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30 June 2020

Ref: LG1930.02 Geotech Rpt 30-06-20.docx

David Wenkart
President Private Hospital Pty. Limited
369-381 President Avenue
Kirrawee NSW 2232

Via Email: dwenkart@machealth.com.au

Dear David,

Geotechnical Investigation
369-381 President Avenue, Kirrawee, NSW

1. Introduction

Land & Groundwater Consulting Pty Ltd (LG) has been engaged by President Private Hospital Pty. Limited to undertake a geotechnical investigation at 369-381 President Avenue, Kirrawee, NSW (the site). The site comprises 5 blocks of land legally identified as Lot 1 in Deposited Plan (DP) 841502, Lots 23 and 24A in DP 26995, and Lots 53 and 54 in DP 29493.

LG understands that the proposed development will comprise the demolition and alteration of existing structures and addition of new structures with 2 basement levels.

2. Scope and Investigation Findings

The additional geotechnical investigation was conducted on 29 May 2020 by Soilsrock Engineering Pty Ltd. The investigation findings are presented in **Appendix A**. Should you have questions or require further information about this report, please contact the undersigned on (02) 9560 9760 or 0415 726 951.

Yours sincerely,

Gonzalo Parra
Managing Director
Mobile: 0415 726 951
Email: gparra@lgconsult.com.au

Appendix A: Geotechnical Investigation Report

GEOTECHNICAL SITE INVESTIGATION REPORT
FOR
PROPOSED ADDITIONS & ALTERATIONS TO
PRESIDENT PRIVATE HOSPITAL
AT
369-381 PRESIDENT AVENUE
KIRRAWEE, NSW 2232

Report Prepared for: PRESIDENT PRIVATE HOSPITAL

Project No: SRE/564/KW/20

Date: 24/06/2020

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Distribution and Revision Register

Document details

- **Project number:** 564
- **Document number:** SRE/564/KW/20
- **Document Title:** Geotechnical Site Investigation Report for Proposed Additions and Alterations to President Private Hospital
- **Site address:** 369-381 President Avenue, Kirrawee NSW 2232
- **Report prepared for:** PRESIDENT PRIVATE HOSPITAL

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The undersigned, on behalf of SOILSROCK ENGINEERING PTY LTD, confirm that this document and all attached documents, drawings, and geotechnical results have been checked and reviewed for errors, omissions and inaccuracies.

For and on behalf of

Soilsrock Engineering Pty Ltd

Jorge Cabaco

BEng MEng MIEAust CPEng RPEQ NER

Principal Geotechnical Engineer

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APPENDICES

1. INTRODUCTION

This document presents the result of a detailed geotechnical investigation carried out by Soilsrock Engineering Pty Ltd (SOILSROCK) to assist the proposed additions and alterations to President Private Hospital at 369 – 381 President Avenue, Kirrawee NSW 2232. The investigation was commissioned on 17th May 2020 by Mr. Gonzalo Parra from LG Consult who is the representative of the property's owners. The works were conducted in accordance with the email Proposal dated of 10th September 2019 and accepted by email dated of 17th May 2020.

The present report assessment comprises a geotechnical inspection and testing of the existing property and is based on the following documents provided:

- Survey drawings: "PLAN OF THE LAND COMPRISED IN PROPOSED LOT 10 AT KIRRAWEE IN THE LGA OF SUTHERLAND" prepared by DUNLOP THORPE & CO. PTY LTD.
- Preliminary Architectural Drawings prepared by IMAGESCAPE DESIGN STUDIOS as follow: "Proposed Site Context Plan & Site Set Out Plan"; "Basement Plan LVL 1, 2, 3 & 4"; "Ground Floor, First Floor, Second Floor & Roof General Arrangement Plan"; "South & East Elevations"; "North and South Elevations"; "Section 38"; "East/West Sections"; "North/South Sections". DWG. No. A 001 to A 026, from A 100 to A 108; from A300 to A 303; A 401 and A 402.

The purpose of this investigation was to evaluate the subsurface conditions across the site as a basis for comments and recommendations on the following: geotechnical model and ground conditions; excavation and preliminary groundwater assessment; excavation conditions and support design, foundations design and bearing pressures including footings, piling, slabs; filling and pavement requirements.

2. PROPOSED DEVELOPMENT

Based on the architectural drawings provided by the client, the subject site is proposed to be redeveloped into a new five-stories private hospital, including four basement levels within the proposed East Carpark near the southern site boundary close to President Avenue. Vehicular access can be made from Hotham Road at the South-East side of the site which will then lead to the entrance of the proposed basement carpark.

In terms of the proposed structure of the building, the hospital will mainly be divided into North Wing, West Wing and East Wing respectively. On the basement level 1 & 2, it will accommodate around 100 car parking spaces, a bicycle area, ramps leading to the basement level 3 & 4 and stairs combined with lifts connecting to the other floor levels. On the basement level 3 & 4, it will accommodate an additional 61 car parking spaces and stairs combined with lifts connecting to the upper floor levels. On the ground floor, the East wing will accommodate 30 beds, a café, a main entrance and patient drop off area near Hotham Road, a courtyard, admin, staff lunch room, staff changing rooms, storage rooms and fire stairs; for the West wing, there will predominantly be a West wing carpark, rehabitation gym, toilets, existing hydrotherapy pool, education room, consultation rooms, office area, Radiology room and OT1 to OT4; for the North Wing, there will be a storage room, a kitchen, tenancy and lifts connecting to other floor levels. Moving on to the first floor, all wings will accommodate a total of 85 beds, three therapy rooms, a lounge area, a dining area and stairs combined with lifts connecting to other floor levels. Finally, on the second floor, there will only be North Wing and East Wing, both wings will accommodate a total of 66 beds and similarly, three therapy rooms, a lounge area, a dining area and stairs combined with lifts connecting to lower floor levels.

Details of the proposed development are shown on the Architectural drawings provided by IMAGESCAPE DESIGN STUDIOS as referred above.

3. SCOPE OF WORKS

The field works for investigation were carried out on 29th May 2020 and consisted of the following:

- Carry out Dial Before You Dig checks for buried services.
- Conduct an electronic scan by specialized subcontractor to locate and locate buried services.
- Conduct an OH&S and walkover survey to access local topography, geology, and existing site conditions, including exposed soil and rock conditions, vegetation, and surface drainage.
- Photographic record of the site conditions.
- 3 x Dynamic Cone Penetrometer tests (DCP1-DCP3) were carried out to maximum depth of 2.78m by using a 9kg Dynamic Cone Penetrometer specialised steel cone device. The testing followed the procedure as per AS 1289-1997, method 6.3.2.
- Drilling of two boreholes (BH1 & BH2) to depths of 8.46m and 11.60m below existing ground level within the site by using a geotechnical hydraulic drill rig track mounted. All boreholes were initially drilled through soils and very weathered rock by Solid Flight

Auger with Standard Penetration Tests (SPT) “N” values at 1.5m intervals to assess strength characteristics of overburden soils on all boreholes. Further rock coring drilling through the weathered rock by NMLC diamond Coring by 74.8mm (75mm) diameter OD, with core size 51.94mm (52mm) diameter was undertaken in both boreholes.

- Recovery of representative rock core for visual and classification assessment and logging.
- Recovery and collection of rock core samples organised into steel core boxes, for core logging analyses.
- Carry 30 x Point Load Tests (Is_{50}) every 0.5m and on selected rock samples for rock quality and strength classification and allowable bearing pressures assessment.

The field work was conducted and supervised by the full-time presence of a geotechnical professional engineer and an engineering assistant from SOILSROCK, who carried out the testing *in-situ* and recorded the results.

4. RESULTS AND ANALYSES OF THE INVESTIGATION

4.1 Site Location and Description

The subject site is located at 369-381 President Avenue, Kirrawee, NSW 2231, which belongs to the Sutherland Shire Council and is legally described as Lot 1 DP 841502, Lot 24A DP 26995, Lot 23 DP 26995, Lot 54 DP 29493 and Lot 53 DP 29493 respectively. The project site is situated within both SP1 (Special Activities) and R2 (Low Density Residential) land zoning areas. It is delimited by Hotham Road at the East, South by President Avenue, West by 383 President Avenue and North by 12 Bidurgal Avenue, 10 Bidurgal Avenue, 8 Bidurgal Avenue, 6 Bidurgal Avenue, 59 Hotham Road and Bidurgal Avenue, Kirrawee NSW. The site has a irregular shape, topography of the site is sloping upward, approximately 20-30^{o} from South-East to North-West with a combined area of approximately 9,519.86m^{2}. The street frontage is located on President Avenue on the southern boundary. Vehicular access can be made on the southern side of the site via President Avenue and eastern side of the site via Hotham road where the existing car parks are located. The project site is currently used for health services purpose. The surrounding land comprise mostly of residential dwellings on the northern and southern vicinity.

4.2 Regional Geology

From the analysis of Geology of Wollongong – Port Hacking 1:100 000 Geological Series Sheet 9029 - 9129, it is indicated that the site is located within a region of Triassic age, underlain by **Hawkesbury Sandstone (Rh)** formation. The Hawkesbury Sandstone is comprised of medium to coarse-grained quartz sandstone, very minor shale, and laminate lenses. A reproduction of the geological map is shown on following ***Figure 1*** and is based on a portion of the Sydney 1:100 000 Geological Series Sheet 9130 (interactive resource provided by the Geological Survey of NSW), which depicts the site geological condition.

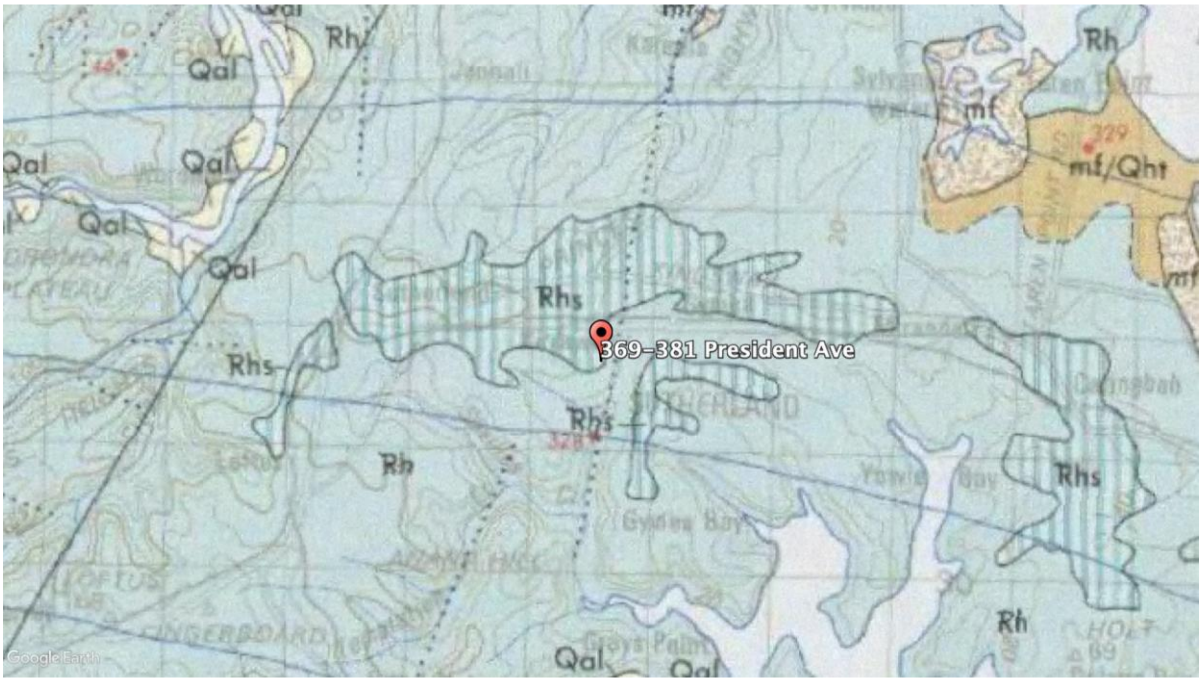

Figure 1 – Portion of the Wollongong – Port Hacking 1:100,000 Geological Series Sheet 9029-9129. Site area location is highlighted in a red/black sign.

