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/22/REV1

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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

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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.
- Architectural Drawings prepared by The George Group for "Proposed Mixed Use Community Centre", DWG's Nos: 40889/01-02; from 4089/03.1 to 4089/3.17; from 4089/4.1 to 4089/4.11; from 4089/5.1 to 4089/5.21; from 4089/06.1 to 4089/06.10; from 4089/07.1 to 4089/07.12; from 4089/08.1 to 4089/08.3; from 4089/09.1 to 4089/09.13; from 4089/10 to 4089/18 Amendment DA, dated of July 2022.

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 two new mixed-use residential and commercial 4 and 5 storey buildings, including two basements level.

Details of the proposed development are shown on the Architectural drawings provided by THE GEORGE GROUP as referred above.

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.

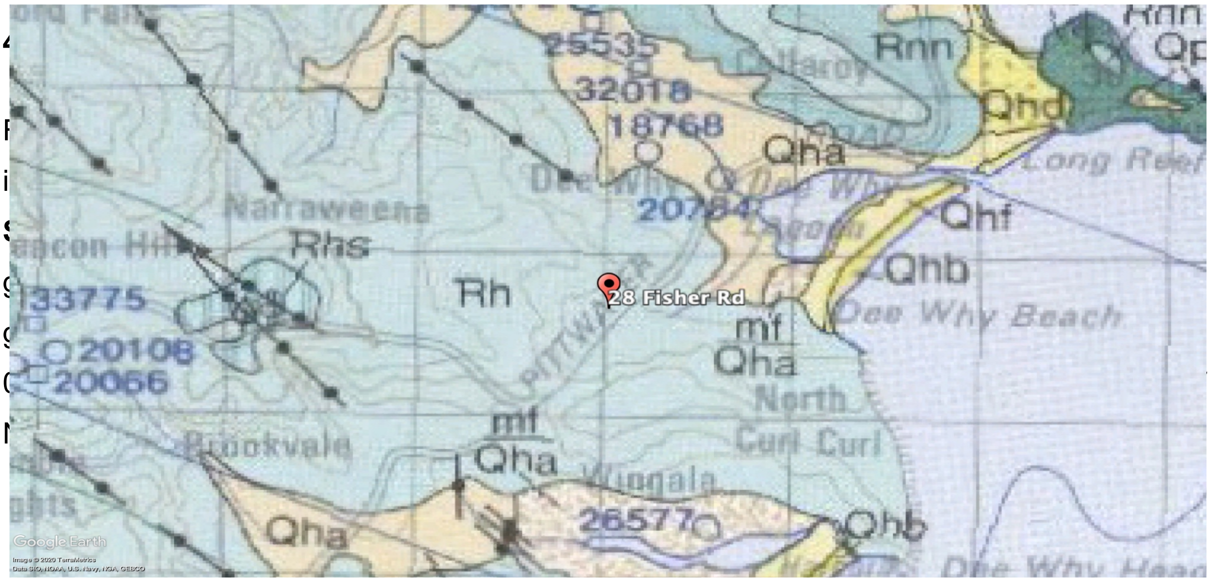

Figure 1 – Portion of the Sydney 1:100,000 Geological Series Sheet 9130. Site area location is highlighted in a red/black sign.

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.

