GEOTECHNICAL SITE INVESTIGATION REPORT
FOR
PROPOSED NEW MIXED-USE
RESIDENTIAL/COMMERCIAL DEVELOPMENT
AT
28 FISHER ROAD & 9 FRANCIS STREET
DEE WHY, NSW 2099

Report Prepared for: THE GEORGE GROUP PTY LTD

Project No: SRE/524/DW/19

Date: 07/05/2020

Soilsrock Engineering Pty Ltd

5/110 Oaks Avenue, Dee Why NSW 2099

M 0457 1150 44 **T** 02 8065 2922

E info@soilsrock.com.au **W** www.soilsrock.com.au

Distribution and Revision Register

Document details

- **Project number:** 524
- **Document number:** SRE/524/DW/19
- **Document Title:** Geotechnical Site Investigation Report for Proposed New Mixed-Use Residential/Commercial Development
- **Site address:** 28 Fisher Road & 9 Francis Street, Dee Why, NSW 2099
- **Report prepared for:** THE GEORGE GROUP PTY LTD

Document status and review

Revision	Prepared by	Reviewed by	Approved by	Date issued
0	AT	JC	J.C	07/05/2020

Distribution of copies

Revision	Electronic	Paper	Issued to
0	1	0	Philip George (THE GEORGE GROUP PTY LTD)

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

soilsrock
geotechnical | environmental | foundations

TABLE OF CONTENTS

APPENDICES

1. INTRODUCTION

This document presents the results of a detailed geotechnical investigation carried out by Soilsrock Engineering Pty Ltd (SOILSROCK) to assist the proposed mixed-use residential/commercial development at 28 Fisher Road & 9 Francis Street, Dee Why, NSW 2099. The investigation was commissioned on 17th April 2020 by Mr. Philip George from The George Group Pty Ltd who is the Architect and representative of the property's owners. The works were conducted in accordance with the Letter Proposal Ref: SRE/524/DW/19 DATED OF 13th June 2019 accepted by email dated of 17th April 2020.

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

- Survey drawings: "DETAIL SURVEY AT 28 FISHER ROAD DEE WHY, NSW" prepared by DA SURVEYS.
- Conceptual Architectural Drawings prepared by The George Group as follow: "Proposed Generic Site Plan"; "Proposed Site Basement Carpark"; "Proposed Site Section indicating general intent of the development (single level basement to around 2.5-3.5m deep maximum).

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 information provided within the correspondences, the subject site is proposed to develop for a new mixed-use residential and commercial 5 storey building, including one lower basement level. No structural drawings and final architectural drawings were provided by the client.

3. SCOPE OF WORKS

The field works for investigation were carried out on 21st April 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 4.15m 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 ranging between approximately 9.97m to 10.90m 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 27 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 28 Fisher Road & 9 Francis Street, Dee Why, NSW 2099, which belongs to the Northern Beaches Council and is legally described as Lot 28 DP 7413 and Lot 43 DP 7413, respectively. The project site is situated within both B4 (mixed-use) and R3 (Medium Density Residential) land zoning areas. It is delimited by Fisher Road at the East, South by 22-26 Fisher Road, West by Francis Street and North by 108/30 Fisher Road and 11 Francis Street. The site is rectangular in shape, topography of the site is relatively flat with a combined area of approximately 1,391.2m^2. The street frontage is located on Fisher Road on the eastern boundary. Vehicular access can be made on the western side of the site via Francis Street where the existing car park is located. The project site is currently used for commercial purpose. The surrounding land comprise mostly of residential and commercial buildings on the northern and southern vicinity.

4.2 Regional Geology

From the analysis of Geology of Sydney 1:100 000 Geological Series Sheet 9130, 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 Sydney 1:100,000 Geological Series Sheet 9130. 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 10.90m. The boreholes BH1 and BH2 were drilled at the eastern site area close to Fisher Road and on the existing car park at the western part of the project site towards Francis Street, 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 (SPT_1)	Refusal Bouncing @ 1.88m	7
3.00 – 3.95 (SPT_2)	-	Refusal Bouncing @ 3.21m

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 due to presence of other hard obstacles like rubbles, flouters, or cobbles.
- NR: Not Recorded – SPT tests were not carried out, only auger drilling.

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	Loose Sand
3.00 – 3.95	NR	Very Dense Sand/ Extremely weathered Sandstone*

Notes:

- NR – Not Recorded - SPT tests were not carried out only auger drilling or rock core drilling,
- ***Residual Sandstone was encountered at 3.2m depth in BH2.

Point Load Strength Index (Is50) testing was carried out on 27 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
0.03 – 0.1	Very Low	1
0.1 – 0.3	Low	8
0.3 – 1.0	Medium	18

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 0.044 MPa to 0.965 MPa in sandstone, corresponding to very low to medium 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 0.88 MPa to 19.3 MPa, and average results of 10.09Mpa.

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

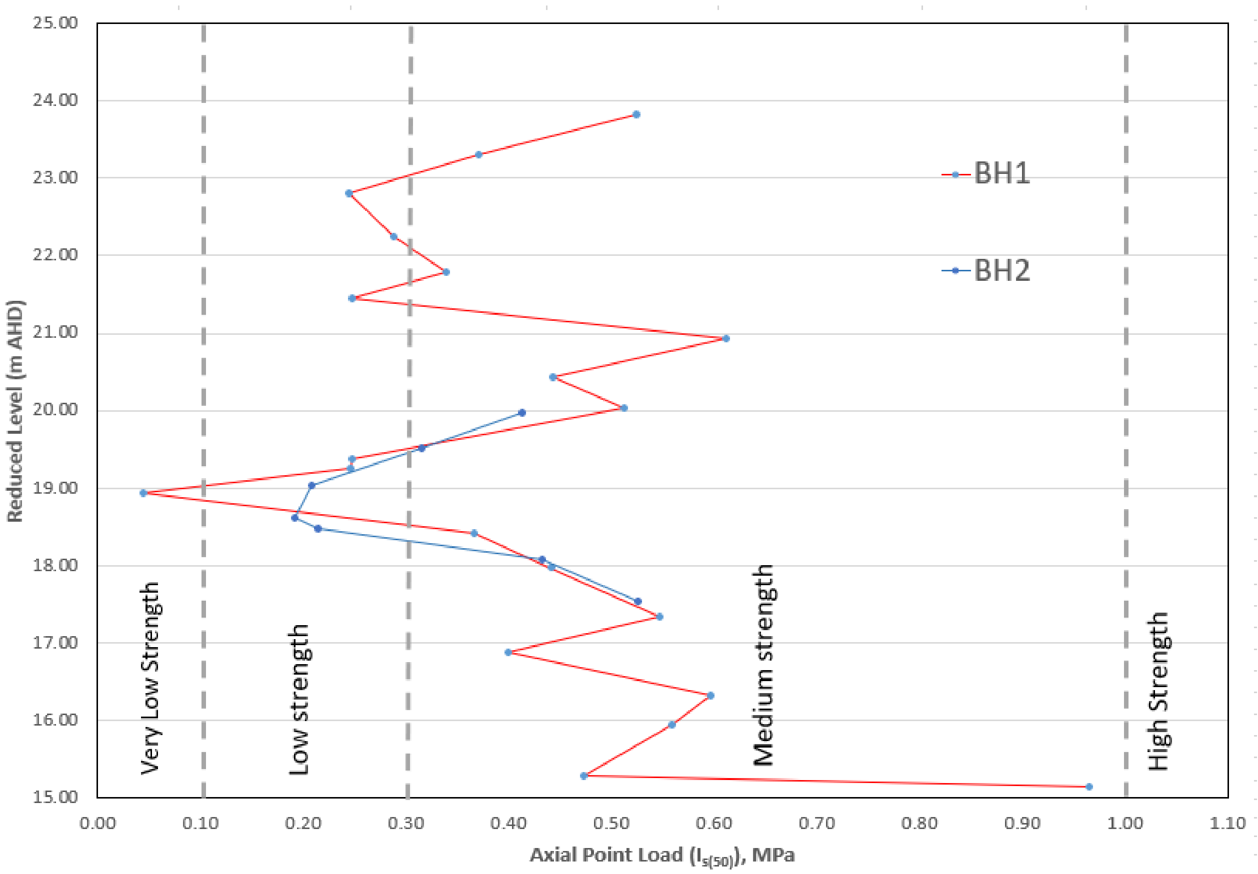


Figure 2 – Axial Point Load Tests Strength Results Plotted against Depth for Boreholes BH1 and BH2.