4.3 Subsurface Investigation

As mentioned above, two boreholes (BH1 & BH2) were drilled on site within the area of the proposed new mixed-use development to investigate the soil and rock ground condition profile to a maximum depth of 11.60m. The boreholes BH1 and BH2 were drilled at the southern site area within the proposed East Wing and on the western part of the site area within the proposed North Wing, respectively.

A summary of result from the site investigation and ground condition encountered along the boreholes are presented in the following ***Table 1 and 2*** and details of the borehole logs and photos of rock coring are given in the ***Appendix D***.

Table 1 – Standard Penetration Tests (SPT) N-Values results within the Boreholes.

Depth (m)	BH1 N-Value (Blows/ 300mm) *	BH2 N-Value (Blows/ 300mm) *
1.5 – 1.95 (SPT1)	31	NR[1]

Notes:

- *SPT values were obtained from the counting blows of the last 300mm of the 450mm carried from the SPT testing.
- "Bouncing" indicates reached top of rock/boulders/very dense sand/concrete/steel or in some cases can be due to presence of other hard obstacles like rubbles, flouters, or cobbles.
- NR: Not Recorded – SPT tests were not carried out, only auger drilling.
- 1: Residual sandstone was encountered at 1.0m at BH2 test location.

Table 2 – Geotechnical subsurface interpretation by SPT results.

Depth (m)	BH1 Soil Type Consistency	BH2 Soil Type Consistency
1.5 – 1.95	Very Dense Sand	NR*

Notes:

- NR – Not Recorded - SPT tests were not carried out only auger drilling or rock core drilling,
- *Residual Sandstone was encountered at 1.0m depth at BH2 test location.

Point Load Strength Index (Is50) testing was carried out on 30 samples of the rock core obtained from the borehole's profiles BH1 and BH2 of the present investigation, to assist rock quality and strength classification.

The result of the tests within the borehole logs referred above, are presented on the following ***Table 3***.

Table 3 - Point Load Strength Index Test Results (BH1, BH2)

Is50 (MPa)	Inferred Rock Strength	No. of Tests
0.1 – 0.3	Low	1
0.3 – 1.0	Medium	8
1.0 – 3.0	High	21

The following ***Figure 2*** presents the axial point load strength results plotted against reduced level. The results of axial point load testing indicated Is(50) results of 0.280 MPa to 2.465 MPa in sandstone, corresponding to low to high strength sandstone. Based on a typical ratio of Is (50) to unconfined compressive strength (UCS) of 1: 16 to 20 in Hawkesbury Sandstone, this corresponds to UCS values of between 5.6 MPa to 49.3 MPa, and average results of 27.45Mpa.

The ***Figure 2*** below indicates that the strength profile generally increases with depth.

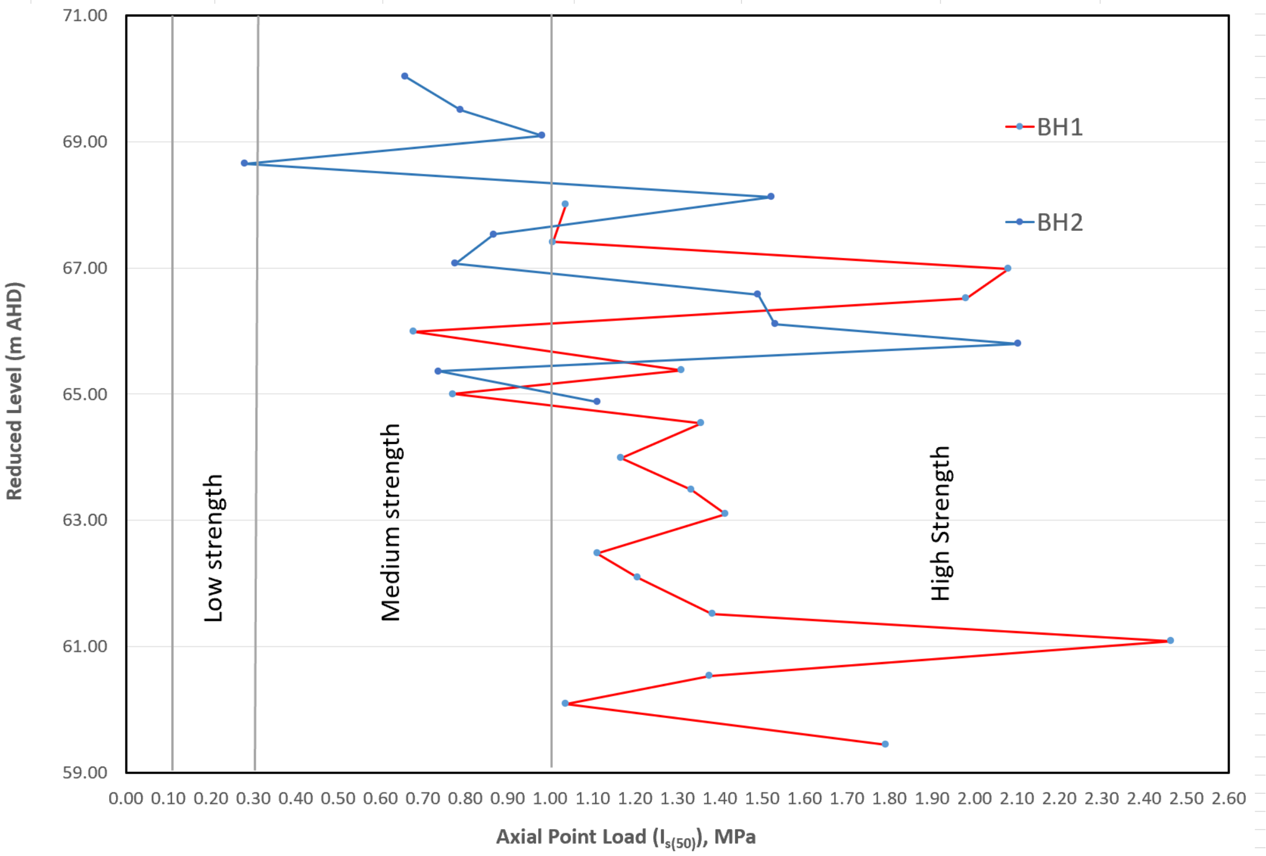

Figure 2 – Axial Point Load Tests Strength Results Plotted against Depth for Boreholes BH1 and BH2.

The following ***Table 4*** provides a summary of the Soil and Rock Profiles in the relevant Boreholes in relation to the present site investigation.

Table 4 – Summary of Soil and Rock Profiles within the Boreholes.

Layer	BH1	BH2
	Depth to top of stratum in boreholes (m) [Reduced Level mAHD]	
Pavement – Concrete	0.00 [70.84]	0.00 [73.15]
Sand – L	0.10 [70.74]	0.10 [73.05]
Sand – MD	0.55 [70.29]	-
Sand – VD	1.00 [69.84]	-
Sandstone – Rock VL	-	1.00 [72.15]
Sandstone – Rock L	-	4.40 [68.75]
Sandstone – Rock M	4.75 [66.09]	2.80 [70.35] / 5.5 [67.65] / 7.50 [65.65]
Sandstone – Rock H	2.60 [68.24] / 6.50 [64.34] / 9.15 [61.69]	4.65 [68.50] / 6.45 [66.70] / 7.90 [65.25]

Notes:

- **Sand Relative Density Description:** VL= Very Loose, L= Loose, MD= Medium Dense, D= Dense, VD= Very Dense.
- **Rock Strength Description:** EL= Extremely Low, VL= Very Low, L= Low, M= Medium, H=High.

In addition, three Dynamic Cone Penetrometer (DCP1 to DCP3) tests were carried out to examine the soil strength to complement the present investigation in relation to subsurface ground conditions.

The following ***Tables 5 and 6*** describe generically the interpolated principal soil strata observed according to the results obtained from the DCP tests conducted on site.

Table 5 - Dynamic Cone Penetrometer tests result.

Depth (m)	DCP 1	DCP 2	DCP 3
0.0 – 0.3	2	3 Bouncing @ 0.3m	3
0.3 – 0.6	15	-	8
0.6 – 0.9	52 Bouncing @0.875m		3
0.9 – 1.2	-		31
1.2 – 1.5			48
1.5 – 1.8			25
1.8 – 2.1			47
2.1 – 2.4			35
2.4 – 2.7			36
2.7 – 3.0			60 Refusal @2.78m

Equipment & Procedure Notes:

- Equipment used: 9kg hammer, 510mm drop distance, conical tip: Standard used: AS1289.6.3.2 - 1997; the total number of blows are considered for 300mm penetration steps.
- 60 defined as “Practical Refusal”, no further penetration and “solid” ringing sound from slide hammer, which may indicate reaching into “Hard” clay layer or “Very Dense” sand layer or on top of bed rock/boulder/obstacles.
- *Bouncing” indicates reached top of rock/boulders/obstacles/concrete/steel or in some cases can be due to presence of a hard obstacle such as steel, rubble, flouters, boulders, cobbles or hard materials.
- * Drop due to self-weight of the device; ** Reached first practical refusal at 2.95m at DCP1 but appears to be only a very thin layer of very dense sand or an hard obstacle.

Table 6 - Geotechnical subsurface interpretation by DCP results

Depth (m)	DCP 1	DCP 2	DCP 3
0.0 – 0.3	Very Loose Sand	Loose Sand	Loose Sand
0.3 – 0.6	Medium Dense Sand	-	
0.6 – 0.9	Very Dense Sand		
0.9 – 1.2	-		Dense Sand
1.2 – 1.5			Very Dense Sand
1.5 – 1.8			Dense Sand
1.8 – 2.1			Very Dense Sand
2.1 – 2.7			Dense Sand
2.7 – 3.0			Very Dense Sand

Notes:

- No sample was provided by DCP test, thus the geotechnical interpretation above is based only on the observation carried through the soil traces left attached to the rods and tip; this interpretation is only indicative, and some soils characteristics can be difficult to identify properly without samples.
- “Probably on top of rock” indicates reached top of rock or in some cases can be due to presence of a hard obstacle such as steel, rubble, flouters, boulders, cobbles, or hard materials.

4.4 Geotechnical Model

A general geotechnical model of the site has been developed for the subsurface characteristics of the soil and rock based on the boreholes campaign which are summarised in the ***Table 7*** below, and in the form of interpreted geotechnical **Cross-Section A-A’** shown in **Appendix C**. The section shows the depth of overlaying soils, together with the interpreted geotechnical boundaries limits for the underlying rock quality.

Table 7 – Interpreted Geotechnical Model.

Unit		Material Description	Thickness of Unit (m)	Top of Unit by Depth (m) [Reduced level- mAHD]
Unit 1		**TOPSOIL/ASPHALT:** The topsoil materials are brown/dark brown silty sand mixed with grass roots, dry and fine-grained.	0.1	0.00 [73.15] / 0.00 [70.84]
Unit 2		**SAND:** The materials are dry, light brown/grey/white, fine to medium-grained silty sand. Loose to Very Dense.	0.9 – 2.5	0.1 [73.05] / 0.1 [70.74]
Unit 3 Bedrock Sandstone	Unit 3A	**SANDSTONE:** Low to Medium Strength, White/Pink Residual Sandstone. Cl. V/IV	0.24-1.8	1.00 [72.15] / 3.45 [69.7]
	Unit 3B	**SANDSTONE:** Medium to High strength, slightly weathered, Class III Sandstone.	0.39 – 1.44	2.70 [68.24] / 4.60 [68.55] / 4.75 [66.09] / 9.10 [61.74]
	Unit 3C	**SANDSTONE:** Medium to High strength, slightly weathered to fresh rock, Class II/I Sandstone.	0.65 – 3.96	2.80 [70.35] / 3.69 [69.46] / 6.04 [67.11] / 3.81 [67.03] / 5.14 [65.70] / 9.43 [61.41]

Notes:

The unit thickness and base of unit values are based on the borehole logs and may not represent extreme (maximum and minimum) values across the site. Rock classification is based on Pells et.al (1998) and Bertuzzi an Pells (2002).

