscaped areas or similar, we recommend the use of a trapezoidal earth pressure distribution of 6HkPa for the soil profile and bedrock, where ‘H’ is the retained height in metres of soil and rock of less than consistent medium strength. These pressures should be assumed to be uniform over the central 50% of the height of the support system, tapering to zero at the crest and toe. Where movements are to be limited, e.g. where neighbouring structures or movement sensitive services are located within 2H of the wall, an increased maximum pressure of 8HkPa should be adopted.

Consideration must also be given to the possible presence of continuous inclined joints which can be present in the siltstone bedrock in Sydney. We recommend an ultimate load case on the walls be considered assuming a joint inclined at between 45° and 65° extending up from bulk excavation level to the top of bedrock of low strength. The shear resistance of such a joint can be calculated using an interface friction angle of 25°. Identifying the presence of such defects from boreholes is not possible and as such, it is not possible to eliminate this design case by completing additional investigation.

A bulk unit weight of 21kN/m^3 should be adopted for the soil and bedrock.

Any surcharge affecting the walls (e.g. traffic loading, construction loads, adjacent high level footings, etc.) should be allowed for in the design using an 'at-rest' earth pressure coefficient, K_0, of 0.5.

The retaining walls are to be designed as 'drained' and measured taken to provide permanent and effective drainage of the ground behind the walls. The subsoil drains should incorporate a non-woven geotextile fabric (e.g. Bidim A34) to act as a filter against subsoil erosion.

Lateral toe restraint of piles socketed into the bedrock below bulk excavation level can be calculated adopting an allowable lateral resistance of 200kPa for bedrock of low or greater strength, though the upper 0.5m of socket below excavation level must be ignored to allow for disturbance or possible over excavation. For piles embedded into bedrock below bulk excavation level, a minimum embedment depth (ignoring the 0.5m allowance above) of 1.0m should apply. Care is required not to over-excavate in front of the piles, and all excavations in front of the walls, such as for footings, tanks, buried services, etc. must be taken into account in the wall design.

Temporary anchors bonded into very low or greater strength bedrock could be designed for an allowable bond of 150kPa. The anchors should have minimum free lengths and bond lengths of 4m and 3m respectively, with the bond length being entirely behind a line drawn upward at 1V in 1H from bulk excavation level. All anchors should be proof loaded to 130% of their working load in the presence of a geotechnical or structural engineer independent of the anchoring contractor, prior to locking off. Lift-off tests should be completed on 20% of the anchors approximately 3 days to 4 days after locking off to confirm the anchors are continuing to hold their load. We recommend that ground anchors be part of a design and construct sub-contract to avoid disputes is anchors fail test loading. We strongly recommend all anchors be thoroughly flushed with air following drilling, to remove any water which could soften the bedrock, further, we also recommend all anchors be installed and grouted without delay following drilling to reduce the potential for softening.

Alternatively, the retaining walls could be designed using computer based analysis methods (e.g. Wallap or Plaxis), which could result in cost savings compared to a design based on the above simplified earth pressure assumptions. Analysis software treating the soil as 'equivalent springs' should not be used for this design. Wallap or Plaxis type analysis methods can model the actual excavation stages, including progressive anchoring/shoring, and outputs include structural actions in the piles, anchor/prop loads, and wall movements. The analysis should be completed by an engineer with a good understanding of soil-structure interaction behaviour, including an understanding of when soil-wall friction should and should not be used etc. Parameters for analysis could be provided following completion of the recommended supplementary investigation.

5.4.3 Conventional Retaining Walls

For retaining walls outside basement excavations, or if a battered excavation will be adopted, we assume these will be constructed within temporarily battered excavations, and then backfilled, or will be supporting fill.

Free standing cantilever walls, with a maximum height of about 2m, and where minor wall movements are tolerable (e.g. when supporting soft landscaped areas) should be designed using a triangular lateral earth pressure distribution with an ‘active’ earth pressure coefficient, K_a, of 0.35 for the soil and weathered bedrock, as well as for any backfill materials, assuming a horizontal retained surface.

Cantilever walls which will be propped or restrained by structures and subsequently backfilled, or where wall movements are to be limited, should be designed using a triangular lateral earth pressure distribution with an 'at rest' earth pressure coefficient, K_0, of 0.5 for the soil, as well as for any backfill materials, assuming a horizontal retained surface.

A bulk unit weight of 21kN/m^3 should be adopted for the soil for the purpose of retaining wall design.

Any surcharge affecting the walls (e.g. traffic loading, construction loads, adjacent high level footings, sloping backfill, etc.) should be allowed for in the design using the appropriate earth pressure coefficient from above.

All retaining walls should be designed as 'drained' and measures taken to provide permanent and effective drainage of the ground behind the walls. The subsoil drains should incorporate a non-woven geotextile fabric (e.g. Bidim A34) to act as a filter against subsoil erosion.

Lateral toe restraint of cantilever walls may be achieved by passive resistance of the soil (engineered fill and residual soils, i.e. not the existing fill) in front of the wall using a triangular lateral earth pressure distribution and a 'passive' earth pressure coefficient, K_P, of 3.0 for the soil and weathered bedrock. We note that significant movement is required in order to mobilise the full passive pressure of the resisting material, and therefore a factor of safety of at least 2 should be adopted to reduce such movements. Any localised excavations in front of the wall should be taken into account in the embedment design. Friction on the base of a wall can be calculated using a friction angle of 28˚ between the retaining wall base and the soil below, provided the base is clean, rough and 'dry' when the retaining wall footing is poured. If walls outside basement excavations will be socketed into or founded on bedrock, higher lateral resistance and base shear resistance may be appropriate, and further geotechnical advice should be sought prior to the retaining wall design being finalised.

5.5 Footings

5.5.1 Main Building

On completion of excavation, bedrock of variable strength and will be exposed at bulk excavation level, and we therefore recommend that all of the buildings be uniformly supported on footings/piles to the bedrock Where bedrock is exposed, or is at shallow depth, say less than 1m, Where bedrock is at greater depth, say outside the footprint of the basements, would be more appropriate. All piles must be founded below a line draw up at 45° from the base of the excavation unless basement retaining walls have been designed for the surcharge loads.

Pad and strip footings founded in, and piles socketed at least 0.3m into, bedrock of at least low strength can be designed based on an allowable bearing pressure 1,000kPa. Pile sockets longer than this 0.3m can be designed based on allowable shaft adhesion of 100kPa in compression and 50kPa in tension (uplift), provided the socket is satisfactorily cleaned and roughened. Due to the variability in the bedrock strength/quality, we do not consider it would be appropriate to design footings for higher bearing pressures.

At least the initial stages or pile drilling and pad footing excavation must be inspected by a geotechnical engineer to confirm that the appropriate foundation material has been achieved.

5.5.2 Light Weight Buildings and Retaining Walls

Structures outside the basements will include low height retaining walls, pavilions in garden areas, and similar structures. Such structures could be founded in engineered fill (refer to Section 5.7.1 below), the natural soils, or the underlying bedrock.

Detailed design advice for footings for structures outside the proposed basement would be provided once architectural details for such structures are known. Indicatively however, an allowable bearing pressure of 100kPa would likely be feasible for shallow footings founded in the soil (either engineered fill or residual clays) for such structures. The clayey soils are reactive and would probably result in shrink swell movements equivalent to a Class H1 or H2 site in accordance with AS2870-2011, following the completion of earthworks. We note however the presence of fill and nearby trees which result in a Class P classification. As such, design of all structures outside the basement must be completed adopting engineering principles. Further recommendations could be provided on the classification when the nature and locations of the structures identified, and the nature of the earthworks is known. Retaining walls and similar longitudinal structures, e.g. walls, should incorporate articulation at reasonably close centres to allow for possible differential movements.

5.6 On-Grade Floor Slabs

Basement on-grade floor slabs are expected to directly overlie bedrock and no particular subgrade preparation will be required.

Slab-on-grade construction is therefore considered appropriate. Underfloor drainage must however be provided. For all buildings, the underfloor drainage should connect with the wall drains, where appropriate, and direct groundwater seepage to a sump(s) for pumped disposal to a stormwater system following obtaining authority approval. Joints in the on-grade floor slabs should incorporate dowels or keys.

If a soil subgrade is present in any portion of the basement, further geotechnical advice must be sought as to appropriate subgrade preparation and on-grade floor slab design considerations.

We recommend that the floor slabs for any portions of the ground floor level which extend outside the basement footprint be designed as suspended, supported on piles founded in the underlying bedrock, to reduce to potential for differential movements between these and the portions of the ground floor level above the basement. To reduce the potential for high swell pressures which can result from the reactive soils at the site, any suspended on-grade slabs should incorporate a collapsible (cardboard) void former not less than 75mm thick.

5.7 Earthworks and External Pavements

5.7.1 Subgrade Preparation and Engineered Fill

All earthworks recommendations provided below should be complemented by reference to AS3798-2007 'Guidelines on Earthworks for Commercial and Residential Developments'.

Site Drainage

The clay subgrade at the site is expected to undergo substantial loss of strength when wet, as indicated by the moderately low CBR values. Furthermore, the clay subgrade is expected to have a high shrink-swell reactive potential. Soils with these characteristics are also often dispersive, though this property has not been confirmed by testing. Therefore, it is important to provide good and effective site drainage both during construction and for long-term site maintenance. The principal aim of the drainage is to promote run-off and reduce ponding. A poorly drained clay subgrade may quickly become untrafficable when wet. The earthworks should be carefully planned and scheduled to maintain good cross-falls during construction.

In the long term, good drainage must be provided to prevent scour and erosion of batter slopes and to protect all structures, including pavements, from adverse effects.

Site and Subgrade Preparation

Following removal of all vegetation within the footprint of any pavements (or other proposed earthworks) on site, and thorough grubbing out of all root balls, all topsoil and root affected soils should also be stripped.

Stripped topsoil/root affected soils must be stockpiled separately as they are considered unsuitable for reuse as engineered fill. They may however be reused for landscaping purposes. Reference should be made to the JKE report for guidance on the offsite disposal of soil.

All existing fill, where present, must also be stripped, but can be stockpiled for reuse as engineered fill.

In any areas where the natural soils are water softened, the water softened material must be boxed out to a sound base and replaced with engineered fill.

Following stripping of all topsoil and existing fill, and excavation to achieve design subgrade levels, the exposed subgrade should proof rolled with at least 6 passes of a minimum 12 tonne deadweight smooth drum roller. The final pass of proof rolling should be witnessed by an experienced geotechnical engineer to check for the presence of possible soft or heaving areas. If any soft or heaving areas are identified, the advice on remediation options would be provided at the time of the inspection, however, it is most likely that locally removing any softened materials to a sound base would be recommended.

If soil softening occurs after rainfall periods, then the clay subgrade should be over-excavated to below the depth of moisture softening and replaced with engineered fill. If the clay subgrade exhibits shrinkage cracking, then the surface must be moistened with a water cart and rolled until the shrinkage cracks are no longer evident, or the layer tyned, moisture conditioned, and recompacted. Care must be taken not to over-water the subgrade as this will result in softening.

All fill used to remediate soft/heaving areas, or raise site levels must be placed as engineered fill.

Engineered Fill

From a geotechnical perspective, the excavated fill, residual soils, and bedrock are considered suitable for reuse as engineered fill on condition that they are 'clean', free of organic matter and contain a maximum particle size not exceeding 70mm. For the excavated bedrock, crushing of oversize particles may be required, but for a siltstone this is commonly achieved below the drum(s) of large rollers.

All clayey fill should be compacted in layers not exceeding 200mm loose thickness to a density ratio strictly between 98% and 102% of SMDD and within 2% of SOMC. Some moisture conditioning may be required in order to meet this specification. Engineered fill comprising well graded granular materials, such as imported crushed sandstone, should be compacted layers not exceeding 200mm loose thickness to achieve a density ratio of not less than 98% of SMDD.

Where filling is completed against temporary batter slopes, horizontal benches must be formed at no greater than 0.4m vertical increments so that the engineered fill is 'keyed' into the slope.

Backfilling of service trenches must be carried out using engineered fill in order to reduce post-construction settlements. Due to the reduced energy output of the compaction plant that can be placed in trenches, backfilling should be carried out in layers not exceeding 100mm loose thickness and compacted using a trench roller, a pad foot roller attachment fitted to an excavator, and/or a vertical rammer compactor (also known as a 'Wacker Packer'). Due to the reduced loose layer thickness, the maximum particle size of the backfill material should also be reduced to not more than 50mm. The compaction specifications provided above are applicable.

As for services trenches, retaining wall backfilling must also be carried out using engineered fill in order to reduce post-construction settlements. Compaction of the engineered backfill should be carried out using a trench roller or hand operated vertical rammer compactor for the lower layers and immediately behind the wall in the upper layers. Elsewhere a small static roller could be used. As per service trenches, backfilling should be carried out in layers not exceeding 100mm loose thickness and the maximum particle size of the backfill material should be reduced to not more than 50mm. The compaction specifications provided above are applicable.

Compaction of engineered fill immediately behind retaining walls is time consuming and not commonly undertaken. The more common method comprises using a single sized hard, durable, free draining aggregate, such as 'Blue Metal' or crushed concrete aggregate (free of fines, brick and tile fragments), which do not require significant compactive effort. Such material should be nominally compacted using a hand operated vibrating plate (sled) compactor in maximum 400mm thick loose layers. A non-woven geotextile filter fabric such as Bidim A34 should be placed as a separation layer immediately above the cut batter slope (prior to backfilling) to control subsoil erosion into the gravel, and this should be wrapped over the surface of the aggregate backfill and capped with at least a 0.3m thick compacted layer of clayey engineered fill, or concrete, to reduce the potential for surface water to enter the retaining wall drainage. Provided the aggregate backfill is placed as recommended above, density testing would not be required in that material.

In-situ density tests must be carried out on the engineered fill to confirm the above specifications are achieved, in accordance with the requirements of Table 8.1 of AS3798-2007.

Level 1 testing of fill placement and compaction in accordance with AS3798-2007 is recommended for all fill which will support proposed structures. Due to a potential conflict of interest, we strongly recommend that the GITA be directly engaged by the client, and not by the earthworks contractor or sub-contractors.

5.7.2 External Pavements

For external pavements, provided the subgrade has been appropriately prepared as described above, a CBR value of 3.5% can be adopted for design, or, a short term Young's Modulus of 35MPa, based on the results of the laboratory testing.

We recommend that all basecourse materials for flexible pavements and sub-base materials for rigid pavements comprise DGB20 which meets the requirements of TfNSW QA Specification 3051 unbound base. The DGB20 material should be compacted in maximum 200mm thick loose layers using a large smooth drum roller to at least 98% of Modified Maximum Dry Density (MMDD). Adequate moisture conditioning to within 2% of Modified Optimum Moisture Content (MOMC) should be provided during placement to reduce the potential for material breakdown. For rigid pavement, a cement stabilised or lean mix concrete subbase could also be considered.

We further recommend that all sub-base materials for flexible pavements comprise DGS40, DGS20 or DGB20 which meets the requirements of TfNSW QA Specification 3051. The sub-base material should be compacted in maximum 200mm thick loose layers using a large smooth drum roller to at least 95% of MMDD. Again, adequate moisture conditioning to within 2% of MOMC should be provided during placement.

The final material and compaction specification must be determined by the pavement designer.

Density tests should be carried out on the granular pavement materials to confirm the above specifications are achieved. The frequency of density testing should be at least three tests per layer, or three tests per visit, whichever requires the most tests. Level 2 testing in accordance with AS3798-2007 would be considered acceptable for the pavement layers. The geotechnical testing authority (GTA) should be directly engaged by the client, and not by the earthworks contractor or sub-contractors.

Subsoil drains should be provided below the perimeter of the proposed pavements, including any internal planters etc. with invert levels at least 200mm below design subgrade level. The drainage trenches should be excavated with a continuous longitudinal fall to appropriate discharge points so as to reduce the risk of water ponding. The subgrade should be graded to promote water flow towards the subsoil drains. Discharge from the subsoil drains should be piped to the stormwater system.

5.8 Earthquake Design

In accordance with Table 4.1 of AS1170.4-2007, the site subsoil class is 'Class C_e – Shallow Soil' as the depth of soil exceeds 3m in areas.

5.9 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 of the comments and recommendations in this report following the completion of architectural design;
- Analysis of shoring walls;
- Quantitative vibration monitoring during rock excavation;
- Proof-testing of anchors;
- Seepage monitoring during excavation; and
- Geotechnical footing inspections.

6 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the design and construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. 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.