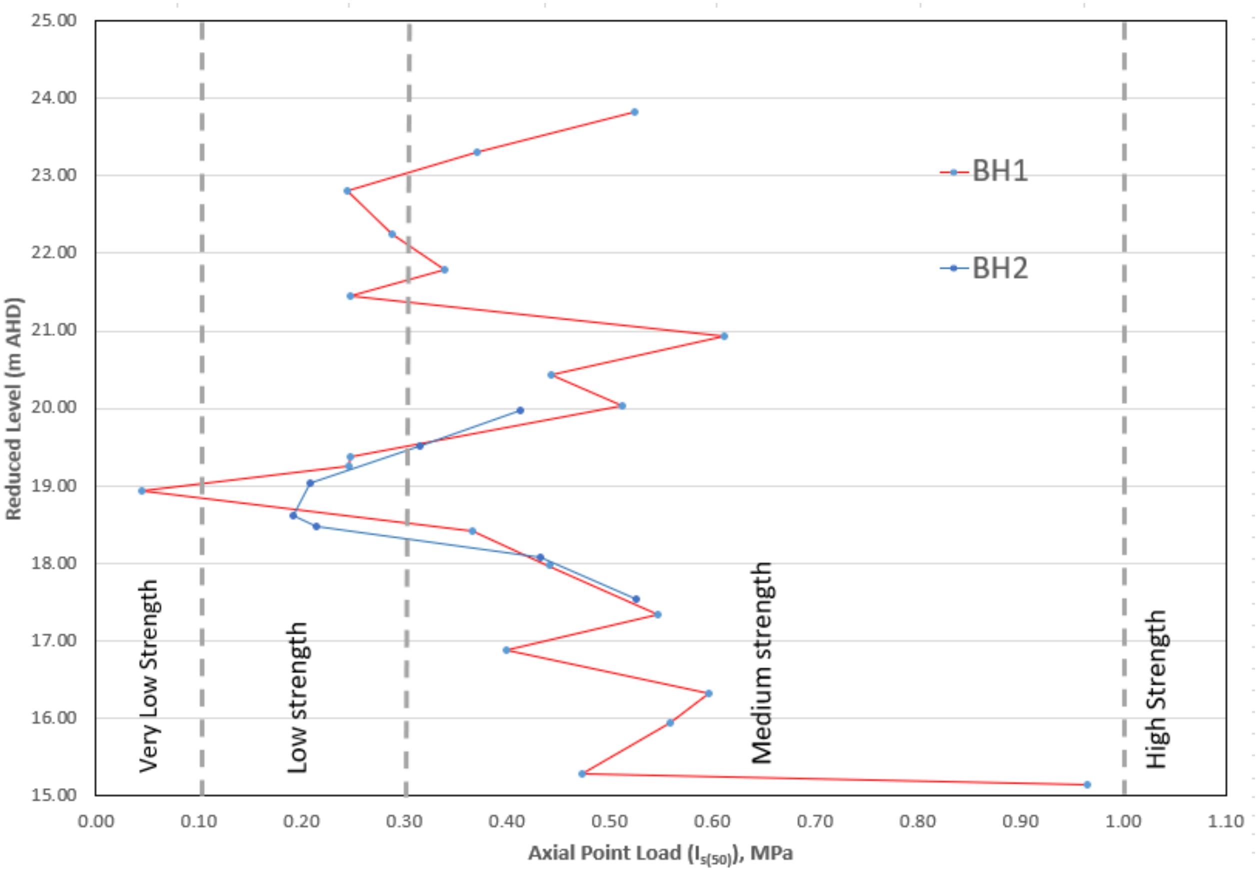

Figure 2 – Axial Point Load Tests Strength Results Plotted against Depth for Boreholes BH1 and BH2.

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

Depth (m)	DCP 1	DCP 2	DCP 3
0.0 – 0.3	Medium Dense Sand	Dense Sand	Very Dense Sand
0.3 – 0.6	Dense Sand	Loose Sand	
0.6 – 0.9	Medium Dense Sand	Medium Dense Sand	
0.9 – 1.2	Dense Sand	Very Dense Sand	Medium Dense Sand
1.2 – 1.5	Very Loose Sand		Dense Sand
1.5 – 1.8	Medium Dense Sand		Very Dense Sand
1.8 – 2.1	Dense Sand		
2.1 – 2.4	Very Dense Sand		-
2.4 – 2.7			
2.7 – 3.0			
3.0 – 3.3		-	
3.3 – 3.6			
3.6 – 3.9			
3.9 - 4.2			

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 borehole drilling and installation of a standpipe water Well installation to monitor groundwater behaviour if required.

5. COMMENTS AND RECOMMENDATIONS

5.1 Excavation and Groundwater Seepage Conditions

As indicated by the architectural drawings provided by the client, maximum excavation depth required is to approximately 9m to construct the two basements 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, groundwater seepage is expected during the excavation for the lower basement level construction to 6.3m deep at RL 21 approximately, which is approximately 2.6m above the bulk excavation level at RL 18.4. It is uncertain if it is expected low or high flows rates of groundwater at this stage, it is recommended to install a standpipe groundwater Well to undertake ground monitoring during minimum 3 weeks including pump out tests if necessary, to determine and confirm groundwater flows and behaviour.

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 two basement car park levels, 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 is 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:

- Anchored Soldier CFA Piles Shoring Wall by minimum 600mm diameter with shotcrete infill, this option considers minimum 600mm diameter piles or larger diameter (depending on the design modelling calculations results) can be installed with appropriate pile spacing to support the soils/weathered rock during the excavation. These shoring bored piles can be used as load bearing piles if founded at appropriate depths.

- Soldier CFA Piles Shoring Wall with minimum 600mm diameter with shotcrete infill and steel bracing. This option is similar to the above with the difference of the anchors which are being replaced by steel bracing and/or props. 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.