The ***Table 8*** below assesses the strength of the relevant soils materials crossed by the DCP tests, based in *situ tests* results, soil classification, visual interpretation, and extrapolation. For detailed description of the subsurface conditions, explanation sheets about geotechnical parameters are presented in ***Appendix A***.

Table 8 – Recommended Geotechnical Design parameters for Soil (Sand).

Depth Range (m)	Material Conditions	Allowable Extrapolated Bearing Pressure (kPa)
0.0 – 0.3	Very Loose	50
0.3 – 0.9	Loose	70
0.9 – 2.7	Dense	360
>2.7	Very Dense	500

Notes:

- The geotechnical parameters interpretation and extrapolation is based and limited to the DCP test carried on site, which are only indicative for design proposes.
- Allowable extrapolated bearing pressures and strength values are only indicative, these will need to be properly confirmed on site in further geotechnical site inspections to confirm properly bearing pressures and soil and rock quality at the locations.
- NR = Not recommended

The interpreted depth at the upper surface of the various bedrock classes are shown in following ***Tables 9 & 10***, it should be noted that the profiles are accurate at borehole location only, and some degree of variation must be expected away from the borehole locations.

Table 9 – Summary of Geotechnical Model for Rock (Sandstone).

Rock Class	Depth to Top of Various Rock Classes in Boreholes (m) [Reduced Levels- m AHD]	
	BH1	BH2
Top of Borehole	0.00 [70.84]	0.00 [73.15]
Sandstone Class V/IV	-	1.00 [72.20] / 3.45 [69.70]
Sandstone Class III	2.70 [68.14] / 4.75 [66.09] / 9.1 [61.74]	4.60 [68.55]
Sandstone Class II	3.81 [67.03] / 5.14 [65.70] / 9.43 [61.41]	2.80 [70.35] / 3.69 [69.46] / 7.50 [65.65]
Sandstone Class I	-	6.04 [67.11]
End of Borehole	11.60 [59.24]	8.46 [64.69]

Notes:

Rock Classification is based on Pells et.al (1998) and Bertuzzi and Pells (2002). Sandstone Classification was adopted.

Table 10 – Recommended Geotechnical Parameters for Rock (Sandstone).

Foundation Stratum	Allowable End Bearing Pressure (kPa)	Ultimate End Bearing Pressure (kPa)	Ultimate Shaft Adhesion (kPa)	Typical Elastic Modulus (MPa)
Sandstone Class V	1,000	3,000	150	50
Sandstone Class IV	2000	4,000	400	100
Sandstone Class III	3,500	15,000	800	350
Sandstone Class II	6000	30,000	1500	900
Sandstone Class I	12000	60,000	3000	2000

Notes:

- Rock Classification and bearing pressures based on P.J.N Pells "Substance and Mass Properties for The Design of Engineering Structures in The Hawkesbury Sandstone" AGM Vol No. 39 September 2004
- Ultimate end bearing pressures values occur at large settlements (>5% of minimum footing dimensions)
- Ultimate shaft adhesion values to depend on clean socket of roughness category R2 or better. Values may have to be reduced because of smear.
- Shaft adhesion applicable to the design of CFA or bored piles, uncased over the rock socket length, where adequate sidewall cleanliness and roughness are achieved.

4.5 Preliminary Groundwater Assessment

Throughout the auguring process, no groundwater was observed to the end of auguring at 2.60m depth within the borehole BH1 and at 2.80m depth within borehole BH2. At deeper levels through the rock core drilling, fluid water circulation was introduced to cut the rock as per normal rock core drilling procedure, therefore groundwater levels detection through rock coring was not possible to evaluate properly.

Likewise, groundwater was not observed through the three DCP tests, however, the DCP2 test detected moist sand material at approximately 0.2m deep, and the DCP3 test also detected moist sand material at 0.9m. For DCP1 test, the materials attached on the DCP rods and conical tip were dry. Groundwater detection by DCP tests could be indicated/interpreted if wet sand materials are attached on the DCP rods and conical tip after its extraction.

Groundwater can only be investigated properly by further geo-hydrological assessment using a proper borehole drilling and water well standpipe installation to monitor groundwater behaviour if required.

5. COMMENTS AND RECOMMENDATIONS

5.1 Excavation and Groundwater Seepage Conditions

As indicated by the preliminary architectural conceptual drawings provided by the client, maximum excavation depth required is to approximately 6-7m to construct the car park basement levels.

Based on the in situ testing the overall excavation it is expected to intersect the sandy soils profile and medium to high rock strength sandstone. Excavation within the soils and Class V/IV rock should be readily achievable using hydraulic excavators with bucket attachments. Excavation in Sandstone Class III or better rock will require the use of heavy ripping equipment, rock-hammers, rock saws etc. Considering the proximity of the neighbouring buildings to the site boundary, and high strength rock excavation, special excavation methods should be employed to reduce the vibrations to minimum levels. Such methods include rock cutting saw “diamond tipped rock cutting saw”, the low noise levels and reduced vibration make them ideal for excavation or demolition projects that are nearby sensitive structures.

Vibration monitoring plan is required to record vibration during excavation works and ensure exaction is carried without damages to the surrounding structures. It is recommended during the excavation and demolition of the existing structures, excavation and construction techniques be adopted without causing more than 5mm/sec vibration limit (Peak Particle Velocity) to the existing and neighbouring residential buildings along the western site boundary.

Regarding the West, North and South sides of the site development areas could not be investigated by geotechnical drilling, the geotechnical ground condition is unknown. Furthermore, it is unclear if medium to high strength rock sandstone would also be encountered during excavation works to the bottom of the bulk excavation levels at those locations. Hence, further investigations after demolition of the existing structures is recommended.

Regarding the adjoining residential house around the site and adjoining roads and footpaths, depending on the demolition and excavation methods, dilapidation survey should be required, so that an accurate record of the existing conditions are documented and mapped prior to the commencement of site works including demolition and excavation. That report surveys will document any defects in the existing buildings so that any claims for damage due to vibration can be accurately assessed.

In addition, a Waste Classification should be carried for all the excavated materials to be disposed in accordance with NSW Environment Protection Authority (EPA) Waste Classification Guidelines Nov 2014, and under the Protection of the Environment Operations Act 1997 (POEO Act). Environmental sampling and chemical laboratory testing will need to be carried out to classify the spoil resulted from the excavation prior to disposal. This includes filling and excavated natural materials (GSW/VENM/ENM) if it is intent to be removed from the site. The type and extent of testing undertaken will depend on the final use or destination of the spoil, and requirements of the site.

5.2 Excavation Support & Shoring Retention Systems

For the construction of the basement car park, vertical excavations are required within the sand’s materials and weathered rock, which are unlikely to be self-supporting for any significant period. Unsupported vertical excavations are not recommended, due to the relatively deep excavations, excavation extend to close site boundaries and rainwater potential issues. Therefore, temporary, and permanent shoring support is required in all the sides of the excavation, except for the side along Hotham Road where the entrance ramp to access the basement car park is located, regarding it is expected that medium to high strength rock is at shallow depths of less than 1m, this side of the site probably can be excavated by using batters of 1 (V): 2(H) if space are allowed.

Shoring Retentions Systems Options

Further to the above prior to excavation commencing, a retaining wall must be installed to maintain the stability of the sands and very low to medium strength rock strength sandstone for the basement’s construction.

There are several retaining wall systems that can be adequate to construct, we do recommend the following options:

- Cantilever contiguous piling wall by CFA (contiguous flight auger) methods, considering diameter piles such 600mm or 750mm dimeter can be considered for excavation depths less than 6m deep to support the sandy soils and low strength sandstone rock without the need to install permanent or temporary anchors. Long sockets into medium to high strength sandstone rock is required. These CFA piles can be used as load bearing piles if founded at appropriate depths. This method would need to consider deep piles to ensure enough embedment in the soil strata to cantilever the maximum excavation high required.

- Anchored contiguous piling wall by CFA (continuous flight auger) with self-drilling temporary anchors methods. This option will allow to significantly reduce the pile depths comparing with the cantilever solution mentioned above. Smaller pile diameters can be considered such as 450mm regarding temporary anchors would be required along all excavation sides. Therefore, written authorization and confirmation by the properties neighbour owners must be obtained to allow its installation and must comply also with Council's anchors policy. Permanent anchors are not required since the retaining walls structures would be only temporary until the concrete slabs and walls of the building are constructed and connected to the temporary retaining structure. Temporary anchors installation below the Hotham Road and President Avenue would need further written authorization from the RMS and/or Council. If ground anchors are not allowed, steel bracing and diagonal props can be considered in lieu of the ground anchors. Steel props and/or bracing can be considered instead of anchors but will bring some issues for slab construction which may delay the construction works.

Conventional strand/bars anchors are not suitable as well as bored piles due to the collapsing nature of the sand materials, instead CFA piles and Self-Drilling Anchors are recommended.

Shotcrete spray could be installed in between piles and at front for final finish if a final and permanent wall is considered.

The ground settlements for the options mentioned above will have no adverse impact on the surrounding properties and infrastructures, providing enough structural elements are considered within the Retaining Piling Shoring Wall Design. Any surcharges load including construction, traffic nearby footings, inclined backfill surface affecting the walls should be considering in the design.

Regarding the constraints of the present geotechnical site investigation, only the west and South part of the entire proposed development area were able to undertake rock core drilling at the time, therefore it is recommended that further investigation to be undertaken at the East and North area of the site to confirm the boreholes and in-situ testing result of the present report are consistent and to gather additional information about the ground condition judging a relatively large-scale development, to formulate an optimal retaining walls and foundation piles design.

Ground Anchors

Temporary ground anchors may need to be used for the temporary lateral restraint of the perimeter piled wall systems during excavation works. It is recommended ground anchors to be designed inclined below the horizontal from 25º to 35º to allow anchorage into the stronger bedrock materials at depth, have a free length equal to their height above the base of the excavation and minimum 3.0m bond length.

Temporary anchors should be proof loaded to 125% of the design working load after installation and locked-off to no more than 80% of the working load. To ensure that lock-off load is maintained and not lost due to creep effects or other causes, periodic checks should be carried out during the construction phase.

To anchor drilling holes must be properly clean and flushed and grouting operations to be undertaken with good anchoring practice using minimum water/cement ratio w/c=0.4 mixed properly in a colloidal high-speed grout mixer.

Also, centralizers must be installed in the anchor's bodies prior installation in the hole to ensure anchors are centralized and has minimum grout cover. It is recommended to carry preliminary anchor testing prior start the anchoring construction works to confirm bond stresses and bond length requirements. Preliminary anchors testing supervised by a qualified geotechnical engineer could allow increased bond stresses to be adopted during construction.

5.3 Foundations – Footings and Piles

Regarding the expected high loads required by the five-storey proposed building, piled footing systems are recommended. Regarding the high loads expected for the project, the piles would be required to be socket into bedrock Sandstone with quality CL. III with minimum allowable bearing pressure of 3,500kPa, subject to rock strength and bearing capacity confirmation/inspection by a professional qualified geotechnical engineer. The boreholes carried for the present site indicates that Sandstone CL. III is achieved below 4.75m deep within BH1 and probably 3.69m deep within BH2. However, ffounding depths must be adjusted and confirmed by the structural loads and foundations type required for the project.

Regarding the nature of the medium to high strength rock sandstone, strip/pad footings are also a suitable foundation type recommended for the site. Strip/pad footings founded within the rock with Cl. III with minimum allowable bearing pressure of 3,500 kPa is also recommended. For foundations located outside of the excavation footprint, CFA piles socketed into the bedrock sandstone, medium strength CL.III with minimum 3,500 kPa allowable bearing pressure are recommended. The allowable bearing capacity for both footings and piles must be confirmed in-situ through a confirmation/inspection carried out by a professional qualified geotechnical engineer to confirm the suitability for the subjected design loads. Founding depths must also be adjusted and confirmed by the structural loads and foundation type required for the project. Once the structural loads and footings/piers sizes have been determined, settlement analyses should be carried out to confirm the suitability of the foundation's solution adopted.