The long term successful performance of floor slabs and pavements is dependent on the satisfactory completion of the earthworks. In order to achieve this, the quality assurance program should not be limited to routine compaction density testing only. Other critical factors associated with the earthworks may include subgrade preparation, selection of fill materials, control of moisture content and drainage, etc. The satisfactory control and assessment of these items may require judgment from an experienced engineer. Such judgment often cannot be made by a technician who may not have formal engineering qualifications and experience. In order to identify potential problems, we recommend that a pre-construction meeting be held so that all parties involved understand the earthworks requirements and potential difficulties. This meeting should clearly define the lines of communication and responsibility.

Occasionally, the subsurface conditions between and below 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 advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Contract Documents and Specifications may be prepared based on our report. However, there may be design features we are not aware of or have not commented on for a variety of reasons. The designers should satisfy themselves that all the necessary advice has been obtained. If required, we could be commissioned to review the geotechnical aspects of contract documents to confirm the intent of our recommendations has been correctly implemented.

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.

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE A

MOISTURE CONTENT, ATTERBERG LIMITS AND LINEAR SHRINKAGE TEST REPORT

Client: JK Geotechnics
Project: Proposed Aged Care and Residential Living Development
Location: Masada College, College Road, St Ives, NSW

Report No.: 37393PN - A
Report Date: 6/03/2025

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	**DEPTH m**	**MOISTURE CONTENT %**	**LIQUID LIMIT %**	**PLASTIC LIMIT %**	**PLASTICITY INDEX %**	**LINEAR SHRINKAGE %**
3	1.10 - 1.30	11.2	-	-	-	-
4	5.30 - 5.70	6.7	-	-	-	-
4	5.90 - 6.10	5.6	-	-	-	-
5	1.50 - 1.95	24.2	59	28	31	16.5*
7	2.60 - 2.90	9.4	-	-	-	-
7	5.30 - 5.60	6.0	-	-	-	-
9	1.20 - 1.40	10.0	-	-	-	-
10	0.50 - 0.95	15.3	61	27	34	12.5
10	2.10 - 2.50	12.8	-	-	-	-
10	3.60 - 3.80	10.0	-	-	-	-
11	3.60 - 3.90	9.7	-	-	-	-
11	4.80 - 5.20	9.9	-	-	-	-

Notes:
- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 27/02/2025.
- Sampled and supplied by client. Samples tested as received.
- * Denotes Linear Shrinkage cracked.

NATA
NATA Accredited Laboratory Number:1327

07/03/2025
Authorised Signature / Date
(D. Treweek)

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SOIL TEST SERVICES
ABN 43 002 145 173

TABLE B
FOUR DAY SOAKED CALIFORNIA BEARING RATIO TEST REPORT

Client:	JK Geotechnics	**Report No.:**	37393PN - B
Project:	Proposed Aged Care and Residential Living Development	**Report Date:**	11/03/2025
Location:	Masada College, College Road, St Ives, NSW	**Page 1 of 1**	

BOREHOLE NUMBER		BH 1	BH 2	BH 11
DEPTH (m)		0.50 - 1.50	0.50 - 1.50	0.50 - 1.50
Surcharge (kg)		9.0	9.0	9.0
Maximum Dry Density (t/m^3)		1.72 STD	1.58 STD	1.61 STD
Optimum Moisture Content (%)		19.5	23.9	21.6
Moulded Dry Density (t/m^3)		1.67	1.54	1.57
Sample Density Ratio (%)		98	98	98
Sample Moisture Ratio (%)		99	101	101
Moisture Contents				
Insitu (%)		18.8	28.7	21.5
Moulded (%)		19.3	24.2	21.8
After soaking and After Test, Top 30mm(%)		28.3	29.1	28.9
Remaining Depth (%)		22.3	25.9	25.7
Material Retained on 19mm Sieve (%)		0	0	0
Swell (%)		2.0	0.5	1.0
C.B.R. value:	@2.5mm penetration	3.5		6
	@5.0mm penetration		3.5	

NOTES: Sampled and supplied by client. Samples tested as received.

- Refer to appropriate Borehole logs for soil descriptions
- Test Methods : AS 1289 6.1.1, 5.1.1 & 2.1.1.
- Date of receipt of sample: 27/02/2025.
- BH's 1 & 11 had insufficient material supplied to complete a 4-point compaction curve.

NATA
NATA Accredited Laboratory Number:1327

11/03/2025
Authorised Signature / Date
(D. Treweek)

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client: EQUITY DEVELOPMENT MANAGEMENT **Ref No:** 37393PN2

Project: PROPOSED INDEPENDENT LIVING AND AGED CARE DEVELOPMENT **Report:** C

Location: 25 COLLEGE CRESCENT, ST IVES, NSW **Report Date:** 28/01/26

BOREHOLE NUMBER	DEPTH (m)	$I_{S\ (50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
101	8.10 - 8.12	0.2	4	A
	8.24 - 8.27	0.6	12	A
	8.41 - 8.43	0.9	18	A
	8.52 - 8.56	1.3	26	A
	8.89 - 8.91	1.3	26	A
	9.05 - 9.08	0.3	6	A
	9.15 - 9.18	1.8	36	A
	9.62 - 9.65	0.7	14	A
102	4.35 - 4.38	1.7	34	A
	4.66 - 4.68	2.3	46	A
	4.96 - 4.98	1.1	22	A
	5.25 - 5.29	0.9	18	A
	5.73 - 5.75	1.8	36	A
	6.00 - 6.02	1.1	22	A
	6.50 - 6.54	1	20	A
	6.90 - 6.93	2.1	42	A
	7.23 - 7.26	1.4	28	A
	7.73 - 7.76	1.6	32	A
	8.29 - 8.32	1.2	24	A
	8.81 - 8.84	1.3	26	A
103	7.30 - 7.33	1.4	28	A
	7.82 - 7.86	0.7	14	A
	8.00 - 8.03	0.6	12	A
	8.30 - 8.35	0.9	18	A
	8.72 - 8.76	1.2	24	A

NOTE: SEE PAGE 3

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client: EQUITY DEVELOPMENT MANAGEMENT **Ref No:** 37393PN2

Project: PROPOSED INDEPENDENT LIVING AND AGED CARE DEVELOPMENT **Report:** C

Location: 25 COLLEGE CRESCENT, ST IVES, NSW **Report Date:** 28/01/26

BOREHOLE NUMBER	DEPTH (m)	$I_{S(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
103	9.38 - 9.41	1.4	28	A
	9.86 - 9.90	1.5	30	A
	10.27 - 10.31	0.8	16	A
	10.67 - 10.71	2.1	42	A
	11.17 - 11.19	0.9	18	A
	11.70 - 11.75	1.5	30	A
	12.18 - 12.22	1.5	30	A
	12.70 - 12.74	1	20	A
	13.09 - 13.12	1.6	32	A
	13.59 - 13.62	2.1	42	A
	14.19 - 14.22	0.7	14	A
	14.50 - 14.53	2.7	54	A
104	7.03 - 7.06	0.1	2	A
	8.60 - 8.63	0.6	12	A
105	6.33 - 6.36	0.2	4	A
	6.67 - 6.70	1.5	30	A
	6.84 - 6.87	1	20	A
	7.37 - 7.40	1.1	22	A
	7.85 - 7.89	1.6	32	A
	8.29 - 8.32	1	20	A
	8.71 - 8.75	1.2	24	A
106	6.29 - 6.32	1.1	22	A
	7.11 - 7.13	1	20	A
	8.53 - 8.57	1.8	36	A
	8.88 - 8.92	3.2	64	A

NOTE: SEE PAGE 3

TABLE C
POINT LOAD STRENGTH INDEX TEST REPORT

Client: EQUITY DEVELOPMENT MANAGEMENT **Ref No:** 37393PN2

Project: PROPOSED INDEPENDENT LIVING AND AGED CARE DEVELOPMENT **Report:** C

Location: 25 COLLEGE CRESCENT, ST IVES, NSW **Report Date:** 28/01/26

BOREHOLE NUMBER	DEPTH (m)	$I_{S(50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
106	9.09 - 9.13	1.4	28	A
107	8.03 - 8.05	0.2	4	A
	8.50 - 8.53	0.02	<1	A
	9.14 - 9.17	0.2	4	A
	9.43 - 9.46	0.3	6	A
	9.77 - 9.79	0.4	8	A
	10.19 - 10.22	1	20	A
	10.56 - 10.58	1	20	A
	10.92 - 10.95	0.7	14	A
	11.11 - 11.16	1.2	24	A
	11.38 - 11.41	1.5	30	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 $I_{S(50)}$.

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CERTIFICATE OF ANALYSIS 374352

Client Details

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

Sample Details

Your Reference	**37393PN, Masada College, College Rd, St Ives NSW**
Number of Samples	3 Soil
Date samples received	28/02/2025
Date completed instructions received	28/02/2025

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.

Report Details

Date results requested by	07/03/2025
Date of Issue	06/03/2025

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Accredited for compliance with ISO/IEC 17025 - Testing. **Tests not covered by NATA are denoted with ***

Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By
Nancy Zhang, Laboratory Manager

Misc Inorg - Soil				
Our Reference		374352-1	374352-2	374352-3
Your Reference	UNITS	4	4	10
Depth		1.5-1.95	4.5-4.9	1.5-1.95
Date Sampled		24/02/2025	24/02/2025	24/02/2025
Type of sample		Soil	Soil	Soil
Date prepared	-	03/03/2025	03/03/2025	03/03/2025
Date analysed	-	03/03/2025	03/03/2025	03/03/2025
pH 1:5 soil:water	pH Units	5.1	5.1	5.5
Chloride, Cl 1:5 soil:water	mg/kg	<10	10	10
Sulphate, SO4 1:5 soil:water	mg/kg	51	120	37
Resistivity in soil*	ohm m	210	130	330

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.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			03/03/2025	[NT]	[NT]	[NT]	[NT]	03/03/2025	[NT]
Date analysed	-			03/03/2025	[NT]	[NT]	[NT]	[NT]	03/03/2025	[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]	108	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	108	[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.

JKGeotechnics

BOREHOLE LOG

Borehole No. **1**

1/1

Client: MONTEFIORE

Project: PROPOSED AGED CARE AND RESIDENTIAL LIVING DEVELOPMENT

Location: MASADA COLLEGE, COLLEGE ROAD, ST IVES, NSW

Job No.: 37393PN **Method:** SPIRAL AUGER **R.L. Surface:** 157.00m

Date: 25/2/25 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** J.F./N.E.S.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLET-ION			0		-	ASPHALTIC CONCRETE: 20mm.t	w<PL			
						FILL: Silty clay, medium to high plasticity, grey mottled orange brown.				
		N = 21 6,9,13			CH	Silty CLAY: high plasticity, light grey mottled orange brown.	w<PL	Hd	>600 >600 >600	RESIDUAL
			1							
		N = 42 14,17.25				as above, but with extremely weathered fabric.	w≈PL		>600 >600 >600	
			2			END OF BOREHOLE AT 1.95m				
			3							
			4							
			5							
			6							
			7							

JKGeotechnics

BOREHOLE LOG

Borehole No.
2
1/1

Client: MONTEFIORE
Project: PROPOSED AGED CARE AND RESIDENTIAL LIVING DEVELOPMENT
Location: MASADA COLLEGE, COLLEGE ROAD, ST IVES, NSW

Job No.: 37393PN **Method:** SPIRAL AUGER **R.L. Surface:** 157.50m
Date: 25/2/25 **Datum:** AHD
Plant Type: JK305 **Logged/Checked by:** J.F./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0			FILL: Silty gravelly sand, fine to medium grained, dark grey and grey, trace of clay nodules.	M			
					N = 8 2,4,4			CH	Silty CLAY: high plasticity, light grey mottled orange brown and red brown.	w>PL	VSt	250 220 270	RESIDUAL
						1				w≈PL	Hd		
					N = 12 5,5,7							>600 >600 >600	
						2			END OF BOREHOLE AT 1.95m				
						3							
						4							
						5							
						6							
						7							

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BOREHOLE LOG

Borehole No.
3
1/1

Client: MONTEFIORE
Project: PROPOSED AGED CARE AND RESIDENTIAL LIVING DEVELOPMENT
Location: MASADA COLLEGE, COLLEGE ROAD, ST IVES, NSW

Job No.: 37393PN **Method:** SPIRAL AUGER **R.L. Surface:** 155.55m
Date: 25/2/25 **Datum:** AHD
Plant Type: JK305 **Logged/Checked by:** J.F./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION	■					0			FILL: Silty clay, low to medium plasticity, dark brown, trace of root fibres.	w≈PL			GRASS COVER
	■			■	N > 19 7,9, 10/100mm REFUSAL			CH	Silty CLAY: high plasticity, light grey mottled red brown, trace of fine to medium grained ironstone gravel.	w≈PL	Hd	>600 >600 >600	RESIDUAL
				■		1		-	SILTSTONE: dark grey.	DW	M-H		ASHFIELD SHALE MODERATE TO HIGH 'TC' BIT RESISTANCE
									END OF BOREHOLE AT 1.5m				'TC' BIT REFUSAL
						2							
						3							
						4							
						5							
						6							
						7							

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BOREHOLE LOG

Borehole No. 4

1/1

Client: MONTEFIORE

Project: PROPOSED AGED CARE AND RESIDENTIAL LIVING DEVELOPMENT

Location: MASADA COLLEGE, COLLEGE ROAD, ST IVES, NSW

Job No.: 37393PN **Method:** SPIRAL AUGER **R.L. Surface:** 155.02m

Date: 24/2/25 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** J.F./N.E.S.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION			0			FILL: Silty clay, low to medium plasticity, dark brown, trace of root fibres.	w≈PL			GRASS COVER APPEARS WELL COMPACTED
		N = 16 4,6,10				as above, but dark brown mottled red brown.	w>PL			
			1							
		N = 14 6,6,8								
			2							
					CH	Silty CLAY: high plasticity, light grey mottled orange brown and red brown.	w>PL	VSt		RESIDUAL
ON 11/3/25		N = 14 4,6,8	3						390 340 360	GROUNDWATER MONITORING WELL INSTALLED TO 6.1m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 3.1m TO 6.1m. CASING 0.1m TO 3.1m. 2mm SAND FILTER PACK 3.3m TO 6.1m. BENTONITE SEAL 2.6m TO 3.3m. BACKFILLED WITH SAND AND CUTTINGS TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.
						as above, but light grey mottled red brown.	w≈PL	Hd	>600 >600 >600	
			4			Silty CLAY: high plasticity, light grey mottled red brown, with extremely weathered fabric.				
		N > 30 7,15, 15/100mm REFUSAL	5						>600 >600 >600 >600	
					-	SILTSTONE: dark grey, with extremely weathered and iron indurated bands.	DW	L		ASHFIELD SHALE LOW 'TC' BIT RESISTANCE WITH MODERATE TO HIGH BANDS
			6					M-H		MODERATE TO HIGH RESISTANCE
						END OF BOREHOLE AT 6.1m				'TC' BIT REFUSAL
			7							

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BOREHOLE LOG

Borehole No. 5

Client: MONTEFIORE
Project: PROPOSED AGED CARE AND RESIDENTIAL LIVING DEVELOPMENT
Location: MASADA COLLEGE, COLLEGE ROAD, ST IVES, NSW

Job No.: 37393PN **Method:** SPIRAL AUGER **R.L. Surface:** 155.61m
Date: 24/2/25 **Datum:** AHD
Plant Type: JK305 **Logged/Checked by:** J.F./N.E.S.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0		FILL: Silty sand, fine to medium grained, dark brown, trace of root fibres.	M			GRASS COVER
	N = 15 4,7,8			FILL: Silty clay, low to medium plasticity, dark grey brown, trace of root fibres.	w>PL			APPEARS WELL COMPACTED
	N = 10 4,4,6	1	CH	Silty CLAY: high plasticity, orange brown mottled red brown.	w<PL	Hd	>600 >600 >600	RESIDUAL
		2		Silty CLAY: high plasticity, light grey mottled red brown.				
	N > 18 9,18/ 150mm REFUSAL	3					>600 >600 >600	
		4	-	SILTSTONE: dark grey, with extremely weathered bands.	DW	L		ASHFIELD SHALE LOW 'TC' BIT RESISTANCE
ON COMPLETION		5				L-M		LOW TO MODERATE RESISTANCE NO SAMPLE RETURN DUE TO SEEPAGE
		6		as above, but with iron indurated bands.		(M)		MODERATE RESISTANCE
		7						

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BOREHOLE LOG

Borehole No. 5

Client: MONTEFIORE

Project: PROPOSED AGED CARE AND RESIDENTIAL LIVING DEVELOPMENT

Location: MASADA COLLEGE, COLLEGE ROAD, ST IVES, NSW

Job No.: 37393PN **Method:** SPIRAL AUGER **R.L. Surface:** 155.61m

Date: 24/2/25 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** J.F./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
									SILTSTONE: dark grey, with extremely weathered bands and iron indurated bands.	DW	(M)		
											M-H		MODERATE TO HIGH RESISTANCE
									END OF BOREHOLE AT 7.7m				'TC' BIT REFUSAL
						8							
						9							
						10							
						11							
						12							
						13							
						14							