Minimum 600mm diameter piles sizes or bigger are recommended, regarding the deep levels of excavation range at the deeper parts of the site from 6m to 9m deep to ensure piles verticality. Special careful and attention must be taken by the piling contractor to ensure that piles verticality is carried properly for piling shoring wall, experienced piling operators and piling site supervisors must be used on site to ensure that verticality issues will not occurs during piling drilling and installation. All designs systems must ensure that the walls can support the ground at the maximum deflections permitted by the design.

If temporary anchors are required within the design option chosen, written authorization and confirmation by the property owners, and Local Council must be obtained to allow its installation and must comply with Council's Policies. 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.

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

All the above solutions are assumed for low flows volumes of groundwater seepage into the excavation. If high flows volumes are confirmed, a shoring piling design option by a fully watertight wall and slab tanked solution by a secant piling wall combined with ground anchors or steel bracing/props and impermeable wall and slab could be necessary (minimum 600mm diameter piles sizes or bigger are recommended, regarding the deep levels of excavation to ensure piles verticality).

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 four-storey and five-storey proposed buildings, 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.

Regarding the expected rock materials at the base of the excavation bored piles are suitable foundation type recommended for the site. Groundwater could also be encountered within the rock fractures which could require a change to 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). If better rock quality is encountered at the base of the excavation, large pad footings can be also considered to be socket in medium strength sandstone rock (CL. III low to medium quality range) with minimum 2.500 kPa allowable bearing pressure. Large pad footings founding depths and sizes would need to be enough to permit enough skin friction and load bearing to support the expected heavy columns loading. If necessary and regarding the expected low rock quality and bearing pressures at the bulk excavation levels, and in order to determine the pad footing size foundation design, further geotechnical investigations by rock coring should be carried out at the location of each pad footing at the base of the bulk excavation level at least 2-3m deep below the underside of the proposed pad footings.

A professional qualified Geotechnical Engineer must inspect the site during piles and/or pad footings construction to confirm allowable bearing pressures are achieved to confirm suitability of design loads. Founding depths must be adjusted and confirmed by the structural loads and foundations type required for the project.

Once the structural loads and pad footings 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 site size, in order to confirm the unknow ground conditions specially in between boreholes and below the footprint of the existing buildings, it is strongly recommended to undertake an additional geotechnical investigation after the building's demolition.
- Regarding the uncertainty of groundwater behaviour flows type it is recommended the installation of standpipe groundwater wells installed to minimum 10m depth for further groundwater monitoring and pump out tests to confirm groundwater behaviour, flows and levels, to confirm the suitability of the piling shoring wall design without a tanking design solution.
- Further geotechnical investigation by rock coring at the location of each pad footing to confirm allowable bearing pressures to assist and confirm pad footings final design.
- 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.

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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**
≤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