All footing/pile excavations should have their base levelled, clean, and free of any loose material prior to pouring and ground bearing pressures should be checked and confirmed on site by a qualified experienced Geotechnical Engineer. The concrete pouring should occur with the minimum delay to avoid deterioration, if delays are anticipated, it is recommended that the base of the footings be protected with a blinding layer of concrete with minimum strength of 25Mpa.

5.4 Subgrade Preparation for Slab on Ground and Pavements

Slab on Ground

Depending on the loads required, slab-on-grade construction is feasible for basement levels, depending on the ground conditions encountered after excavation, subgrade preparation could be required.

Following bulk excavation, if Sandstone of medium strength is encountered below the basement level, subgrade preparation will not be necessary unless if there is over-excavation requiring replacement levels with engineering fill. However, it is recommended to apply a blinding and levelling granular layer of sand with minimum 100mm thick above the subgrade rock materials prior installation of any plastic membrane and concrete slab specified by the design engineer.

If the subgrade encountered comprises soil or extremely low to very low strength sandstone, a well compacted granular course material (with maximum particle size of 37.5mm) subgrade with maximum 150mm thick layers of crushed recycled concrete or crushed sandstone

(DGB20 or similar) layers it is recommended to install and be properly compacted. The subgrade layers should be compacted using a vibratory roller (minimum 8 tonnes deadweight) to target density ratio of 98% of SMDD. Moistening of each layer will facilitate compaction.

Density/compaction tests should be carried out on each layer to confirm the above specification has been achieved in accordance with AS3798 Guidelines on Earthworks for Commercial and Residential Developments. A qualified geotechnical engineering should supervise on site the subgrade preparation at minimum Level 2 Inspection and Testing as defined in AS3798, Soilsrock Engineering can supervise, testing and certify the works if required.

Pavements

For pavement design, minimum CBR values of the subgrade material must be determined by the design engineer depending on the pavement design type considered.

Depending on the pavement type design, the subgrade depth shall be compacted to achieve minimum relative compaction of minimum dry density ratio of 100% obtained from Standard Compactive Effort "SMDD – Standard Maximum Dry Density", following the same compaction methodology described for slab on ground subgrade preparation.

Above the well compacted subgrade materials a subbase granular course material layer with minimum 150mm thickness by crushed concrete or crushed sandstone (DGB20 or similar) should be installed. Subbase layers should be also compacted using the same compaction methods described above. Final thickness of subbase should be determined by the pavement design. All pavements subgrade and subbase preparation geotechnical inspection and testing minimum level 2 geotechnical inspection and testing should be allowed for all pavements accordingly with AS3798 Guidelines on Earthworks for Commercial and Residential Developments.

5.5 Engineering Fill

If backfill is to support landscaped areas and backfill retaining walls, an engineered fill should be carried comprising 'clean' sandy soils, free of organic matter and contain a maximum particle size of 37.5mm. The engineered fill should be placed in a controlled and engineered manner compacted using a vibrating plate compactor and/or trench roller in layers not more than 150mm for non-sand materials not containing gravel-sized, or not more than 300mm for sand materials for controlled fill following AS2870-2011. Compaction should achieve minimum density index (ID) of 70%, to be proof tested by "DCP" tests Dynamic Cone Penetrometer as described in AS1289.6.3.3.

5.6 Final Comments

Following the above, further geotechnical input is required and summarized as follow:

- Further Geotechnical Investigation of minimum 2 to 3 additional boreholes (along the West, South and Northeast parts of the site property) must be carried out after demolishing the existing buildings to confirm ground conditions interpretation of the present report. Therefore, any retaining wall and foundation piles design carried previously should also be revised and adjusted if required.
- Develop and concept a Piling Shoring Retaining Wall Design solution prior excavation works.
- Dilapidation reports to the adjoining buildings and roads infrastructure prior excavation works.
- Geotechnical monitoring program to control and ensure low vibrations to the neighbor buildings prior start and during the demolition and excavation works if required.
- Geotechnical depths inspections to confirm piling socket for retaining walls stability during construction works.
- Geotechnical monitoring to the wall deflections during excavation works along all wall's sides.
- Geotechnical site inspections to footings and piles to determine and confirm ground bearing pressures during constructions works.
- Geotechnical site inspections for anchoring installation and testing if required.
- Density tests to control all engineered fill material if required.
- Geotechnical site inspections and compaction tests to confirm density targets for subgrade preparation and subbase installation below slab-on-grade and pavements.

Further to the results of the present investigation, and geotechnical recommendations above, providing the works are carried accordingly with this report, experienced qualified professional geotechnical engineer inspect the site to approve the founding levels and carry proper in situ tests, and good engineering and building construction practice is maintained the proposed development is suitable for the site.

Regarding the soils and rock depths with the geotechnical allowable bearing capacities recommended above could vary across the site, the founding depth for foundations and geotechnical conditions for excavation support to be constructed could also vary. Therefore, it is recommended, that an experienced professional and qualified geotechnical engineer inspect the site during the excavation works and foundations installation, should approve the founding levels.

6. LIMITATIONS

The site geotechnical investigation undertaken for the present report is an estimate and interpretation of the characteristics of the soil and rock of the subsurface conditions encountered during the test locations investigated. Geological and geotechnical conditions can be unpredictable or can reveal unforeseen conditions, in other test locations investigated no matter how comprehensive the investigation is.

This present report analyses forms an engineering model interpretation and opinion of the actual subsurface conditions of the points where the tests were carried. The selected in-situ tests results are indicative of the actual conditions encountered. Recommendations are given based on the data testing results and visual interpretation carried by professional geotechnical and geological engineers from this office. Interpretation of the present report by others may differ from the interpretation given, there is the risk the report may be misinterpreted and Soilsrock cannot be held responsible for that reason.

Geotechnical reports rely on factual interpreted and judgement of information based on professional visual interpretation of soils and rock samples, in situ tests and sampling tests, which has some uncertainty due to changing unexpected ground conditions and it is far less exact than other design disciplines. Soilsrock Engineering accepts no responsibility if different unexpected ground conditions occur in locations where the investigations were not carried out.

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APPENDIX A

Geotechnical Explanatory Notes

APPENDIX A – GEOTECHNICAL EXPLANATORY NOTES

The following geotechnical notes are provided, to give a better understanding of the description and classification methods and field procedures used for the interpretation and compilation of this report which is entirely based on the AS 1726-1993 – Geotechnical Investigations.

INVESTIGATIONS METHODS

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 3m for a backhoe and up to 6m for a large excavator. A potential disadvantage of this investigation method is the larger area of disturbance to the site. Samples can be taken from the test pits for soils testing and analyses.

Large Diameter Augers

Boreholes can be drilled using a rotating plate or short spiral auger, generally 3000mm or large in diameter commonly mounted on a standard piling rig. The cuttings are returned to the surface at intervals (generally not more than 0.5m) 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 90-125mm 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 mixed with soils from the sides of the hole. Information from the drilling (as a 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.

Dynamic Cone Penetrometer Tests

Dynamic penetrometer tests (DCP) are carried out by driving a steel rod into the ground using a standard weight of hammer falling a specified distance. As the rood penetrates the soil the number of blows required to penetrate each successive 300mm depth are recorded. Normally there is a depth limitation of 1.2m, but this may be extended in certain conditions by the use of extension rods. A 16mm diameter rod with a 20mm diameter cone end is driven using a 9kg hammer dropping 510mm (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. Also, Correlations with SPT tests can be made for Cohesion less and cohesive soils.

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 Proposes – Test 6.3.1.

The test is carried out in a borehole by driving a 50mm diameter split sample tube under the impact of a 63kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments equal to 450mm in total. The first 150mm increment it not considered for the so-called "N" value (standard penetration resistance), which is taken from the number of blows of the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm may not be practicable and the test will be discontinued. The results are represented in the following example:

- In the case where full penetration is obtained with successive blow counts for each 150mm as follow:
  - 1st Increment (150mm) = 2 blows
  - 2nd Increment (150mm) = 8 blows
  - 3rd Increment (150mm) = 15 blows
  - Representation – 2,8,15 "N" Value = 23
- In the case where the test is discontinued before the full penetration:
  - 1st Increment (150mm) = 20 blows
  - 2nd Increment (100mm) = 40 blows – test interrupted
  - 3rd Increment (150mm) = not carried – test refusal
  - Representation – 20, 40/100 mm "N" Value = 40

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

Correlation between DCP vs SPT for Cohesionless Soils

DCP (Blows/300mm)	SPT Value (Blows/300mm)	RELATIVE DENSITY
0-3	0-4	Very Loose
3-9	4-10	Loose
9-24	10-30	Medium Dense
24-45	30-50	Dense
>45	>50	Very Dense

Correlation Between DCP vs SPT for Cohesive Soils

DCP (Blows/300mm)	SPT Value (Blows/300mm)	CONSISTENCY
0-3	0-2	Very Soft
3-6	2-5	Soft
6-9	5-10	Medium/Firm
9-21	10-20	Stiff
21-36	20-40	Very Stiff
>36	>40	Hard

Continuous Diamond Core Drilling

A continuous core sample can be obtained using a diamond tipped core barrel, usually with a 50mm 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.

Sampling

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

DESCRIPTION AND CLASSIFICATIONS METHODS FOR SOILS AND ROCK

Descriptions include strength or density, colour, structure, soil or rock type and inclusions.

SOIL DESCRIPTIONS

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	0.6 – 63
Sand	0.075 – 0.6
Silt	0.002 – 0.075
Clay	<0.002

Type	Sand & Gravel Particle size
Coarse gravel	36mm – 19mm
Medium gravel	19mm – 6.7mm
Fine gravel	6.7mm – 2.36mm
Coarse sand	2.36mm – 600μm
Medium sand	600μm – 212μm
Fine sand	212μm – 75μm

The proportions of secondary constituents of soils are described as:

Coarse grained soils		Fine grained soils	
%Fines	Modifier	%Coarse	Modifier
<u><</u>5	Omit, or use 'trace'	<u><</u>15	Omit, or use 'trace'
>5 - <u><</u>12	Describe as 'with clay/silt' as applicable	>15 - <u><</u>30	Describe as 'with clay/silt' as applicable
>12	Describe as 'with silty/clayey' as applicable	>30	Describe as 'with silty/clayey' as applicable

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 specified range;
- Uniformly graded – an excess of a particular particle size;
- Gap graded – a deficiency of a particular particle size with the range.

Cohesive Soils

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

Description	Abbreviation	Undrained shears strength (kPa)
Very soft	vs	<u><</u>12
Soft	s	>12 – <u><</u>25
Firm	f	>25 – <u><</u>50
Stiff	st	>50 – <u><</u>100
Very stiff	vst	>100 – <u><</u>200
Hard	h	>200

Cohesionless Soils

Cohesionless soils, such as clean sands, are classified on the basics 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	<u><</u>15
Loose	l	>15 – <u><</u>35
Medium dense	md	>35 – <u><</u>65
Dense	d	>65 – <u><</u>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;
- Transported soils – formed somewhere else and transported by nature to the site;
- Filling – moved by man.

Transported soils may be further subdivided into:

- Alluvium – river deposits;
- Lacustrine – lake deposits;
- Aeolian – wind deposits;
- Littoral – beach deposits;
- Estuarine – tidal river deposits;
- Talus – coarse colluvium;
- Slop wash or Colluvium – transported downslope by gravity assisted by water. Often includes angular rock fragments and boulders.

ROCK DESCRIPTIONS

Rock Strength

Rock strength is defined by the Point Load Strength (Is50) and 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 test procedure is described by Australian Standards 1726. The terms used to describe rocks strength are as follow:

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

*Assumes a ratio of 20:1 for UCS to $IS_{(50)}$

Degree of Weathering

The degree of weathering of rocks is classified as follows:

Term	Abbreviation	Description
Residual	RS	Soil developed on extremely weathered rock; the mass structure and substance are no longer evident.
Extremely weathered	XW	Rock is weathered to such an extent that it has 'soil' properties, i.e. it either disintegrates or can be remoulded in water, but the texture of the original rock is still evident.
Distinctly weathered	DW	Staining and discolouration of rock substance has taken place.
Slightly weathered	SW	Rock substance is slightly discoloured but shows little or no change of strength from fresh rock.
Fresh	FR	No signs of decomposition or staining.