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BOREHOLE LOG

Borehole No. **6**

1/2

Client: MONTEFIORE
Project: PROPOSED AGED CARE AND RESIDENTIAL LIVING DEVELOPMENT
Location: MASADA COLLEGE, COLLEGE ROAD, ST IVES, NSW

Job No.: 37393PN **Method:** SPIRAL AUGER **R.L. Surface:** 156.16m
Date: 25/2/25 **Datum:** AHD
Plant Type: JK305 **Logged/Checked by:** J.F./N.E.S.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION		0		FILL: Silty clay, medium to high plasticity, dark brown, trace of root fibres.	w<PL			
	N = 23 6,11,12		CH	Silty CLAY: high plasticity, light grey mottled red brown.	w≈PL	Hd	>600 >600 >600	RESIDUAL
	N > 13 14,13/ 100mm REFUSAL		-	Extremely Weathered siltstone: silty CLAY, high plasticity, light grey and grey.	XW	Hd	>600 >600 >600	ASHFIELD SHALE
		2		SILTSTONE: dark grey.	DW	VL-L		VERY LOW TO LOW 'TC' BIT RESISTANCE
ON 11/3/25		3						
		4				L-M		LOW TO MODERATE RESISTANCE
						M-H		MODERATE TO HIGH RESISTANCE
		5		END OF BOREHOLE AT 4.6m				'TC' BIT REFUSAL GROUNDWATER MONITORING WELL INSTALLED TO 4.6m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 3.1m TO 4.6m. CASING 0.1m TO 3.1m. 2mm SAND FILTER PACK 3.0m TO 4.6m. BENTONITE SEAL 2.5m TO 3.0m. BACKFILLED WITH SAND AND CUTTINGS TO THE SURFACE.
		6						
		7						

Samples columns: ES, U50, DB, DS

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BOREHOLE LOG

Borehole No.
6
2/2

Client: MONTEFIORE
Project: PROPOSED AGED CARE AND RESIDENTIAL LIVING DEVELOPMENT
Location: MASADA COLLEGE, COLLEGE ROAD, ST IVES, NSW

Job No.: 37393PN **Method:** SPIRAL AUGER **R.L. Surface:** 156.16m
Date: 25/2/25 **Datum:** AHD
Plant Type: JK305 **Logged/Checked by:** J.F./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
													COMPLETED WITH A CONCRETED GATIC COVER.
						8							
						9							
						10							
						11							
						12							
						13							
						14							

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BOREHOLE LOG

Borehole No. 7

1/1

SDUP2: 0-0.3m

Client: MONTEFIORE

Project: PROPOSED AGED CARE AND RESIDENTIAL LIVING DEVELOPMENT

Location: MASADA COLLEGE, COLLEGE ROAD, ST IVES, NSW

Job No.: 37393PN **Method:** SPIRAL AUGER **R.L. Surface:** 154.08m

Date: 24/2/25 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** J.F./N.E.S.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			0			FILL: Silty clay, medium to high plasticity, dark brown, trace of root fibres.	w≈PL			GRASS COVER
		N = 19 5,9,10			CH	Silty CLAY: high plasticity, dark grey mottled red brown.	w>PL	VSt- Hd	320 390 440	RESIDUAL
			1			as above, but with extremely weathered fabric.	w≈PL			
		N = 26 9,11,15							>600 >600 >600	
			2		-	Extremely Weathered siltstone: silty CLAY, high plasticity, dark grey and light grey, with iron indurated bands.	XW	Hd	>600 >600 >600	ASHFIELD SHALE
						SILTSTONE: grey and dark grey, with iron indurated bands.	DW	VL-L		VERY LOW TO LOW 'TC' BIT RESISTANCE
			3			SILTSTONE: dark grey and grey, with extremely weathered and iron indurated bands.		L		LOW RESISTANCE WITH MODERATE BANDS
ON COMPLET-ION			4							
			5					M		MODERATE RESISTANCE
			6			END OF BOREHOLE AT 6.1m				'TC' BIT REFUSAL
			7							

JKGeotechnics

BOREHOLE LOG

Borehole No. **8**

1/1

Client: MONTEFIORE
Project: PROPOSED AGED CARE AND RESIDENTIAL LIVING DEVELOPMENT
Location: MASADA COLLEGE, COLLEGE ROAD, ST IVES, NSW

Job No.: 37393PN **Method:** SPIRAL AUGER **R.L. Surface:** 155.84m
Date: 25/2/25 **Datum:** AHD
Plant Type: JK305 **Logged/Checked by:** J.F./N.E.S.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			0			FILL: Silty clay, medium plasticity, dark brown, trace of root fibres.	w≈PL			GRASS COVER
		N = 16 5,7,9			CH	Silty CLAY: high plasticity, light grey mottled orange brown and red brown.	w≈PL	Hd	450 460 400	RESIDUAL
		N = 27 10,13,14	1			as above, but with extremely weathered fabric.			>600 >600 >600	
			2		-	SILTSTONE: dark grey, with extremely weathered and iron indurated bands.	DW	VL		ASHFIELD SHALE VERY LOW 'TC' BIT RESISTANCE WITH LOW BANDS VERY LOW RESISTANCE WITH MODERATE BANDS
ON COMPLETION			6					M		HIGH RESISTANCE
						END OF BOREHOLE AT 6.3m				'TC' BIT REFUSAL
			7							

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BOREHOLE LOG

Borehole No. 9

Client: MONTEFIORE

Project: PROPOSED AGED CARE AND RESIDENTIAL LIVING DEVELOPMENT

Location: MASADA COLLEGE, COLLEGE ROAD, ST IVES, NSW

Job No.: 37393PN **Method:** SPIRAL AUGER **R.L. Surface:** 156.53m

Date: 25/2/25 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** J.F./N.E.S.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION			0			FILL: Silty clay, low to medium plasticity, dark brown, trace of fine to medium grained ironstone gravel.	w<PL			GRASS COVER
		N > 7 16,7/50mm REFUSAL			-	Extremely Weathered siltstone: silty CLAY, high plasticity, light grey and grey.	XW	Hd		ASHFIELD SHALE
			1			SILTSTONE: dark grey, with iron indurated bands.	DW	M-H		MODERATE TO HIGH 'TC' BIT RESISTANCE HIGH RESISTANCE
						END OF BOREHOLE AT 1.4m				'TC' BIT REFUSAL
			2							
			3							
			4							
			5							
			6							
			7							

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BOREHOLE LOG

Borehole No.
10
1/1
SDUP1: 0-0.1m

Client: MONTEFIORE
Project: PROPOSED AGED CARE AND RESIDENTIAL LIVING DEVELOPMENT
Location: MASADA COLLEGE, COLLEGE ROAD, ST IVES, NSW

Job No.: 37393PN **Method:** SPIRAL AUGER **R.L. Surface:** 157.78m
Date: 24/2/25 **Datum:** AHD
Plant Type: JK305 **Logged/Checked by:** J.F./N.E.S.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION			0			FILL: Silty clay, low to medium plasticity, dark grey brown, trace of root fibres.	w<PL			GRASS COVER
		N = 19 5,9,10			CH	Silty CLAY: high plasticity, light grey mottled red brown.	w<PL	Hd	>600 >600 >600	RESIDUAL
			1							
		N = 29 10,13,16				as above, but with extremely weathered fabric.			>600 >600 >600	
			2		-	SILTSTONE: dark grey, with iron indurated bands.	DW	L-M		ASHFIELD SHALE LOW TO MODERATE 'TC' BIT RESISTANCE
			3					M-H		MODERATE TO HIGH RESISTANCE WITH VERY HIGH BANDS
			4			END OF BOREHOLE AT 3.8m				'TC' BIT REFUSAL
			5							
			6							
			7							

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BOREHOLE LOG

Borehole No. **11**

Client: MONTEFIORE

Project: PROPOSED AGED CARE AND RESIDENTIAL LIVING DEVELOPMENT

Location: MASADA COLLEGE, COLLEGE ROAD, ST IVES, NSW

Job No.: 37393PN **Method:** SPIRAL AUGER **R.L. Surface:** 157.05m

Date: 24/2/25 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** J.F./N.E.S.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0		FILL: Silty clay, medium to high plasticity, dark grey brown, trace of root fibres.	w<PL			GRASS COVER
	N = 23 6,10,13		CH	Silty CLAY: high plasticity, light grey mottled orange brown.	w≈PL	Hd	>600 >600 >600	RESIDUAL
		1		as above, but with extremely weathered fabric.				
	N = 42 10,18,24		-	Extremely Weathered siltstone: silty CLAY, high plasticity, light grey and dark grey, with iron indurated bands.	XW	Hd	>600 >600 >600	ASHFIELD SHALE
		2		SILTSTONE: dark grey, with extremely weathered and iron indurated bands.	DW	VL-L		VERY LOW TO LOW 'TC' BIT RESISTANCE
ON 11/3/25		3		SILTSTONE: dark grey, with iron indurated bands.		L-M		LOW TO MODERATE RESISTNCE
		4				M-H		MODERATE TO HIGH RESISTANCE
ON COMPLETION		5 / 6				H		HIGH RESISTANCE
		7		END OF BOREHOLE AT 6.2m				'TC' BIT REFUSAL GROUNDWATER MONITORING WELL INSTALLED TO 6.2m.

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BOREHOLE LOG

Borehole No. 11

Client: MONTEFIORE

Project: PROPOSED AGED CARE AND RESIDENTIAL LIVING DEVELOPMENT

Location: MASADA COLLEGE, COLLEGE ROAD, ST IVES, NSW

Job No.: 37393PN **Method:** SPIRAL AUGER **R.L. Surface:** 157.05m

Date: 24/2/25 **Datum:** AHD

Plant Type: JK305 **Logged/Checked by:** J.F./N.E.S.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
													CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 3.1m TO 6.2m. CASING 0.1m TO 3.1m. 2mm SAND FILTER PACK 3.0m TO 6.2m. BENTONITE SEAL 2.6m TO 3.0m. BACKFILLED WITH SAND AND CUTTINGS TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.
						8							
						9							
						10							
						11							
						12							
						13							
						14							

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BOREHOLE LOG

Borehole No.
101

SDUP109: 0-0.1m

Client: THE SIR MOSES MONTEFIORE JEWISH HOME
Project: PROPOSED INDEPENDENT LIVING AND AGED CARE DEVELOPMENT
Location: 25 COLLEGE CRESENT, ST IVES, NSW

Job No.: 37393PN **Method:** SPIRAL AUGER **R.L. Surface:** ~155.2 m
Date: 19/1/26 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** A.R./J.L.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING		0		FILL: Silty clay, medium to high plasticity, brown, with roots, trace of fine to medium grained gravel.	w>PL			GRASS COVER
	N = 15 7,8,7		CH	Silty CLAY: high plasticity, light orange brown and light grey, trace of fine grained sub-rounded sandstone gravel.	w>PL	VSt	380 330	SCREEN: 11.9kg, 0-0.1m, NO FCF; SCREEN: 10.88kg, 0.1-0.2m, NO FCF; RESIDUAL
	N = 24 10,11,13			Silty CLAY: high plasticity, grey.	w~PL	Hd	480 550	
				as above, but with extremely weathered siltstone bands.				
			-	Extremely Weathered siltstone: silty CLAY, high plasticity, dark grey and grey.	XW	Hd		ASHFIELD SHALE
				SILTSTONE: grey.	DW	VL - L		VERY LOW 'TC' BIT RESISTANCE
								MODERATE BANDS
								HIGH BANDS
				REFER TO CORED BOREHOLE LOG				

RL (m AHD): 155, 154, 153, 152, 151, 150, 149
Depth (m): 1, 2, 3, 4, 5, 6

SAMPLES: ES, U50, DB, DS

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 37393PN STIVES.GPJ <<DrawingFile>> 10/03/2026 17:18 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

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CORED BOREHOLE LOG

Borehole No.
101

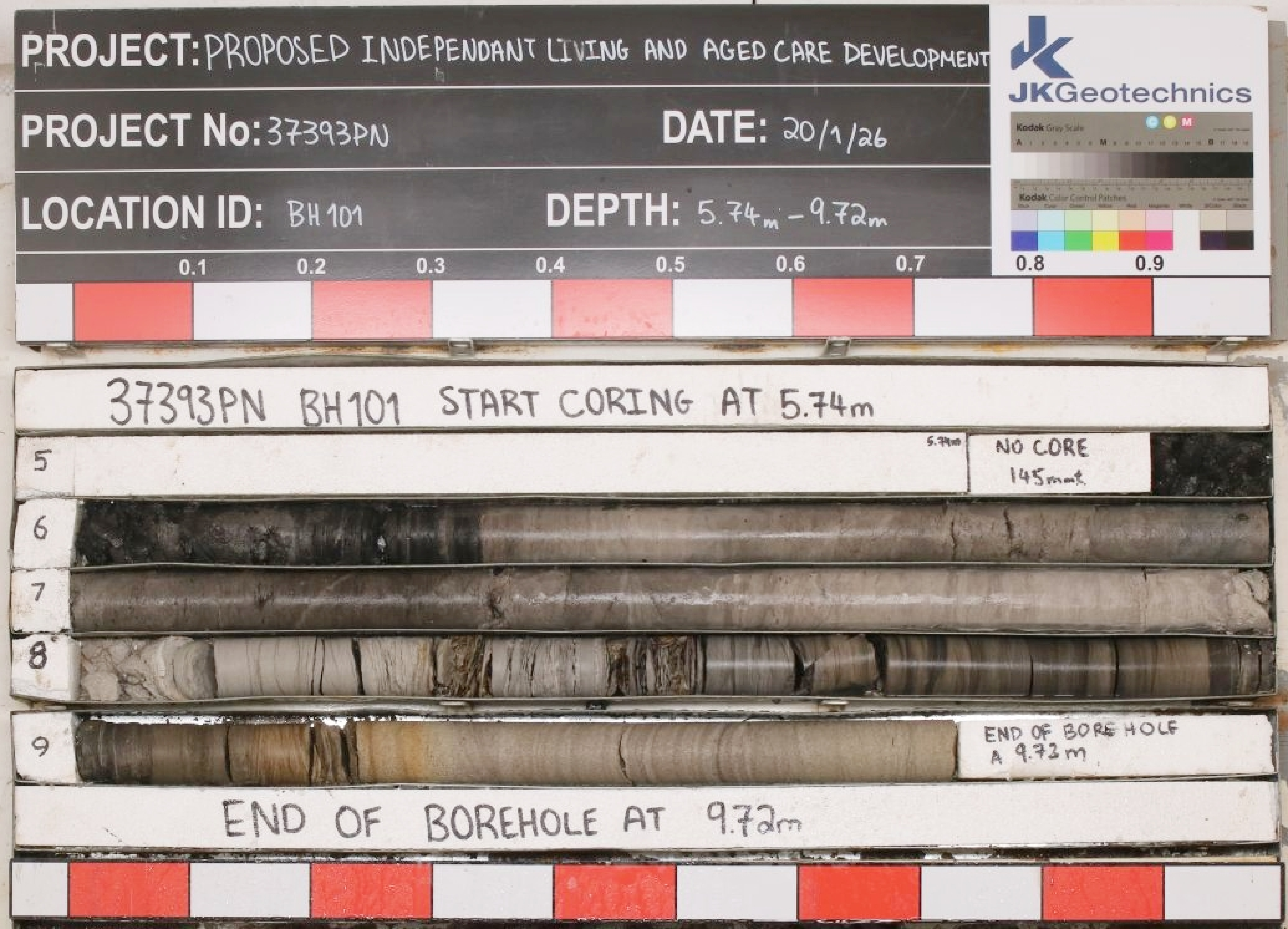
SDUP109: 0-0.1m

Client: THE SIR MOSES MONTEFIORE JEWISH HOME
Project: PROPOSED INDEPENDENT LIVING AND AGED CARE DEVELOPMENT
Location: 25 COLLEGE CRESENT, ST IVES, NSW

Job No.: 37393PN | **Core Size:** NMLC | **R.L. Surface:** ~155.2 m
Date: 19/1/26 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK305 | **Bearing:** N/A | **Logged/Checked By:** A.R./J.L.