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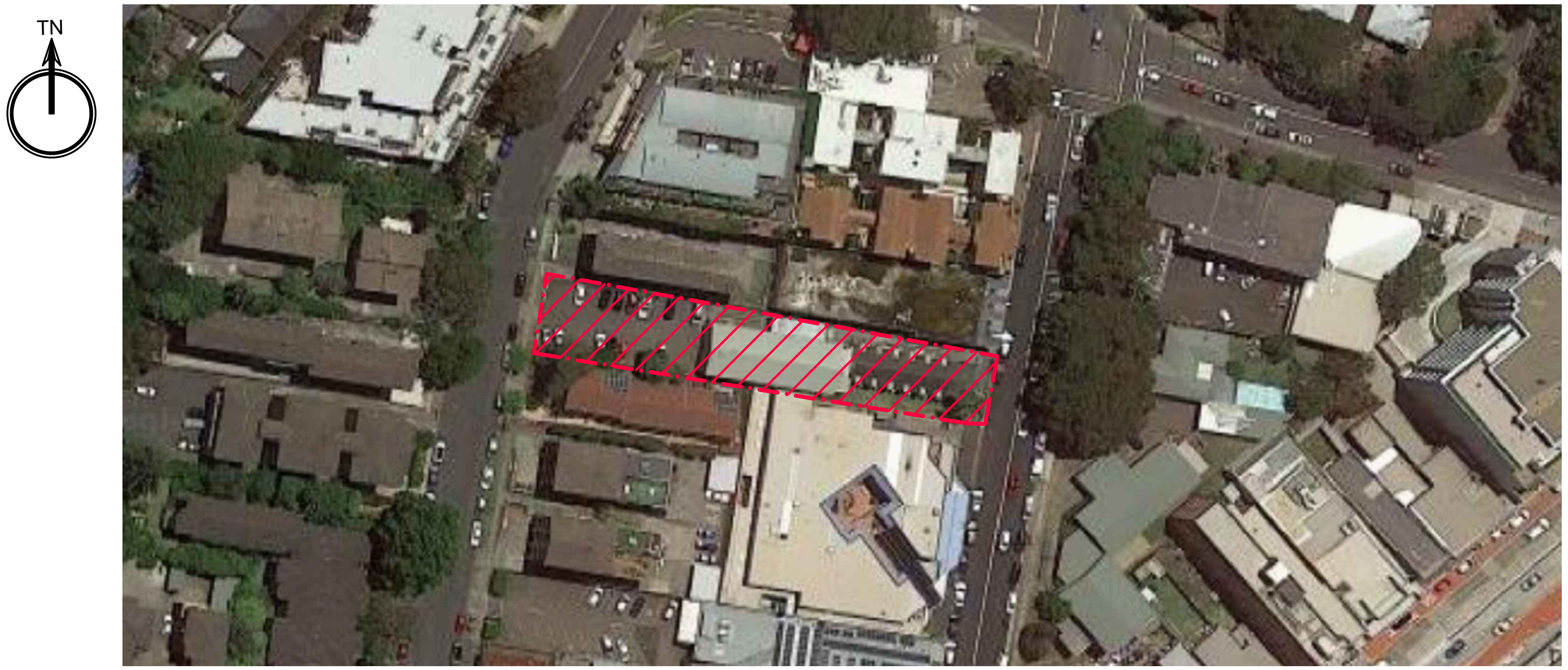
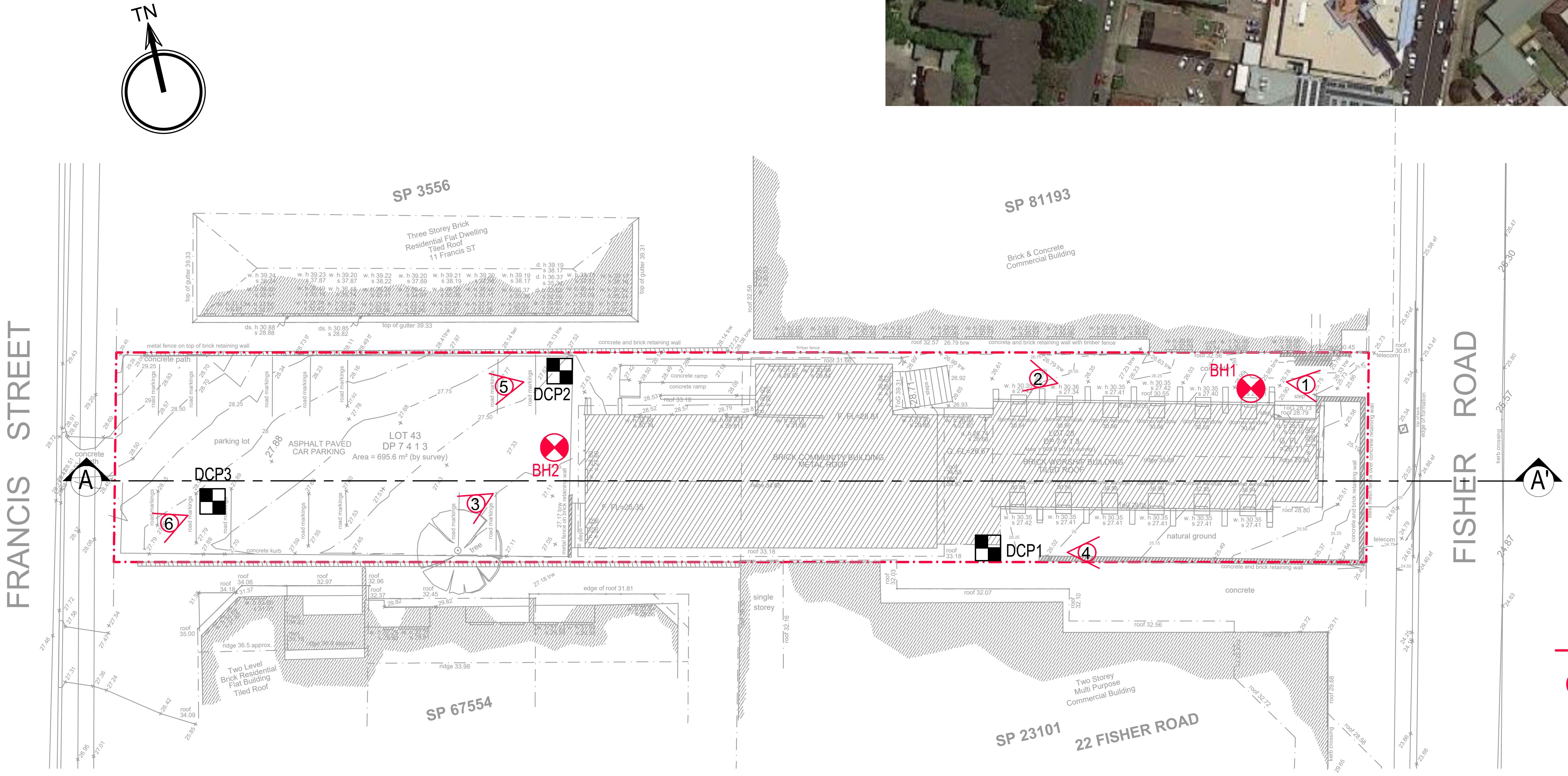
GRAPHIC LOG SYMBOLS