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 <20mm
Highly fragmented	Core lengths of 20 – 40mm with some fragments
Fractured	Core lengths of 40 – 200mm with some shorter and longer sections
Slightly Fractured	Core lengths of 200 – 400mm with some shorter and longer sections
Unbroken	Core lengths mostly >1000mm

Rock Quality Designation

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

$$RQD\ \% = \frac{cumulative\ length\ of\ 'sound'\ coresections\ \geq 100mm\ long}{total\ drilled\ lenght\ of\ section\ being\ assessed}$$

Where 'sound' rock is assessed to be rock of low strength or better. 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 or RQD.

Rock Quality Designation

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

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

LOG SYMBOLS

Moisture Condition - Cohesive Soils:

MC > PL – Moisture content estimated to be greater than plastic limit
MC = PL - Moisture content estimated to be approximately equal to plastic limit
MC < PL - Moisture content estimated to be less than plastic limit

Moisture Condition - Cohesionless Soils:

D – Dry – Runs freely through fingers
M – Moist – Does not run freely but no free water visible on soil surface
W – Wet – Free water visible on soil surface

Strength (Consistency) - Cohesive Soils:

VS – Very Soft – Unconfined compressive strength less than 25 kPa
S – Soft – Unconfined compressive strength 25-50 kPa
F – Firm – Unconfined compressive strength 50-100 kPa
St – Stiff – Unconfined compressive strength 100-200 kPa
VSt – Very Stiff – Unconfined compressive strength 200-400 kPa
H – Hard - Unconfined compressive strength greater than 400 kPa

Density Index/Relative Density - Cohesionless Soils

Symbol	Density Index (ID)	Range %	SPT “N” Value Range (Blows/300mm)
VL	Very Loose	<15	0-4
L	Loose	15-35	4-10
MD	Medium Dense	35-65	10-30
D	Dense	65-85	30-50
VD	Very Dense	>85	>50

SOILS

■ PAVING

▥ TOP SOIL

▥ FILL

▤ CLAY (CL, CH)

░ SILT (ML, MH)

□ SAND (SP, SW)

▩ GRAVEL

▧ SANDY CLAY

▨ SILTY SAND

▧ CLAYEY SAND

▤ SILTY CLAY

ROCKS

▤ SILTSTONE

▩ CLAYEY GRAVEL

🟨 SANDSTONE

🟩 SHALE

DEFECTS AND INCLUSIONS

▥ CLAY SEAM

▧ SHEARED OR CRUSHED SEAM

▩ BRECCIATED OR SHATTERED SEAM/ZONE

▦ IRONSTONE GRAVEL

░ ORGANIC MATERIAL

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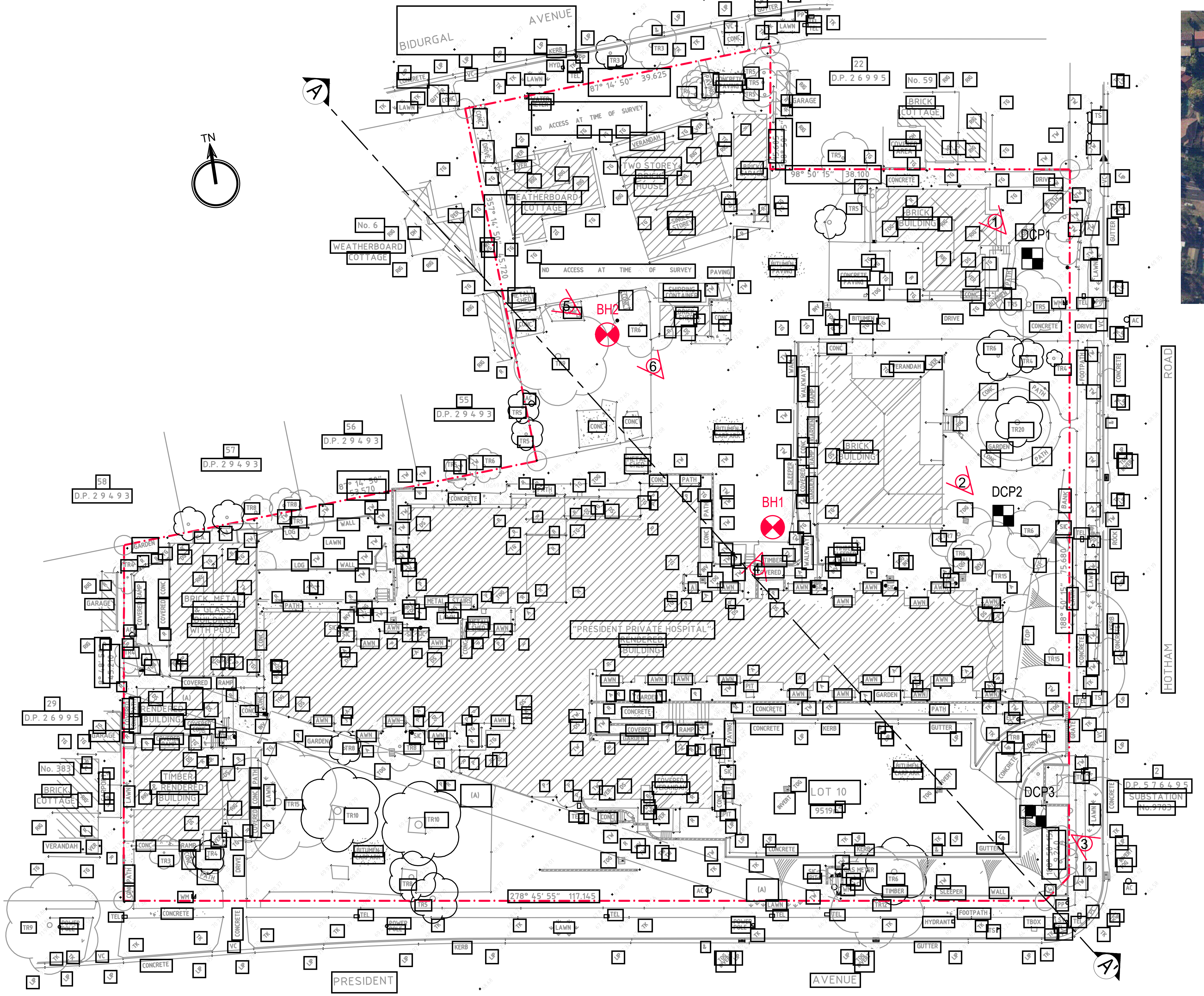
GRAPHIC LOG SYMBOLS

APPENDIX B

DCP tests, Boreholes & Photos Location Plan



Source:
SURVEY DRAWING BY
REGISTERED SURVEYORS NSW
Project N. 41452
Dated 13 / 07 / 15

This Drawing is Used to Illustrate
Boreholes Location Only, and Must Not
Be Used For Any Other Purpose,
Copyright Of Source Drawing Remains
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LEGEND

- · -	SITE BOUNDARY (APPROX.)
▚	DCP (Dynamic Cone Penetrometer)
⊗	BOREHOLE
◁①	PHOTO NUMBER WITH DIRECTION OF VIEW
Ⓐ	CROSS SECTION

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CLIENT:

PRESIDENT PRIVATE HOSPITAL

TITLE:

BOREHOLES, DCP TESTS & PHOTOS LOCATION PLAN
ADDITIONS & ALTERATIONS

369-381 PRESIDENT AVENUE
KIRRAWEE, NSW 2232

Revision	Date

DATE: 24/06/2020

SCALE: 1/500

PROJECT No:
SRE/564/KW/20

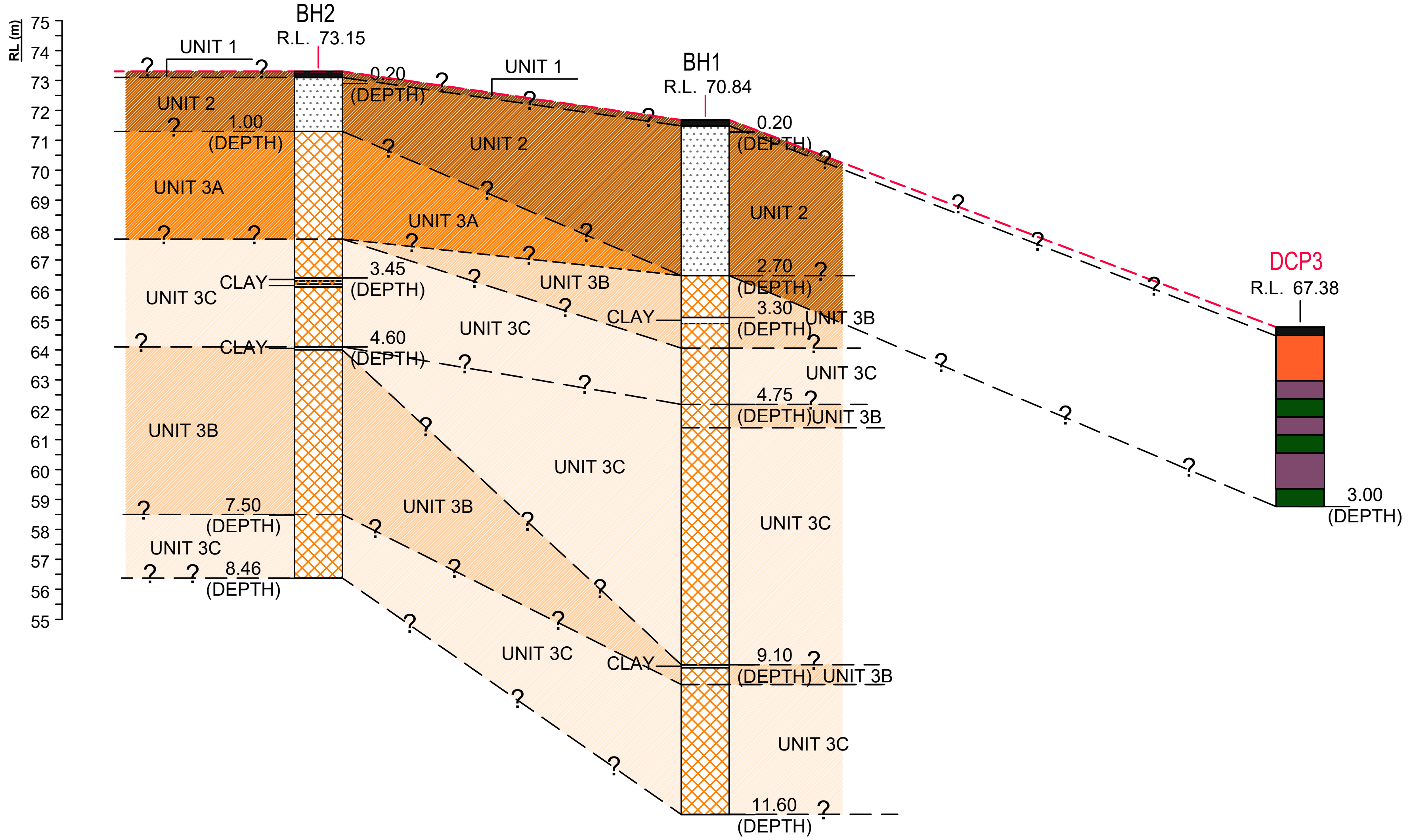
CHECKED BY: JC

DESIGNED BY: MJ

Drawing No: G01

APPENDIX C

Cross Section A-A’



CROSS SECTION A-A'

BOREHOLE LEGEND

-·-·- BASEMENT BEL.
- - ? - - GROUND LEVEL (approx.)
——?—— GEOTECHNICAL UNIT LIMITS
■ TOP SOIL / ASPHALT
?
▩ CONCRETE PAVING
▥ FILL
▤ CLAY
▩ SILT
▦ SAND
▩ GRAVEL
▨ SANDY CLAY
▧ SILTY SAND
▨ CLAYEY SAND
▤ SILTY CLAY
▩ GRAVELY CLAY
▩ CLAYEY GRAVEL
▩ SANDSTONE
▤ SHALE

DCP LEGEND

🟥 VERY LOOSE SAND
🟧 LOOSE SAND
🟫 MEDIUM DENSE SAND
🟪 DENSE SAND
🟩 VERY DENSE SAND

UNIT LEGEND

▨ UNIT 1 - TOP SOILS - SILTY SAND
▨ UNIT 2 - LOOSE TO VERY DENSE SAND
▨ UNIT 3A - SANDSTONE CL. V/IV
▨ UNIT 3B - SANDSTONE CL. III
▨ UNIT 3C - SANDSTONE - CL. II/I

IMPORTANT NOTE:

The geotechnical cross sections presented are a result of a geotechnical interpretation and analyses at the Boreholes location carried only. An inferred correlation of geotechnical units limits between boreholes are carried directly. However, in between boreholes where boreholes were not carried those geotechnical units limits could change and vary. The present geotechnical cross section interpretation its only indicative.