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 $I_s(50)$ (VL -0.1, L -0.3, M -1, H -3, VH -10, EH)	DEFECT DETAILS: SPACING (mm) (600, 200, 60, 20)	DEFECT DETAILS: DESCRIPTION: Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness (Specific / General)	Formation
		150			START CORING AT 5.74m						
					NO CORE 0.16m					(5.90m) Cr, 0°, 200 mm.t	
100% RETURN		149	6		SILTSTONE: dark grey and grey, bedded at 0-10°.	HW	VL			(6.11m) CS, 100 mm.t (6.25m) Be, 0°, P, S, Cn	Ashfield Shale
		148	7		Extremely Weathered siltstone: silty CLAY, medium to high plasticity, light grey.	XW	Hd				Ashfield Shale
		147	8		SANDSTONE: fine grained, light grey, with extremely weathered bands, bedded at 0-10°.	MW	L - M	0.20 0.60 0.90 1.3		(8.09m) CS, 0°, 10 mm.t (8.11m) Be, 0°, P, R, Cn (8.14m) Be, 0°, P, R, Cn (8.15m) Be, 0°, P, Clay Vn, 1 mm.t (8.20m) Be x 2, 0°, P, R, Cn (8.23m) Be, 0°, P, R, Cn (8.30m) Be, 0°, Ir, Fe Sn (8.35-8.57m) Be, 0 - 10°, P, R, Cn, & Fe, Sn, 3-10mm.t (8.62m) J, 65°, Ca Cn	Hawkesbury Sandstone
					SANDSTONE: fine grained, light grey, dark grey and brown, with siltstone laminae, bedded at 0-5°.	MW	H	1.3			Hawkesbury Sandstone
			9				M	0.30 1.8		(9.15m) J, Fe Ct	Hawkesbury Sandstone
		146			SANDSTONE: fine to medium grained, light grey and yellow brown.	SW	M - H	0.70		(9.20m) Be x 3, 0 - 10°, P, R, Fe Sn	Hawkesbury Sandstone
		145	10		END OF BOREHOLE AT 9.72 m						
		144	11								

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


PROJECT: PROPOSED INDEPENDANT LIVING AND AGED CARE DEVELOPMENT
JKGeotechnics
PROJECT No: 37393PN
DATE: 20/1/26
Kodak Gray Scale
Kodak Color Control Patches
LOCATION ID: BH 101
DEPTH: 5.74m - 9.72m
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
37393PN BH101 START CORING AT 5.74m
5
5.74m
NO CORE
145mm
6
7
8
9
END OF BORE HOLE
A 9.72m
END OF BOREHOLE AT 9.72m

JKGeotechnics

BOREHOLE LOG

Borehole No.
102

SDUP110: 0-0.1m

Client: THE SIR MOSES MONTEFIORE JEWISH HOME
Project: PROPOSED INDEPENDENT LIVING AND AGED CARE DEVELOPMENT
Location: 25 COLLEGE CRESENT, ST IVES, NSW

Job No.: 37393PN **Method:** SPIRAL AUGER **R.L. Surface:** ~152.2 m
Date: 19/1/26 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** A.R./J.L.

Groundwater Record	Field Tests	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING			FILL: Silty clay, low plasticity, brown, with roots and root fibres.	w~PL			GRASS COVER, TOP 100mm ROOT AFFECTED SCREEN: 10.86kg 0-0.1m, NO FCF
ON 30/1/26	N = 11 4,4,7	CH	Silty CLAY: high plasticity, light grey brown mottled orange brown and red brown.	w~PL	VSt	380 400	SCREEN: 10.91kg 0.1-0.5m, NO FCF RESIDUAL
ON 20/2/26	N = 14 5,7,7		Silty CLAY: high plasticity, grey and orange brown, with iron indurated bands.		VSt - Hd	400 500	
			Silty CLAY: high plasticity, light grey mottled red brown.				
		-	SILTSTONE: grey, with iron indurated bands.	DW	(VL)		ASHFIELD SHALE
			SANDSTONE: fine to coarse grained, light brown and light grey brown, with iron indurated bands.		L M - H		HAWKESBURY SANDSTONE LOW 'TC' BIT RESISTANCE WITH MODERATE TO HIGH BANDS MODERATE TO HIGH RESISTANCE
			REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 8.59m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 2.0m TO 8.59m. CASING 0m TO 2.0m. 2mm SAND FILTER PACK 1.5m TO 8.59m. BENTONITE SEAL 0.15m TO 1.5m. COMPLETED WITH A CONCRETED GATIC COVER.

SAMPLES: ES, U50, DB, DS

RL (m AHD): 152, 151, 150, 149, 148, 147, 146

Depth (m): 1, 2, 3, 4, 5, 6

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
102
2 / 2

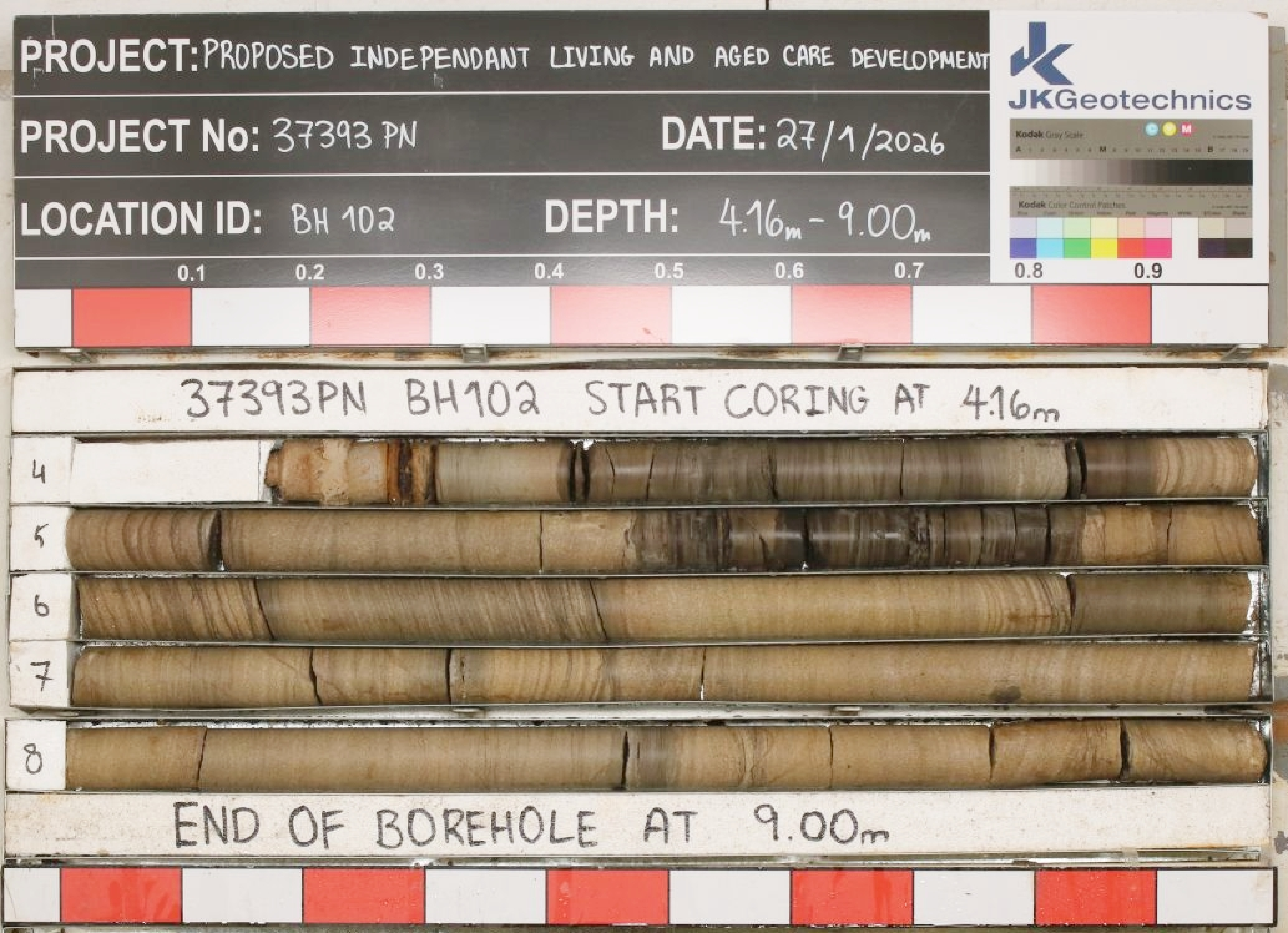
SDUP110: 0-0.1m

Client:	THE SIR MOSES MONTEFIORE JEWISH HOME
Project:	PROPOSED INDEPENDENT LIVING AND AGED CARE DEVELOPMENT
Location:	25 COLLEGE CRESENT, ST IVES, NSW

Job No.: 37393PN	**Core Size:** NMLC	**R.L. Surface:** ~152.2 m
Date: 19/1/26	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK305	**Bearing:** N/A	**Logged/Checked By:** A.R./J.L.

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 $I_s(50)$ (VL -0.1, L -0.3, M -1, H -3, VH -10, EH)	DEFECT DETAILS: SPACING (mm) (600, 200, 60, 20)	DEFECT DETAILS: DESCRIPTION: Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness (Specific / General)	Formation
					START CORING AT 4.16m						
100% RETURN		148, 147, 146, 145	5, 6, 7		SANDSTONE: fine to medium grained, light grey, grey and orange brown, with siltstone bands and laminations, bedded at 0-10°, with occasional cross-bedding to 20°.	MW / SW	L - M / H / M / H	1.7, 2.3, 1.1, 0.90, 1.8, 1.1, 1.0, 2.1		(4.22m) CS, 0°, 25 mm.t (4.25-4.31m) Be x 4, 0°, P, R, Fe Sn, & Clay FILLED, 2mm.t (4.53m) Vn, 0°, 1mm.t (4.59m) Be, 0°, P, R, Cn (4.84m) Be, 0°, Ir, R, Cn (4.85m) Be, 0°, P, R, Clay Ct (5.54m) Be, 0°, P, R, Fe Sn (5.56m) J, 50°, C, R, Fe Sn (5.69m) Cr, 0°, 15 mm.t (6.16m) Be, 15°, P, R, Fe Sn (6.45m) Be, 10°, P, R, Cn (6.83m) Be, 0°, P, R, Cn	Hawkesbury Sandstone
		144	8		SANDSTONE: fine to medium grained, light brown, bedded at 0-10°, with occasional cross-bedding to 20°.			1.4, 1.6, 1.2, 1.3	600 200 60 20	(8.43m) Be, 0°, P, R, Fe Sn	
		143, 142	9, 10		END OF BOREHOLE AT 9.00 m				600 200 60 20		

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


PROJECT: PROPOSED INDEPENDANT LIVING AND AGED CARE DEVELOPMENT
PROJECT No: 37393 PN
DATE: 27/1/2026
LOCATION ID: BH 102
DEPTH: 4.16m - 9.00m
JKGeotechnics
Kodak Gray Scale
Kodak Color Control Patches
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
37393PN BH102 START CORING AT 4.16m
4
5
6
7
8
END OF BOREHOLE AT 9.00m

JKGeotechnics

BOREHOLE LOG

Borehole No. 103

SDUP106: 0-0.1m

Client: THE SIR MOSES MONTEFIORE JEWISH HOME
Project: PROPOSED INDEPENDENT LIVING AND AGED CARE DEVELOPMENT
Location: 25 COLLEGE CRESENT, ST IVES, NSW

Job No.: 37393PN **Method:** SPIRAL AUGER **R.L. Surface:** ~155.4 m
Date: 16/1/26 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** A.R./J.L.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING		0–0.4		FILL: Silty clay, high plasticity, brown mottled grey brown, with root fibres.	w<PL			GRASS COVER SCREEN: 10.26kg, 0-0.1m, NO FCF
	N = 11 6,7,4	0.4–1.4		FILL: Silty clay, medium to high plasticity, brown, trace of root fibres.			>600 >600 >600	SCREEN: 10.6kg, 0.1-0.4m, NO FCF APPEARS MODERATELY TO WELL COMPACTED SCREEN: 4.71kg(<10L), 0.4-1.4m, NO FCF
	N = 19 9,8,11	1.4–2.2	CH	Silty CLAY: high plasticity, grey brown and brown.	w<PL	Hd	>600 >600 >600	RESIDUAL
		2.2–3.0		as above, but light grey and light red brown.				
	N = 25 9,9,16	3.0–4.2		Silty CLAY: high plasticity, grey mottled orange brown, with extremely weathered bands.			>600 >600 >600	
		4.2–5.0	-	Extremely Weathered siltstone: silty CLAY, high plasticity, grey mottled orange brown and red brown.	XW	Hd		ASHFIELD SHALE
		5.0–5.5		SILTSTONE: grey.	DW	L		LOW 'TC' BIT RESISTANCE
		5.5–5.8				M		MODERATE RESISTANCE
		5.8–		as above, but with iron indurated bands.		M - H		MODERATE TO HIGH RESISTANCE

JKGeotechnics

BOREHOLE LOG

Borehole No.
103

SDUP106: 0-0.1m

Client: THE SIR MOSES MONTEFIORE JEWISH HOME
Project: PROPOSED INDEPENDENT LIVING AND AGED CARE DEVELOPMENT
Location: 25 COLLEGE CRESENT, ST IVES, NSW

Job No.: 37393PN **Method:** SPIRAL AUGER **R.L. Surface:** ~155.4 m
Date: 16/1/26 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** A.R./J.L.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
									-	SILTSTONE: grey, with iron indurated bands.	DW	M - H		
						148				REFER TO CORED BOREHOLE LOG				
							8							
						147								
							9							
						146								
							10							
						145								
							11							
						144								
							12							
						143								
							13							
						142								

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CORED BOREHOLE LOG

Borehole No.
103
3 / 4

SDUP106: 0-0.1m

Client: THE SIR MOSES MONTEFIORE JEWISH HOME
Project: PROPOSED INDEPENDENT LIVING AND AGED CARE DEVELOPMENT
Location: 25 COLLEGE CRESENT, ST IVES, NSW

Job No.: 37393PN | **Core Size:** NMLC | **R.L. Surface:** ~155.4 m
Date: 16/1/26 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK305 | **Bearing:** N/A | **Logged/Checked By:** A.R./J.L.

START CORING AT 7.17m

Water Loss\Level	Depth (m)	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS – DESCRIPTION: Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness (Specific)	Formation
100% RETURN	7.17–7.40	SANDSTONE: fine grained, brown, bedded at 0-10°.	MW	H	1.4	(7.20m) Jh, 80°, C, Fe Sn; (7.36m) Be, 0°, P, R, Fe Sn	Ashfield Shale
	7.40–8.22	SILTSTONE: dark grey, trace of fine grained sandstone laminae, bedded at 0-5°		M	0.70, 0.60	(7.40m) Be, 0°, P, R, Fe Sn; (7.44m) CS, 0°, 20 mm.t; (7.61m) J, 40°, Un, R, Cn; (7.68m) CS, 0°, 50 mm.t; (7.74m) J, 30°, C, Cn; (7.87m) J, 40°, P & C, Cn; (8.18m) CS, 0°, 40 mm.t	Ashfield Shale
	8.22–12.10	SANDSTONE: fine to medium grained, light brown and yellow brown, bedded at 0-5°, with cross-bedding to 20°.	SW	H (8.22–10.12), M (10.12–10.48), H (10.48–11.09), M (11.09–11.70), H (11.70–)	0.90, 1.2, 1.4, 1.5, 0.80, 2.1, 0.90, 1.5	(8.42m) Be, 5°, P, Clay Vn; (10.12m) Be, 20°, P, R, Cn; (10.14m) Be, 20°, P, R, Cn	Hawkesbury Sandstone
	12.10–13.76	as above, but with carbonaceous laminations.		H	1.5, 1.0, 1.6, 2.1	(12.36m) Be, 10°, P, R, Cb Vn; (13.50m) Jh, 70°, C	Hawkesbury Sandstone
	13.76–	SANDSTONE: as below		M			Hawkesbury Sandstone

RL (m AHD): 148 … 142

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FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