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 [26.00]	0.00 [27.30]
Sand – L	-	1.50 [25.80]
Sand – D	0.30 [25.70]	-
Sand – VD	1.50 [24.50]	3.00 [24.30]
Sandstone – Rock EL	6.90 [19.10] / 10.80 [15.20]	3.20 [24.10]
Sandstone – Rock L	3.10 [22.90] / 4.45 [21.55] / 6.50 [19.50]	8.00 [19.30]
Sandstone – Rock M	1.92 [24.08] / 4.00 [22.00] / 4.95 [21.05] / 7.16 [18.84] / 9.20 [16.80] / 10.85 [15.15]	7.06 [20.24] / 9.10 [18.20]

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	8	14	32
0.3 – 0.6	12	3	24
0.6 – 0.9	8	8	16
0.9 – 1.2	13	16	8
1.2 – 1.5	0*	25	15
1.5 – 1.8	5	27	29
1.8 – 2.1	14	28	32 Bouncing @ 1.95m
2.1 – 2.4	26	23	-
2.4 – 2.7	40	41	
2.7 – 3.0	60**	60 Bouncing @ 3.0m	
3.0 – 3.3	47	-	
3.3 – 3.6	40		
3.6 – 3.9	53		
3.9 – 4.2	60 Practical Refusal @ 4.15m		

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

<table>
<tr><th>Depth (m)</th><th>DCP 1</th><th>DCP 2</th><th>DCP 3</th></tr>
<tr><td>0.0 – 0.3</td><td>Medium Dense Sand</td><td>Dense Sand</td><td rowspan="3">Very Dense Sand</td></tr>
<tr><td>0.3 – 0.6</td><td>Dense Sand</td><td>Loose Sand</td></tr>
<tr><td>0.6 – 0.9</td><td>Medium Dense Sand</td><td>Medium Dense Sand</td></tr>
<tr><td>0.9 – 1.2</td><td>Dense Sand</td><td rowspan="7">Very Dense Sand</td><td>Medium Dense Sand</td></tr>
<tr><td>1.2 – 1.5</td><td>Very Loose Sand</td><td>Dense Sand</td></tr>
<tr><td>1.5 – 1.8</td><td>Medium Dense Sand</td><td rowspan="2">Very Dense Sand</td></tr>
<tr><td>1.8 – 2.1</td><td>Dense Sand</td></tr>
<tr><td>2.1 – 2.4</td><td rowspan="7">Very Dense Sand</td><td rowspan="7">-</td></tr>
<tr><td>2.4 – 2.7</td></tr>
<tr><td>2.7 – 3.0</td></tr>
<tr><td>3.0 – 3.3</td><td rowspan="4">-</td></tr>
<tr><td>3.3 – 3.6</td></tr>
<tr><td>3.6 – 3.9</td></tr>
<tr><td>3.9 - 4.2</td></tr>
</table>

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		**SAND:** The materials are dry, light brown/grey/orange, fine-grained clayey sand. Loose to Very Dense.	1.9 – 3.2	0.00 [25.70]
Unit 2 Bedrock Sandstone	Unit 2A	**SANDSTONE:** Extremely low to Very Low Strength, Residual Sandstone, Class V Sandstone.	0.26 – 3.8	3.20 [24.10] / 6.90 [19.10]
	Unit 2B	**SANDSTONE:** Low to medium strength, slightly to highly weathered, Class IV Sandstone.	0.16 – 2.18	5.24 [20.76] / 6.50 [19.50] / 7.00 [20.30] / 7.50 [19.80] / 8.75 [17.25]
	Unit 2C	**SANDSTONE:** Low to medium strength, slightly to highly weathered, Class III Sandstone.	0.2 – 3.34	1.90 [24.10] / 5.90 [20.10] / 7.16 [18.84] / 7.30 [20.00] / 8.91 [17.09] / 9.68 [17.62]

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	Medium Dense	200
0.3 – 0.6	Loose	50
0.6 – 1.2	Medium Dense	200
1.2 – 1.5	Very Loose	NR
1.5 – 1.8	Medium Dense	100
1.8 – 2.1	Dense	300
>2.1	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 [26.00]	0.00 [27.30]
Sandstone Class V	6.90 [19.1]	3.20 [24.1]
Sandstone Class IV	5.24 [20.76] / 6.50 [19.50] / 8.75 [17.25]	7.00 [20.30] / 7.50 [19.80]
Sandstone Class III	1.92 [24.08] / 5.90 [20.10] / 7.16 [18.84] / 8.91 [17.09]	7.30 [20.00] / 9.68 [17.62]
End of Borehole	10.90 [15.10]	9.97 [17.33]

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

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 1.90m depth within the borehole BH1. 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.

During the drilling by auger for borehole BH2, groundwater was observed at approximately 6.3m depth. Through the DCP tests groundwater was not observed, however, the DCP1 detected moist sand material at 3.9m deep, and the DCP2 detected also moist sand material at 3.7m. For DCP3 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 3-4m to construct the lower basement level car park.

Based on the in situ testing the overall excavation it is expected to intersect the sandy soils profile and extremely low rock strength sandstone, and very low to medium rock strength sandstone materials. 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.

Accordingly, with the preliminary groundwater assessment described above and as indicated, high flow groundwater seepage is not expected during the excavation for the lower basement level construction to 3-4m deep. However low rates of groundwater seepage could occur at the base of the excavation, following heavy rain events, which could recharge the groundwater in deep. Groundwater behaviour could be confirmed by installing a stand-pipe water well and carry further groundwater monitoring if required.

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 lower basement car park level, 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 Francis Street where the entrance ramp to access the basement car park is located, this side of the site can be excavated by using batters of 1 (V): 2(H) if space are allowed.

<u>Shoring Retentions Systems Options</u>

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:

– <u>Cantilever Contiguous CFA Piles (Contiguous Flight Auger) Wall with minimum 600/750mm diameter,</u> this option considers minimum 600/750mm diameter piles (depending on the design modelling calculations results) can be installed as a cantilever depending on the piles spacing considered to support the soils/rock without the need to install permanent or temporary anchors or propping systems. These CFA piles can be used as load bearing piles if founded at appropriate depths.

This method would need to consider deep piles regardless, to ensure enough embedment in a competent rock stratum to cantilever the maximum excavation high required of approximately 3-4m. Shotcrete spray could be installed in between piles and at front for final finish, if a permanent wall is considered.

- Anchored Contiguous CFA Piles (Contiguous Flight Auger) Wall with 450mm or 600mm diameter combined by temporary anchors or props. This option will allow to reduce significantly the pile depths comparing with the cantilever solution mentioned above. Similarly, with the option above, these CFA piles can be used as load bearing piles if founded at appropriate depths.

  Temporary anchors would be required along the surrounding existing properties, therefore, written authorization and confirmation by the property owners must be obtained to allow its installation and must comply with Council's Policies. Temporary anchors below to the existing roads are also required depending on the length and inclination of the anchors design, an authorization by the Council or RMS would be required.

  Permanent anchors are not required since the retaining wall structures would be only temporary until the concrete slabs and permanent walls of the building are constructed and connected to the retaining structure. Props can be considered instead of anchors but will bring some issues for slabs construction which can delay the construction works. Shotcrete spray could be installed in between piles and at front for final finish if a permanent wall is considered.