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CLIENT:

PRESIDENT PRIVATE HOSPITAL

TITLE:

CROSS SECTION A-A'
369-381 PRESIDENT AVENUE
KIRRAWEE, NSW 2232

Revision	Date

DATE: 24/06/2020
SCALE: NTS
PROJECT No: SRE/564/KW/20
CHECKED BY: JC
DESIGNED BY: MJ
Drawing No: G02

APPENDIX D

Borehole Logs

GEOTECHNICAL BOREHOLE LOG

CLIENT:	PRESIDENT PRIVATE HOSPITAL	**BOREHOLE NO:**	BH1
PROJECT:	ADDITIONS & ALTERATIONS TO PRESIDENT PRIVATE HOSPITAL	**PAGE:**	1 of 3
LOCATION:	369-381 PRESIDENT AVENUE, 61-65 HOTHAM ROAD, 2-4 BIDURGAL AVENUE KIRRAWEE NSW	**DATE STARTED:**	29/05/2020
DATE:	19/06/2020	**DATE COMPLETED:**	29/05/2020
PROJECT NO:	SRE/564/KW/20	**LOGGED BY:**	AT

Equipment:	BG RIG 3 - HANJIN	**Hole Diameter:**	90mm	**Coring Size:**	-	**RL Surface:**	70.84m
Driller:	BG Drilling	**Drilling Method:**	Solid Flight Auger	**Inclination:**	90˚	**Easting:**	-
						Northing:	-

METHOD	GROUNDWATER RECORD		Field Tests SPT	Sample ID	DEPTH R.L (m)	DEPTH (m)	GRAPHIC LOG	SOIL MATERIAL DESCRIPTION	SOILS CLASSIFICATION MOISTURE CONTENT	 STRENGTH (Consistency, Relative Density)	 DENSITY INDEX	REMARKS AND ADDITIONAL OBSERVATION
SOLID FLIGHT AUGER WITH TC BIT	NO GROUNDWATER OBSERVED	Dry through the Completion of Augering						**ASPHALT:** 100mm thickness of Asphalt	-	-	-	
					70.3	0.5		**SILTY SAND:** Dark grey to light brown silty sands, medium grained.	D	-	L	
								SILTY SAND: Brown silty sands, medium grained.	D	-	M	
					69.8	1.0		**SAND:** White sands, fine grained.	D	-	M	
			SPT1 (13,13,18) Np = 31		69.3 68.8 68.3	1.5 2.0 2.5		**SAND:** White, reddish grey sands, fine grained.	D	-	D to VD	Residual Soils (Sandstone) Low to Medium TC Bit Resistance
					67.8 67.3 66.8 66.3 65.8 65.3 64.8	3.0 3.5 4.0 4.5 5.0 5.5 6.0		**END OF AUGERING @ 2.6m** **PLEASE REFER TO CORE BOREHOLE LOG**				

Comments:		**CHECKED BY:**	JC		
A General Remark:		**APPROVED BY:**	JC	**DATE:**	19/06/2020

GEOTECHNICAL BOREHOLE LOG

CLIENT:	PRESIDENT PRIVATE HOSPITAL	**BOREHOLE NO:**	**BH1**
PROJECT:	ADDITIONS & ALTERATIONS TO PRESIDENT PRIVATE HOSPITAL	**PAGE:**	2 of 3
LOCATION:	369-381 PRESIDENT AVENUE, 61-65 HOTHAM ROAD, 2-4 BIDURGAL AVENUE KIRRAWEE NSW	**DATE STARTED:**	29/05/2020
DATE:	19/06/2020	**DATE COMPLETED**	29/05/2020
PROJECT NO:	SRE/564/KW/20	**LOGGED BY:**	AT

Equipment:	BG RIG 8 - HANJIN	**Hole Diameter:**	76mm	**Coring Size:**	50mm	**RL Surface:**	70.84m
						Easting:	-
Driller:	BG Drilling	**Drilling Method:**	NMLC	**Inclination:**	90˚	**Northing:**	-

METHOD	GROUNDWATER RECORD	BARREL LIFT	TOTAL CORE RECOVERY (TCR)	DEPTH R.L (m)	DEPTH (m)	GRAPHIC LOG	ROCK MATERIAL DESCRIPTION	WEATHERING (RS XW HW MW SW FR)	INFERRED STRENGTH (EL VL L M H VH EH) ● Axial	$Is_{(50)}$ (MPa)	RQD% (0-25 25-50 50-75 75-90 >90)	DEFECT SPACING (mm) (0-60 60-200 200-600 > 600)	DEFECT DESCRIPTION / ADDITIONAL OBSERVATION
				68.8	2.0								
				68.3	2.5		**START CORING @ 2.6m**						
					2.6		**SANDSTONE:** Light grey to light brown sandstone, highly weathered. High strength.	HW	H	1.04		0-60; > 600	(2.65m): Cz(~400mm)
				67.8	3.0								
					3.3		**CLAY:** Reddish brown to dark grey clay seam.						(~3.34m): Cn, 0°, Pl, Sm8, CD, Cn
				67.3	3.5		**SANDSTONE:** Light grey to grey sandstone, highly weathered. High strength.	HW	H	1.01		200-600	
					3.6		**SANDSTONE:** Light grey, reddish grey sandstone, moderately weathered. High strength.	MW	H	2.08		200-600; > 600	(~3.81m): Jt, 15°-20°,PL, Sm8,OP
				66.8	4.0		**SANDSTONE:** Reddish grey, yellowish grey sandstone, moderately weathered. High strength.	MW	H	1.98		> 600	
				66.3	4.5								
					4.85		**SANDSTONE:** Light brown to ligh grey sandstone, slightly weathered. Medium to high strength.	SW	M-H	0.68		> 600; 200-600	(~4.85m): Pt,0°,Pl,Sm8,OP,Cn
				65.8	5.0							> 600	(~5.14m): Pt,0°,Pl,Sm8,Fl,Ct,Qz
				65.3	5.5					1.31			
					5.7		**SANDSTONE:** Light grey sandstone, fresh. Medium to high strength.	FR	M-H	0.77		> 600	
				64.8	6.0					1.36			
				64.3	6.5								

Comments — **CHECKED BY:** JC

A General Remark — **APPROVED BY:** JC **DATE:** 19/06/2020

GEOTECHNICAL BOREHOLE LOG

CLIENT: PRESIDENT PRIVATE HOSPITAL
PROJECT: ADDITIONS & ALTERATIONS TO PRESIDENT PRIVATE HOSPITAL
LOCATION: 369-381 PRESIDENT AVENUE, 61-65 HOTHAM ROAD, 2-4 BIDURGAL AVENUE KIRRAWEE NSW
DATE: 19/06/2020
PROJECT NO: SRE/564/KW/20

BOREHOLE NO:	BH1
PAGE:	3 of 3
DATE STARTED:	29/05/2020
DATE COMPLETED	29/05/2020
LOGGED BY:	AT

Equipment:	BG RIG 8 - HANJIN	**Hole Diameter:**	76mm	**Coring Size:**	50mm
Driller:	BG Drilling	**Drilling Method:**	NMLC	**Inclination:**	90˚

RL Surface:	70.84m
Easting:	-
Northing:	-

ROCK CLASSIFICATION: WEATHERING (RS, XW, HW, MW, SW, FR); INFERRED STRENGTH (EL, VL, L, M, H, VH, EH; ● Axial); Is(50) (MPa); RQD% (0-25, 25-50, 50-75, 75-90, >90); DEFECT SPACING (mm) (0-60, 60-200, 200-600, > 600)

METHOD	GROUNDWATER RECORD	BARREL LIFT	TOTAL CORE RECOVERY (TCR)	DEPTH R.L (m)	DEPTH (m)	ROCK MATERIAL DESCRIPTION	$I_{S(50)}$ (MPa)	DEFECT DESCRIPTION / ADDITIONAL OBSERVATION
						SANDSTONE: Light grey sandstone, fresh. High strength.		
							1.17	
				63.8	7.0			
								(~7.06m): Pt.0°.Pl.Sm8.OP.Cn
							1.33	
				63.3	7.5			
							1.41	
				62.8	8.0			
							1.11	
								(~8.48m): Pt,0°,Pl,Sm8,OP,St
				62.3	8.5			
							1.21	
				61.8	9.0			
						CLAY: Reddish brown to dark grey clay seam.		Cn, 0°, Pl, Sm8, CD, Cn
							1.38	
						SANDSTONE: Light grey sandstonee, fresh. High strength.		
								(~9.43m): Jt, 30°, Un, Sm5, OP, St
				61.3	9.5			
							2.46	
				60.8	10.0			
							1.38	
				60.3	10.5			
							1.04	
				59.8	11.0			
							1.79	
				59.3	11.5			
					11.6			
				70.8		**END OF CORING @ 11.60m**		

Comments

A General Remark

CHECKED BY: AT
APPROVED BY: JC **DATE:** 19/06/2020

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CLIENT:	PRESIDENT PRIVATE HOSPITAL	**TITLE:**	Rock Core Photograph
PROJECT:	ADDITIONS & ALTERATIONS TO PRESIDENT PRIVATE HOSPITAL	**BOREHOLE NO:**	BH1
ADDRESS:	369-381 PRESIDENT AVENUE, KIRRAWEE NSW 2232	**SCALE:**	NTS
PROJECT NO:	SRE/564/KW/20	**DATE RECORDED:**	29/05/2020

CORING STARTED AT 2.6m

CORING TERMINATED AT 11.6m

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GEOTECHNICAL BOREHOLE LOG

CLIENT:	PRESIDENT PRIVATE HOSPITAL	**BOREHOLE NO:**	BH2
PROJECT:	ADDITIONS & ALTERATIONS TO PRESIDENT PRIVATE HOSPITAL	**PAGE:**	1 of 3
LOCATION:	369-381 PRESIDENT AVENUE, 61-65 HOTHAM ROAD, 2-4 BIDURGAL AVENUE KIRRAWEE NSW	**DATE STARTED:**	29/05/2020
DATE:	19/06/2020	**DATE COMPLETED:**	29/05/2020
PROJECT NO:	SRE/564/KW/20	**LOGGED BY:**	AT

Equipment:	BG RIG 3 - HANJIN	**Hole Diameter:**	90mm	**Coring Size:**	-	**RL Surface:**	73.15m
Driller:	BG Drilling	**Drilling Method:**	Solid Flight Auger	**Inclination:**	90˚	**Easting:**	-
						Northing:	-