APPENDIX B

DCP tests, Boreholes & Photos Location Plan





LEGEND

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

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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

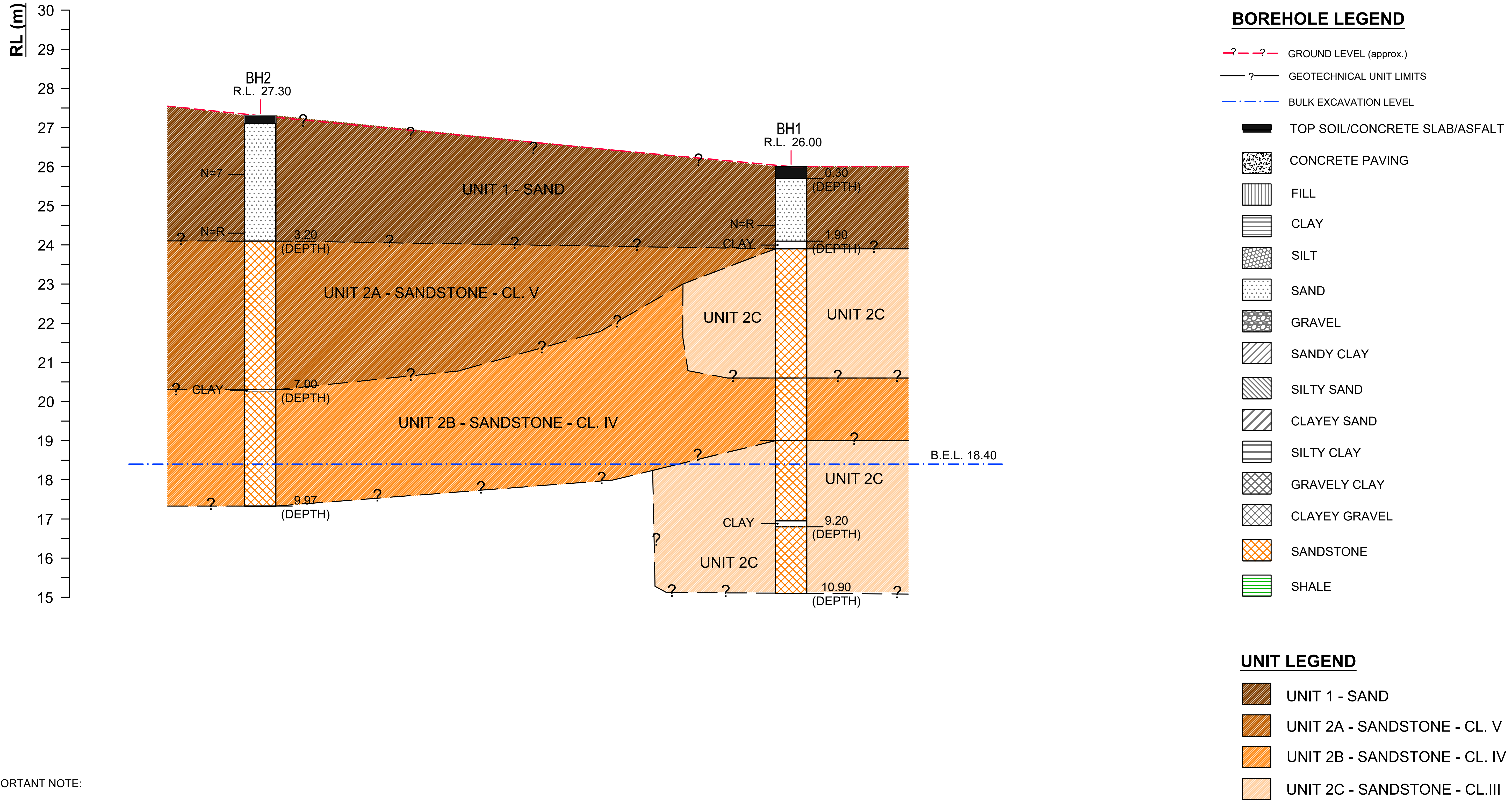
CHECKED BY: JC

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.