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CORED BOREHOLE LOG

SDUP106: 0-0.1m

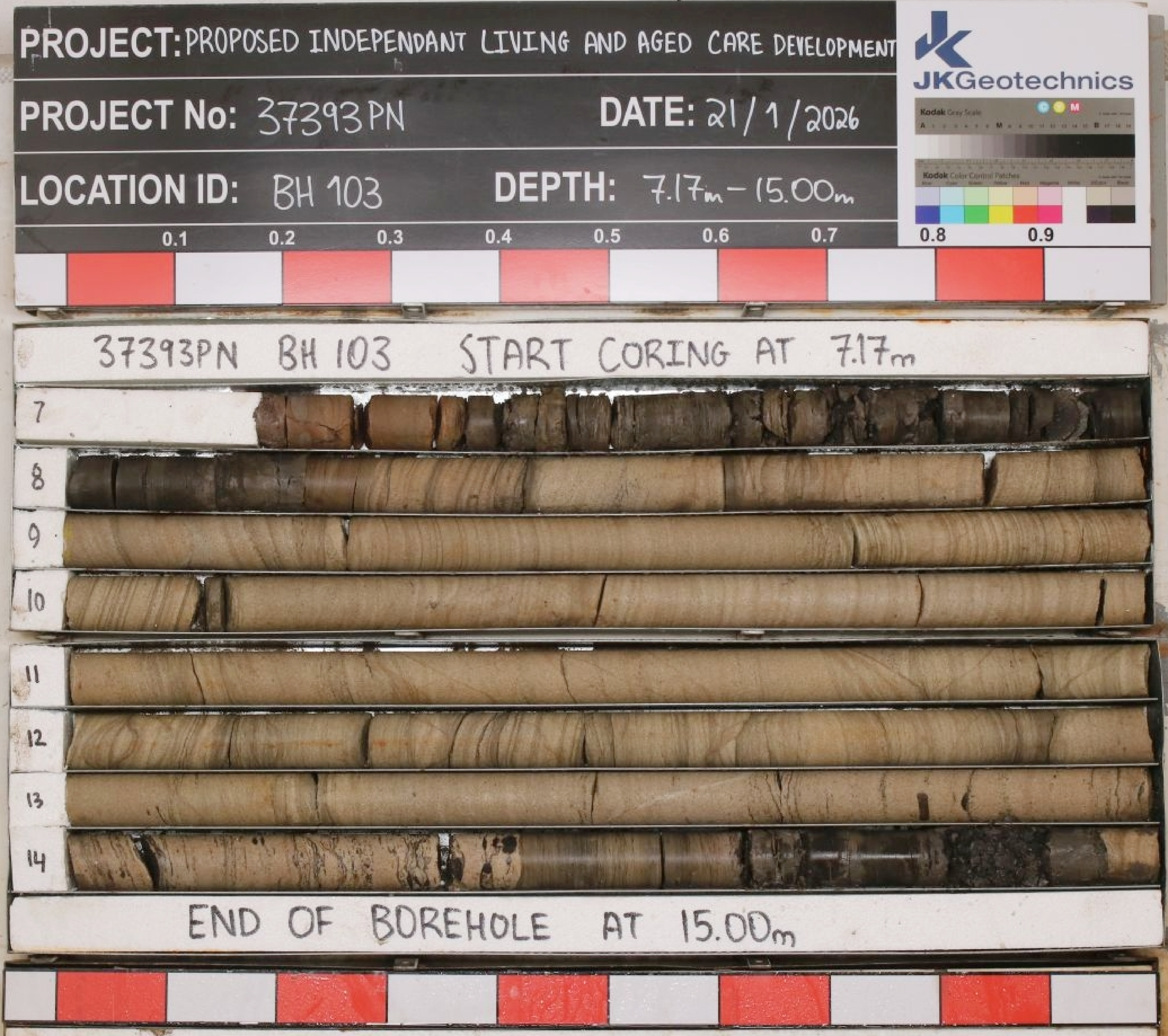
Client: THE SIR MOSES MONTEFIORE JEWISH HOME
Project: PROPOSED INDEPENDENT LIVING AND AGED CARE DEVELOPMENT
Location: 25 COLLEGE CRESENT, ST IVES, NSW

Job No.: 37393PN **Core Size:** NMLC **R.L. Surface:** ~155.4 m
Date: 16/1/26 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK305 **Bearing:** N/A **Logged/Checked By:** A.R./J.L.

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 $I_s(50)$	DEFECT DETAILS: Spacing (mm)	DEFECT DETAILS: Description (Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness) Specific / General	Formation
100% RETURN		141			SANDSTONE: fine to coarse grained, brown, with siltstone laminae and gravel inclusions up to 20mm, cross-bedded at 10-20°.	MW	M	0.70		(14.04m) Be, 15°, P, R, Clay Vn	
					SANDSTONE: fine grained, light brown, with carbonaceous laminations, bedded at 0-10°.	MW - HW	H / L - M	2.7		(14.78m) Cr, 0°, 120 mm.t	
					SILTSTONE: dark grey, bedded at 0-5°.					(14.90m) CS, 0°, 50 mm.t	
			15		SANDSTONE: fine to medium grained, brown.	MW	H				
		140			END OF BOREHOLE AT 15.00 m						
			16								
		139									
			17								
		138									
			18								
		137									
			19								
		136									
			20								
		135									

Strength scale: VL -0.1, L -0.3, M -1, H -3, VH -10, EH. Spacing scale: 600, 200, 60, 20.

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


PROJECT: PROPOSED INDEPENDANT LIVING AND AGED CARE DEVELOPMENT
JKGeotechnics
PROJECT No: 37393PN
DATE: 21/1/2026
LOCATION ID: BH 103
DEPTH: 7.17m – 15.00m
Kodak Gray Scale
Kodak Color Control Patches
0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9
37393PN BH 103 START CORING AT 7.17m
7
8
9
10
11
12
13
14
END OF BOREHOLE AT 15.00m

JKGeotechnics

BOREHOLE LOG

Client: THE SIR MOSES MONTEFIORE JEWISH HOME
Project: PROPOSED INDEPENDENT LIVING AND AGED CARE DEVELOPMENT
Location: 25 COLLEGE CRESENT, ST IVES, NSW

Job No.: 37393PN **Method:** SPIRAL AUGER **R.L. Surface:** ~156.0 m
Date: 19/1/26 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** A.R./J.L.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	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							FILL: Silty clay, medium to high plasticity, brown, with fine to medium grained sand, and root fibres.	w>PL			GRASS COVER SCREEN: 10.23kg, 0-0.1m, NO FCF
						CH	FILL: Silty clay, high plasticity, brown, with iron indurated bands.	w~PL	VSt		SCREEN: 10.69kg, 0.1-0.25m, NO FCF
		N = 16 7,7,9					Silty CLAY: high plasticity, light grey mottled light orange brown.	w~PL		380 390	RESIDUAL
			155	1		-	Extremely Weathered siltstone: silty CLAY, high plasticity, light grey, with iron indurated bands.	XW	Hd		ASHFIELD SHALE
			154	2			SILTSTONE: grey, with extremely weathered bands.	DW	VL - L		VERY LOW TO LOW 'TC' BIT RESISTANCE
ON 30/1/26 & 20/2/26			153	3							
			152	4			as above, but with iron indurated bands.		L - M		LOW TO MODERATE RESISTANCE
			151	5							
			150	6			REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 8.85m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 1.8m TO 8.85m. CASING 0m TO 1.8m. 2mm SAND FILTER PACK 1.0m TO 8.85m. BENTONITE SEAL 0m TO 1.0m. COMPLETED WITH A CONCRETED GATIC COVER.

JKGeotechnics

CORED BOREHOLE LOG

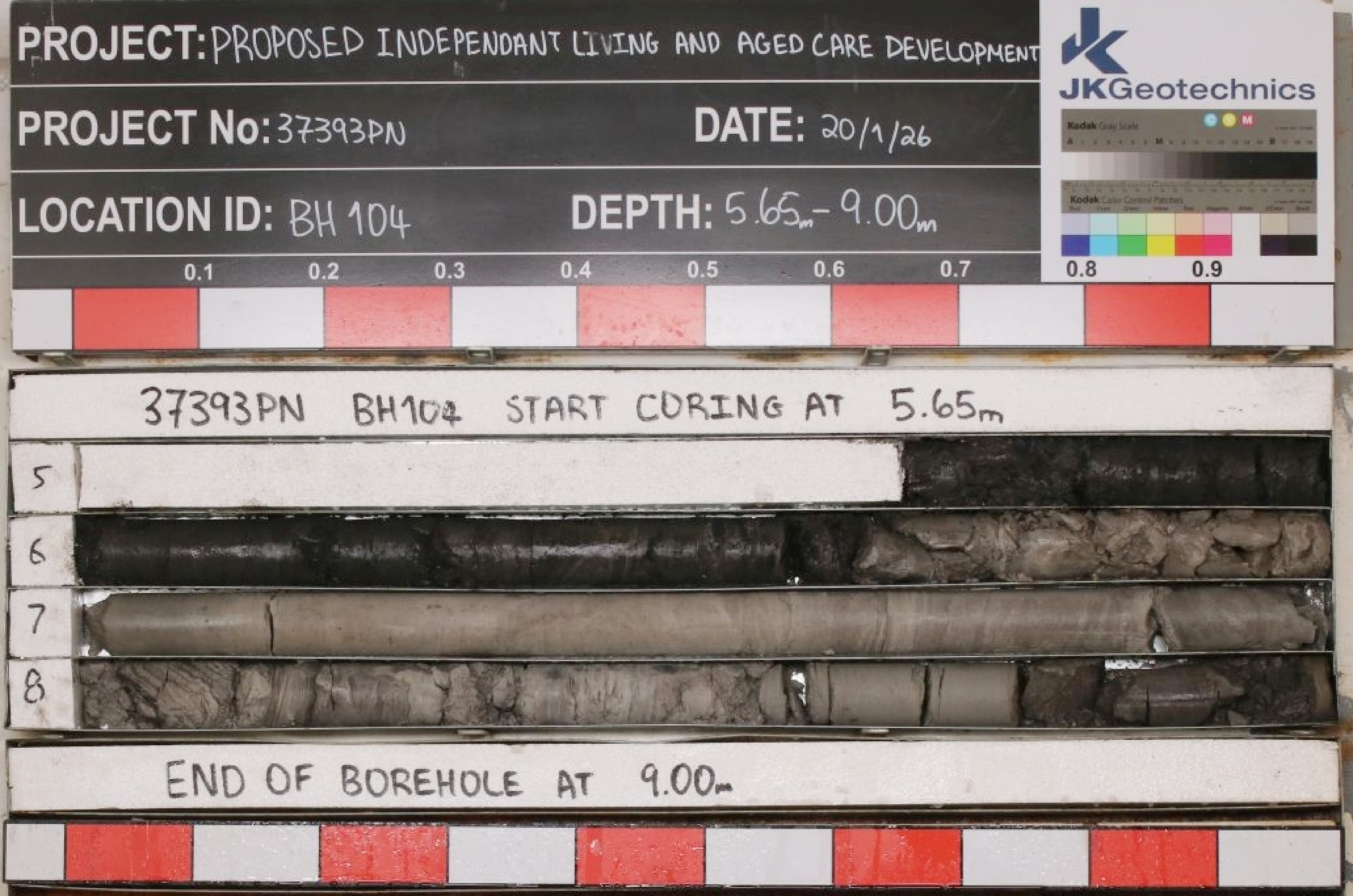
Borehole No.
104
2 / 2

Client: THE SIR MOSES MONTEFIORE JEWISH HOME
Project: PROPOSED INDEPENDENT LIVING AND AGED CARE DEVELOPMENT
Location: 25 COLLEGE CRESENT, ST IVES, NSW

Job No.: 37393PN	**Core Size:** NMLC	**R.L. Surface:** ~156.0 m
Date: 19/1/26	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK305	**Bearing:** N/A	**Logged/Checked By:** A.R./J.L.

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 $I_s(50)$ (VL -0.1, L -0.3, M -1, H -3, VH -10, EH)	DEFECT DETAILS: SPACING (mm) (600, 200, 60, 20)	DEFECT DETAILS: DESCRIPTION (Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness) Specific / General	Formation
					START CORING AT 5.65m						
100% RETURN		150	6		Extremely Weathered siltstone: silty CLAY, high plasticity, dark grey.	XW	Hd				Ashfield Shale
		149	7		SILTSTONE: light grey, bedded at 0°.	HW	VL - L	0.10			
		148	8		Extremely Weathered siltstone: silty CLAY, medium to high plasticity, light grey.	XW	Hd				
					SANDSTONE: fine grained, light grey.	SW	M	0.60			
		147	9		Extremely Weathered siltstone: silty CLAY, medium to high plasticity, dark grey.	XW	Hd				
					END OF BOREHOLE AT 9.00 m						
		146	10								
		145	11								

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


PROJECT: PROPOSED INDEPENDANT LIVING AND AGED CARE DEVELOPMENT
PROJECT No: 37393PN
DATE: 20/1/26
LOCATION ID: BH 104
DEPTH: 5.65m - 9.00m
JKGeotechnics
Kodak Gray Scale
Kodak Color Control Patches
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
37393PN BH104 START CORING AT 5.65m
5
6
7
8
END OF BOREHOLE AT 9.00m

JKGeotechnics

BOREHOLE LOG

Borehole No.
105
1 / 2

Client: THE SIR MOSES MONTEFIORE JEWISH HOME
Project: PROPOSED INDEPENDENT LIVING AND AGED CARE DEVELOPMENT
Location: 25 COLLEGE CRESENT, ST IVES, NSW

Job No.: 37393PN **Method:** SPIRAL AUGER **R.L. Surface:** ~155.4 m
Date: 20/1/26 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** A.R./J.L.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING		0		FILL: Silty clay, high plasticity, brown, with root fibres.	w~PL			GRASS COVER; SCREEN: 10.02kg 0-0.1m, NO FCF; SCREEN: 10.93kg 0.1-0.2m, NO FCF
	N = 14 6,7,7	0.2	CH	Silty CLAY: high plasticity, orange brown, with fine to medium grained ironstone gravel.	w~PL	Hd	>600	RESIDUAL
	N=SPT 12/ 150mm REFUSAL	1.1		Silty CLAY: high plasticity, grey mottled light orange brown, with fine to medium grained ironstone gravel, and extremely weathered bands.	w<PL		>600	
		2.0	-	SILTSTONE: light grey and dark grey.	DW	VL		ASHFIELD SHALE; VERY LOW 'TC' BIT RESISTANCE
		2.5				L		LOW RESISTANCE
		3.0		SILTSTONE: dark grey, with iron indurated bands.		L - M		LOW TO MODERATE RESISTANCE
		3.5				M		MODERATE RESISTANCE
		5.8		REFER TO CORED BOREHOLE LOG				

RL (m AHD): 155, 154, 153, 152, 151, 150, 149
SAMPLES: ES, U50, DB, DS

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
105

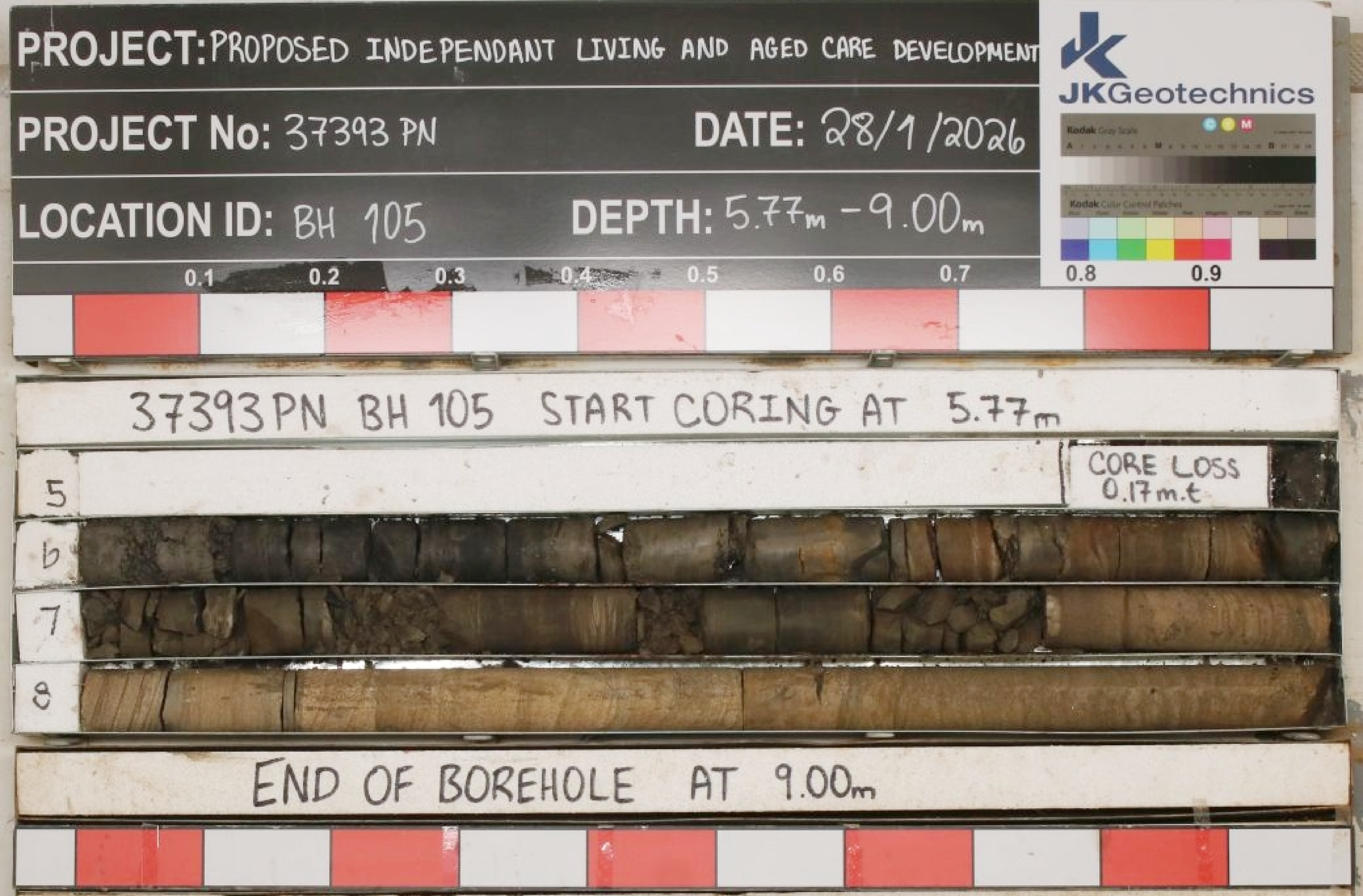
Client: THE SIR MOSES MONTEFIORE JEWISH HOME
Project: PROPOSED INDEPENDENT LIVING AND AGED CARE DEVELOPMENT
Location: 25 COLLEGE CRESENT, ST IVES, NSW

