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Integrated Practical Solutions

Report on Geotechnical Investigation

Proposed Shop Top Housing Development
57-69 Strathallen Avenue, Northbridge

Prepared for
SJD NB Pty Ltd

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The undersigned, on behalf of Douglas Partners Pty Ltd, confirm that this document and all attached drawings, logs and test results have been checked and reviewed for errors, omissions and inaccuracies.

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Report on Geotechnical Investigation
Proposed Shop Top Housing Development
57-69 Strathallen Avenue, Northbridge

1. Introduction

This report presents the results of a geotechnical investigation undertaken for a proposed shop top housing development at 57-69 Strathallen Avenue, Northbridge. The investigation was commissioned in an email dated 13 April 2023 by Alex Zhao of SJD Property Group Pty Ltd, on behalf of SJD NB Pty Ltd and was undertaken in accordance with Douglas Partners' proposal P221953.000 dated 6 April 2023.

It is understood that the proposed development includes multi-storey residential and commercial buildings with possibly two basement levels.

The field work for the investigation was undertaken in conjunction with a preliminary contamination investigation by DP, which has been reported separately (Ref: Reports 221953.00.R.001.Rev0).

The objective of the geotechnical investigation was to provide information on subsurface conditions for preliminary planning and design purposes.

2. Site Description

The site is occupied by single and two-storey brick commercial buildings, including a dry cleaner at the northern end of the site, and an asphaltic concrete (AC) car park fronting Strathallen Avenue. The site is bounded by mixed residential buildings with basement level(s) and commercial buildings to the north, Baringa Road to the south, Strathallen Avenue to the west and residential buildings to the east. The basement to the north and residence to the east extend along the common site boundaries. A Sydney Water sewer intersects the central part of the site in an approximate north-south direction and extends to the sewer main below Baringa Road.

A survey plan issued by Beveridge Williams in October 2022 (Ref. Project 2202145, Drawing DET-001, Version A) shows the site currently consists of six separate lots over a combined area of approximately 2400 m^2. The ground surface level across the site ranges from approximately reduced level (RL) 85 m relative to Australian Height Datum (AHD) at the southern boundary to RL 88.5 m at the northern end of the carpark, representing an average slope to the south of approximately 4 - 5 degrees.

3. Published Data

3.1 Geology

The Sydney 1:100,000 Sydney Soil Landscape Sheet and Geological Series Sheets indicate that the site is underlain by Hawkesbury Sandstone, which comprises medium to coarse grained quartz sandstone with very minor shale and laminite lenses.

The geotechnical investigation confirmed the presence of Hawkesbury Sandstone.

3.2 Hydrogeology

Based on the regional topography and limited groundwater data (from the two wells installed during the investigation), groundwater is inferred to be flowing to the south-east direction, towards Flat Rock Creek and Sydney Harbour.

3.3 Acid Sulfate Soils

Reference to acid sulfate soil mapping sourced from Willoughby Local Environmental Plan 2012 indicates that the site is located in an area classified as Class 5, which indicates that acid sulfate soils are not typically found in Class 5 areas. Areas classified as Class 5 are located within 500 m of adjacent Classes 1, 2, 3 or 4 land.

Reference to the 1:25,000 Acid Sulfate Soils (ASS) Risk Mapping Data published by the CSIRO indicates that the site is located within an area of low probability of occurrence for ASS.

4. Field Work

4.1 Field Work Methods

The field work for the geotechnical investigation was undertaken on 14 April and 26 April 2023, and included:

- drilling of three boreholes (BH1 to BH3) to approximate depths of between 6.0 m and 15.0 m. The boreholes were drilled using spiral flight augers in the soil and, in BH1 and BH3, diamond core drilling techniques in the rock to recover rock core samples for point-load strength testing and logging of defects. Standard penetration tests were undertaken within the soil strata to assess the in-situ strength of the soil;
- installation of two groundwater wells to 9.5 m depth in BH1 and 6.0 m depth in BH2 to allow for subsequent measurement of the water level (and sampling of groundwater for the preliminary contamination investigation). The well in BH1 was screened in rock to determine groundwater levels in the rock profile and the well in BH2 was screened in soil to determine groundwater levels in the soil profile. Groundwater observations were also made during auger drilling of the boreholes;

- measurement of groundwater using dip tape measurements and data loggers in monitoring wells at BH1 and BH2 for about one week on May / June 2023;
- permeability testing of the rock within the screened section of the monitoring wells at BH1 and BH2; and
- co-ordination of field work and logging and collection of soil / rock samples by a geotechnical engineer.

The locations of the boreholes are shown on Drawing 1 in Appendix B.

4.2 Field Work Results

Details of the subsurface conditions encountered in the boreholes are provided in the logs in Appendix C, together with notes explaining descriptive terms and classification methods.

The subsurface materials encountered in the boreholes are described as follows:

PAVEMENT:	Asphaltic concrete to a depth of 0.05 m;
FILL:	Gravelly sand, silty sand and sandy gravelly clay fill to depths of between 0.4 m and 0.9 m;
RESIDUAL SOIL:	Sandy clay (extremely weathered sandstone) in BH1 only extending to the top of rock at 1.38 m;
BEDROCK:	Initially very low to low strength sandstone to depths of between 3.9 m and 4.4 m becoming medium strength sandstone to between 7.3 m and 8.0 m, then high strength sandstone extending to the bottom of the boreholes at 12.15 m in BH1 and 15.03 m in BH3. A siltstone band was encountered in BH1 at 3.6 m depth. The bedrock was initially fractured and slightly fractured becoming slightly fractured to unbroken in BH1 and BH3 below depths of 4.3 m and 2.5 m, respectively.

No free groundwater was observed whilst auger drilling the boreholes, however drilling fluid loss occurred in BH1 at a depth of 4.4 m, near the shale / sandstone interface and core loss zone, possibly indicating the presence of open rock defects.

After drilling was completed, the groundwater wells in BH1 and BH2 were developed on 18 April 2023 and measurements were subsequently taken on 21 April 2023. The groundwater levels are shown in Table 1.

Table 1: Summary of Groundwater Level Measurements on 21 April 2023

Well ID	Ground Level * m (AHD)	SWL m (bgl)	SWL m (AHD)
BH1	87.6	3.05	84.55
BH2	87.1	3.70	83.40

Notes: *Surveyed by dGPS
AHD – Australian Height Datum
SWL – Standing Water Level
bgl – Below ground level

The results of the groundwater levels measured with data loggers in the wells at BH1 and BH2 are provided in Appendix E. The groundwater level was at depths of about 6.7 m/RL 80.9 m in BH1 and 3.9 m/RL 83.2 m in BH2, both within the rock profile over the approximate one-week monitoring period that did not experience any rainfall. The groundwater wells remain in ground and the data loggers were removed.

The groundwater level in BH1 varied significantly from RL 84.55 m in April to RL 80.9 m in May/June 2023. The earlier measurement may not represent stabilised levels, or the fluctuation may be driven be transient perched seepage flows through rock fractures.

The results of the permeability tests indicate that the permeability of the rock tested is about 1×10^{-7} m/s and 2×10^{-6} m/s at BH1 and BH2, respectively.

5. Laboratory Testing

Two soil samples were analysed to assess the aggressivity of the soil to buried steel and concrete structures. The results of the aggressivity laboratory testing are summarised in Table 2. The detailed laboratory test reports are included in Appendix D.

Table 2: Laboratory Test Results for Soil Aggressivity to Buried Concrete and Steel

Sample ID	Sample Description	pH	EC (1) (µS/cm)	Chloride (mg/kg)	Sulfate (mg/kg)
BH1 (0.7 m – 1.0 m)	Sandy CLAY	8.6	100	10	110
BH2 (0.6 m – 0.8 m)	Silty SAND	8.6	110	30	71

Notes: (1) EC = Electrical Conductivity.
(2) Each analyte was tested as a 1:5 mixture of soil:water.

Selected samples of the rock core were tested in the laboratory to determine the Point Load Strength Index (Is_{50}) values to assist with the rock strength classification. The results of the testing are shown on the borehole logs at the appropriate depth. The Is_{50} values for the rock ranged from approximately less than 0.1 MPa to 1.8 MPa, indicating that the rock samples tested had an unconfined compressive strength of about 2 MPa to 36 MPa when using a typical correlation of 20:1 (UCS: Is_{50}), consistent with very low to high strength rock.

6. Geotechnical Model

The subsurface profile at this site includes fill and residual clay or extremely weathered sandstone overlying very low to low strength sandstone bedrock with an approximate 1 m thick, very low to medium strength siltstone band, grading to medium and high strength sandstone with depth.

A geotechnical cross-section showing the interpreted subsurface profile across the site is shown in Drawing 2 in Appendix B. The section shows the interpreted geotechnical boundaries for the fill, natural soil and underlying sandstone rock. These profiles are accurate at the borehole locations only and variations must be expected away from the boreholes. The strata units or layers have been shown on the cross-sections by inferred strata boundaries only. The cross-sections indicate that variable very low to low strength sandstone and shale overlies more consistent medium and high strength sandstone at about RL 82.0 m to RL 83.5 m.

The permeability of the rock is expected to be in the order of 1×10^{-7} m/s and 2×10^{-6} m/s, which is variable, and ultimately depends on the rock defects present.

The groundwater level was at depths of about 6.7 m/RL 80.9 m in BH1 and 3.9 m/RL 83.2 m in BH2, both within the rock profile over the approximate one-week monitoring period in May/ June 2023, which is noted to not experience any rainfall. The groundwater flow direction appears to be in a northerly direction towards Sailors Bay Creek. The initial field dip-measurements of groundwater levels taken on 21 April 2023 showed a higher groundwater level in BH1, resulting in the groundwater flow direction to be in a south-easterly direction towards Flat Rock Creek. The variability in the groundwater levels and respective flow directions between those measured in April 2023 and in May/June 2023 may be associated with transient flow paths that develop from the rock defects and groundwater storage within the rock defects, over the relatively short period of groundwater monitoring. Long term groundwater level monitoring is recommended to further assess the groundwater conditions.