Earth Pressures

For the design of shoring system, limit the deformation and deflection occurring outside the excavation are the major consideration in selecting earth pressures.

Earth pressures will be affecting the excavation faces retained regarding they are temporarily or permanently retained, from the ground surface along the sands down to the weathered rock materials. The ***Table 11*** below provides preliminary coefficient of lateral earth pressures for retaining design support which are based on horizontal ground surface for the soils and rock horizons encountered during the geotechnical investigation.

Table 11 – Preliminary Coefficients of Lateral Earth Pressure for Excavation Support

Material	Bulk Unit Weight (kN/m^3)	Effective Friction Angle (°)	Coefficient of Lateral Earth Pressure at Rest (K_0)	Coefficient of Active Earth Pressure (K_a)	Coefficient of Passive Earth Pressure (Kp)
Unit 1 – Sand	19	25	0.58	0.41	2.46
Unit 2A - Class V Rock – Sandstone	20	26	0.56	0.39	2.56
Unit 2B - Class IV Rock – Sandstone	22	28	0.53	0.36	2.77
Unit 2C - Class III Rock – Sandstone	23	34	0.41	0.26	3.85

Any surcharges load including construction, traffic nearby footings, inclined backfill surface affecting the walls should be considering in the design. Drainage of the ground behind impermeable walls should be provided otherwise the wall should be designed for full hydrostatic pressures.

For passive restraint, rock sockets below the bulk excavation level, should have a minimum length of three pile diameters below the lowest level of any nearby excavation and socket into competent rock strength.

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.

The following ***Table 12*** presents the allowable average bond stresses at the grout-rock interface for design of temporary ground anchors to install for the support of piled wall systems.

Table 12 – Geotechnical Anchor Design Bond Stresses.

Material Description	Allowable Average Bond Stress (kPa)
Class V Rock - Sandstone	150
Class IV Rock – Sandstone	250
Class III Rock – Sandstone	800

To apply the parameters above it is assumed that the anchor drilling holes are 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 good quality with minimum 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 7.20m deep within BH1 and probably 9.7m deep within BH2. However, founding depths must be adjusted and confirmed by the structural loads and foundations type required for the project.

Regarding the nature of the sandy soils CFA piles are the suitable foundation type recommended for the site. Groundwater could also be encountered within the rock fractures which could be easily overcome using CFA type piles. However, if casings are used to ensure the holes not collapsing, bored piles could be also considered, but will be probably a slow process, and careful consideration must be taken during the pile constructions to ensure the holes are dewatered and clean prior the concrete pour (groundwater present within the rock fractures could be an issue for bored piles).

CFA piles must be designed taking into consideration skin friction, end bearing, and should also be socket into minimum rock CL. III with minimum 3,500 kPa allowable bearing pressure as mentioned above. 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 6-8 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.

For pavements designs where the subgrade is clay material a depth of 500mm should be considered for static/medium loads and rigid pavement types. For static/light loads and rigid/flexible pavement types 750mm subgrade depth should be 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:

- Regarding the geotechnical inconsistency of the two boreholes, the reasonable distance between boreholes location and reasonable size, in order to confirm the unknow ground conditions specially in between boreholes and below the foot print of the existing buildings, it is strongly recommended to undertake an additional geotechnical investigation after the buildings demolition.

- Carry pit tests along the northern side of the site to check the foundations levels of the footings of the adjoining neighbor building, to confirm if underpinning works are necessary to carry out. These pit tests could be carried after demolition and be part of the scope for the additional geotechnical investigation mentioned above.
- Develop and concept a Piling Shoring Retaining Wall Design solution prior excavation works.
- Dilapidation reports to the adjoining building 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.

This document is for use only of the party to whom it is addressed and for no other purpose. No responsibility is accepted to any third party who may use or rely on the whole or any part of the content of this document. If this report is altered in any way, or not reproduced in full, no responsibility will be taken for this. This is report is only valid upon all costs related with the field works and reporting has been settled and released by the client.

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**
≤5	Omit, or use 'trace'	≤15	Omit, or use 'trace'
>5 - ≤12	Describe as 'with clay/silt' as applicable	>15 - ≤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	≤12
Soft	s	>12 – ≤25
Firm	f	>25 – ≤50
Stiff	st	>50 – ≤100
Very stiff	vst	>100 – ≤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	≤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;
- 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

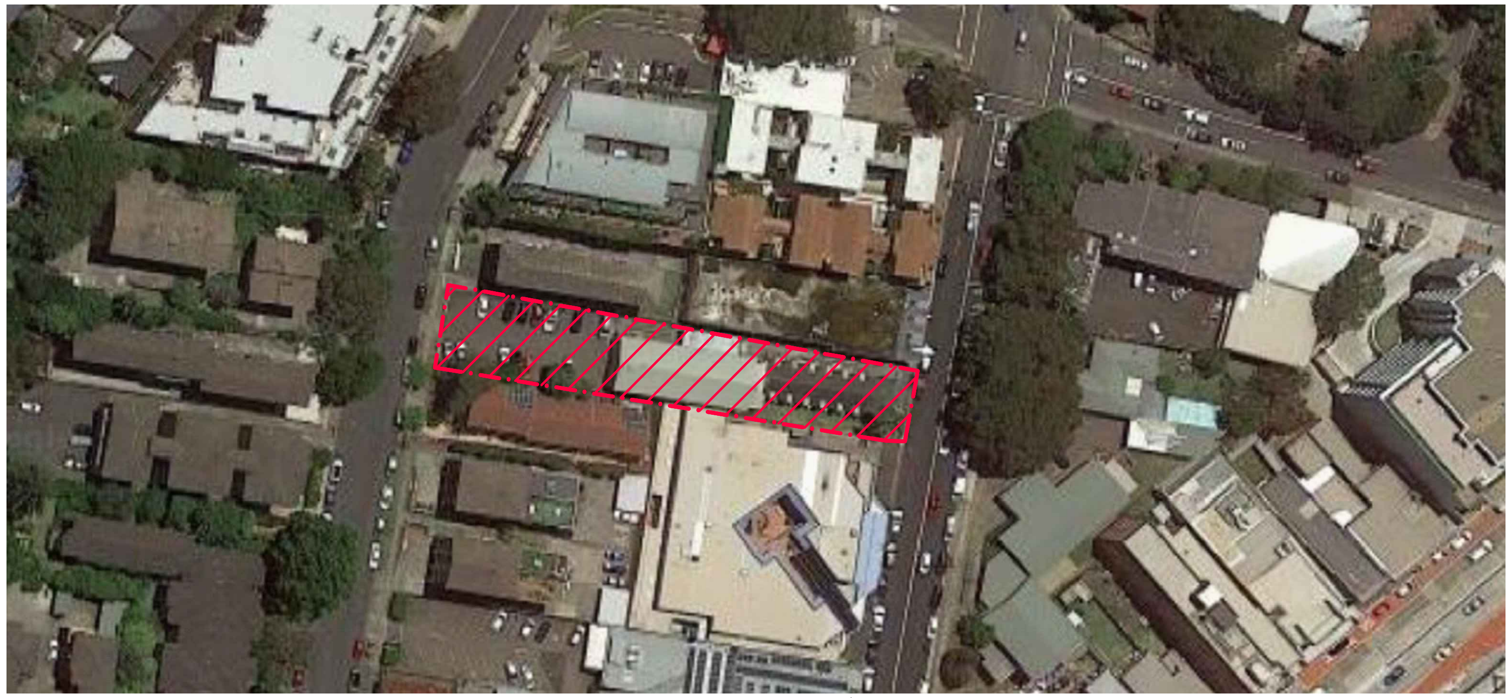
Soilsrock Engineering Pty. Ltd.
2A/32 Fisher Road, Dee Why, NSW 2099
M: 0457 115 044 | T: (02) 8065 2922
E-mail: info@soilsrock.com.au
www.soilsrock.com.au