Job No.: 37393PN	**Core Size:** NMLC	**R.L. Surface:** ~155.4 m
Date: 20/1/26	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK305	**Bearing:** N/A	**Logged/Checked By:** A.R./J.L.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS – DESCRIPTION: Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness (Specific / General)	Formation
		150		START CORING AT 5.77m					
100% RETURN				NO CORE 0.17m					
			6	SILTSTONE: dark grey, bedded at 0-10°.	HW	L	0.20	(6.00m) CS, 0°, 130 mm.t (6.25m) J, 30°, C, Cn (6.30m) Be, 0°, P, Vn, 1 mm.t (6.41m) J, 40°, P, R, Cn	Ashfield Shale
		149		SANDSTONE: fine grained, grey and red brown, bedded at 0-10°.	MW	H	1.5 1.0 1.1	(6.53m) Be, 0°, P, Clay Ct (6.57m) Be, 5°, Ir, R, Fe Sn (6.58m) CS, 0°, 25 mm.t (6.60m) CS, 0°, 20 mm.t (6.62m) J, 60°, C, R, Cn (6.73m) Be, 10°, P, R, Fe Sn (6.94m) Be, 20°, C, R, Cn (6.98m) Jh, 45° (7.05m) J, 45°, C, R, Cn (7.07m) J, 90°, P, R, Cn (7.13m) J, 30°, C, R, Fe Sn (7.20m) CS, 0°, 80 mm.t (7.43m) CS, 0°, 70 mm.t (7.63m) J, 80 - 90°, Ir, R, Fe Sn	Hawkesbury Sandstone
		148	7						
		147	8	SANDSTONE: fine to medium grained, light grey, bedded at 0-10°, with occasional cross-bedding to 20°, trace of siltstone laminae.			1.6 1.0 1.2	(8.08m) Be, 0°, P (8.18m) Be, 0°, P (8.52m) Be, 0°, P	
		146	9	END OF BOREHOLE AT 9.00 m					
		145	10						
		144	11						

Point load scale: VL -0.1, L -0.3, M -1, H -3, VH -10, EH
SPACING (mm): 600, 200, 60, 20

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


PROJECT: PROPOSED INDEPENDANT LIVING AND AGED CARE DEVELOPMENT
JKGeotechnics
PROJECT No: 37393 PN
DATE: 28/1/2026
LOCATION ID: BH 105
DEPTH: 5.77m - 9.00m
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
37393PN BH 105 START CORING AT 5.77m
5
CORE LOSS 0.17m.t
6
7
8
END OF BOREHOLE AT 9.00m

JKGeotechnics

BOREHOLE LOG

Borehole No. **106**

SDUP105: 0-0.1m

Client: THE SIR MOSES MONTEFIORE JEWISH HOME
Project: PROPOSED INDEPENDENT LIVING AND AGED CARE DEVELOPMENT
Location: 25 COLLEGE CRESENT, ST IVES, NSW

Job No.: 37393PN **Method:** SPIRAL AUGER **R.L. Surface:** ~156.6 m
Date: 15/1/26 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** A.R./J.L.

Groundwater Record	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING			0		FILL: Silty clay, low to medium plasticity, brown, trace of fine grained sand.	w<PL			GRASS COVER
	N=SPT 20/ 100mm REFUSAL	156		CH	Silty CLAY: high plasticity, light grey and light red brown, with extremely weathered fabric.	w<PL	Hd		SCREEN: 10.23kg, 0-0.1m NO FCF SCREEN: 10.66kg 0.1-0.2m, NO FCF RESIDUAL
		155	1	-	SILTSTONE: grey and dark grey, with occasional iron indurated bands and extremely weathered bands.	DW	L - M		ASHFIELD SHALE LOW TO MODERATE 'TC' BIT RESISTANCE HIGH BANDS
		154, 153	2, 3		SILTSTONE: dark grey.		M		MODERATE TO HIGH RESISTANCE
		152, 151	4, 5						
		150	6		REFER TO CORED BOREHOLE LOG				

SAMPLES: ES, U50, DB, DS

JKGeotechnics

CORED BOREHOLE LOG

Borehole No. 106

2 / 2

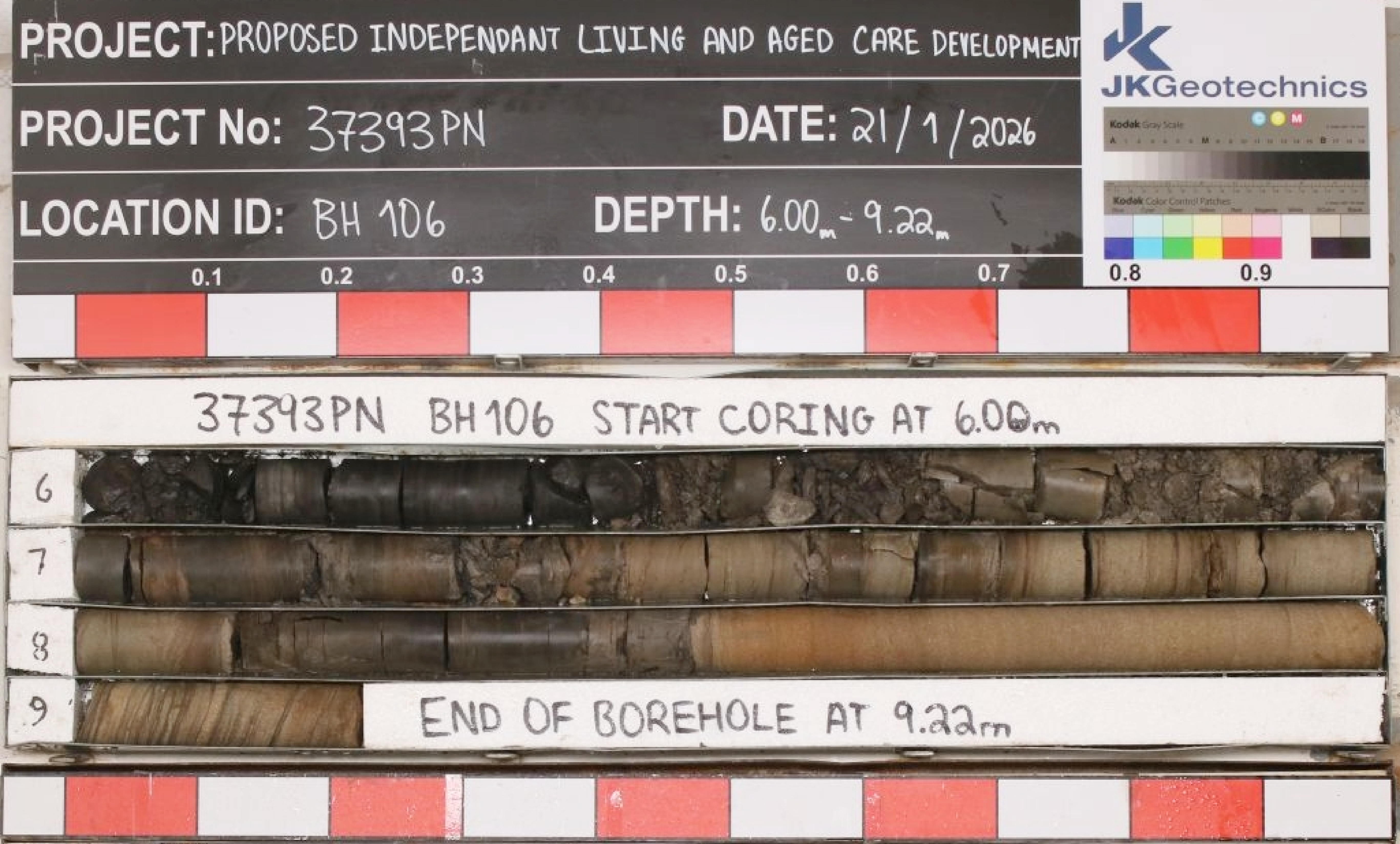
SDUP105: 0-0.1m

Client: THE SIR MOSES MONTEFIORE JEWISH HOME
Project: PROPOSED INDEPENDENT LIVING AND AGED CARE DEVELOPMENT
Location: 25 COLLEGE CRESENT, ST IVES, NSW

Job No.: 37393PN	**Core Size:** NMLC	**R.L. Surface:** ~156.6 m
Date: 15/1/26	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK305	**Bearing:** N/A	**Logged/Checked By:** A.R./J.L.

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 $I_s(50)$ (VL -0.1, L -0.3, M -1, H -3, VH -10, EH)	SPACING (mm) (600, 200, 60, 20)	DEFECT DETAILS – DESCRIPTION (Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness) Specific / General	Formation
		151			START CORING AT 6.00m						
100% RETURN		150	6 – 7		SILTSTONE: grey, with sandstone laminae, bedded at 0-5°.	HW	VL H VL	1.1		(6.00m) Cr, 0°, 140 mm.t (6.18m) J, 30°, Ir, R, Clay Ct (6.34m) J, 65°, P, R, Cn (6.44m) XWS, 0°, 60 mm.t (6.51m) Be, 0°, P, R, Cn (6.53m) XWS, 0°, P, 110 mm.t (6.63m) Ji, 60 - 90°, C (6.74m) Cr, 0°, 110 mm.t (6.86m) J, 70°, P, R, Cn (6.88m) Cr, 0°, 30 mm.t (6.98m) Be, 0°, P	Ashfield Shale
			7			MW	H	1.0		(7.19m) Be, 0°, P, Cn (7.28m) Cr, 0°, 60 mm.t (7.38m) Be, 10°, C, R, Cn	
		149			SANDSTONE: fine grained, light brown and brown, with siltstone bands, bedded at 0-10°.					(7.56m) CS, 0°, 20 mm.t (7.64m) Be, 0°, P, R, Cn (7.70m) Be, 0°, P, R, Cn	Hawkesbury Sandstone
			8		SILTSTONE: grey to dark grey, bedded at 0-5°.	SW				(8.16m) J, 90°, P, R, Cn (8.42m) CS, 5°, 30 mm.t	
		148	9		SANDSTONE: fine to medium grained, light brown and brown, bedded at 0°, with occasional cross-bedding to 25°.	MW		1.8 3.2 1.4		(8.48m) Be, 0°, P	
		147	10		END OF BOREHOLE AT 9.22 m						
		146	11								
		145									

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


PROJECT: PROPOSED INDEPENDANT LIVING AND AGED CARE DEVELOPMENT
JKGeotechnics
PROJECT No: 37393PN
DATE: 21/1/2026
LOCATION ID: BH 106
DEPTH: 6.00m - 9.22m
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
37393PN BH106 START CORING AT 6.00m
6
7
8
9
END OF BOREHOLE AT 9.22m

JKGeotechnics

BOREHOLE LOG

Borehole No.
107

SDUP101: 0-0.1m

Client: THE SIR MOSES MONTEFIORE JEWISH HOME
Project: PROPOSED INDEPENDENT LIVING AND AGED CARE DEVELOPMENT
Location: 25 COLLEGE CRESENT, ST IVES, NSW

Job No.: 37393PN **Method:** SPIRAL AUGER **R.L. Surface:** ~157.6 m
Date: 15/1/26 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** A.R./J.L.

Groundwater Record: DRY ON COMPLETION OF AUGERING; (4.2m) ON 30/1/26; (4.3m) ON 20/2/26

Field Tests	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
		FILL: Silty clay topsoil, low to medium plasticity, dark grey and brown, trace of fine grained sand, fine to medium grained sandstone gravel, tile fragments, slag and root fibres.	w<PL			GRASS COVER
						SCREEN: 10.11kg, 0-0.1m, NO FCF
N = 12 5,6,6	CH	Silty CLAY: high plasticity, grey mottled light red brown.	w>PL	VSt	250 280	SCREEN: 10.86kg, 0.1-0.15m, NO FCF
						RESIDUAL
N = 29 12,14,15		as above, but with extremely weathered fabric.	w<PL	Hd	>600	
	-	SILTSTONE: dark grey, with iron indurated bands.	DW	VL - L		ASHFIELD SHALE
						VERY LOW TO LOW 'TC' BIT RESISTANCE
				L		MODERATE BANDS
						HIGH BANDS
		REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 11.4m DEPTH. FOR DETAILS OF INSTALLATION, REFER TO CORED BOREHOLE LOG.

Depth (m)	RL (m AHD)
0	~157.6
1	156.6
2	155.6
3	154.6
4	153.6
5	152.6
6	151.6

RL axis ticks: 157, 156, 155, 154, 153, 152, 151

SAMPLES: ES, U50, DB, DS

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
107

SDUP101: 0-0.1m

Client: THE SIR MOSES MONTEFIORE JEWISH HOME
Project: PROPOSED INDEPENDENT LIVING AND AGED CARE DEVELOPMENT
Location: 25 COLLEGE CRESENT, ST IVES, NSW

Job No.: 37393PN | **Core Size:** NMLC | **R.L. Surface:** ~157.6 m
Date: 15/1/26 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK305 | **Bearing:** N/A | **Logged/Checked By:** A.R./J.L.

Column headings: 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 $I_s(50)$ (VL -0.1, L -0.3, M -1, H -3, VH -10, EH) | DEFECT DETAILS: SPACING (mm) (600, 200, 60, 20) | DEFECT DETAILS: DESCRIPTION (Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness) Specific / General | Formation