Due to the site elevation (i.e., >80 m AHD) and regional topography, the measured groundwater is expected to be associated with perched seepage rather than a regional water table. Groundwater seepage is expected at the soil/rock interface and through rock defects, and water levels will fluctuate and temporarily rise following periods of extended rainfall.

7. Comments

7.1 Proposed Development

It is understood that a multi-storey, commercial and residential building is being proposed with a two-level basement being considered. It is anticipated that approximately 7 m – 8 m of excavation is required for the basement levels.

7.2 Earthworks

7.2.1 Excavation Conditions

Excavations will initially intersect fill, natural soil and possibly very low strength sandstone which should be readily removed using conventional earthmoving equipment such as excavators. Excavation of low strength and medium strength rock, and possibly high strength rock, would require hydraulic rock hammers to break up the rock before removal. Alternatively, excavation of medium and high strength rock may be possible with ripping by a heavy bulldozer, although a bulldozer may not be cost-effective for the relatively small basement footprint. Productivity within medium and high strength rock may be low (even with large dozers) and therefore some pre-splitting or rock hammering may be necessary to improve efficiency. Earthworks contractors should make their own assessment of the machinery required and can inspect the rock core at our office which will be held for 6 months.

Detailed excavations for service trenches, footings, lift pits and along the site boundary within medium strength (or stronger) rock could be carried out using a rotary rock saw with diamond teeth, rock hammers or rotary milling heads.

7.2.2 Groundwater

Groundwater was encountered between approximate levels of RL 81.0 m and RL 84.5 m AHD, which is expected to be above the lowest basement floor level about 7 – 8 m deep. Groundwater seepage is expected to enter the excavation at the soil / rock interface and through rock joints and defects in the basement floor and walls. This seepage may be relatively minor and possibly no existent during dry periods and will likely increase temporarily after periods of rainfall.

During construction and in the long term, it is anticipated that seepage into the excavation could be controlled by perimeter and subfloor drainage connected to a sump-and-pump system. On this basis, a drained basement may be considered for this site from a geotechnical perspective. The suitability of the water collected from dewatering operations, however, is subject to confirmation testing of groundwater quality (see DP report 221953.00.R.001.Rev0) and approval from regulatory authorities. It is understood from the contamination report that the groundwater is contaminated and therefore groundwater treatment would be required prior to disposal.

Installation of additional groundwater monitoring wells and long-term groundwater monitoring is required to further assess groundwater conditions for detailed design purposes and as part of WaterNSW guidelines. This should occur following the demolition of existing structures to allow greater site coverage and access for a drilling rig to install the wells.

7.2.3 Disposal of Excavated Material

All excavated materials will need to be disposed of in accordance with the provisions of the current legislation and guidelines including the Waste Classification Guidelines (EPA, 2014). Reference should be made to DP's contamination report (Ref. 221953.00.R.001.Rev0) for details on the preliminary contamination status of the soils.

7.2.4 Acid Sulfate Soils

Acid sulfate soils are typically encountered in low-lying (generally below RL 5 m AHD), water-logged, estuarine or marine soil deposits of recent Holocene Age, and can include organic deposits.

Given the acid sulfate soil risk mapping, the site topography (i.e. above RL 86 m AHD), DP's experience in the site area, and the soil conditions encountered in the boreholes, ASS are not expected at the site and therefore an acid sulfate soil management plan is not required.

7.2.5 Dilapidation Surveys

Dilapidation (building condition) reports should be undertaken on surrounding properties that may be affected by the excavations prior to commencing work on the site to document any existing defects so that any claims for damage due to construction related activities can be accurately assessed.

7.2.6 Vibrations

During excavation, it will be necessary to use appropriate methods and equipment to keep ground vibrations at adjacent buildings and structures within acceptable limits. The level of acceptable vibration is dependent on various factors including the type of building structure (e.g. reinforced concrete, brick, etc.), its structural condition, the frequency range of vibrations produced by the construction equipment, the natural frequency of the building and the vibration transmitting medium.

A ground vibration limit of 8 mm/sec vector sum peak particle velocity (VSPPV) is commonly adopted at the foundation level of existing buildings/structures for both architectural and human comfort considerations, although this vibration limit may need to be reduced if there are sensitive buildings, structures or equipment in the area and close to Baringa Road where Sydney Water may nominate a lower vibration limit for the protection of their assets. It is noted that vibration levels above 2 - 3 mm/sec may be strongly perceptible to occupants of adjacent buildings.

As the magnitude of vibration transmission is site specific, it is recommended that a vibration trial be undertaken at the commencement of rock excavation. The trial may indicate that smaller or different types of excavation equipment should be used for bulk (or detailed) excavation purposes.

7.3 Excavation Support

7.3.1 Batter Slopes

Although it is expected that shoring will be used around the basement perimeter, suggested temporary and permanent batter slopes for unsupported excavations up to a maximum height of 3 m are shown in Table 3. Deeper excavations and / or steeper batters will require further geotechnical review and input. The batters recommended in Table 3 are also subject to assessment of jointing in the rock by a geotechnical engineer. If adverse jointing is present in the rock then flatter batters or stabilisation may be required. If surcharge loads are applied near the crest of the slope then further geotechnical review and probably flatter batters or stabilisation using rock bolts or soil nails may be required.

Table 3: Recommended Safe Batter Slopes for Exposed Material

Exposed Material	Maximum Temporary Batter Slope (H : V)	Maximum Permanent Batter Slope (H : V)
Fill and Natural Soil	1 : 1	2 : 1
Variable Very Low and Low Strength Bedrock	0.75 : 1	1 : 1
Consistent Medium Strength (or Stronger) Sandstone	Vertical*	Vertical*

Note: * Subject to jointing assessment by experienced Geotechnical Engineer/Engineering Geologist at 1.5 m depth intervals

Unlike medium strength (or stronger) sandstone, weaker sandstone and shale is expected to deteriorate and break down in the long-term if left exposed to the weather. It is therefore recommended that excavations exposing soil, weak sandstone and shale should be covered with mesh reinforced shotcrete pinned to the face with dowels for long term protection to erosion.

7.3.2 Retaining Walls

The proposed basement excavation may extend up to or close to the site boundaries. Vertical excavations will require retaining structures both during construction and as part of the final structure.

Shoring support methods generally require tie-back anchors for stability, particularly where limiting ground movements behind the wall is essential. The legal implications of the use of rock anchors extending onto neighbouring properties and public land will need to be considered. Approval should be sought from Council and adjacent property owners.

A soldier pile/infill panel wall system consisting of bored, rock socketed piles, at typical intervals of 2 – 3 m centres is considered to be a suitable shoring system for this site. As excavation proceeds, structurally reinforced, shotcrete infill panels, or similar, are constructed in between the piles. A row of ground anchors may be required to provide additional lateral support. Excavation drops of 1.5 m depth should be inspected by a geotechnical engineer to confirm subsurface conditions and to check whether any additional stabilisation or support is required.

A more rigid contiguous pile wall consisting of closely spaced, or almost touching, bored, rock-socketed piles may be required where movement sensitive structures are to be supported or where surcharge loads from buildings or similar are present, such as along part of the northern boundary where high-level footings of the adjoining multistorey building may exist (depending upon the basement depth) and along part of the eastern boundary where the neighbouring, one to two-storey residential building at the common boundary. The wall may form part of the final structure, sealed by a shotcrete panel facing that is constructed as the bulk excavation progresses.

It may be possible to terminate the shoring piles in competent medium strength (or stronger), slightly fractured to unbroken sandstone rock which was encountered in both cored boreholes at depths between 4.4 m and 4.6 m. However, the lowest risk option is to extend piles to below the bulk excavation level as adverse joints can form unstable wedges of rock that may undermine piles at higher level.

Additionally, both pile wall systems above can often be designed to also provide foundation support for the perimeter of the structure provided piles extend to below the bulk excavation level. The piles are drilled with a minimum “toe in” design to provide lateral restraint below the base of the excavation based on the passive resistance of the rock in which the pile is socketed.

The drilling of the shoring piles may require the use of a high-powered piling rig capable of drilling into medium and high strength sandstone. Prospective piling contractors should be asked to make their own assessment on the type of equipment required to achieve the design requirements and pile depths.

7.3.3 Shoring Design

Excavations braced, anchored or propped, either temporarily or permanently, will be subject to earth pressures above the top of medium strength rock.

The preliminary design of cantilevered or single propped/anchored walls may be based on the parameters provided in Table 4, with a triangular earth pressure distribution (i.e. with zero pressure at the ground surface) calculated using an active earth pressure coefficient (k_a) where some wall movement is acceptable, or an “at-rest” earth pressure coefficient (k_o) where wall movement is to be reduced such as near adjacent structures. The pressure coefficients in Table 4 assume a level ground surface behind the top of the wall.

Table 4: Recommended Earth Pressure Coefficients and Bulk Unit Weights

Material	Earth Pressure Coefficient		Bulk Unit Weight (kN/m^3)
	Active (K_a)	At Rest (K_o)	
Filling or Natural Soil	0.4	0.6	20
Extremely Low to Very Low Strength Bedrock	0.2	0.3	22
Medium Strength (or Stronger) Bedrock	0*	0*	23

Note: * assuming no adverse joints present in the rock

Where more than one row of temporary anchors are used to support shoring walls, preliminary design should be based on a trapezoidal earth pressure distribution. Where the wall movement is to be minimised, the maximum pressure could be calculated using 6H kPa (H equals the depth to the top of medium strength or greater rock). Where there are no movement-sensitive structures in close proximity to the excavation, the maximum pressure could be calculated using 4H kPa. The pressure distribution should increase from zero at the ground surface to the maximum value at a depth of 0.25 H and decrease from the maximum at a depth of 0.75 H back to zero at the base of the excavation.

All surcharge loads should be allowed for in the retaining wall design including building footings, inclined slopes behind the wall, traffic and construction related activities.

Retaining / shoring walls should be designed for full hydrostatic pressures unless appropriate drainage systems are implemented in the design.