GRAPHIC LOG SYMBOLS

soilsrock
geotechnical | environmental | foundations

APPENDIX B

DCP tests, Boreholes & Photos Location Plan



LEGEND

- SITE BOUNDARY (APPROX.)
- BOREHOLE
- DCP (Dynamic Cone Penetrometer)
- PHOTO NUMBER WITH DIRECTION OF VIEW
- CROSS SECTION

soilsrock
geotechnical | environmental | foundations

Soilsrock Engineering Pty. Ltd
2A/32 Fisher Road, Dee Why, NSW 2099
M: 0457 115 044 | T: (02)8065 2922
Email: info@soilsrock.com.au
www.soilsrock.com.au

CLIENT:

THE GEORGE GROUP PTY LTD

TITLE:

DCP'S, BORE HOLES & PHOTOS LOCATION PLAN
PROPOSED NEW MIXED USE
RESIDENTIAL/COMMERCIAL
28 FISHER ROAD & 9 FRANCIS STREET
DEE WHY, NSW 2099

Revision	Date

DATE: 11/05/2020

SCALE: NTS

PROJECT No: SRE/524/DW/20

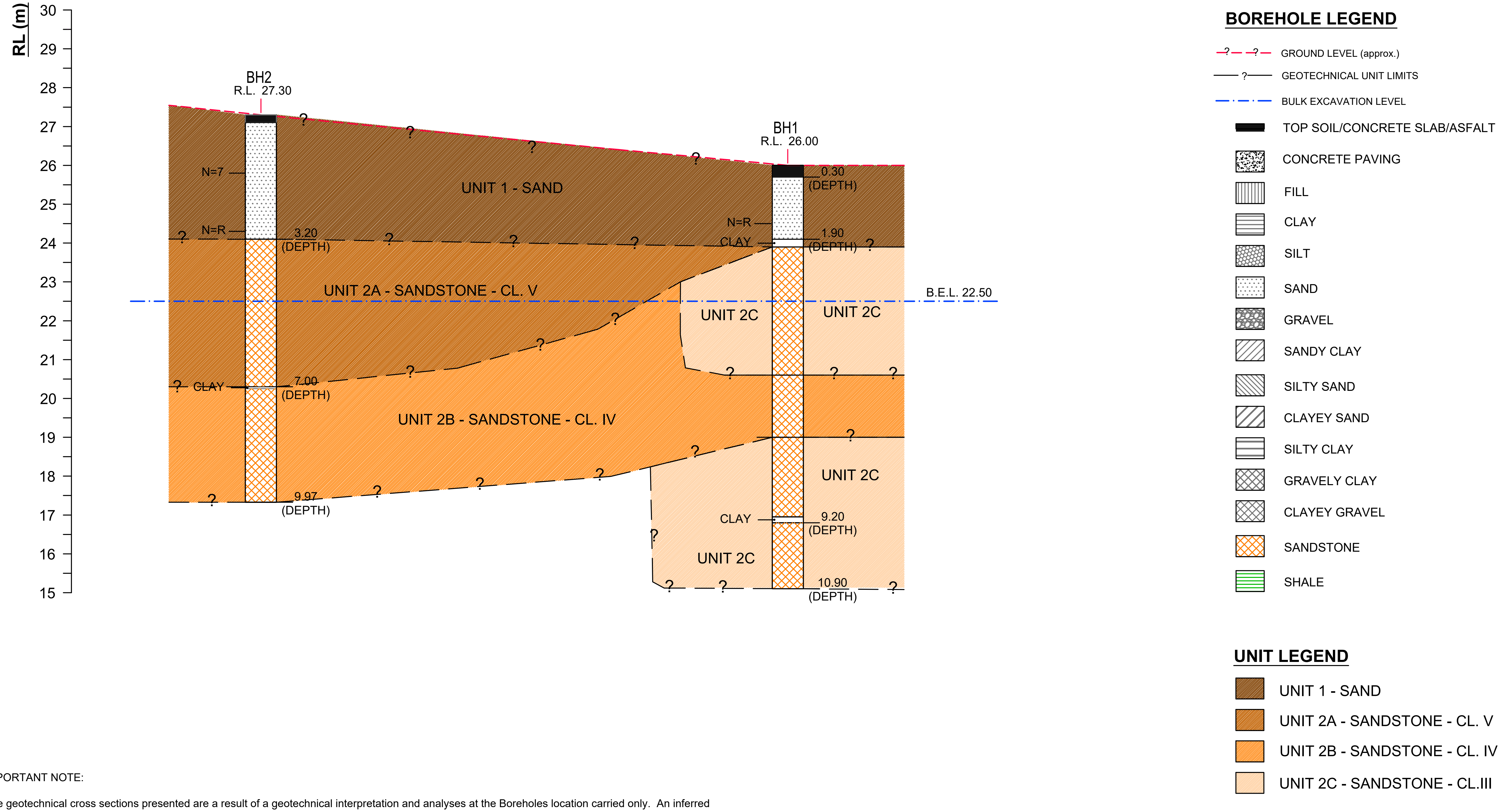
CHECKED BY: JC

DESIGNED BY: MJ

Drawing No: G01

APPENDIX C

Cross Section A-A’



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.

Soilsrock Engineering Pty. Ltd
2A/32 Fisher Road, Dee Why, NSW 2099
M: 0457 115 044 | T: (02)8065 2922
Email: info@soilsrock.com.au
www.soilsrock.com.au

CLIENT: THE GEORGE GROUP PTY LTD	TITLE: SECTION A-A' PROPOSED NEW MIXED USE RESIDENTIAL/COMMERCIAL 28 FISHER ROAD & FRANCIS STREET DEE WHY, NSW 2099	Revision / Date	DATE: 11/05/2020	CHECKED BY: JC
			SCALE: NTS	DESIGNED BY: MJ
			PROJECT No: SRE/524/DW/20	Drawing No: G02

APPENDIX D

Borehole Logs

GEOTECHNICAL BOREHOLE LOG

soilsrock

CLIENT:	THE GEORGE GROUP PTY LTD	**BOREHOLE NO:**	**BH1**
PROJECT:	PROPOSED NEW MIXED USE RESIDENTIAL/COMMERCIAL BUILDING DEVELOPMENT	**PAGE:**	1 of 3
LOCATION:	28 FISHER ROAD & 9 FRANCIS STREET, DEE WHY NSW 2099	**DATE STARTED:**	21/04/2020
DATE:	29/04/2020	**DATE COMPLETED:**	21/04/2020
PROJECT NO:	SRE/524/DW/20	**LOGGED BY:**	AT

Equipment:	BG RIG - HANJIN	**Hole Diameter:**	90mm	**Coring Size:**	-	**RL Surface:**	26m
						Easting:	-
Driller	BG Drilling	**Drilling Method:**	Solid Flight Auger	**Inclination:**	90˚	**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)	DENSITY INDEX	REMARKS AND ADDITIONAL OBSERVATION
SOLID FLIGHT AUGER WITH TC BIT+A14:A133	NO GROUNDWATER OBSERVED	Dry through the Completion of Augering						**PAVING:** Concrete (~300mm)	–	–	–	–
					25.5	0.5		**SAND:** Dark brown/Light brown silty sand, fine-grained.	D			LOW TC BIT RESISTANCE
					25.0	1.0						
					24.5	1.5						
			SPT1 (6, 9, 10/70mm) N = R					**SAND:** Light brown/grey clayey sand, fine-grained.		VD		
					24.0	2.0		**END OF AUGERING @ 1.9m** **PLEASE REFER TO CORE BOREHOLE LOG**				
					23.5	2.5						
					23.0	3.0						
					22.5	3.5						
					22.0	4.0						
					21.5	4.5						
					21.0	5.0						
					20.5	5.5						
					20.0	6.0						