START CORING AT 6.06m

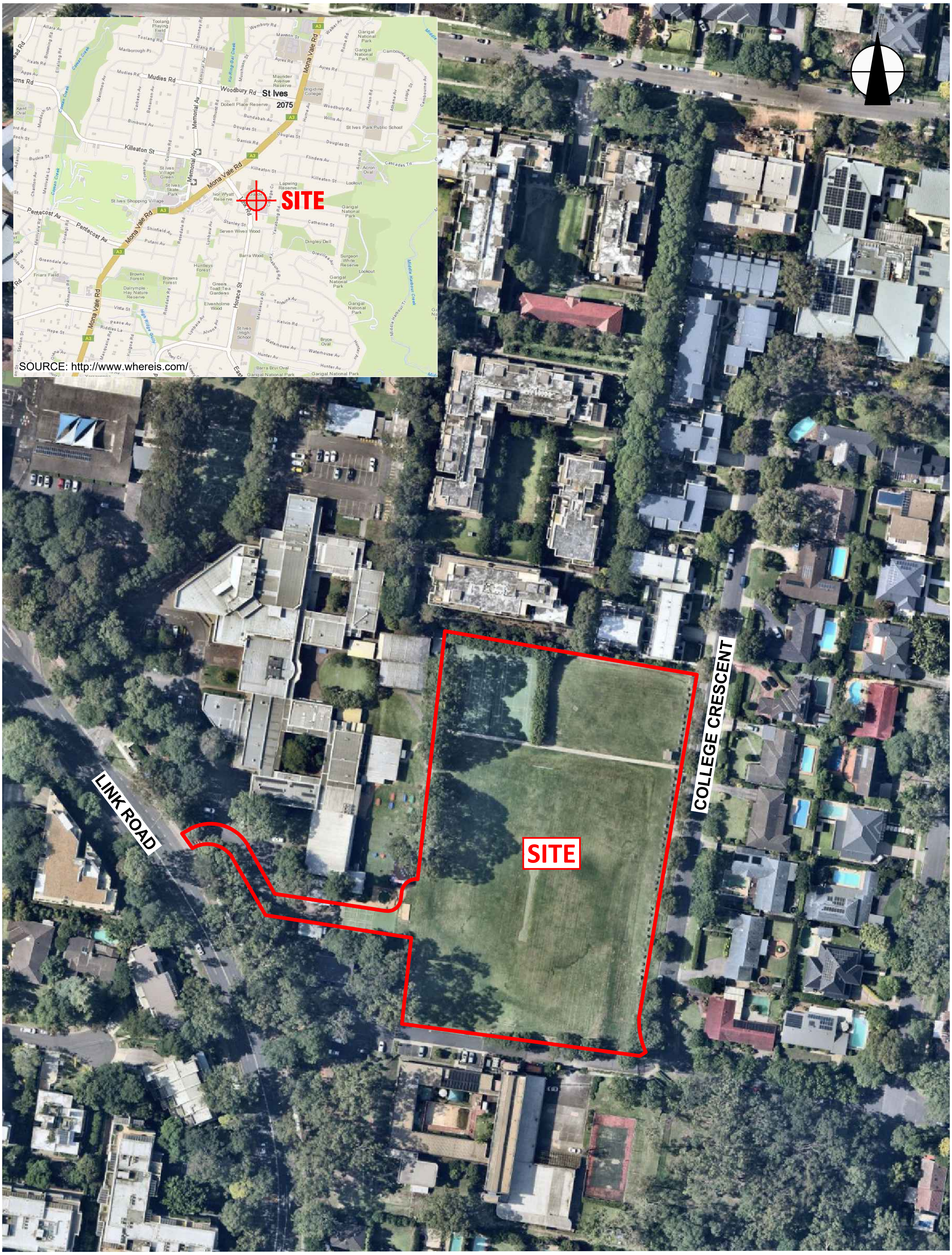
Water Loss\Level: 100% RETURN

RL (m AHD): 151, 150, 149, 148, 147, 146, 145
Depth (m): 7, 8, 9, 10, 11, 12

Core Description	Weathering	Strength	Point Load $I_s(50)$	Defects (Specific)	Formation
NO CORE 0.14m					
SILTSTONE: dark grey and grey brown, bedded at 0-15°.	HW	VL		(6.22m) Cr, 5°, 70 mm.t (6.33m) Be, 0°, P, R, Fe Sn (6.35m) Be, 15°, P, Fe Sn (6.36m) CS, 0°, 10 mm.t (6.39m) J, 30°, P-C, Cn (6.42m) Be, 0°, P, e, Fe Sn (6.47m) Be, 0°, P, Cn (6.50m) Be, 0°, P, R, Fe Sn (6.54m) CS, 0°, P, 20 mm.t (6.60m) Be, 0°, P, S-R, Cn (6.64m) J, 90°, P, R, Fe Sn (6.70m) Be, 5°, P, R, Fe Sn (6.72m) Cr, 0°, 100 mm.t (6.90m) Cr, 0°, 300 mm.t (7.23m) Cr, 0°, 70 mm.t (7.30m) XWS, 0°, 158 mm.t (7.34m) J, 40°, C, Fe Sn	Ashfield Shale
Extremely Weathered siltstone: silty CLAY, medium plasticity, grey, with low strength bands.	XW	Hd	0.20 0.020	(9.00m) J, 40°, Ir, R, Cn	Ashfield Shale
SANDSTONE: fine grained, light brown and grey brown, bedded at 0-10°.	MW	L	0.20 0.30 0.40		Hawkesbury Sandstone
SANDSTONE: fine to medium grained, light brown and brown, bedded at 0-5°, with occasional cross-bedding at 20°.		M - H	1.0 1.0 0.70 1.2 1.5	(9.86m) CS, 0°, 60 mm.t (10.34m) Be, 20°, P, R, Cn (10.42m) Be, 0°, P, Fe Sn (10.77m) Be, 20°, P, Clay, 3 mm.t	Hawkesbury Sandstone
END OF BOREHOLE AT 11.50 m				GROUNDWATER MONITORING WELL INSTALLED TO 11.4m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 2.0m TO 11.4m. CASING 0m TO 2.0m. 2mm SAND FILTER PACK 1.5m TO 11.4m. BENTONITE SEAL 0.2m TO 1.5m. BACKFILLED WITH SAND AND CUTTINGS TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.	

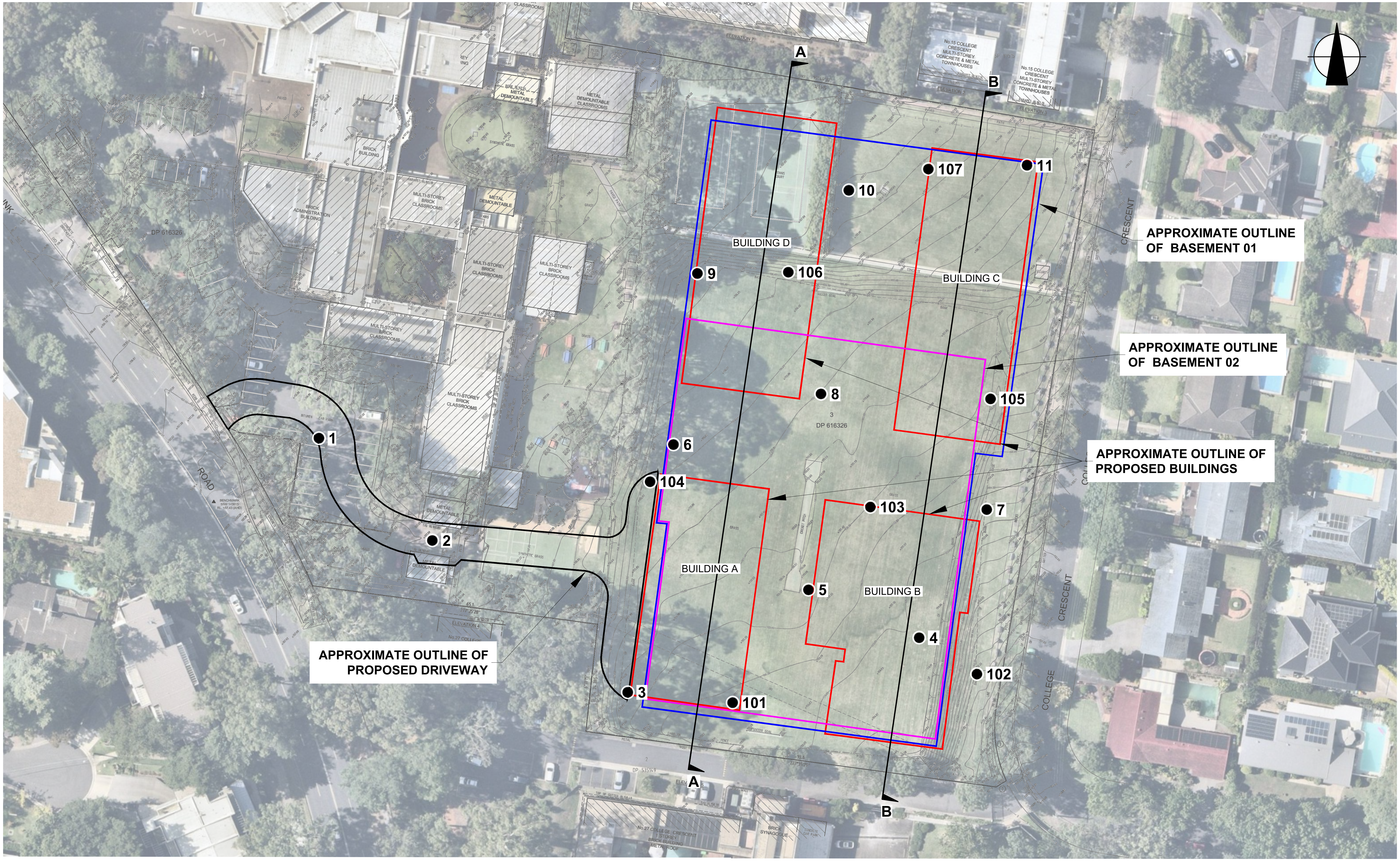
FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


PROJECT: PROPOSED INDEPENDANT LIVING AND AGED CARE DEVELOPMENT
JKGeotechnics
PROJECT No: 37393PN
DATE: 21/1/2026
LOCATION ID: BH 107
DEPTH: 6.05m - 11.50m
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
37393PN BH107 START CORING AT 6.05m
6
NO CORE: 0.14m
7
8
9
10
11
END OF BOREHOLE AT 11.50m



AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM	Title: **SITE LOCATION PLAN**
	Location: MASADA COLLEGE, COLLEGE CRESCENT, ST IVES, NSW
	Report No: 37393PN — Figure No: 1
This plan should be read in conjunction with the JK Geotechnics report.	JKGeotechnics

PLOT DATE: 3/02/2026 12:11:28 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\37000'S\37393PN ST IVES\CAD\37393PN.DWG



NOTES:
1. BOREHOLES 1 TO 11 ARE FROM OUR PREVIOUS INVESTIGATION.
2. BOREHOLES 101 TO 107 ARE FROM OUR CURRENT INVESTIGATION.
3. REFER TO FIGURE 3 FOR CROSS SECTION A-A.
4. REFER TO FIGURE 4 FOR CROSS SECTION B-B.

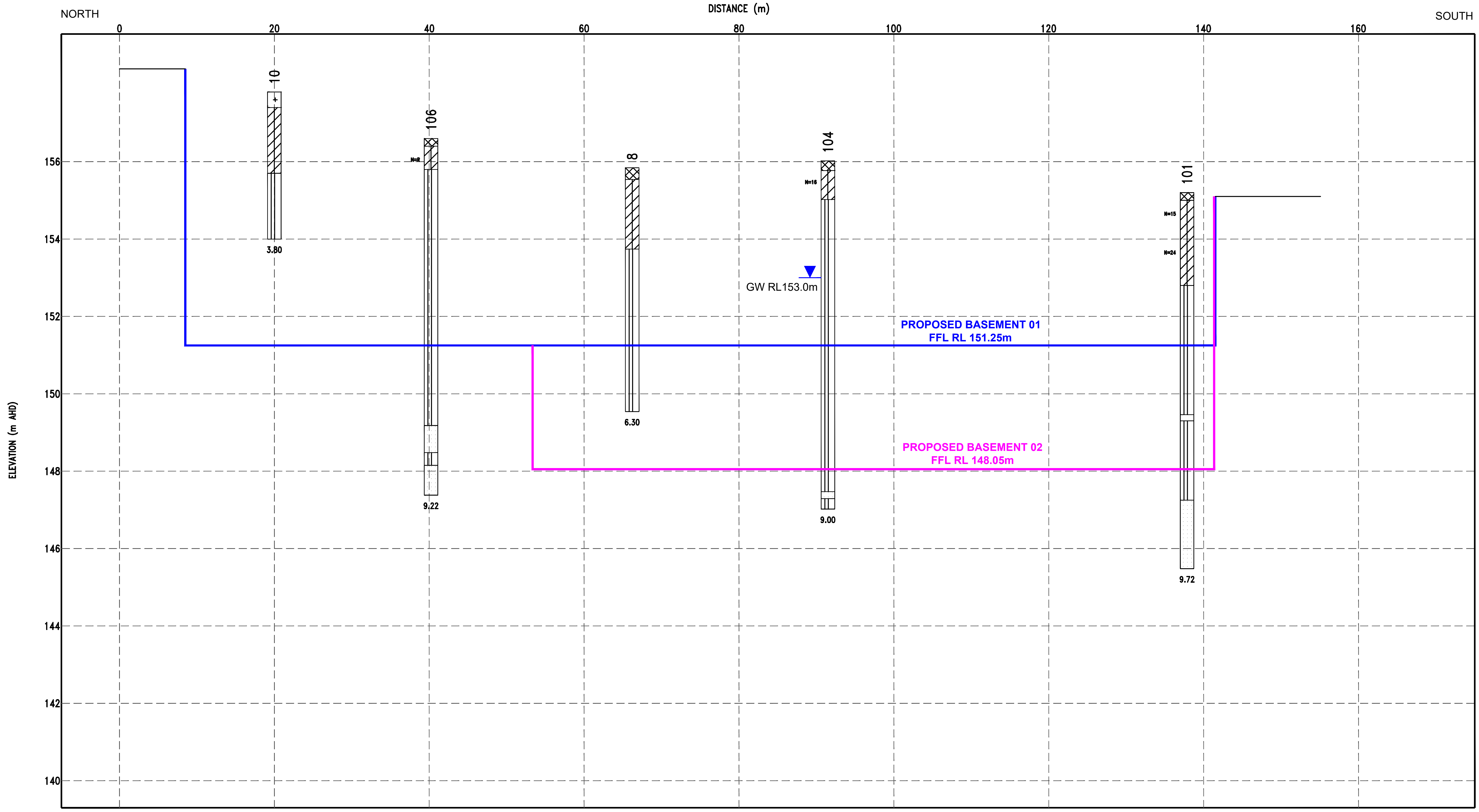
AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

SCALE 0 8 16 24 32 40 METRES 1:800 @A3

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

Title:	BOREHOLE LOCATION PLAN
Location:	MASADA COLLEGE, COLLEGE CRESCENT, ST IVES, NSW
Report No:	37393PN
Figure No:	2

JKGeotechnics



MATERIAL GRAPHIC

NO CORE
FILL
SILTY CLAY (CL, CI, CH)
SANDSTONE
DOLERITE, DIORITE
SILTSTONE

0 5 10 15 20 25
H:SCALE 1:500 @A3 METRES
0 1 2 3 4 5
V:SCALE 1:100 @A3 METRES

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

Title: **SECTION A-A**
GRAPHICAL BOREHOLE SUMMARY

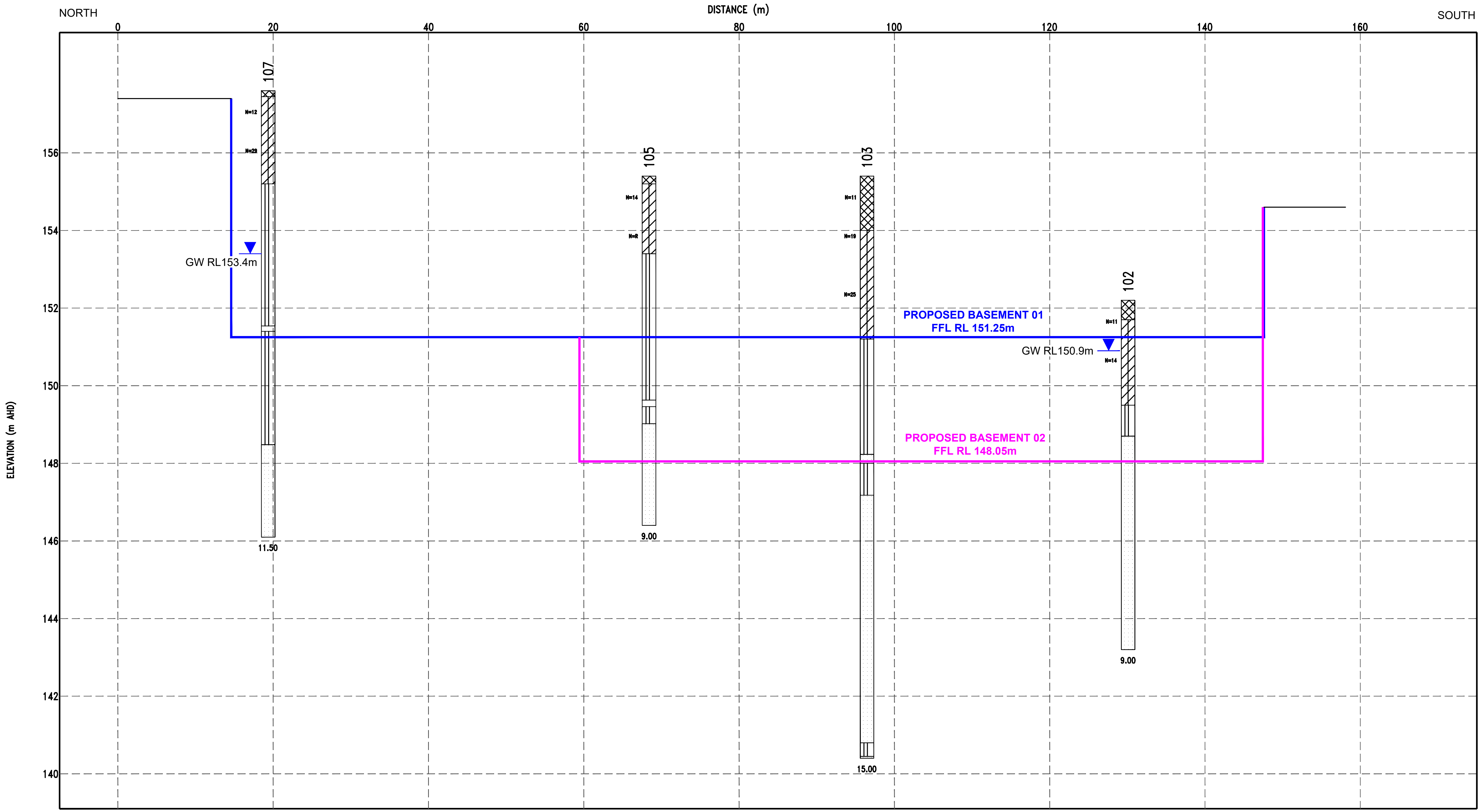
Location: MASADA COLLEGE, COLLEGE CRESCENT, ST IVES, NSW