The final or detailed design of retaining walls for deep basements should be undertaken using interactive computer programs such as WALLAP or FLAC, which can take due regard of soil-structure interaction during the progressive stages of wall construction, anchoring and bulk excavation.

Precise survey monitoring of the shoring walls should be undertaken to measure baseline readings prior to any excavation and then at regular 1.5 m vertical drops in the excavation until the bulk excavation level is reached and then to at least one month after the shoring walls are fully supported by the building slabs to check that the actual shoring wall movements are within the predicted wall movements. Survey monitoring of adjacent roads and buildings should also be undertaken by a registered surveyor. A more detailed geotechnical monitoring plan outlining responsibilities of various parties, hold points and contingency plans should be prepared for construction monitoring once the final shoring design has been completed.

7.3.4 Passive Resistance

Passive resistance for shoring piles founded in rock below the base of the bulk excavation (including allowance for services and / or footings) may be based on the ultimate passive restraint pressure of 4,000 kPa in medium strength or stronger rock. This ultimate value represents the pressure mobilised at high displacements and therefore it will be necessary to incorporate a factor of safety of at least 2 to limit wall movement. The top 0.5 m of the pile socket should be ignored due to possible disturbance and over-excavation and adverse rock joints.

7.3.5 Ground Anchors

The preliminary design of temporary ground anchors for the support of shoring / retaining systems may be carried out on the basis of the parameters and maximum bond stresses given in Table 5.

Table 5: Bond Stresses for Anchor Design

Material Description	Maximum Allowable Bond Stress (kPa)	Maximum Ultimate Bond Stress (kPa)
Very Low to Low Strength Bedrock	75	150
Medium Strength (or Stronger) Bedrock	500	1000

The parameters given in Table 5 assume that the drilled holes are clean and adequately flushed. The anchors should be bonded behind a line drawn up at 45 degrees from the base of the shoring or the top of medium (or stronger) strength sandstone, whichever is shallower. ‘Lift-off’ tests should be carried out to confirm the anchor capacities. It is suggested that ground anchors should be proof loaded to 125% of the design working load and locked-off at no higher than 80% of the working load.

It is anticipated that the building will support the shoring walls over the long term and therefore the ground anchors are expected to be temporary only. The use of permanent anchors would require careful attention to corrosion protection including full column grouting and the use of an internal corrugated sheathing over the full length of the anchor. A detailed specification would need to be prepared for the installation and stressing of permanent anchors.

7.4 Foundations

It is expected that a bulk excavation level requiring excavations up to 7 - 8 m deep will likely expose medium and / or high strength sandstone. Therefore, shallow pad footings below the basement level are expected to be suitable to support column loads typical for a multi-storey residential building.

Preliminary design of footings may be based on the parameters provided in Table 6 but will need to be confirmed with detailed investigations including additional rock-cored boreholes. For bored piles that extend below the basement level, if required, shaft adhesion values for uplift (tension) may be taken as being equal to 70% of the shaft adhesion values for compression in Table 6.

Table 6: Preliminary Design Parameters for Foundation Design

Foundation Stratum	Maximum Allowable Pressure		Maximum Ultimate Pressure	
	End Bearing (kPa)	Shaft Adhesion (Compression) (kPa)	End Bearing (kPa)	Shaft Adhesion (Compression) (kPa)
Medium Strength Bedrock	3,500	350	15,000	600
High Strength Bedrock	6,000	500	30,000	1,000

Footings should be founded below a 45 degree line drawn up from the toe of any adjacent excavations.

Foundations proportioned on the basis of the allowable bearing pressure in Table 6 would be expected to experience total settlements of less than 1% of the footing width under the applied working load, with differential settlements between adjacent columns expected to be less than half of this value.

Spoon testing should be undertaken in at least 50% of foundations proportioned on the basis of an allowable bearing pressure of 6000 kPa. The purpose of “spoon” testing is to check that no significant weak seams exist within a depth of 1.5 times the minimum footing dimension below the foundation level.

All footings should be inspected by a geotechnical engineer to confirm that foundation conditions are suitable for the design parameters.

7.5 Soil Aggressivity

The laboratory test results indicate that the samples tested are non-aggressive to both buried concrete and steel elements in accordance with the provisions of Australian Standard AS 2159 – 2009 Piling – Design and installation.

7.6 Seismic Design

In accordance with Australian Standard AS 1170 - 2007 Structural Design Actions, Part 4: Earthquake Actions in Australia, based on the current borehole information, a site subsoil Class B_e (Rock site) is considered to be appropriate given the depth to very low strength or stronger rock is less than 3 m. AS 1170 nominates a hazard factor (Z) of 0.08 for Sydney.

7.7 Pavements

If access roads and pavements around the perimeter of the basement are being proposed, it is recommended that the fill material is removed and replaced as engineered filling, compacted to 100% relative to Standard compaction.

Preliminary design of road pavements or pavements may be based on a California bearing ratio (CBR) of 3% for clayey soil.

It is recommended that laboratory testing be undertaken to assess the CBR value for pavement design at a later stage once subgrade levels are confirmed.

7.8 Management and Mitigation of Adverse Impacts to Surrounding Properties

This report provides some preliminary management and mitigation strategies from a geotechnical perspective to reduce the potential risk of adverse impacts to surrounding properties. Such strategies during construction include groundwater level monitoring, vibration trials and geotechnical monitoring of machinery, batter slopes, survey monitoring of shoring walls and adjacent roads and buildings. Provided "good" engineering and construction practices are implemented and the geotechnical advice within this report is followed, then the risk of adverse impacts on surrounding properties would be reduced and low.

7.9 Further Investigation

Once the existing buildings are demolished and details of the basement footprint(s), basement depths and building column loads are known, it is recommended that further geotechnical investigation including rock-cored boreholes, three triangulated groundwater monitoring wells with at least three months of groundwater level monitoring and laboratory testing for pavements is completed. Further investigation is to confirm the subsurface and groundwater conditions as well as to refine geotechnical design parameters and comments for the proposed development.

Further assessment of rock permeability and inflow assessment may be required together with a specialised engineering assessment (SEA) for Sydney Water assets. Details of existing footings for neighbouring buildings should also be confirmed prior to detailed design.

8. Limitations

Douglas Partners (DP) has prepared this report for this project at 57-69 Strathallen Avenue, Northbridge in accordance with DP's proposal dated 6 April 2023 and acceptance received from Alex Zhao dated 13 April 2023. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of SJD NB Pty Ltd for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and / or their agents.

The results provided in the report are indicative of the sub-surface conditions on the site only at the specific sampling and / or testing locations, and then only to the depths investigated and at the time the work was carried out. Sub-surface conditions can change abruptly due to variable geological processes and also as a result of human influences. Such changes may occur after DP's field testing has been completed.

DP's advice is based upon the conditions encountered during this investigation. The accuracy of the advice provided by DP in this report may be affected by undetected variations in ground conditions across the site between and beyond the sampling and / or testing locations. The advice may also be limited by budget constraints imposed by others or by site accessibility.

The assessment of atypical safety hazards arising from this advice is restricted to the (geotechnical / environmental / groundwater) components set out in this report and based on known project conditions and stated design advice and assumptions. While some recommendations for safe controls may be provided, detailed 'safety in design' assessment is outside the current scope of this report and requires additional project data and assessment.

This report must be read in conjunction with all of the attached and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by DP. This is because this report has been written as advice and opinion rather than instructions for construction.

Douglas Partners Pty Ltd

Appendix A

About This Report

About this Report

Introduction

These notes have been provided to amplify DP's report in regard to classification methods, field procedures and the comments section. Not all are necessarily relevant to all reports.

DP's reports are based on information gained from limited subsurface excavations and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

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Borehole and Test Pit Logs

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

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes or pits, the frequency of sampling, and the possibility of other than 'straight line' variations between the test locations.

Groundwater

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

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole 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. They may not be the same at the time of construction as are indicated in the report; and
- 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 first be washed out of the hole if water measurements are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or 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 a perched water table.

Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP will be pleased to review the report and the sufficiency of the investigation work.

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

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

If these occur, DP will be pleased to assist with investigations or advice to resolve the matter.

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, DP requests that it be immediately notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

Information for Contractual Purposes
Where information obtained from this report 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. DP would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection
The company will always be pleased to provide engineering inspection services for geotechnical and environmental aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

Appendix B

Drawings

LOCALITY MAP

Notes:
1. Drawing projection GDA94 / MGA zone 56
2. Latest available aerial imagery sourced from metromap.com accessed 26/4/2023
3. Test locations shown are approximate only