Comments:	**CHECKED BY:**	JC		
A General Remark:	**APPROVED BY:**	JC	**DATE:**	**8/05/2020**

GEOTECHNICAL BOREHOLE LOG

CLIENT:	THE GEORGE GROUP PTY LTD	**BOREHOLE NO:**	**BH1**
PROJECT:	PROPOSED NEW MIXED USE RESIDENTIAL/COMMERCIAL BUILDING DEVELOPMENT	**PAGE:**	2 of 3
LOCATION:	28 FISHER ROAD & 9 FRANCIS STREET, DEE WHY NSW 2099	**DATE STARTED:**	21/04/2020
DATE:	29/04/2020	**DATE COMPLETED**	21/04/2020
PROJECT NO:	SRE/524/DW/20	**LOGGED BY:**	AT

Equipment:	BG RIG - HANJIN	**Hole Diameter:**	76mm	**Coring Size:**	50mm	**RL Surface:**	26.00m
						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	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)	DEFECT DESCRIPTION / ADDITIONAL OBSERVATION
ROCK CORING NMLC							**START CORING @ 1.9m**						
							CLAY: Red silty clay, low plasticity, dry.						
		1st	100%	24.0	2.0		**SANDSTONE:** Light red/ Light yellow, moderately weathered sandstone, medium to low strength.	MW	M	0.52	>90	> 600	
				23.5	2.5					0.37			
				23.0	3.0					0.25			
				22.5	3.5		**SANDSTONE:** Red/Light brown, highly weathered sandstone, low to medium strength.	HW	L–M	0.29			
				22.0	4.0					0.34			
				21.5	4.5					0.25			(~4.59m): Jt.0°.Pl.Sm8.FL.Oz (~25mm
		2nd	100%	21.0	5.0					0.61			
				20.5	5.5		**SANDSTONE:** Light grey/Light brown, moderately weathered sandstone, medium strength.	MW	M	0.44			(~5.46m): Pt. 0°. Pl. Sm8. OP. Cn (~5.66m): Pt. 0°. Pl. Sm8. OP. St
				20.0	6.0		**SANDSTONE:** White/Light brown, slightly weathered sandstone, medium strength.	SW	M	0.51			
				19.5	6.5								

Comments

A General Remark

CHECKED BY: JC

APPROVED BY: JC **DATE:** 8/05/202

GEOTECHNICAL BOREHOLE LOG

soilsrock

CLIENT:	THE GEORGE GROUP PTY LTD	BOREHOLE NO:	BH1
PROJECT:	PROPOSED NEW MIXED USE RESIDENTIAL/COMMERCIAL BUILDING DEVELOPMENT	PAGE:	3 of 3
LOCATION:	28 FISHER ROAD & 9 FRANCIS STREET, DEE WHY NSW 2099	DATE STARTED:	21/04/2020
DATE:	29/04/2020	DATE COMPLETED	21/04/2020
PROJECT NO:	SRE/524/DW/20	LOGGED BY:	AT

Equipment:	BG RIG - HANJIN	Hole Diameter:	76mm	Coring Size:	50mm	RL Surface:	26.00m
						Easting:	-
Driller:	BG Drilling	Drilling Method:	NMLC	Inclination:	90˚	Northing:	-

METHOD: ROCK CORING NMLC

BARREL LIFT	TOTAL CORE RECOVERY (TCR)
2nd	100%
3rd	100%

DEPTH R.L (m): 19.0, 18.5, 18.0, 17.5, 17.0, 16.5, 16.0, 15.5, 15.0, 14.5

DEPTH (m): 7.0, 7.5, 8.0, 8.5, 9.0, 9.5, 10.0, 10.5, 11.0, 11.5

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)

ROCK MATERIAL DESCRIPTION	$Is_{(50)}$ (MPa)
SANDSTONE: Brown/Orange, highly weathered sandstone, low strength.	0.25
SANDSTONE: Dark grey to Light grey/Light brown, slightly weathered sandstone, extremely low to low strength.	0.25, 0.04
SANDSTONE: Light grey to Light brown, slightly weathered to moderately weathered sandstone, medium strength.	0.37, 0.44, 0.55
SANDSTONE: Grey/Light brown, moderately weathered sandstone, medium strength.	0.4
CLAY: Dark grey clay, low to medium plasticity, dry. (~15mm	
SANDSTONE: Light grey/Light brown, slightly weathered sandstone, medium strength.	0.6, 0.56
SANDSTONE: Light grey/Brown, moderately weathered sandstone, medium strength.	0.47
SANDSTONE: Light grey, extremely weathered sandstone.	0.97
SANDSTONE: Light grey, slightly weathered sandstone.	
END OF CORING @ 10.90m	

DEFECT DESCRIPTION / ADDITIONAL OBSERVATION

- (~6.69m): Pt. 0°. Pl. Sm8. OP. St
- (~7.75m): FZ (~25mm), Pl, Sm8, OP,
- (~8.75m): Pt. 0°. Pl. Sm8. CD. Cn
- (~8.91m): Pt. 0°. Pl. Sm8. OP. St
- (~9.64m): Pt. 0°. Pl. Sm8. OP. Cn
- (~10.29m): Jt. 0°.Pl.Sm8. FL. Clv(~15
- (~10.85m): Jt. 0°. Pl. Sm8. OP. Cn

Comments		CHECKED BY:	AT		
A General Remark		APPROVED BY:	JC	DATE:	8/05/202

soilsrock
geotechnical engineering consultants

CLIENT:	THE GEORGE GROUP PTY LTD	**TITLE:**	Rock Core Photograph
PROJECT:	PROPOSED NEW MIXED USE RESIDENTIAL/COMMERCIAL BUILDING DEVELOPMENT	**BOREHOLE NO:**	BH1
ADDRESS:	28 FISHER ROAD & 9 FRANCIS STREET, DEE WHY NSW 2099	**SCALE:**	NTS
PROJECT NO:	SRE/524/DW/20	**DATE RECORDED:**	21/04/2020

CORING STARTED AT 1.90m

CORING TERMINATED AT 10.90m

GEOTECHNICAL BOREHOLE LOG

CLIENT:	THE GEORGE GROUP PTY LTD	**BOREHOLE NO:**	BH2
PROJECT:	PROPOSED NEW MIXED USE RESIDENTIAL/COMMERCIAL BUILDING DEVELOPMENT	**PAGE:**	1 of 3
LOCATION:	28 FISHER ROAD & 9 FRANCIS STREET, DEE WHY NSW 2099	**DATE STARTED:**	21/04/2020
DATE:	29/04/2020	**DATE COMPLETED:**	21/04/2020
PROJECT NO:	SRE/524/DW/20	**LOGGED BY:**	AT

Equipment:	BG RIG - HANJIN	**Hole Diameter:**	90mm	**Coring Size:**	-	**RL Surface:**	27.3m
						Easting:	-
Driller:	BG Drilling	**Drilling Method:**	Solid Flight Auger	**Inclination:**	90˚	**Northing:**	-

METHOD	GROUNDWATER RECORD	Field Tests SPT	Sample ID	DEPTH R.L (m)	DEPTH (m)	SOIL MATERIAL DESCRIPTION	MOISTURE CONTENT	STRENGTH (Consistency, Relative Density)	DENSITY INDEX	REMARKS AND ADDITIONAL OBSERVATION
SOLID FLIGHT AUGER WITH TC BIT+A14:A133	NO GROUNDWATER OBSERVED; Dry through the Completion of Augering					ASPHALT (~30mm)	D		–	LOW TC BIT RESISTANCE
				26.8	0.5	**SAND:** Dark brown/Yellow silty sand, fine-grained.		–		
				26.3	1.0					
				25.8	1.5					
		SPT1 (3, 3, 4) N = 7				**SAND:** Red/Brown clayey sand, fine-grained.		L		
				25.3	2.0	**SAND:** Orange/Brown clayey sand, fine-grained.				
				24.8	2.5					
				24.3	3.0					
		SPT2 (11, 4/140mm) N = R				**SAND:** Orange/Light grey sand, fine-grained.		VD		
				23.8	3.5	**SANDSTONE:** Red/Light grey, extremely weathered sandstone, residual properties, very weak strength.		–		
				23.3	4.0					MEDIUM TC BIT RESISTANCE
				22.8	4.5					
				22.3	5.0					LOW TC BIT RESISTANCE
				21.8	5.5					
				21.3	6.0					