Report No: 37393PN

Figure No: 3

JKGeotechnics

PLOT DATE: 11/03/2026 3:25:23 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\37000'S\37393PN ST IVES\CAD\37393PN.DWG



MATERIAL GRAPHIC

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

H:SCALE 1:500 @A3 METRES (0, 5, 10, 15, 20, 25)

V:SCALE 1:100 @A3 METRES (0, 1, 2, 3, 4, 5)

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

Title:	SECTION B-B GRAPHICAL BOREHOLE SUMMARY
Location:	MASADA COLLEGE, COLLEGE CRESCENT, ST IVES, NSW
Report No:	37393PN
Figure No:	4

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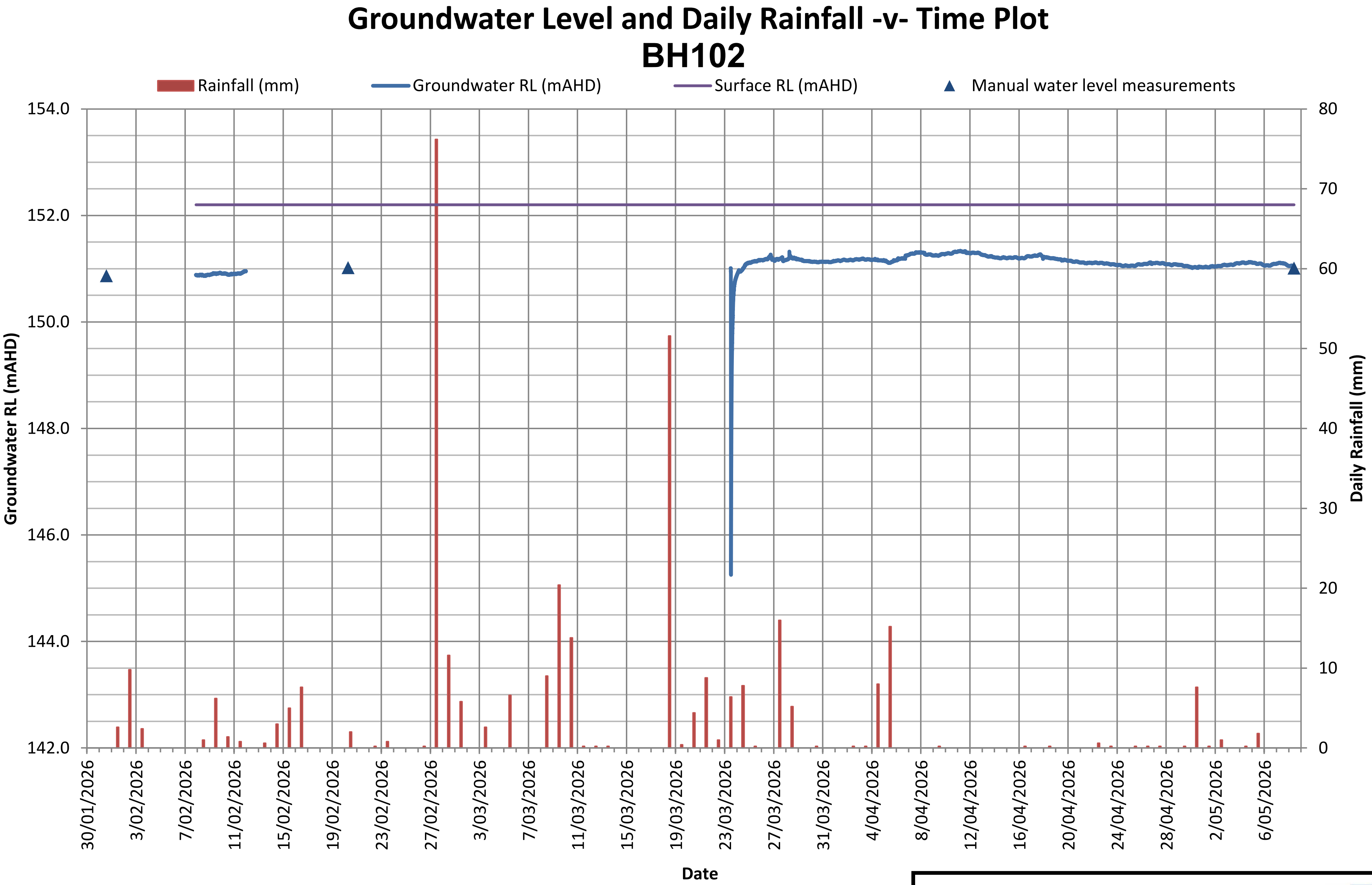
PLOT DATE: 11/03/2026 3:25:48 PM DWG FILE: J:\6F GEOTECHNICAL JOBS\37000's\37393PN ST IVES\CAD\37393PN.DWG



Rainfall data from: Wahroonga (Ada Avenue), Station No. 66211

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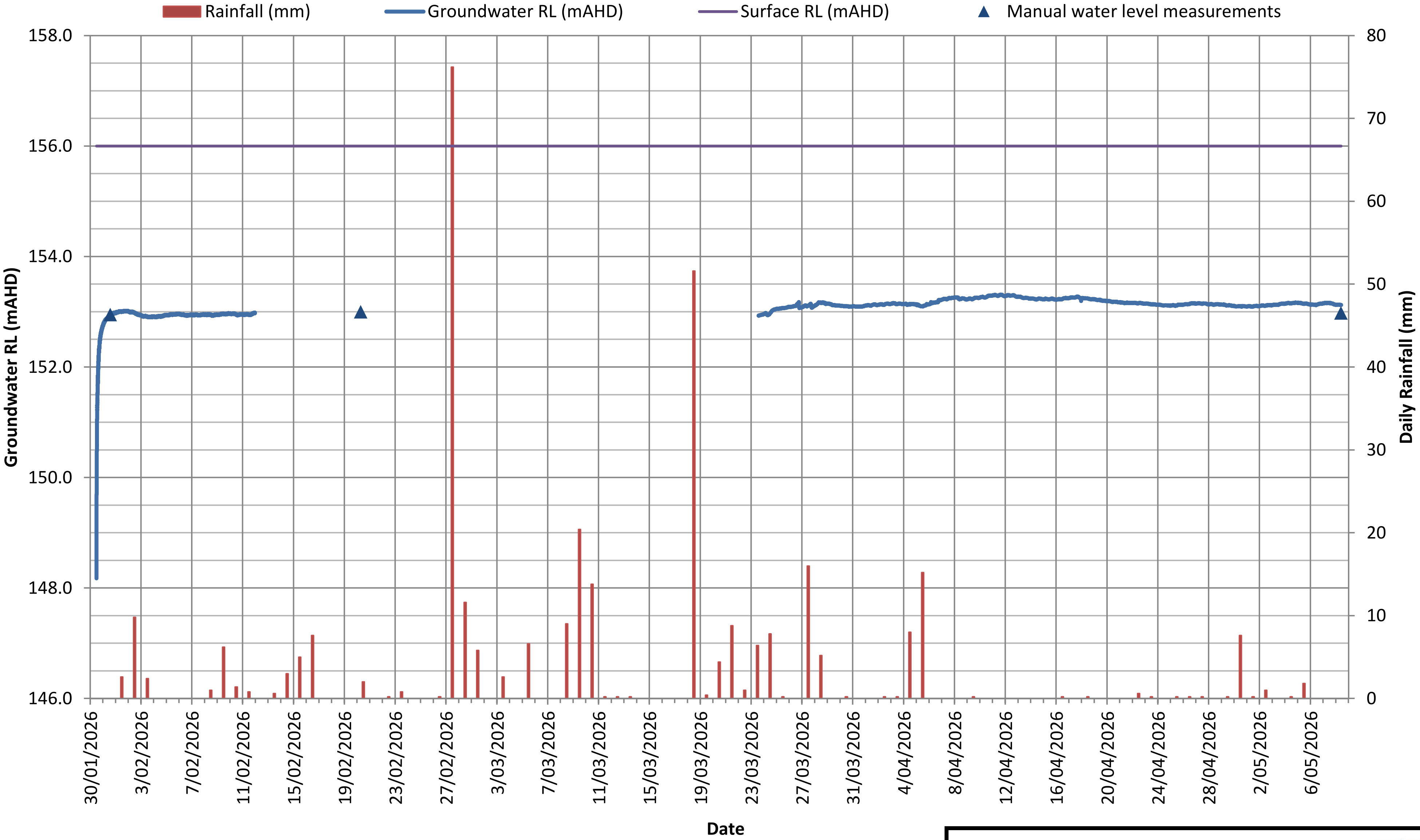
Report No. 37393PN Figure No. 5

File Name: 37393PN BH104 - GW vs Rainfall Graph
Date Printed: 21/05/2026

Groundwater Level and Daily Rainfall -v- Time Plot
BH104



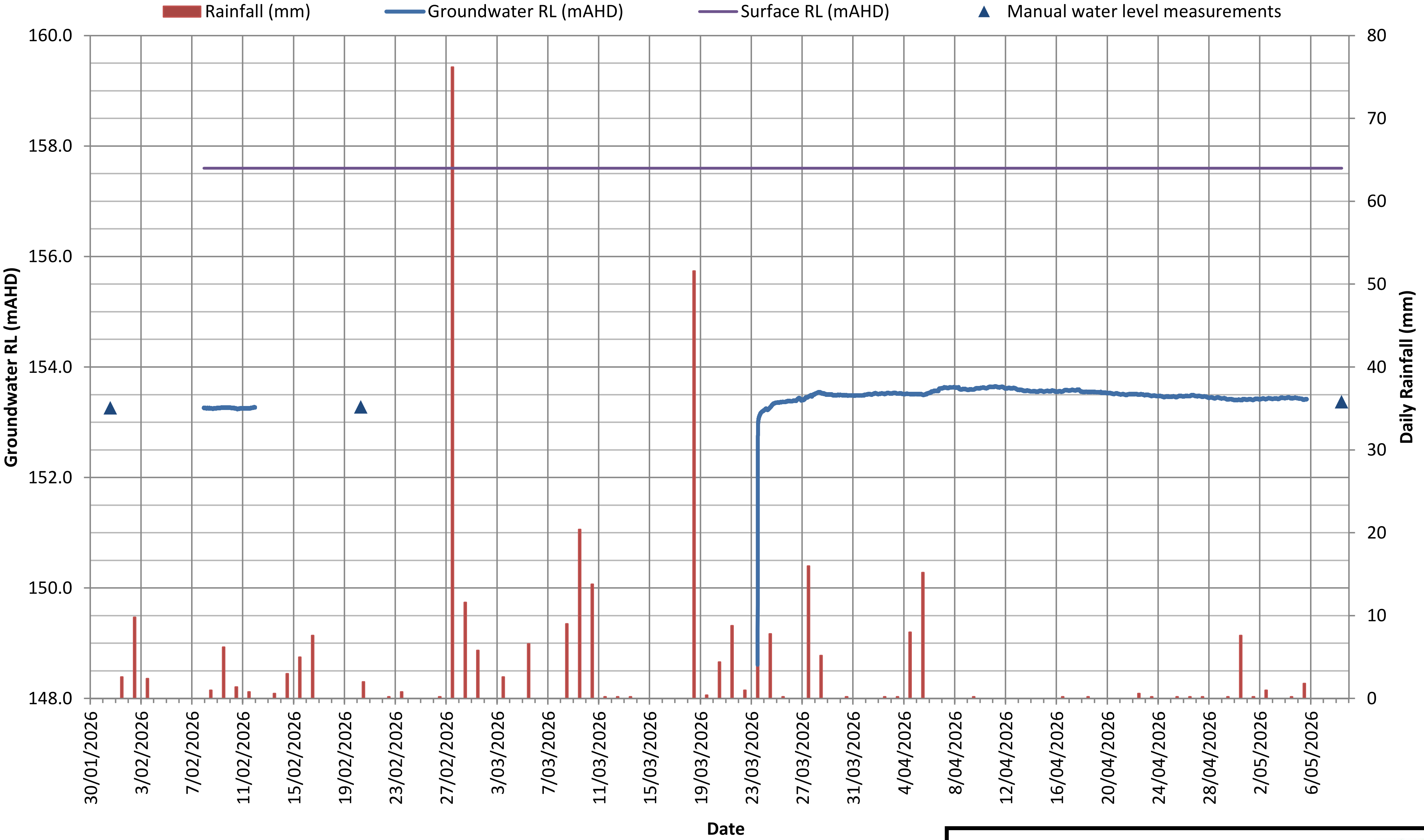
Rainfall data from: Wahroonga (Ada Avenue), Station No. 66211

JKGeotechnics

Report No. 37393PN Figure No. 6

File Name: 37393PN BH107 - GW vs Rainfall Graph
Date Printed: 21/05/2026

Groundwater Level and Daily Rainfall -v- Time Plot
BH107



Rainfall data from: Wahroonga (Ada Avenue), Station No. 66211

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Report No. 37393PN Figure No. 7

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_D), horizontal stress index (K_D), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_o), 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_o).

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 <u>or</u> where only a limited investigation has been completed <u>or</u> 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 65% 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	

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	% < 0.075mm
ine 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	–	–	–	–

Laboratory Classification Criteria

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

$$C_U = \frac{D_{60}}{D_{10}} \quad and \quad C_C = \frac{(D_{30})^2}{D_{10}\,D_{60}}$$

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:

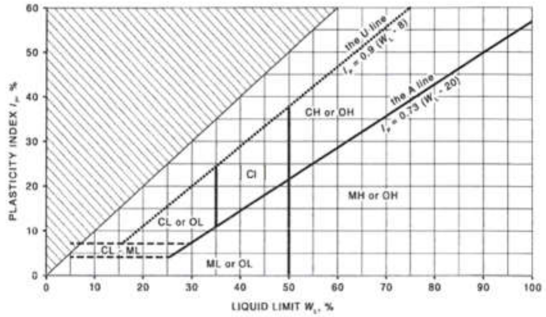
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

2 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.

3 Clay soils with liquid limits > 35% and ≤ 50% may be classified as being of medium plasticity.

4 The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

Modified Casagrande Chart for Classifying Silts and Clays according to their Behaviour



LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record		Standing water level. Time delay following completion of drilling/excavation may be shown.
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.
		Groundwater seepage into borehole or test pit noted during drilling or excavation.
Samples	ES U50 DB DS ASB ASS SAL	Sample taken over depth indicated, for environmental analysis. Undisturbed 50mm diameter tube sample taken over depth indicated. Bulk disturbed sample taken over depth indicated. Small disturbed bag sample taken over depth indicated. Soil sample taken over depth indicated, for asbestos analysis. Soil sample taken over depth indicated, for acid sulfate soil analysis. 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 = 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.
	VNS = 25 PID = 100	Vane shear reading in kPa of undrained shear strength. Photoionisation detector reading in ppm (soil sample headspace test).
Moisture Condition (Fine Grained Soils)	w > PL $w \approx$ PL w < PL $w \approx$ LL w > LL	Moisture content estimated to be greater than plastic limit. Moisture content estimated to be approximately equal to plastic limit. Moisture content estimated to be less than plastic limit. Moisture content estimated to be near liquid limit. Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D M W	DRY – runs freely through fingers. MOIST – does not run freely but no free water visible on soil surface. WET – free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS S F St VSt Hd Fr ( )	VERY SOFT – unconfined compressive strength ≤ 25kPa. SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa. FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa. STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa. VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa. HARD – unconfined compressive strength > 400kPa. FRIABLE – strength not attainable, soil crumbles. Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.
Density Index/ Relative Density (Cohesionless Soils)		**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.
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

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

<table>
<tr><th colspan="2">Term</th><th colspan="2">Abbreviation</th><th>Definition</th></tr>
<tr><td colspan="2">Residual Soil</td><td colspan="2">RS</td><td>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.</td></tr>
<tr><td colspan="2">Extremely Weathered</td><td colspan="2">XW</td><td>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.</td></tr>
<tr><td>Highly Weathered</td><td rowspan="2">Distinctly Weathered (Note 1)</td><td>HW</td><td rowspan="2">DW</td><td>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.</td></tr>
<tr><td>Moderately Weathered</td><td>MW</td><td>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, but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Slightly Weathered</td><td colspan="2">SW</td><td>Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Fresh</td><td colspan="2">FR</td><td>Rock shows no sign of decomposition of individual minerals or colour changes.</td></tr>
</table>

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

			Guide to Strength	
Term	**Abbreviation**	**Uniaxial Compressive Strength (MPa)**	**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

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.