Legend

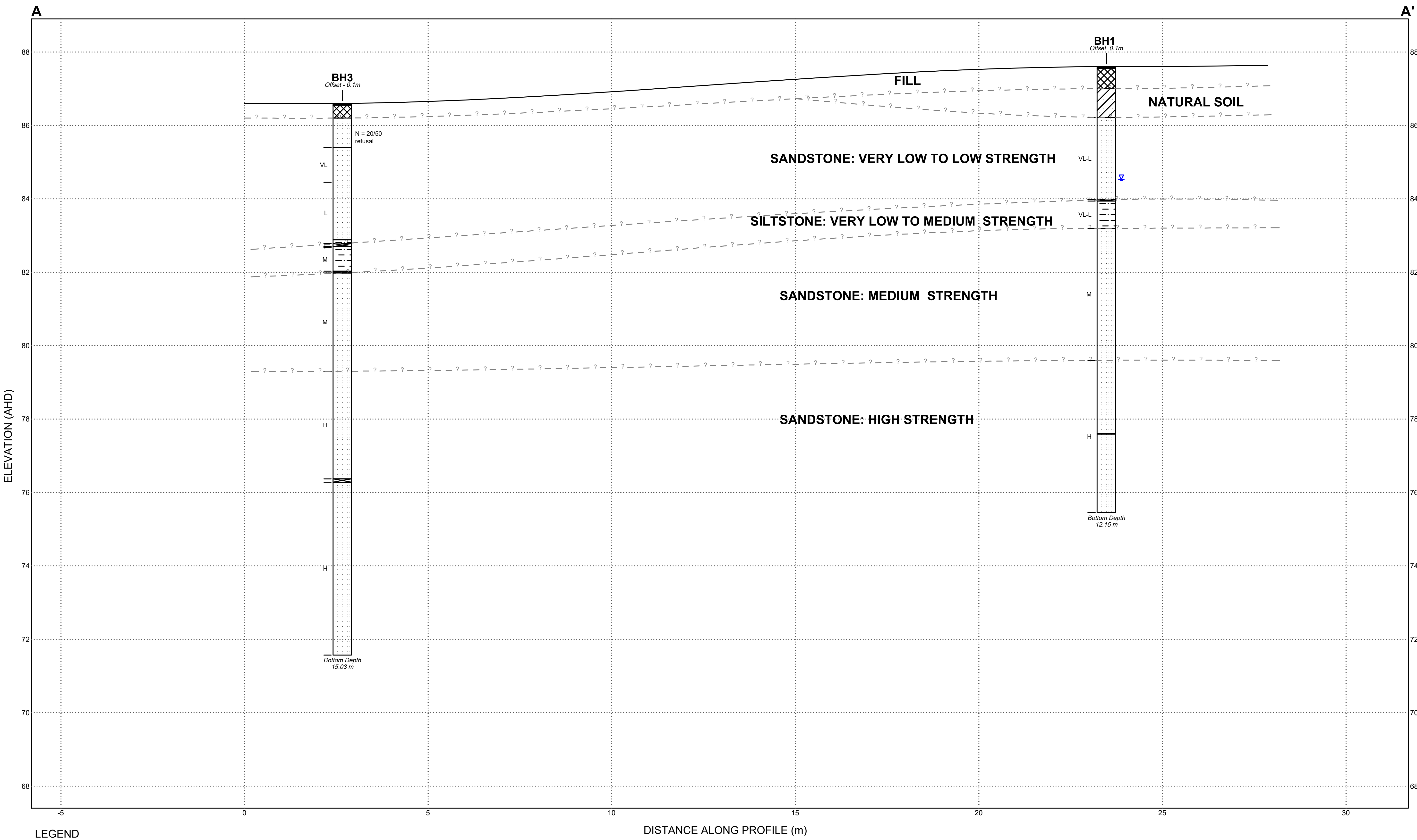
- Site Boundary
- Lot
- Approximate Sewer Alignment
- Borehole / Monitoring Well Location
- Geological Cross Section

0 5 10 15 20 m

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: SJD NB Pty Ltd		TITLE:	**Site Locality and Test Location Plan**
OFFICE: Sydney	DRAWN BY: JJH / MN		**Due Dilligence Geotechnical Investigation**
SCALE: 1:300 @ A3	DATE: 17.10.2023		**57-59 Strathallen Avenue, Northbridge**

PROJECT No:	221953.01
DRAWING No:	1
REVISION:	0



LEGEND

- Core Loss
- Asphaltic Concrete
- Filling
- Sandstone
- Sandy Clay
- Siltstone

NOTES:

1. Subsurface conditions are accurate at the borehole locations only. Variations in subsurface conditions may occur between borehole locations.
2. Strata layers and rock classification shown are generalised and each layer can include bands of lower and higher strength rock and also bands of less or more fractured rock.
3. Summary logs only and should be read in conjunction with detailed logs.

ROCK STRENGTH

- VL - Very Low
- L - Low
- M - Medium
- H - High

TESTS / OTHER

- N - Standard penetration test value
- — ? — — - Interpreted geotechnical boundary
- Water level

0 2

Horizontal Scale (metres)

Natural Scale

Douglas Partners Geotechnics / Environment / Groundwater	CLIENT: SJD NB Pty Ltd	TITLE: **Interpreted Geotechnical Cross-Section A-A'**	PROJECT No: 221953.01
	OFFICE: Sydney / DRAWN BY: MN	**Due Diligence Geotechnical Investigation**	DRAWING No: 2
	SCALE: 1:100 (H) 1:100 (V) @ A3 / DATE: 17.10.2023	**57-69 Strathallen Avenue, Northbridge**	REVISION: 0

Appendix C

Borehole Logs

Sampling

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

Disturbed samples taken during drilling provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing it to obtain a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Test Pits

Test pits are usually excavated with a backhoe or an excavator, allowing close examination of the in-situ soil if it is safe to enter into the pit. The depth of excavation is limited to about 3 m for a backhoe and up to 6 m for a large excavator. A potential disadvantage of this investigation method is the larger area of disturbance to the site.

Large Diameter Augers

Boreholes can be drilled using a rotating plate or short spiral auger, generally 300 mm or larger in diameter commonly mounted on a standard piling rig. The cuttings are returned to the surface at intervals (generally not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube samples.

Continuous Spiral Flight Augers

The borehole is advanced using 90-115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are disturbed and may be mixed with soils from the sides of the hole. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively low reliability, due to the remoulding, possible mixing or softening of samples by groundwater.

Non-core Rotary Drilling

The borehole is advanced using a rotary bit, with water or drilling mud being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from the rate of penetration. Where drilling mud is used this can mask the cuttings and reliable identification is only possible from separate sampling such as SPTs.

Continuous Core Drilling

A continuous core sample can be obtained using a diamond tipped core barrel, usually with a 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in weak rocks and granular soils), this technique provides a very reliable method of investigation.

Standard Penetration Tests

Standard penetration tests (SPT) are used as a means of estimating the density or strength of soils and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, Methods of Testing Soils for Engineering Purposes - Test 6.3.1.

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm 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 150 mm of, say, 4, 6 and 7 as:

  4,6,7
  N=13

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

  15, 30/40 mm

The results of the SPT tests can be related empirically to the engineering properties of the soils.

Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests

Dynamic penetrometer tests (DCP or PSP) are carried out by driving a steel rod into the ground using a standard weight of hammer falling a specified distance. As the rod penetrates the soil the number of blows required to penetrate each successive 150 mm depth are recorded. Normally there is a depth limitation of 1.2 m, but this may be extended in certain conditions by the use of extension rods. Two types of penetrometer are commonly used.

- Perth sand penetrometer - a 16 mm diameter flat ended rod is driven using a 9 kg hammer dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands and is mainly used in granular soils and filling.
- Cone penetrometer - a 16 mm diameter rod with a 20 mm diameter cone end is driven using a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). This test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various road authorities.

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are generally based on Australian Standard AS1726:2017, Geotechnical Site Investigations. In general, the descriptions include strength or density, colour, structure, soil or rock type and inclusions.

The soil group symbol classifications are given as follows based on two major soil divisions:

- Coarse-grained soils
- Fine-grained soils

Major Divisions				Description	
				Group Symbol*	Typical Name
COARSE-GRAINED SOILS	More than 65% by dry mass, (excluding that larger than 63 mm) is greater than 0.075 mm	GRAVEL	More than 50% of coarse grains are greater than 2.36 mm	GW	Well graded gravels and gravel-sand mixtures, little or no fines.
				GP	Poorly graded gravels and gravel-sand mixtures, little or no fines.
		GRAVELLY SOILS		GM	Silty gravels, gravel-sand-silt mixtures.
				GC	Clay gravels, gravel-sand-clay mixtures.
		SAND	More than 50% of coarse grains are less than 2.36 mm	SW	Well graded sands and gravelly sands, little or no fines.
				SP	Poorly graded sands and gravelly sands, little or no fines.
		SANDY SOILS		SM	Silty sand, sand-silt mixtures.
				SC	Clayey sands, sand-clay mixtures.

* For coarse grained soils where the fines content is between 5% and 12%, the soil shall be given a dual classification eg GP-GM.

FINE-GRAINED SOILS	More than 35% by dry mass, (excluding that larger than 63 mm) is less than 0.075 mm	Liquid Limit less than 35%	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity
		35% <LL< 50%	CI	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
		Liquid Limit greater than 50%	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity.
			Pt	Peat muck and other highly organic soils.

Soil Types

Soil types are described according to the predominant particle size, qualified by the grading of other particles present:

Type	Particle size (mm)
Boulder	>200
Cobble	63 - 200
Gravel	2.36 - 63
Sand	0.075 - 2.36
Silt	0.002 - 0.075
Clay	<0.002

The sand and gravel sizes can be further subdivided as follows:

Type	Particle size (mm)
Coarse gravel	19 - 63
Medium gravel	6.7 - 19
Fine gravel	2.36 – 6.7
Coarse sand	0.6 - 2.36
Medium sand	0.21 - 0.6
Fine sand	0.075 - 0.21

Definitions of grading terms used are:

- Well graded - a good representation of all particle sizes
- Poorly graded - an excess or deficiency of particular sizes within the specified range
- Uniformly graded - an excess of a particular particle size
- Gap graded - a deficiency of a particular particle size with the range

The proportions of secondary constituents of soils are described as follows:

In fine grained soils (>35% fines)

Term	Proportion of sand or gravel	Example
And	Specify	Clay (60%) and Sand (40%)
Adjective	>30%	Sandy Clay
With	15 – 30%	Clay with sand
Trace	0 - 15%	Clay, trace sand

In coarse grained soils (>65% coarse)
- with clays or silts

Term	Proportion of fines	Example
And	Specify	Sand (70%) and Clay (30%)
Adjective	>12%	Clayey Sand
With	5 - 12%	Sand with clay
Trace	0 - 5%	Sand, trace clay

In coarse grained soils (>65% coarse)
- with coarser fraction

Term	Proportion of coarser fraction	Example
And	Specify	Sand (60%) and Gravel (40%)
Adjective	>30%	Gravelly Sand
With	15 - 30%	Sand with gravel
Trace	0 - 15%	Sand, trace gravel

The presence of cobbles and boulders shall be specifically noted by beginning the description with ‘Mix of Soil and Cobbles/Boulders’ with the word order indicating the dominant first and the proportion of cobbles and boulders described together.