Comments:		**CHECKED BY:**	JC		
A General Remark:		**APPROVED BY:**	JC	**DATE:**	8/05/202

GEOTECHNICAL BOREHOLE LOG

soilsrock

CLIENT:	THE GEORGE GROUP PTY LTD	**BOREHOLE NO:**	BH2
PROJECT:	PROPOSED NEW MIXED USE RESIDENTIAL/COMMERCIAL BUILDING DEVELOPMENT	**PAGE:**	2 of 3
LOCATION:	28 FISHER ROAD & 9 FRANCIS STREET, DEE WHY NSW 2099	**DATE STARTED:**	21/04/2020
DATE:	29/04/2020	**DATE COMPLETED:**	21/04/2020
PROJECT NO:	SRE/524/DW/20	**LOGGED BY:**	AT

Equipment:	BG RIG - HANJIN	**Hole Diameter:**	90mm	**Coring Size:**	-	**RL Surface:**	27.3m
						Easting:	-
Driller:	BG Drilling	**Drilling Method:**	Solid Flight Auger	**Inclination:**	90˚	**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 AUGERING; GROUNDWATER OBSERVED AT ~6.3m			20.8	6.5		**SANDSTONE:** Red/Light grey, extremely weathered sandstone, residual properties, very weak strength.	D	–	–	LOW TC BIT RESISTANCE
				20.3	7.0			W			MEDIUM TC BIT RESISTANCE
							END OF AUGERING @ 7.0m **PLEASE REFER TO CORE BOREHOLE LOG**				
				19.8	7.5						
				19.3	8.0						
				18.8	8.5						
				18.3	9.0						
				17.8	9.5						
				17.3	10.0						
				16.8	10.5						
				16.3	11.0						
				15.8	11.5						
				15.3	12.0						

Comments:		**CHECKED BY:**	JC	
A General Remark:		**APPROVED BY:**	JC	**DATE:** 8/05/2020

GEOTECHNICAL BOREHOLE LOG

CLIENT:	THE GEORGE GROUP PTY LTD	**BOREHOLE NO:**	**BH2**
PROJECT:	PROPOSED NEW MIXED USE RESIDENTIAL/COMMERCIAL BUILDING DEVELOPMENT	**PAGE:**	3 of 3
LOCATION:	28 FISHER ROAD & 9 FRANCIS STREET, DEE WHY NSW 2099	**DATE STARTED:**	21/04/2020
DATE:	29/04/2020	**DATE COMPLETED**	21/04/2020
PROJECT NO:	SRE/524/DW/20	**LOGGED BY:**	AT

Equipment:	BG RIG - HANJIN	**Hole Diameter:**	76mm	**Coring Size:**	50mm	**RL Surface:**	27.30m
						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	Is(50) (MPa)	DEFECT DESCRIPTION / ADDITIONAL OBSERVATION
				20.3	7.0	**START CORING @ 7.0m**		
	GROUNDWATER	1st	100%			**CLAY:** Red/Brown clay, low to medium plasticity, dry.		
						SANDSTONE: Light grey, slightly weathered sandstone, medium strength.		(~7.16m): Pt, 0°, Pl, Sm8, OP, Cn
						SANDTSONE: Yellow/Light brown, moderately weathered sandstone, medium strength.	0.41	
				19.8	7.5			
						SANDSTONE: Ligth grey, slightly weathered sandstone, low to medium strength.	0.32	
				19.3	8.0		0.21	
				18.8	8.5		0.19	(~8.68m):Pt,0°,Pl, Sm8,FL,Cly(~10mm)
							0.21	
				18.3	9.0		0.43	
				17.8	9.5			
						SANDSTONE: Light brown to Brown/Red, moderately to highly weathered sandstone, medium strength.	0.53	(~9.94m): Jt, 35°, Pl, Sm8, CD, Cn
				17.3	10.0	**END CORING @ 9.97m**		
				16.8	10.5			
				16.3	11.0			
				15.8	11.5			

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

Comments

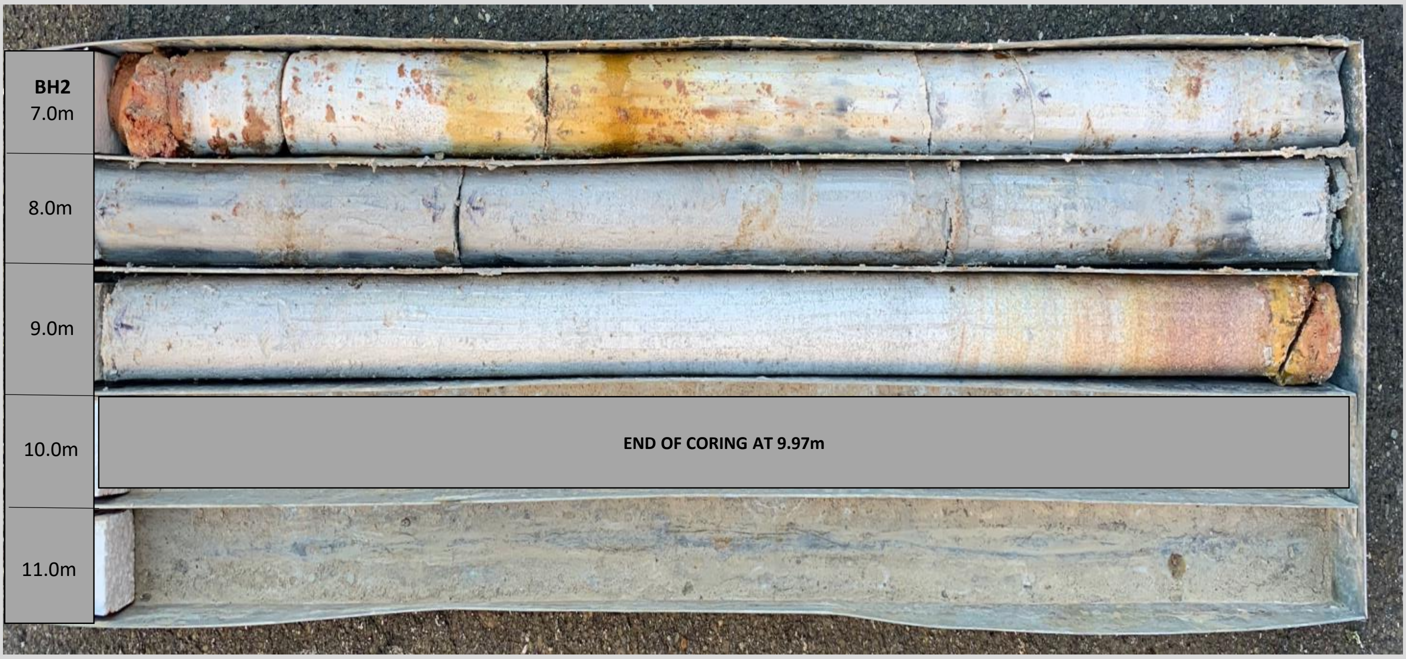
A General Remark

CHECKED BY: JC
APPROVED BY: JC **DATE:** 8/05/202

soilsrock
geotechnical engineering consultants

CLIENT:	THE GEORGE GROUP PTY LTD	**TITLE:**	Rock Core Photograph
PROJECT:	PROPOSED NEW MIXED USE RESIDENTIAL/COMMERCIAL BUILDING DEVELOPMENT	**BOREHOLE NO:**	BH2
ADDRESS:	28 FISHER ROAD & 9 FRANCIS STREET, DEE WHY NSW 2099	**SCALE:**	NTS
PROJECT NO:	SRE/524/DW/20	**DATE RECORDED:**	21/04/2020