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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
1	12/08/2022

DATE: 11/05/2020
SCALE: NTS
PROJECT No:
SRE/524/DW/20

CHECKED BY: JC
DESIGNED BY: MJ
Drawing No: G02

APPENDIX D

Borehole Logs

GEOTECHNICAL BOREHOLE LOG

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			REMARKS AND ADDITIONAL OBSERVATION
									MOISTURE CONTENT	STRENGTH (Consistency, Relative Density)	DENSITY INDEX	
SOLID FLIGHT AUGER WITH TC BIT+A14:A133	NO GROUNDWATER OBSERVED	Dry through the Completion of Augering					■	**PAVING:** Concrete (~300mm)	–	–	–	–
					25.5 25.0 24.5	0.5 1.0 1.5	░	**SAND:** Dark brown/Light brown silty sand, fine-grained.	D			LOW TC BIT RESISTANCE
			SPT1 (6, 9, 10/70mm) N = R				░	**SAND:** Light brown/grey clayey sand, fine-grained.		VD		
					24.0 23.5 23.0 22.5 22.0 21.5 21.0 20.5 20.0	2.0 2.5 3.0 3.5 4.0 4.5 5.0 5.5 6.0		**END OF AUGERING @ 1.9m PLEASE REFER TO CORE BOREHOLE LOG**				

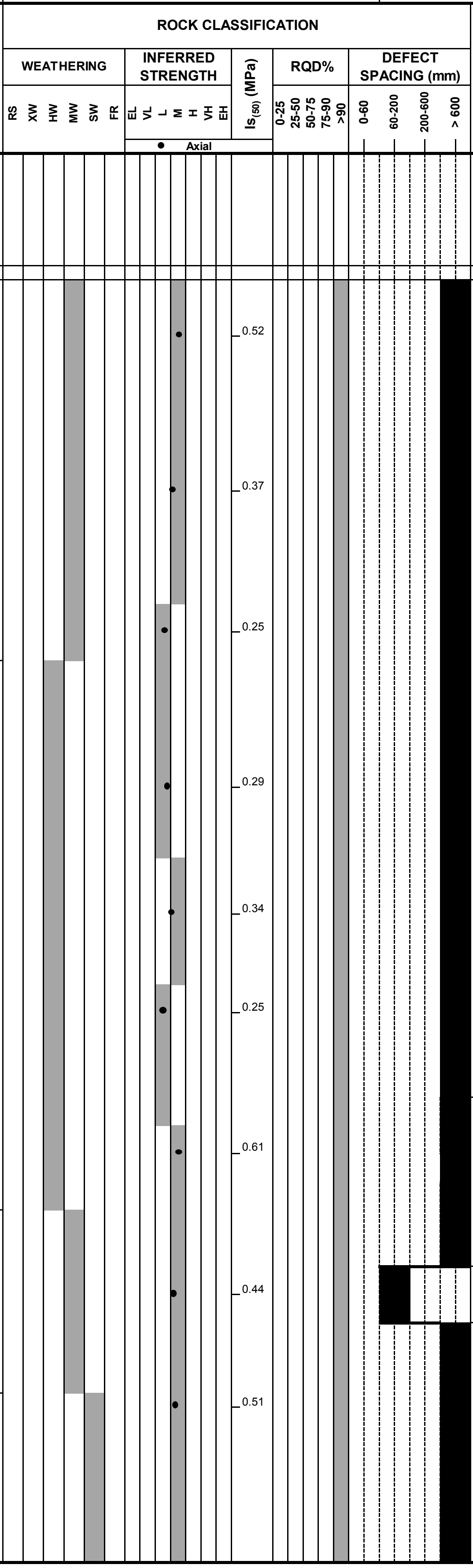
Comments:	**CHECKED BY:**	JC		
A General Remark:	**APPROVED BY:**	JC	**DATE:**	8/05/2020