Cohesive Soils

Cohesive soils, such as clays, are classified on the basis of undrained shear strength. The strength may be measured by laboratory testing, or estimated by field tests or engineering examination. The strength terms are defined as follows:

Description	Abbreviation	Undrained shear strength (kPa)
Very soft	VS	<12
Soft	S	12 - 25
Firm	F	25 - 50
Stiff	St	50 - 100
Very stiff	VSt	100 - 200
Hard	H	>200
Friable	Fr	-

Cohesionless Soils

Cohesionless soils, such as clean sands, are classified on the basis of relative density, generally from the results of standard penetration tests (SPT), cone penetration tests (CPT) or dynamic penetrometers (PSP). The relative density terms are given below:

Relative Density	Abbreviation	Density Index (%)
Very loose	VL	<15
Loose	L	15-35
Medium dense	MD	35-65
Dense	D	65-85
Very dense	VD	>85

Soil Origin

It is often difficult to accurately determine the origin of a soil. Soils can generally be classified as:

- Residual soil - derived from in-situ weathering of the underlying rock;
- Extremely weathered material – formed from in-situ weathering of geological formations. Has soil strength but retains the structure or fabric of the parent rock;
- Alluvial soil – deposited by streams and rivers;
- Estuarine soil – deposited in coastal estuaries;
- Marine soil – deposited in a marine environment;
- Lacustrine soil – deposited in freshwater lakes;
- Aeolian soil – carried and deposited by wind;
- Colluvial soil – soil and rock debris transported down slopes by gravity;
- Topsoil – mantle of surface soil, often with high levels of organic material.
- Fill – any material which has been moved by man.

Moisture Condition – Coarse Grained Soils

For coarse grained soils the moisture condition should be described by appearance and feel using the following terms:

- Dry (D) Non-cohesive and free-running.
- Moist (M) Soil feels cool, darkened in colour.
  Soil tends to stick together.
  Sand forms weak ball but breaks easily.
- Wet (W) Soil feels cool, darkened in colour.
  Soil tends to stick together, free water forms when handling.

Moisture Condition – Fine Grained Soils

For fine grained soils the assessment of moisture content is relative to their plastic limit or liquid limit, as follows:

- 'Moist, dry of plastic limit' or 'w <PL' (i.e. hard and friable or powdery).
- 'Moist, near plastic limit' or 'w ≈ PL (i.e. soil can be moulded at moisture content approximately equal to the plastic limit).
- 'Moist, wet of plastic limit' or 'w >PL' (i.e. soils usually weakened and free water forms on the hands when handling).
- 'Wet' or 'w ≈LL' (i.e. near the liquid limit).
- 'Wet' or 'w >LL' (i.e. wet of the liquid limit).

Rock Descriptions

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Rock Strength

Rock strength is defined by the Unconfined Compressive Strength and it refers to the strength of the rock substance and not the strength of the overall rock mass, which may be considerably weaker due to defects.

The Point Load Strength Index $Is_{(50)}$ is commonly used to provide an estimate of the rock strength and site specific correlations should be developed to allow UCS values to be determined. The point load strength test procedure is described by Australian Standard AS4133.4.1-2007. The terms used to describe rock strength are as follows:

Strength Term	Abbreviation	Unconfined Compressive Strength MPa	Point Load Index * $Is_{(50)}$ MPa
Very low	VL	0.6 - 2	0.03 - 0.1
Low	L	2 - 6	0.1 - 0.3
Medium	M	6 - 20	0.3 - 1.0
High	H	20 - 60	1 - 3
Very high	VH	60 - 200	3 - 10
Extremely high	EH	>200	>10

* Assumes a ratio of 20:1 for UCS to $Is_{(50)}$. It should be noted that the UCS to $Is_{(50)}$ ratio varies significantly for different rock types and specific ratios should be determined for each site.

Degree of Weathering

The degree of weathering of rock is classified as follows:

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

Rock Descriptions

Degree of Fracturing

The following classification applies to the spacing of natural fractures in diamond drill cores. It includes bedding plane partings, joints and other defects, but excludes drilling breaks.

Term	Description
Fragmented	Fragments of <20 mm
Highly Fractured	Core lengths of 20-40 mm with occasional fragments
Fractured	Core lengths of 30-100 mm with occasional shorter and longer sections
Slightly Fractured	Core lengths of 300 mm or longer with occasional sections of 100-300 mm
Unbroken	Core contains very few fractures

Rock Quality Designation

The quality of the cored rock can be measured using the Rock Quality Designation (RQD) index, defined as:

$$\text{RQD \%} = \frac{\text{cumulative length of 'sound' core sections > 100 mm long}}{\text{total drilled length of section being assessed}}$$

where 'sound' rock is assessed to be rock of low strength or stronger. The RQD applies only to natural fractures. If the core is broken by drilling or handling (i.e. drilling breaks) then the broken pieces are fitted back together and are not included in the calculation of RQD.

Stratification Spacing

For sedimentary rocks the following terms may be used to describe the spacing of bedding partings:

Term	Separation of Stratification Planes
Thinly laminated	< 6 mm
Laminated	6 mm to 20 mm
Very thinly bedded	20 mm to 60 mm
Thinly bedded	60 mm to 0.2 m
Medium bedded	0.2 m to 0.6 m
Thickly bedded	0.6 m to 2 m
Very thickly bedded	> 2 m

Symbols & Abbreviations

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Introduction

These notes summarise abbreviations commonly used on borehole logs and test pit reports.

Drilling or Excavation Methods

C	Core drilling
R	Rotary drilling
SFA	Spiral flight augers
NMLC	Diamond core - 52 mm dia
NQ	Diamond core - 47 mm dia
HQ	Diamond core - 63 mm dia
PQ	Diamond core - 81 mm dia

Water

▷	Water seep
▽	Water level

Sampling and Testing

A	Auger sample
B	Bulk sample
D	Disturbed sample
E	Environmental sample
U_{50}	Undisturbed tube sample (50mm)
W	Water sample
pp	Pocket penetrometer (kPa)
PID	Photo ionisation detector
PL	Point load strength Is(50) MPa
S	Standard Penetration Test
V	Shear vane (kPa)

Description of Defects in Rock

The abbreviated descriptions of the defects should be in the following order: Depth, Type, Orientation, Coating, Shape, Roughness and Other. Drilling and handling breaks are not usually included on the logs.

Defect Type

B	Bedding plane
Cs	Clay seam
Cv	Cleavage
Cz	Crushed zone
Ds	Decomposed seam
F	Fault
J	Joint
Lam	Lamination
Pt	Parting
Sz	Sheared Zone
V	Vein

Orientation

The inclination of defects is always measured from the perpendicular to the core axis.

h	horizontal
v	vertical
sh	sub-horizontal
sv	sub-vertical

Coating or Infilling Term

cln	clean
co	coating
he	healed
inf	infilled
stn	stained
ti	tight
vn	veneer

Coating Descriptor

ca	calcite
cbs	carbonaceous
cly	clay
fe	iron oxide
mn	manganese
slt	silty

Shape

cu	curved
ir	irregular
pl	planar
st	stepped
un	undulating

Roughness

po	polished
ro	rough
sl	slickensided
sm	smooth
vr	very rough

Other

fg	fragmented
bnd	band
qtz	quartz

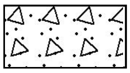
Symbols & Abbreviations

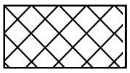
Graphic Symbols for Soil and Rock

General

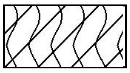
 Asphalt

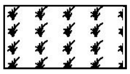
 Road base

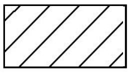
 Concrete

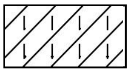
 Filling

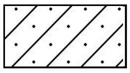
Soils

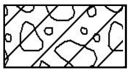
 Topsoil

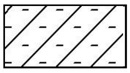
 Peat

 Clay

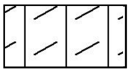
 Silty clay

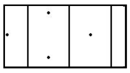
 Sandy clay

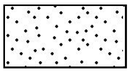
 Gravelly clay

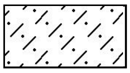
 Shaly clay

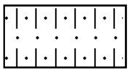
 Silt

 Clayey silt

 Sandy silt

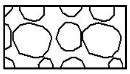
 Sand

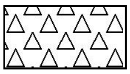
 Clayey sand

 Silty sand

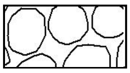
 Gravel

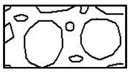
 Sandy gravel

 Cobbles, boulders

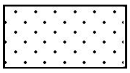
 Talus

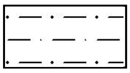
Sedimentary Rocks

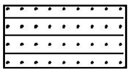
 Boulder conglomerate

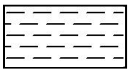
 Conglomerate

 Conglomeratic sandstone

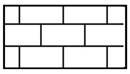
 Sandstone

 Siltstone

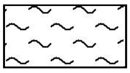
 Laminite

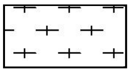
 Mudstone, claystone, shale

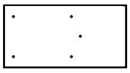
 Coal

 Limestone

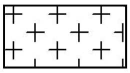
Metamorphic Rocks

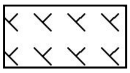
 Slate, phyllite, schist

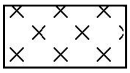
 Gneiss

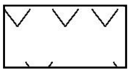
 Quartzite

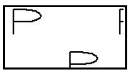
Igneous Rocks

 Granite

 Dolerite, basalt, andesite

 Dacite, epidote

 Tuff, breccia

 Porphyry

BOREHOLE LOG

CLIENT: SJD NB Pty Ltd
PROJECT: Proposed Shop Top Housing Development
LOCATION: 57-69 Strathallen Avenue, Northbridge