METHOD	GROUNDWATER RECORD		Field Tests SPT	Sample ID	DEPTH R.L (m)	DEPTH (m)	GRAPHIC LOG	SOIL MATERIAL DESCRIPTION	SOILS CLASSIFICATION MOISTURE CONTENT	SOILS CLASSIFICATION STRENGTH (Consistency, Relative Density)	SOILS CLASSIFICATION DENSITY INDEX	REMARKS AND ADDITIONAL OBSERVATION
SOLID FLIGHT AUGER WITH TC BIT+A14:A133	NO GROUNDWATER OBSERVED	Dry through the Completion of Augering						**TOPSOIL:** Brown/Dark brown silty sand with grass roots.	–	–	–	LOW TC BIT RESISTANCE
					72.7 72.2	0.5 1.0		**SAND:** Light brown/Grey silty sand, fine-grained.	D			
					71.7 71.2 70.7	1.5 2.0 2.5		**SANDSTONE:** White/Pink residual sandstone, medium strength.	–			MEDIUM TO HIGH TC BIT RESISTANCE
					70.2 69.7 69.2 68.7 68.2 67.7 67.2	3.0 3.5 4.0 4.5 5.0 5.5 6.0		**END OF AUGERING @ 2.8m** **PLEASE REFER TO CORE BOREHOLE LOG**				

Comments:		**CHECKED BY:**	JC		
A General Remark:		**APPROVED BY:**	JC	**DATE:**	**19/06/2020**

GEOTECHNICAL BOREHOLE LOG

CLIENT:	PRESIDENT PRIVATE HOSPITAL	**BOREHOLE NO:**	**BH2**
PROJECT:	ADDITIONS & ALTERATIONS TO PRESIDENT PRIVATE HOSPITAL	**PAGE:**	2 of 3
LOCATION:	369-381 PRESIDENT AVENUE, 61-65 HOTHAM ROAD, 2-4 BIDURGAL AVENUE KIRRAWEE NSW	**DATE STARTED:**	29/05/2020
DATE:	19/06/2020	**DATE COMPLETED**	29/05/2020
PROJECT NO:	SRE/564/KW/20	**LOGGED BY:**	AT

Equipment:	BG RIG 3 - HANJIN	**Hole Diameter:**	76mm	**Coring Size:**	50mm	**RL Surface:**	73.15m
						Easting:	-
Driller:	BG Drilling	**Drilling Method:**	NMLC	**Inclination:**	90°	**Northing:**	-

METHOD	GROUNDWATER RECORD	BARREL LIFT	TOTAL CORE RECOVERY (TCR)	DEPTH R.L (m)	DEPTH (m)	ROCK MATERIAL DESCRIPTION	WEATHERING (RS XW HW MW SW FR)	INFERRED STRENGTH (EL VL L M H VH EH) ● Axial	$Is_{(50)}$ (MPa)	RQD% (0-25 25-50 50-75 75-90 >90)	DEFECT SPACING (mm) (0-60 60-200 200-600 > 600)	DEFECT DESCRIPTION / ADDITIONAL OBSERVATION
						START CORING @ 2.8m						
		1st	100%	70.2	3.0	**SANDSTONE:** Light grey/Purple sandstone, moderately weathered, medium strength.	MW	M	0.66	>90	> 600	
						SANDSTONE: Light brown/Light grey sandstone, slightly weathered, medium strength.	SW	M		>90	> 600	(~3.45m): Pt, 0°, Pl, Sm8, FL, Ct, Cly.
				69.7	3.5	**CLAY:** Light grey clay, medium to high plasticity.						
						SANDSTONE: Light grey/White sandstone, slightly weatehred, medium strength.	SW	M		>90	60-200	(~3.56m): Pt, 0°, Pl, Sm8, FL, Ct, Cly.
						CLAY: Light grey/Grey clay, medium to high plasticity.						
						SANDSTONE: Light grey/Light brown sandstone, slightly weathered, medium strength.	SW	M	0.79	>90	60-200 > 600	(~3.69m): Jt, 10°, Pl, Sm8, OP, St.
				69.2	4.0	**SANDSTONE:** Light grey/Red sandstone, slightly weathered, low to medium strength.	SW	M L	0.98 0.28	>90	> 600	
				68.7	4.5							(~4.59m): Pt, 0°, Pl, Sm8, FL, Ct, Cly.
						CLAY: Dark grey/Light grey clay, medium to high plasticity.						
						SANDSTONE: Light grey/Orange sandstone, slightly weathered, high strength.	SW	H	1.52	>90	> 600	
				68.2	5.0							(~5.26m): Pt, 0°, Pl, Sm8, OP, Cn.
		2nd	100%	67.7	5.5	**SANDSTONE:** Light grey/Dark grey sandstone, fresh rock, medium to high strength.	FR	M	0.87	>90	200-600	(~5.61m): Jt, 20°, Pl, Sm8, OP, St.
				67.2	6.0				0.78			(~6.04m): Pt, 0°, Pl, Sm8, OP, St.
				66.7	6.5			H	1.49		> 600	
				66.2	7.0				1.53			
									2.1			
				65.7	7.5							

Comments

A General Remark

CHECKED BY: JC

APPROVED BY: JC **DATE:** 19/06/2020

GEOTECHNICAL BOREHOLE LOG

CLIENT:	PRESIDENT PRIVATE HOSPITAL	**BOREHOLE NO:**	BH2
PROJECT:	ADDITIONS & ALTERATIONS TO PRESIDENT PRIVATE HOSPITAL	**PAGE:**	3 of 3
LOCATION:	369-381 PRESIDENT AVENUE, 61-65 HOTHAM ROAD, 2-4 BIDURGAL AVENUE KIRRAWEE NSW	**DATE STARTED:**	29/05/2020
DATE:	19/06/2020	**DATE COMPLETED**	29/05/2020
PROJECT NO:	SRE/564/KW/20	**LOGGED BY:**	AT

Equipment:	BG RIG 3 - HANJIN	**Hole Diameter:**	76mm	**Coring Size:**	50mm	**RL Surface:**	73.15m
Driller:	BG Drilling	**Drilling Method**	NMLC	**Inclination:**	90˚	**Easting:**	-
						Northing:	-

METHOD	GROUNDWATER RECORD	BARREL LIFT	TOTAL CORE RECOVERY (TCR)	DEPTH R.L (m)	DEPTH (m)	GRAPHIC LOG	ROCK MATERIAL DESCRIPTION	WEATHERING RS XW HW MW SW FR	INFERRED STRENGTH EL VL L M H VH EH	$Is_{(50)}$ (MPa)	RQD% 0-25 25-50 50-75 75-90 >90	DEFECT SPACING (mm) 0-60 60-200 200-600 > 600	DEFECT DESCRIPTION / ADDITIONAL OBSERVATION
		2nd	100%	65.2	8.0		**SANDSTONE:** Light grey/Dark grey sandstone, fresh rock, medium to high strength.	FR	M H	0.74 1.11	>90	> 600	
				64.7	8.5		**END OF CORING @ 8.46m**						
				64.2	9.0								
				63.7	9.5								
				63.2	10.0								
				62.7	10.5								
				62.2	11.0								
				61.7	11.5								
				61.2	12.0								
				60.7	12.5								

● Axial

Comments

A General Remark

CHECKED BY: AT

APPROVED BY: JC **DATE:** 19/06/2020

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CLIENT:	PRESIDENT PRIVATE HOSPITAL	**TITLE:**	Rock Core Photograph
PROJECT:	ADDITIONS & ALTERATIONS TO PRESIDENT PRIVATE HOSPITAL	**BOREHOLE NO:**	BH2
ADDRESS:	369-381 PRESIDENT AVENUE, KIRRAWEE NSW 2232	**SCALE:**	NTS
PROJECT NO:	SRE/564/KW/20	**DATE RECORDED:**	29/05/2020

CORING STARTED AT 2.80m

CORING TERMINATED AT 8.46m

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APPENDIX E

Point Load Test Index Results

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CLIENT:	PRESIDENT PRIVATE HOSPITAL	**PAGE**	1 of 1
PROJECT:	PROPOSED ADDITION & ALTERATION TO PRESIDENT PRIVATE HOSPITA	**TESTED BY:**	A.T
LOCATION:	369-381 PRESIDENT AVENUE, KIRRAWEE NSW 2232	**CHECKED BY:**	J.C
PROJECT NO:	SRE/564/KW/20	**DATE OF RECORDED:**	17/06/2020
TEST METHOD:	RMS T223	**DATE OF ISSUE:**	17/06/2020

POINT LOAD STRENGTH INDEX TEST RESULT REPORT - BH1

SAMPLE ID	DEPTH (m)	Test Type	SAMPLE DESCRIPTION			DIMENSIONS			RESULTS				
			ROCK TYPE	STRUCTURE	MOISTURE	D (mm)	W (mm)	De (mm)	LOAD, P (kN)	FAILURE MODE	Is (MPa)	$Is_{(50)}$ (MPa)	Estimated Strength
BH01-A1	2.84	A	SS	MA	AR	40.00	50.00	50.46	2.63	3	1.033	1.037	H
BH01-A2	3.43	A	SS	MA	AR	42.00	50.00	51.71	2.65	3	0.991	1.006	H
BH01-A3	3.86	A	SS	MA	AR	42.00	50.00	51.71	5.48	3	2.050	2.081	H
BH01-A4	4.32	A	SS	MA	AR	35.00	50.00	47.20	4.53	3	2.033	1.981	H
BH01-A5	4.85	A	SS	MA	AR	40.00	50.00	50.46	1.72	3	0.675	0.678	M
BH01-A6	5.46	A	SS	MA	AR	40.00	50.00	50.46	3.32	3	1.304	1.309	H
BH01-A7	5.84	A	SS	MA	AR	45.00	50.00	53.52	2.14	3	0.747	0.770	M
BH01-A8	6.30	A	SS	MA	AR	40.00	50.00	50.46	3.44	3	1.351	1.356	H
BH01-A9	6.85	A	SS	MA	AR	40.00	50.00	50.46	2.96	3	1.162	1.167	H
BH01-A10	7.36	A	SS	MA	AR	40.00	50.00	50.46	3.38	3	1.327	1.333	H
BH01-A11	7.74	A	SS	MA	AR	35.00	50.00	47.20	3.23	3	1.450	1.413	H
BH01-A12	8.36	A	SS	MA	AR	35.00	50.00	47.20	2.54	3	1.140	1.111	H
BH01-A13	8.75	A	SS	MA	AR	40.00	50.00	50.46	3.06	3	1.202	1.207	H
BH01-A14	9.32	A	SS	MA	AR	42.00	50.00	51.71	3.64	3	1.361	1.382	H
BH01-A15	9.75	A	SS	MA	AR	40.00	50.00	50.46	6.25	3	2.454	2.465	H
BH01-A16	10.31	A	SS	MA	AR	40.00	50.00	50.46	3.49	3	1.371	1.376	H
BH01-A17	10.75	A	SS	MA	AR	40.00	50.00	50.46	2.63	3	1.033	1.037	H
BH01-A18	11.40	A	SS	MA	AR	42.00	50.00	51.71	4.72	3	1.765	1.792	H

NOTATION

Moisture
(W) Wet
(M) Moist
(D) Dry
(AD) As Drilled
(AR) **As Received**

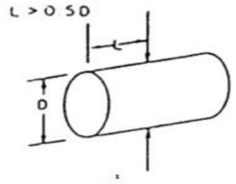
Rock Type
(SS) Sandstone
(ST) Siltstone
(SH) Shale
(G) Granitic
(MSS) Meta Sandstone
(MST) Meta Siltstone

Structure
(MA) Massive
(BE) Bedded
(IB) Interbedded
(LA) Laminated
(CR) Crystalline

Failure Mode
1 - Fracture through fabric oblique to bedding
2 - Fracture along bedding
3 - Fracture through rock mass
4 - Fracpture influenced by pre-existing: (J) Joint Plan, (M) Microfracture, (F) Foliation, (V) Vein
5 - Partial fracture or Chip (Invalid Result)