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)	ROCK MATERIAL DESCRIPTION	DEFECT DESCRIPTION / ADDITIONAL OBSERVATION
ROCK CORING NMLC						**START CORING @ 1.9m**	
		1st	100%			**CLAY:** Red silty clay, low plasticity, dry.	
				24.0	2.0	**SANDSTONE:** Light red/ Light yellow, moderately weathered sandstone, medium to low strength.	
				23.5	2.5		
				23.0	3.0		
						SANDSTONE: Red/Light brown, highly weathered sandstone, low to medium strength.	
				22.5	3.5		
				22.0	4.0		
				21.5	4.5		(~4.59m): Jt.0°.Pl.Sm8.FL.Qz (~25mm
		2nd	100%	21.0	5.0		
						SANDSTONE: Light grey/Light brown, moderately weathered sandstone, medium strength.	(~5.46m): Pt. 0°. Pl. Sm8. OP. Cn
				20.5	5.5		(~5.66m): Pt. 0°. Pl. Sm8. OP. St
						SANDSTONE: White/Light brown, slightly weathered sandstone, medium strength.	
				20.0	6.0		
				19.5	6.5		



Comments **CHECKED BY:** JC

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

GEOTECHNICAL BOREHOLE LOG

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	GROUNDWATER RECORD	BARREL LIFT	TOTAL CORE RECOVERY (TCR)	DEPTH R.L (m)	DEPTH (m)	ROCK MATERIAL DESCRIPTION	Is(50) (MPa)	DEFECT DESCRIPTION / ADDITIONAL OBSERVATION
ROCK CORING NMLC		2nd	100%			**SANDSTONE:** Brown/Orange, highly weathered sandstone, low strength.		
							0.25	(~6.69m): Pt. 0°. Pl. Sm8. OP. St
						SANDSTONE: Dark grey to Light grey/Light brown, slightly weathered sandstone, extremely low to low strength.	0.25	
				19.0	7.0			
							0.04	
						SANDSTONE: Light grey to Light brown, slightly weathered to moderately weathered sandstone, medium strength.		
				18.5	7.5		0.37	
								(~7.75m): FZ (~25mm), Pl, Sm8, OP,
		3rd	100%	18.0	8.0		0.44	
				17.5	8.5			
							0.55	
								(~8.75m): Pt. 0°. Pl. Sm8. CD. Cn
						SANDSTONE: Grey/Light brown, moderately weathered sandstone, medium strength.		(~8.91m): Pt. 0°. Pl. Sm8. OP. St
				17.0	9.0		0.4	
						CLAY: Dark grey clay, low to medium plasticity, dry. (~15mm		
						SANDSTONE: Light grey/Light brown, slightly weathered sandstone, medium strength.		
				16.5	9.5			
							0.6	(~9.64m): Pt. 0°. Pl. Sm8. OP. Cn
				16.0	10.0			
							0.56	
						SANDSTONE: Light grey/Brown, moderately weathered sandstone, medium strength.		(~10.29m): Jt. 0°.Pl.Sm8. FL. Clv(~15
				15.5	10.5			
							0.47	
						SANDSTONE: Light grey, extremely weathered sandstone.	0.97	(~10.85m): Jt. 0°. Pl. Sm8. OP. Cn
						SANDSTONE: Light grey, slightly weathered sandstone,		
						END OF CORING @ 10.90m		
				15.0	11.0			
				14.5	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)

Comments

A General Remark

CHECKED BY: AT

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

SOILSROCK ENGINEERING PTY LTD │ ABN 83 155 012 614

GEOTECHNICAL │ ENVIRONMENTAL │ FOUNDATIONS

www.soilsrock.com.au │ info@soilsrock.com.au

GEOTECHNICAL BOREHOLE LOG

CLIENT: THE GEORGE GROUP PTY LTD
PROJECT: PROPOSED NEW MIXED USE RESIDENTIAL/COMMERCIAL BUILDING DEVELOPMENT
LOCATION: 28 FISHER ROAD & 9 FRANCIS STREET, DEE WHY NSW 2099
DATE: 29/04/2020
PROJECT NO: SRE/524/DW/20

BOREHOLE NO: BH2
PAGE: 1 of 3
DATE STARTED: 21/04/2020
DATE COMPLETED: 21/04/2020
LOGGED BY: AT

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

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

Comments:
A General Remark:

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

GEOTECHNICAL BOREHOLE LOG

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	 STRENGTH (Consistency, Relative Density)	 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 20.3	6.5 7.0	▨	**SANDSTONE:** Red/Light grey, extremely weathered sandstone, residual properties, very weak strength.	D W	–	–	LOW TC BIT RESISTANCE MEDIUM TC BIT RESISTANCE
				19.8 19.3 18.8 18.3 17.8 17.3 16.8 16.3 15.8 15.3	7.5 8.0 8.5 9.0 9.5 10.0 10.5 11.0 11.5 12.0		**END OF AUGERING @ 7.0m** **PLEASE REFER TO CORE BOREHOLE LOG**				