SURFACE LEVEL: 87.6 AHD
EASTING: 334419
NORTHING: 6257320
DIP/AZIMUTH: 90°/--

BORE No: BH1
PROJECT No: 221953.01
DATE: 14/4/2023
SHEET 1 OF 2

Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
0.05	ASPHALTIC CONCRETE					
0.6	FILL/Sandy Gravelly CLAY: low to medium plasticity, orange-brown and grey, fine sand, fine to coarse igneous (roadbase) gravel, trace sandstone and ironstone gravel, w~PL		E			No odour
			E/S			Slight hydrocarbon odour
1.38	Sandy CLAY CI-CH: medium to high plasticity, pale grey mottled red-brown, fine to medium grained sand, w~PL, extremely weathered sandstone At 1.0m: slight hydrocarbon odour					
	SANDSTONE: medium to coarse grained, pale grey, orange and red-brown, low strength, highly weathered with extremely weathered band, slightly fractured, Hawkesbury Sandstone	1.54m: B0°-10°, ir, ro, cln 1.7m: J15°, pl, ro, cln 1.84m: B5°, pl, ro, cln 2.33m: B10°, pl, ro, cln 2.56m: B5°, pl, ro, cln	C	100	100	PL(A) = 0.3 PL(A) = 0.2
3.66		3.62m: CORE LOSS: 40mm				PL(A) = 0.1
4.1	SILSTONE: dark grey and grey, with approximately 40% interbedded fine grained sandstone, very low to low strength, highly weathered, fractured, Hawkesbury Sandstone	3.66m: Ds, 20 mm 3.9m: B0°, pl, ro, cln 4.06m: B0°, irr, ro fe st 4.09m: J0°-20°, cu, he, fe st 4.12m: Cs, 30 mm 4.23m: Cs, 70 mm 4.3m: B10°, pl, he, fe st	C	99	48	PL(A) = 0.9
	SANDSTONE: fine to coarse grained, red-brown, pale grey and pale grey-brown, medium strength, highly weathered, very thinly bedded, slightly fractured to unbroken, Hawkesbury Sandstone From 4.67 m, slightly weathered	5.69m: B0°, pl, cly co 3 mm 5.78m: B5°, pl, ro, cln				PL(A) = 0.8
		6.52m: B0°, pl, cly co 3 mm	C	100	100	PL(A) = 1 PL(A) = 0.9
	Below 8.0 m: high strength	8.26m: B0°-10°, cu, ro, cln 9.04m: B0°, pl, ro, cln				PL(A) = 1.5
10.0			C	100	100	PL(A) = 1.6

RIG: GEO205 **DRILLER:** Ground Test **LOGGED:** RAS **CASING:** HW to 1.3 m
TYPE OF BORING: Solid flight Auger to 1.38 m, NMLC coring to 12.15 m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: Loss of drilling water return below 4.4 m

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: SJD NB Pty Ltd
PROJECT: Proposed Shop Top Housing Development
LOCATION: 57-69 Strathallen Avenue, Northbridge

SURFACE LEVEL: 87.6 AHD
EASTING: 334419
NORTHING: 6257320
DIP/AZIMUTH: 90°/--

BORE No: BH1
PROJECT No: 221953.01
DATE: 14/4/2023
SHEET 2 OF 2

Depth (m)	Description of Strata	Degree of Weathering	Rock Strength	Fracture Spacing (m)	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
	SANDSTONE: fine to coarse grained, red-brown, pale grey and pale grey-brown, high strength, slightly weathered, very thinly bedded, slightly fractured to unbroken, Hawkesbury Sandstone	SW	High		10.03m: B10°, irr, ro, cln 11.07m: B0°-20°, irr, ro, cln 11.71m: B5°, irr, ro, cly	C	100	100	PL(A) = 1.5 PL(A) = 1.8
12.15	Bore discontinued at 12.15m Target depth reached								

Depth scale: 11, 12, 13, 14, 15, 16, 17, 18, 19. RL scale: 77, 76, 75, 74, 73, 72, 71, 70, 69, 68.

RIG: GEO205 **DRILLER:** Ground Test **LOGGED:** RAS **CASING:** HW to 1.3 m
TYPE OF BORING: Solid flight Auger to 1.38 m, NMLC coring to 12.15 m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: Loss of drilling water return below 4.4 m

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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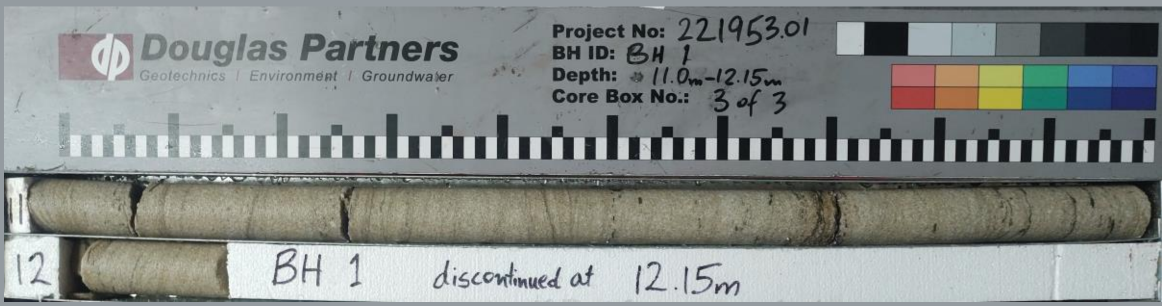
BORE: BH1 PROJECT: NORTHBRIDGE APRIL 2023

1.38-6.00m

BORE: BH1 PROJECT: NORTHBRIDGE APRIL 2023

6.00-11.00m

BORE: BH1 PROJECT: NORTHBRIDGE APRIL 2023



11.00-12.15m

BOREHOLE LOG

CLIENT: SJD NB Pty Ltd
PROJECT: Proposed Shop Top Housing Development
LOCATION: 57-69 Strathallen Avenue, Northbridge

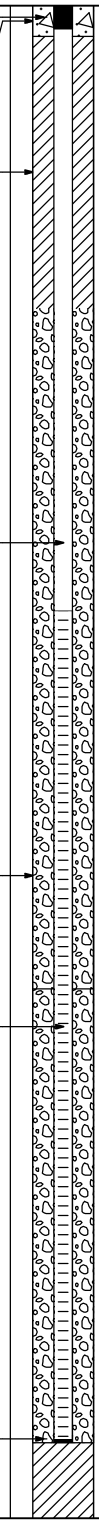
SURFACE LEVEL: 87.6 AHD
EASTING: 334419
NORTHING: 6257320
DIP/AZIMUTH: 90°/--

BORE No: BH1
PROJECT No: 221953.01
DATE: 14/4/2023
SHEET 1 OF 2

Depth (m)	Description of Strata	Type	Depth	Results & Comments
0.05	ASPHALTIC CONCRETE			
0.6	FILL/Sandy Gravelly CLAY: low to medium plasticity, orange-brown and grey, fine sand, fine to coarse igneous (roadbase) gravel, trace sandstone and ironstone gravel, w~PL	E	0.3 – 0.5	No odour
1.38	Sandy CLAY CI-CH: medium to high plasticity, pale grey mottled red-brown, fine to medium grained sand, w~PL, extremely weathered sandstone At 1.0m: slight hydrocarbon odour	E/S	0.7 – 1.0	Slight hydrocarbon odour
3.66	SANDSTONE: medium to coarse grained, pale grey, orange and red-brown, low strength, highly weathered with extremely weathered band, slightly fractured, Hawkesbury Sandstone	C	1.38 – 3.19	
			1.67	PL(A) = 0.3
			2.51	PL(A) = 0.2
		C	3.19 – 6.15	
			3.45	PL(A) = 0.1
4.1	SILSTONE: dark grey and grey, with approximately 40% interbedded fine grained sandstone, very low to low strength, highly weathered, fractured, Hawkesbury Sandstone			
10.0	SANDSTONE: fine to coarse grained, red-brown, pale grey and pale grey-brown, medium strength, highly weathered, very thinly bedded, slightly fractured to unbroken, Hawkesbury Sandstone From 4.67 m, slightly weathered Below 8.0 m: high strength		4.57	PL(A) = 0.9
			5.34	PL(A) = 0.8
		C	6.15 – 9.12	
			6.33	PL(A) = 1
			7.51	PL(A) = 0.9
			8.7	PL(A) = 1.5
		C	9.12 –	
			9.65	PL(A) = 1.6

RL: 87, 86, 85, 84, 83, 82, 81, 80, 79, 78

Well Construction Details



- Gatic cover
- Concrete 0-0.2m
- Bentonite 0.2-2.0m
- Blank pipe 0.1-4.0m
- Gravel 2.0-9.5m
- Machine slotted PVC screen 3.5.0-9.5m
- End cap

RIG: GEO205 **DRILLER:** Ground Test **LOGGED:** RAS **CASING:** HW to 1.3 m
TYPE OF BORING: Solid flight Auger to 1.38 m, NMLC coring to 12.15 m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: Loss of drilling water return below 4.4 m

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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BOREHOLE LOG

CLIENT: SJD NB Pty Ltd
PROJECT: Proposed Shop Top Housing Development
LOCATION: 57-69 Strathallen Avenue, Northbridge

SURFACE LEVEL: 87.6 AHD
EASTING: 334419
NORTHING: 6257320
DIP/AZIMUTH: 90°/--

BORE No: BH1
PROJECT No: 221953.01
DATE: 14/4/2023
SHEET 2 OF 2

RL	Depth (m)	Description of Strata	Graphic Log	Sampling & In Situ Testing: Type	Depth	Sample	Results & Comments	Water	Well Construction Details
77, 76	11, 12	SANDSTONE: fine to coarse grained, red-brown, pale grey and pale grey-brown, high strength, slightly weathered, very thinly bedded, slightly fractured to unbroken, Hawkesbury Sandstone		C	10.72		PL(A) = 1.5		Backfill with bentonite 9.5-12.15m
					11.27		PL(A) = 1.8		
	12.15	Bore discontinued at 12.15m Target depth reached			12.15				
75, 74, 73, 72, 71, 70, 69, 68	13, 14, 15, 16, 17, 18, 19								

RIG: GEO205 **DRILLER:** Ground Test **LOGGED:** RAS **CASING:** HW to 1.3 m
TYPE OF BORING: Solid flight Auger to 1.38 m, NMLC coring to 12.15 m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: Loss of drilling water return below 4.4 m

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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BOREHOLE LOG

CLIENT: SJD NB Pty Ltd
PROJECT: Proposed Shop Top Housing Development
LOCATION: 57-69 Strathallen Avenue, Northbridge