CORING STARTED AT 7.00m



CORING TERMINATED AT 9.97m

SOILSROCK ENGINEERING PTY LTD │ ABN 83 155 012 614
GEOTECHNICAL │ ENVIRONMENTAL │ FOUNDATIONS
www.soilsrock.com.au │ info@soilsrock.com.au

APPENDIX E

Point Load Test Index Results

CLIENT:	THE GEORGE GROUP PTY LTD	**PAGE**	1 of 1
PROJECT:	PROPOSED NEW MIXED-USE BUILDING DEVELOPMENT	**TESTED BY:**	A.T
LOCATION:	28 FISHER ROAD & 9 FRANCIS STREET, DEE WHY NSW 2099	**CHECKED BY:**	J.C
PROJECT NO:	SRE/524/DW/19	**DATE OF RECORDED:**	21/04/2020
TEST METHOD:	RMS T223	**DATE OF ISSUE:**	21/04/2020

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 UCS (Mpa)	Estimated Strength
BH01-A1	2.17	A	SS	MA	AR	40.00	50.00	50.46	1.33	3	0.522	0.524	10.49	M
BH01-A2	2.69	A	SS	MA	AR	35.00	50.00	47.20	0.85	3	0.381	0.372	7.43	M
BH01-A3	3.2	A	SS	MA	AR	45.00	50.00	53.52	0.68	3	0.237	0.245	4.90	L
BH01-A4	3.75	A	SS	MA	AR	40.00	50.00	50.46	0.73	3	0.287	0.288	5.76	L
BH01-A5	4.21	A	SS	MA	AR	40.00	50.00	50.46	0.86	3	0.338	0.339	6.78	M
BH01-A6	4.55	A	SS	MA	AR	45.00	50.00	53.52	0.69	3	0.241	0.248	4.97	L
BH01-A7	5.06	A	SS	MA	AR	40.00	50.00	50.46	1.55	3	0.609	0.611	12.22	M
BH01-A8	5.56	A	SS	MA	AR	30.00	50.00	43.70	0.90	3	0.471	0.444	8.87	M
BH01-A9	5.96	A	SS	MA	AR	30.00	50.00	43.70	1.04	3	0.545	0.513	10.25	M
BH01-A10	6.63	A	SS	MA	AR	45.00	50.00	53.52	0.69	3	0.241	0.248	4.97	L
BH01-A11	6.74	A	SS	MA	AR	30.00	50.00	43.70	0.50	3	0.262	0.246	4.93	L
BH01-A12	7.06	A	SS	MA	AR	30.00	50.00	43.70	0.09	3	0.047	0.044	0.89	VL
BH01-A13	7.58	A	SS	MA	AR	40.00	50.00	50.46	0.93	3	0.365	0.367	7.33	M
BH01-A14	8.03	A	SS	MA	AR	40.00	50.00	50.46	1.12	3	0.440	0.442	8.83	M
BH01-A15	8.66	A	SS	MA	AR	30.00	50.00	43.70	1.11	3	0.581	0.547	10.94	M
BH01-A16	9.12	A	SS	MA	AR	45.00	50.00	53.52	1.11	3	0.387	0.400	7.99	M
BH01-A17	9.68	A	SS	MA	AR	30.00	50.00	43.70	1.21	3	0.634	0.596	11.93	M
BH01-A18	10.06	A	SS	MA	AR	40.00	50.00	50.46	1.42	3	0.558	0.560	11.20	M
BH01-A19	10.72	A	SS	MA	AR	30.00	50.00	43.70	0.96	3	0.503	0.473	9.46	M
BH01-A20	10.85	A	SS	MA	AR	45.00	50.00	53.52	2.68	3	0.935	0.965	19.29	M

NOTATION

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

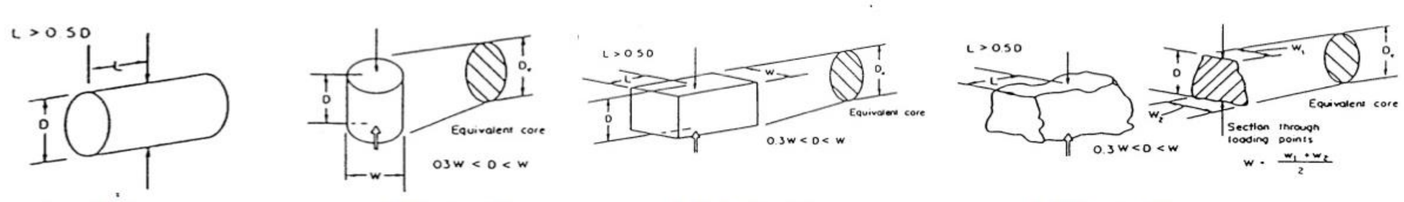
Rock Type
(SS) Sandstone
(ST) Siltstone
(SH) Shale
(G) Granitic
(MSS) Meta Sandstone
(MST) Meta Siltstone

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 | A: Axial Test | B: Block Test | I: Irregular Lump Test



L > 0.5 D | 0.3W < D < W | 0.3W < D < W, L > 0.5 D | 0.3 W < D < W, L > 0.5 D, W = (W1 + W2) / 2

CLIENT:	THE GEORGE GROUP PTY LTD	**PAGE**	1 of 1
PROJECT:	PROPOSED NEW MIXED-USE BUILDING DEVELOPMENT	**TESTED BY:**	A.T
LOCATION:	28 FISHER ROAD & 9 FRANCIS STREET, DEE WHY NSW 2099	**CHECKED BY:**	J.C
PROJECT NO:	SRE/524/DW/19	**DATE OF RECORDED:**	21/04/2020
TEST METHOD:	RMS T223	**DATE OF ISSUE:**	21/04/2020

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 UCS (Mpa)	Estimated Strength
BH01-A1	7.32	A	SS	MA	AR	40.00	50.00	50.46	1.05	3	0.412	0.414	8.28	M
BH01-A2	7.78	A	SS	MA	AR	40.00	50.00	50.46	0.80	3	0.314	0.315	6.31	M
BH01-A3	8.26	A	SS	MA	AR	43.00	50.00	52.32	0.56	3	0.205	0.209	4.18	L
BH01-A4	8.69	A	SS	MA	AR	30.00	50.00	43.70	0.39	3	0.204	0.192	3.84	L
BH01-A5	8.83	A	SS	MA	AR	35.00	50.00	47.20	0.49	3	0.220	0.214	4.29	L
BH01-A6	9.23	A	SS	MA	AR	42.00	50.00	51.71	1.14	3	0.426	0.433	8.66	M
BH01-A7	9.76	A	SS	MA	AR	45.00	50.00	53.52	1.46	3	0.510	0.525	10.51	M

NOTATION

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

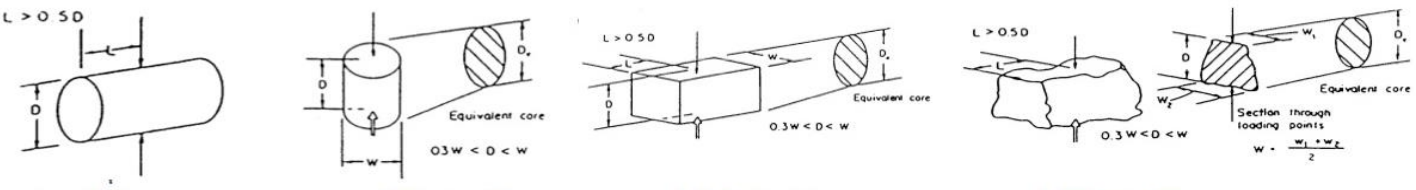
Rock Type
- (SS) Sandstone
- (ST) Siltstone
- (SH) Shale
- (G) Granitic
- (MSS) Meta Sandstone
- (MST) Meta Siltstone