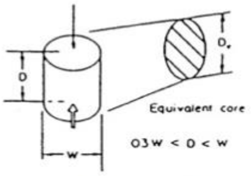
Test Type

D: Diametral Test



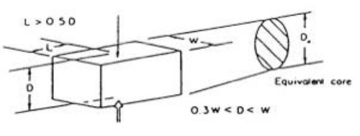
L > 0.5 D

A: Axial Test



0.3W < D < W

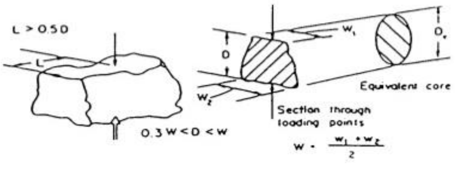
B: Block Test



0.3W < D < W

L > 0.5 D

I: Irregular Lump Test



0.3 W < D < W

L > 0.5 D W = (W1 + W2) / 2

CLIENT: PRESIDENT PRIVATE HOSPITAL
PROJECT: PROPOSED ADDITION & ALTERATION TO PRESIDENT PRIVATE HOSPITA
LOCATION: 369-381 PRESIDENT AVENUE, KIRRAWEE NSW 2232
PROJECT NO: SRE/564/KW/20
TEST METHOD: RMS T223

PAGE 1 of 1
TESTED BY: A.T
CHECKED BY: J.C
DATE OF RECORDED: 17/06/2020
DATE OF ISSUE: 17/06/2020

POINT LOAD STRENGTH INDEX TEST RESULT REPORT - BH2

SAMPLE ID	DEPTH (m)	Test Type	SAMPLE DESCRIPTION			DIMENSIONS			RESULTS				
			ROCK TYPE	STRUCTURE	MOISTURE	D (mm)	W (mm)	De (mm)	LOAD, P (kN)	FAILURE MODE	Is (MPa)	$Is_{(50)}$ (MPa)	Estimated Strength
BH01-A1	3.12	A	SS	MA	AR	40.00	50.00	50.46	1.67	3	0.656	0.659	M
BH01-A2	3.65	A	SS	MA	AR	40.00	50.00	50.46	2.00	3	0.785	0.789	M
BH01-A3	4.06	A	SS	MA	AR	40.00	50.00	50.46	2.49	3	0.978	0.982	M
BH01-A4	4.50	A	SS	MA	AR	40.00	50.00	50.46	0.71	3	0.279	0.280	L
BH01-A5	5.03	A	SS	MA	AR	40.00	50.00	50.46	3.86	3	1.516	1.522	H
BH01-A6	5.62	A	SS	MA	AR	40.00	50.00	50.46	2.20	3	0.864	0.868	M
BH01-A7	6.08	A	SS	MA	AR	40.00	50.00	50.46	1.97	3	0.774	0.777	M
BH01-A8	6.57	A	SS	MA	AR	40.00	50.00	50.46	3.78	3	1.484	1.491	H
BH01-A9	7.04	A	SS	MA	AR	40.00	50.00	50.46	3.88	3	1.524	1.530	H
BH01-A10	7.35	A	SS	MA	AR	42.00	50.00	51.71	5.54	3	2.072	2.104	H
BH01-A11	7.79	A	SS	MA	AR	40.00	50.00	50.46	1.87	3	0.734	0.737	M
BH01-A12	8.28	A	SS	MA	AR	40.00	50.00	50.46	2.82	3	1.107	1.112	H

NOTATION

Moisture
(W) Wet
(M) Moist
(D) Dry
(AD) As Drilled
(AR) **As Received**

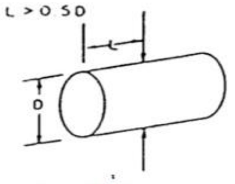
Rock Type
(SS) Sandstone
(ST) Siltstone
(SH) Shale
(G) Granitic
(MSS) Meta Sandstone
(MST) Meta Siltstone

Structure
(MA) Massive
(BE) Bedded
(IB) Interbedded
(LA) Laminated
(CR) Crystalline

Failure Mode
1 - Fracture through fabric oblique to bedding
2 - Fracture along bedding
3 - Fracture through rock mass
4 - Fracpture influenced by pre-existing: (J) Joint Plan, (M) Microfracture, (F) Foliation, (V) Vein
5 - Partial fracture or Chip (Invalid Result)

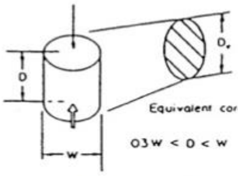
Test Type

D: Diametral Test



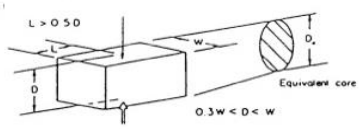
L > 0.5 D

A: Axial Test



0.3W < D < W

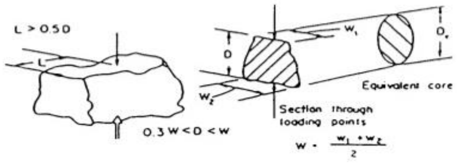
B: Block Test



0.3W < D < W

L > 0.5 D

I: Irregular Lump Test



0.3 W < D < W

L > 0.5 D   W = (W1 + W2) / 2

APPENDIX F

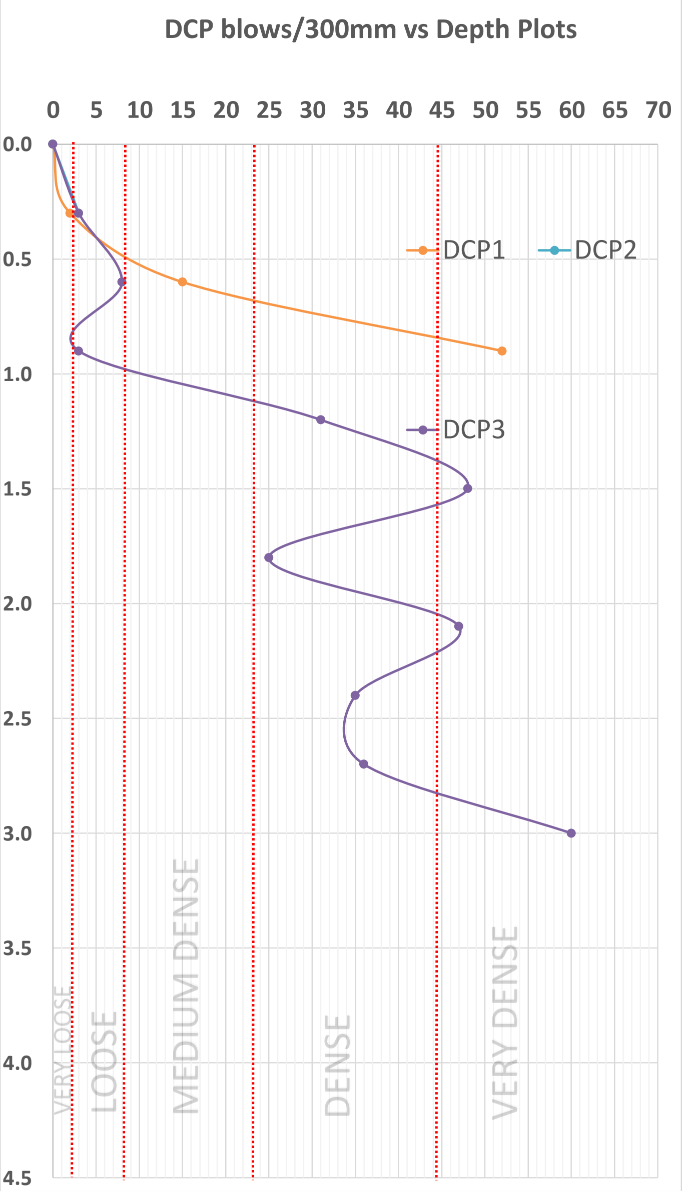
DCP Tests Graphics

Dynamic Cone Penetrometer Test (DCP)

Client:	PRESIDENT PRIVATE HOSPITAL	**Page:**	1 of 1
Project:	PROPOSED ADDITION & ALTERATION TO PRESIDENT PRIVATE HOSPITAL		
Location:	369-381 PRESIDENT AVENUE, KIRRAWEE NSW 2232	**Date Started:**	29/05/2020
Date:	23/06/2020	**Date Completed:**	29/05/2020
Project No.:	SRE/564/KW/20	**Logged/Checked by:**	AT/JC

Equipment: 9kg Dynamic Cone Penetrometer **Standards:** AS 1289.6.3.2 - 1997

Item	Depth (m)	Np (blows/300mm)			
		DCP1	DCP2	DCP3	
1	0.0 - 0.3	2	3	3	
2	0.3 - 0.6	15	Bouncing @ 0.3m	8	
3	0.6 - 0.9	52		3	
4	0.9 - 1.2	Bouncing @ 0.875m		31	
5	1.2 - 1.5			48	
6	1.5 - 1.8			25	
7	1.8 - 2.1			47	
8	2.1 - 2.4			35	
9	2.4 - 2.7			36	
10	2.7 - 3.0			60	
				Refusal @ 2.78m	



Note: * Reach practical refusal 60 blows per 300mm

"Bouncing" indicates reached top of rock/boulders/obstacles/concrete/steel or in some cases can be due to presence of a hard obstacle such as steel, rubble, flouters, boulders, cobbles or hard materials

APPENDIX G

Site Photographs

CLIENT:	PRESIDENT PRIVATE HOSPITAL	**PAGE:**	1 of 1
PROJECT:	GEOTECHNICAL SITE INVESTIGATION REPORT FOR PROPOSED ADDITIONS & ALTERATIONS TO PRESIDENT PRIVATE HOSPITAL	**DATE RECORD:**	29/05/2020
LOCATION:	369-381 PRESIDENT AVENUE, KIRRAWEE NSW 2232		
DATE:	23/06/2020	**LOGGED BY:**	AT
PROJECT NO.:	SRE/564/KW/20	**CHECKED BY:**	JC

SITE PHOTOGRAPHS

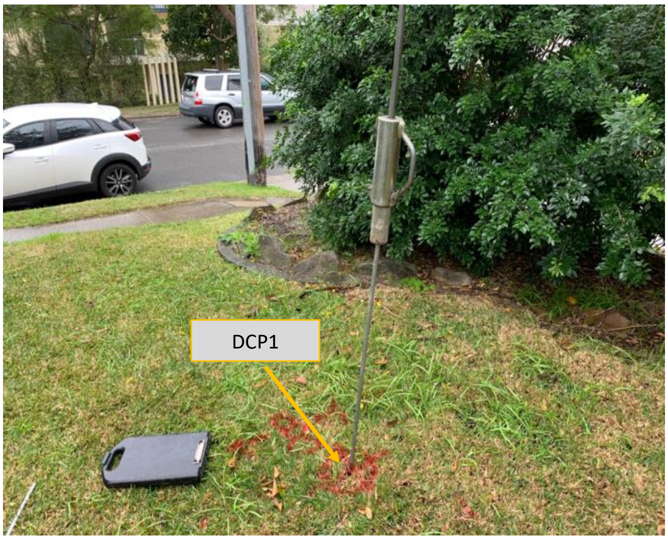


Photo 1 - South-East view to DCP1 test location.

Photo 2 - South-East view to DCP2 test location**.**

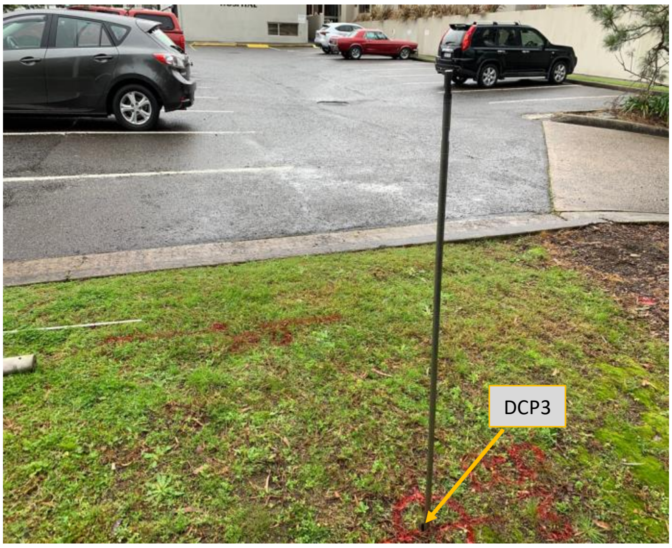


Photo 3 - North-East view to DCP3 test location.

Photo 4 - North-East view to BH1 test location**.**

Photo 5 - South-East view to BH2 test location.

Photo 6 - South-East view to the existing car park.