SURFACE LEVEL: 87.1 AHD
EASTING: 334417
NORTHING: 6257301
DIP/AZIMUTH: 90°/--

BORE No: BH2
PROJECT No: 221953.01
DATE: 14/4/2023
SHEET 1 OF 1

Depth (m)	Description of Strata
0.05	ASPHALTIC CONCRETE
0.5	FILL/Gravelly SAND: fine to medium, orange-brown and grey, fine to coarse igneous (roadbase) gravel, with clay nodules mixed in, trace sandstone and ironstone gravel, w~PL
0.9	FILL/Silty SAND: fine to medium, grey, trace igneous and sandstone gravel, apparently well compacted, moist
6.0	SANDSTONE: medium to coarse grained, yellow-brown and red-brown, apparently very low to medium strength, distinctly weathered, Hawkesbury Sandstone
	Bore discontinued at 6.0m Target depth reached

Sampling & In Situ Testing

Type	Depth	Sample	Results & Comments
E*	0.3 - 0.5		No odour
E	0.6 - 0.8		No odour
E	1.0 - 1.3		No odour

Well Construction Details

- Gatic cover
- Concrete 0-0.2m
- Backfill with spoil 0.2-1.8m
- Blank pipe 0.1-3.0m
- Bentonite 1.8-2.3m
- Gravel 2.3-6.0m
- Machine slotted PVC screen 3.0-6.0m
- End cap

RIG: GEO205 **DRILLER:** Ground Test **LOGGED:** SP **CASING:** Uncased
TYPE OF BORING: Solid flight Auger to 6.0m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: *Replicate sample BH0D1 taken from 0.3 - 0.5m

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: SJD NB Pty Ltd
PROJECT: Proposed Shop Top Housing Development
LOCATION: 57-69 Strathallen Avenue, Northbridge

SURFACE LEVEL: 86.6 AHD
EASTING: 334407
NORTHING: 6257303
DIP/AZIMUTH: 90°/--

BORE No: BH3
PROJECT No: 221953.01
DATE: 26/4/2023
SHEET 1 OF 2

Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
0.05	ASPHALTIC CONCRETE: 50mm thick					
0.4	FILL/Gravelly SAND: medium to coarse, dark brown, fine igneous gravel, moist		A A/S			20/50 refusal
1.2	SANDSTONE: medium to coarse grained, red-brown and pale grey, very low strength, Hawkesbury Sandstone		A S			
3.72	SANDSTONE: medium to coarse grained, pale grey, red-brown and orange, bedding dipping 0°-10°, very low strength, highly weathered, slightly fractured, Hawkesbury Sandstone Between 2.15m - 2.21m: indurated iron-stained band	1.87m: J40°, pl, ro, cln 1.94m: B5°, pl, ro, cln 2.07m: J45°, pl, ro, cln 2.13m: J10°, pl, ro, cln 2.17-2.21m: B(x2)5°, pl, he, fe stn 2.48m: B5°, pl, ro, cln 3.65m: B0°, pl, ro, cln	C	100	40	PL(A) = 0 PL(A) = 0.3 PL(A) = 0.23
3.9		3.82m: CORE LOSS: 80mm				
4.62	SILTSTONE: dark grey and pale grey, 40% interbedded fine grained sandstone, medium strength, slightly weathered, slightly fractured, Hawkesbury Sandstone	3.95m: B0°, pl, ro, cln 4.57m: CORE LOSS: 50mm	C	97	98	PL(A) = 0.61
	SANDSTONE: medium to coarse grained, pale grey, bedding dipping 0°-10°, with siltstone laminations, medium strength, slightly weathered, slightly fractured to unbroken, Hawkesbury Sandstone Below 7.3m: high strength	5.05m: B5°, pl, ro, cln 5.18m: B0°-15°, cu, ro, cln 5.24m: B15°, pl, ro, cln 5.59m: B5°, pl, ro, cln 6.03m: B0°, pl, ro, cln 6.08m: B10°, pl, ro, cln 7.13m: B0°, pl, ro, cln 7.29m: B15°, cu, ro, cln 8.84m: B0°, pl, ro, cln	C	100		PL(A) = 0.91 PL(A) = 0.94 PL(A) = 0.8 PL(A) = 0.81
		9.12m: B5°, pl, ro, cln 9.66-9.78m: B(x4)0°-10°, pl, ro, cln	C	97	90	PL(A) = 0.98

RIG: Hanjin D4B8 **DRILLER:** Rockwell **LOGGED:** RAS **CASING:** HW to 15.03m
TYPE OF BORING: Solid Flight Auger to 1.0m, HQ Coring to 15.03m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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BOREHOLE LOG

CLIENT: SJD NB Pty Ltd
PROJECT: Proposed Shop Top Housing Development
LOCATION: 57-69 Strathallen Avenue, Northbridge

SURFACE LEVEL: 86.6 AHD
EASTING: 334407
NORTHING: 6257303
DIP/AZIMUTH: 90°/--

BORE No: BH3
PROJECT No: 221953.01
DATE: 26/4/2023
SHEET 2 OF 2

Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
10.23 10.32	SANDSTONE: medium to coarse grained, pale grey, bedding dipping 0°-10°, with siltstone laminations, high strength, slightly weathered, slightly fractured to unbroken, Hawkesbury Sandstone	10.11m: B0°, pl, ro, cln 10.19m: B10°, pl, ro, cln 10.23m: CORE LOSS: 90mm 10.44m: B0°, ir, ro, cln 11.47m: Cs 15mm	C	97	90	PL(A) = 0.93 PL(A) = 1
		13.44m: B10°, pl, ro, cln 14.09m: B0°, ir, ro, cln 14.64-14.65m: B(x2)0°, pl, he 14.82m: B0°, pl, ro, cly	C	100	100	PL(A) = 1.3 PL(A) = 1.2 PL(A) = 1.2
15.03	Bore discontinued at 15.03m Target depth reached					

RIG: Hanjin D4B8 **DRILLER:** Rockwell **LOGGED:** RAS **CASING:** HW to 15.03m
TYPE OF BORING: Solid Flight Auger to 1.0m, HQ Coring to 15.03m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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BORE: BH3 PROJECT: NORTHBRIDGE APRIL 2023

1.20-5.00m

BORE: BH3 PROJECT: NORTHBRIDGE APRIL 2023

5.00-9.00m

BORE: BH3 PROJECT: NORTHBRIDGE APRIL 2023

9.00-13.00m

BORE: BH3 PROJECT: NORTHBRIDGE APRIL 2023

13.00-15.03m

Appendix D

Laboratory Test Results

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CERTIFICATE OF ANALYSIS 321010-A

Client Details

Client	Douglas Partners Pty Ltd
Attention	Richard Smith
Address	96 Hermitage Rd, West Ryde, NSW, 2114

Sample Details

Your Reference	**221953.00 Northbridge**
Number of Samples	Additional Testing
Date samples received	17/04/2023
Date completed instructions received	01/05/2023

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Please refer to the last page of this report for any comments relating to the results.

Report Details

Date results requested by	09/05/2023
Date of Issue	09/05/2023

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

Asbestos Approved By
Analysed by Asbestos Approved Analyst: Lucy Zhu
Authorised by Asbestos Approved Signatory: Lucy Zhu

Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil			
Our Reference		321010-A-2	321010-A-4
Your Reference	UNITS	BH01	BH02
Depth		0.7-1	0.6-0.8
Date Sampled		14/04/2023	14/04/2023
Type of sample		Soil	Soil
Date prepared	-	09/05/2023	09/05/2023
Date analysed	-	09/05/2023	09/05/2023
pH 1:5 soil:water	pH Units	8.6	8.6
Electrical Conductivity 1:5 soil:water	µS/cm	100	110
Chloride, Cl 1:5 soil:water	mg/kg	10	30
Sulphate, SO4 1:5 soil:water	mg/kg	110	71

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. 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 25°C in accordance with APHA latest edition 2510 and Rayment & Lyons.
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	-			09/05/2023	[NT]	[NT]	[NT]	[NT]	09/05/2023	[NT]
Date analysed	-			09/05/2023	[NT]	[NT]	[NT]	[NT]	09/05/2023	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	100	[NT]
Electrical Conductivity 1:5 soil:water	µS/cm	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	102	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	104	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	114	[NT]

Result Definitions

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

Quality Control Definitions

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

Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.

The recommended maximums for analytes in urine are taken from “2018 TLVs and BEIs”, as published by ACGIH (where available). Limit provided for Nickel is a precautionary guideline as per Position Paper prepared by AIOH Exposure Standards Committee, 2016.

Guideline limits for Rinse Water Quality reported as per analytical requirements and specifications of AS 4187, Amdt 2 2019, Table 7.2

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

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

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

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

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

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

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

Report Comments

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

Appendix E

Results of Groundwater Level Monitoring and Permeability Tests

Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au

Permeability Testing - Rising Head Test Report

Client:	SJD NB Pty Ltd	Project No:	221953.01
Project:	Due Diligence Geotechnical Investigation	Test date:	24-May-23
Location:	57-69 Strathallen Avenue, Northbridge	Tested by:	MVB

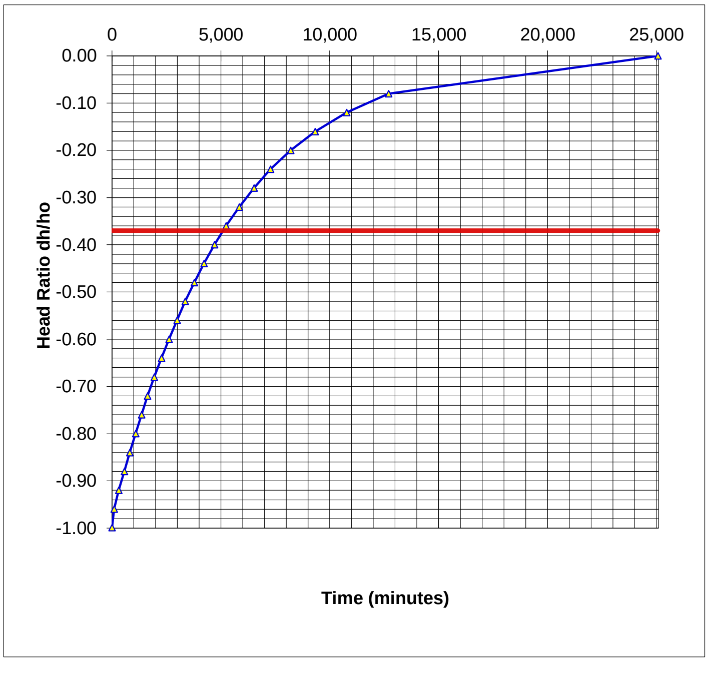
Test Location		**Test No.**	BH1	
Description:	Monitoring well in borehole	Easting:	334419	m
Material type:	Sandstone	Northing	6257320	m
		Surface Level:	87.6	m AHD