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 — A: Axial Test — B: Block Test — I: Irregular Lump Test



D: L > 0.5 D

A: 0.3W < D < W

B: 0.3W < D < W, L > 0.5 D

I: 0.3 W < D < W, L > 0.5 D, W = (W1 + W2) / 2

APPENDIX F

DCP Tests Graphics

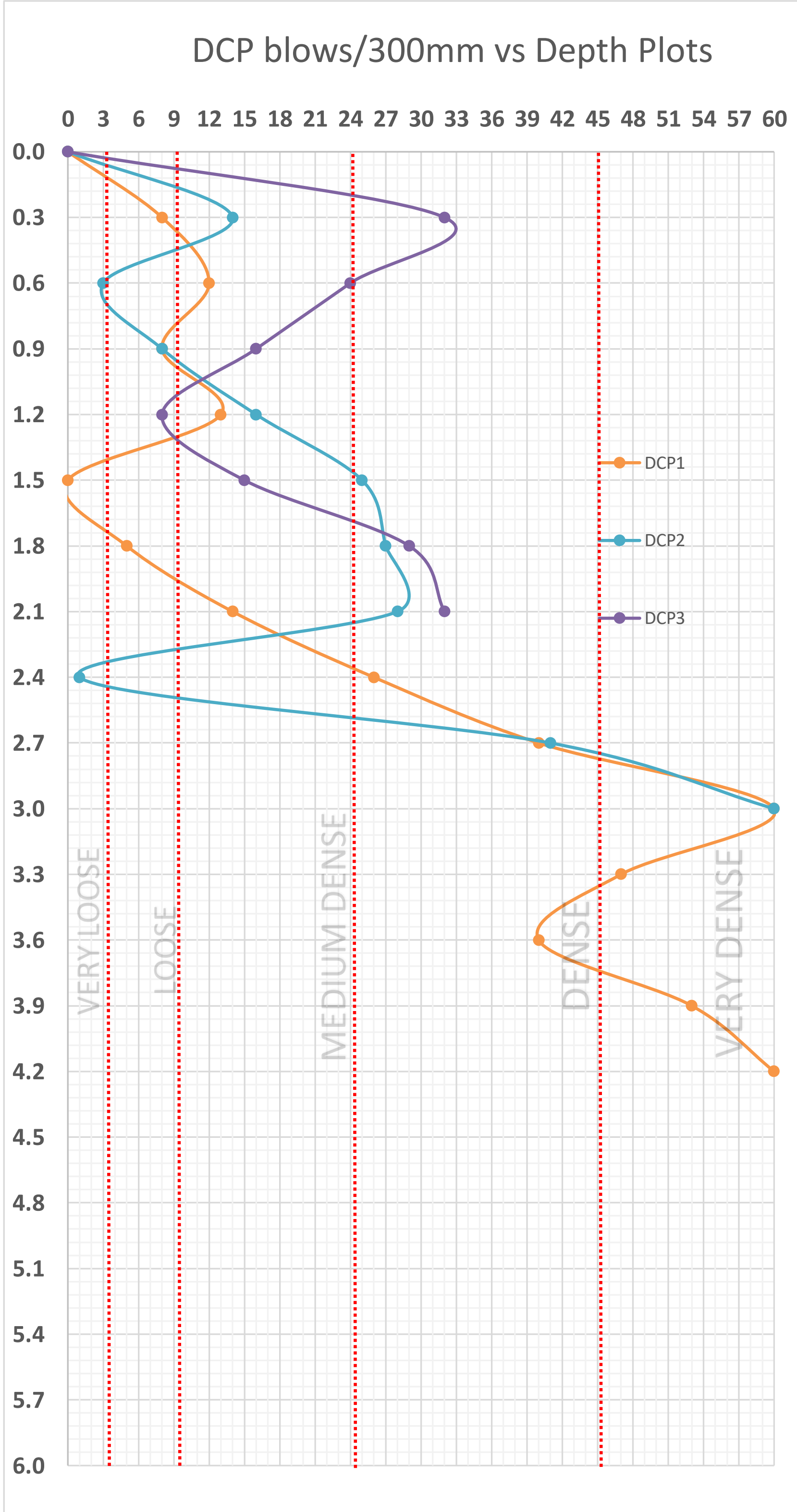
Dynamic Cone Penetrometer Test (DCP)

Client:	THE GEORGE GROUP PTY LTD	**Page:**	1 of 1
Project:	GEOTECHNICAL SITE INVESTIGATION REPORT FOR PROPOSED NEW MIXED-USE DEVELOPMENT	**Date Started:**	21/04/2020
Location:	28 FISHER ROAD & 9 FRANCIS STREET, DEE WHY NSW 2099	**Date Completed:**	21/04/2020
Date:	5/05/2020	**Logged/Checked by:**	AT / JC
Project No.:	SRE/524/DW/19		

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	8	14	32	
2	0.3 - 0.6	12	3	24	
3	0.6 - 0.9	8	8	16	
4	0.9 - 1.2	13	16	8	
5	1.2 - 1.5	0	25	15	
6	1.5 - 1.8	5	27	29	
7	1.8 - 2.1	14	28	32	
8	2.1 - 2.4	26	23		
9	2.4 - 2.7	40	41		
10	2.7 - 3.0	60	60		
11	3.0 - 3.3	47			
12	3.3 - 3.6	40			
13	3.6 - 3.9	53			
14	3.9 - 4.2	60			
15					
16					
17					
18					
19					
20					
21					
22					



Note: * Reached practical refusal 60 blows per 300mm

APPENDIX G

Site Photographs

CLIENT:	THE GEORGE GROUP PTY LTD	**PAGE:**	1 of 1
PROJECT:	GEOTECHNICAL SITE INVESTIGATION REPORT FOR PROPOSED NEW MIXED-USE BUILDING DEVELOPMENT	**DATE RECORD:**	21/04/2020
LOCATION:	28 FISHER ROAD & 9 FRANCIS STREET, DEE WHY NSW 2099		
DATE:	6/05/2020	**LOGGED BY:**	AT
PROJECT NO.:	SRE/524/DW/19	**CHECKED BY:**	JC

SITE PHOTOGRAPHS

Photo 1 - West view to BH1 test location**.**

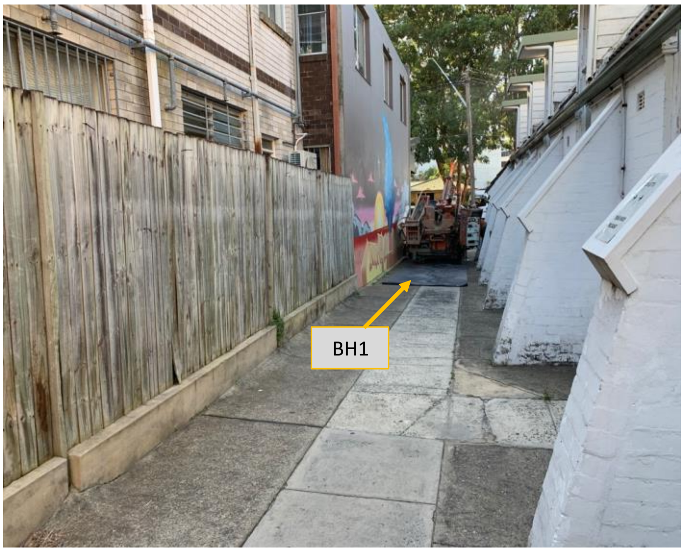


Photo 2 - East view to BH1 test location**.**

Photo 3 - North-East view to BH2 test location.

Photo 4 - South-West view to DCP1 test location**.**

Photo 5 - South-East view to DCP2 test location.

Photo 6 - North-East view to DCP3 test location.