Details of Well Installation

Well casing diameter (2r)	76	mm	Head before test commenced	0.764	m
Well screen diameter (2R)	76	mm	Head after recharge	2.319	m
Length of well screen (Le)	7.5	m	Casing stick up above ground	-	m

Test Results

Time (s)	Head (m)	Change in Head: dH (m)	dH/Ho
0	0.764	-1.555	-1
2	0.766	-1.553	-1.00
98	0.827	-1.492	-0.96
308	0.889	-1.430	-0.92
564	0.951	-1.368	-0.88
816	1.013	-1.306	-0.84
1080	1.076	-1.243	-0.80
1352	1.138	-1.181	-0.76
1630	1.199	-1.119	-0.72
1942	1.262	-1.057	-0.68
2272	1.324	-0.995	-0.64
2618.00	1.386	-0.933	-0.60
2986.00	1.449	-0.870	-0.56
3362.00	1.511	-0.808	-0.52
3774.00	1.573	-0.746	-0.48
4218.00	1.635	-0.683	-0.44
4702.00	1.697	-0.622	-0.40
5238.00	1.759	-0.560	-0.36
5842.00	1.821	-0.497	-0.32
6518.00	1.884	-0.435	-0.28
7280.00	1.946	-0.373	-0.24
8208.00	2.008	-0.311	-0.20
9324.00	2.070	-0.249	-0.16
10768.00	2.132	-0.186	-0.12
12706.00	2.195	-0.124	-0.08
25070.00	2.319	0.00	0.00



To = 85.05 mins
5103 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity	**k =**	**1.0E-07**	m/sec
	=	**0.036**	cm/hour

Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au

Permeability Testing - Rising Head Test Report

Client:	SJD NB Pty Ltd	Project No:	221953.01
Project:	Due Diligence Geotechnical Investigation	Test date:	24-May-23
Location:	57-69 Strathallen Avenue, Northbridge	Tested by:	MVB

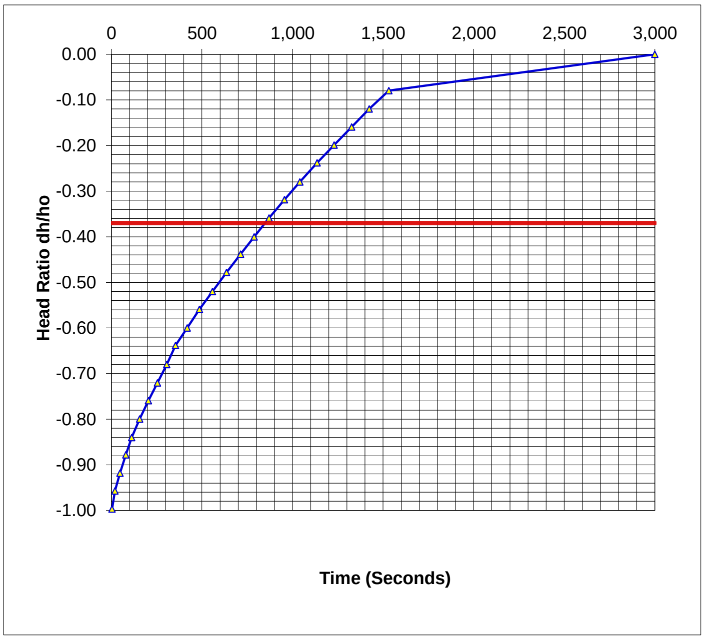
Test Location		**Test No.**	BH2	
Description:	Monitoring well in borehole	Easting:	334417	m
Material type:	Sandstone	Northing	6257301	m
		Surface Level:	87.1	m AHD

Details of Well Installation

Well casing diameter (2r)	110	mm	Head before test commenced	1.650	m
Well screen diameter (2R)	110	mm	Head after recharge	2.190	m
Length of well screen (Le)	3.7	m	Casing stick up above ground	-	m

Test Results

Time (s)	Head (m)	Change in Head: dH (m)	dH/Ho
0	1.650	-0.540	-1
4	1.652	-0.538	-1.00
20	1.673	-0.516	-0.96
48	1.694	-0.495	-0.92
80	1.716	-0.474	-0.88
112	1.737	-0.453	-0.84
156	1.759	-0.431	-0.80
204	1.780	-0.410	-0.76
254	1.801	-0.389	-0.72
306	1.823	-0.367	-0.68
354	1.845	-0.344	-0.64
418.00	1.866	-0.324	-0.60
486.00	1.888	-0.302	-0.56
558.00	1.909	-0.281	-0.52
636.00	1.932	-0.258	-0.48
714.00	1.953	-0.237	-0.44
788.00	1.974	-0.216	-0.40
870.00	1.996	-0.194	-0.36
956.00	2.018	-0.172	-0.32
1040.00	2.039	-0.151	-0.28
1136.00	2.061	-0.129	-0.24
1230.00	2.082	-0.107	-0.20
1326.00	2.104	-0.086	-0.16
1424.00	2.125	-0.065	-0.12
1532.00	2.147	-0.043	-0.08
3000.00	2.190	0.00	0.00



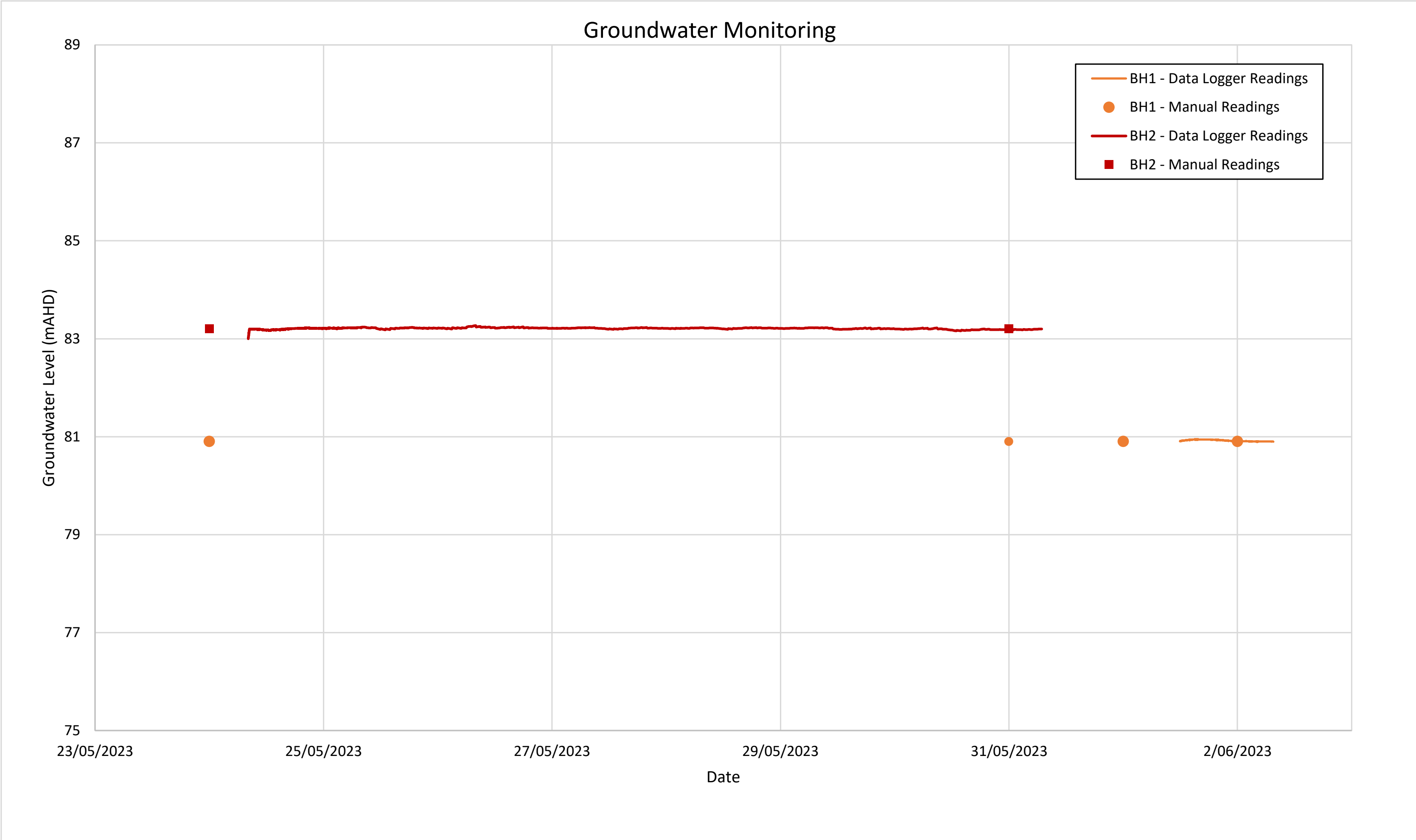
To = 847 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity	**k =**	**2.0E-06**	m/sec
	=	**0.731**	cm/hour



Groundwater Monitoring
89
87
85
83
81
79
77
75
Groundwater Level (mAHD)
23/05/2023
25/05/2023
27/05/2023
29/05/2023
31/05/2023
2/06/2023
Date
BH1 - Data Logger Readings
BH1 - Manual Readings
BH2 - Data Logger Readings
BH2 - Manual Readings