Comments:

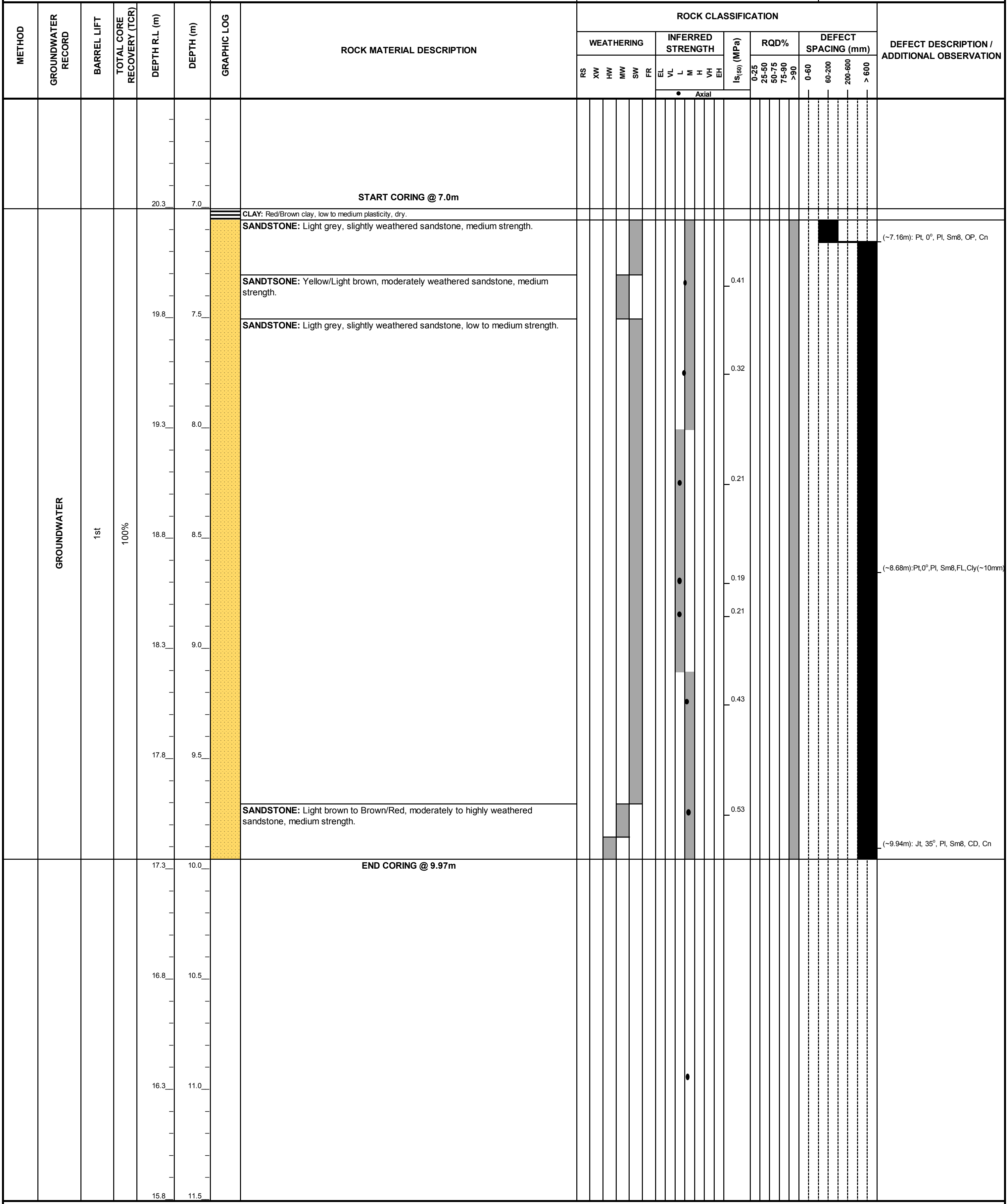
A General Remark:

CHECKED BY:	JC		
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:**	-



DEPTH R.L (m)	DEPTH (m)	ROCK MATERIAL DESCRIPTION	Is(50) (MPa)	DEFECT DESCRIPTION / ADDITIONAL OBSERVATION
20.3	7.0	**START CORING @ 7.0m**		
		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 0.21 0.19 0.21 0.43	(~8.68m):Pt,0°,Pl, Sm8,FL,Cly(~10mm)
		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**		

Comments

A General Remark

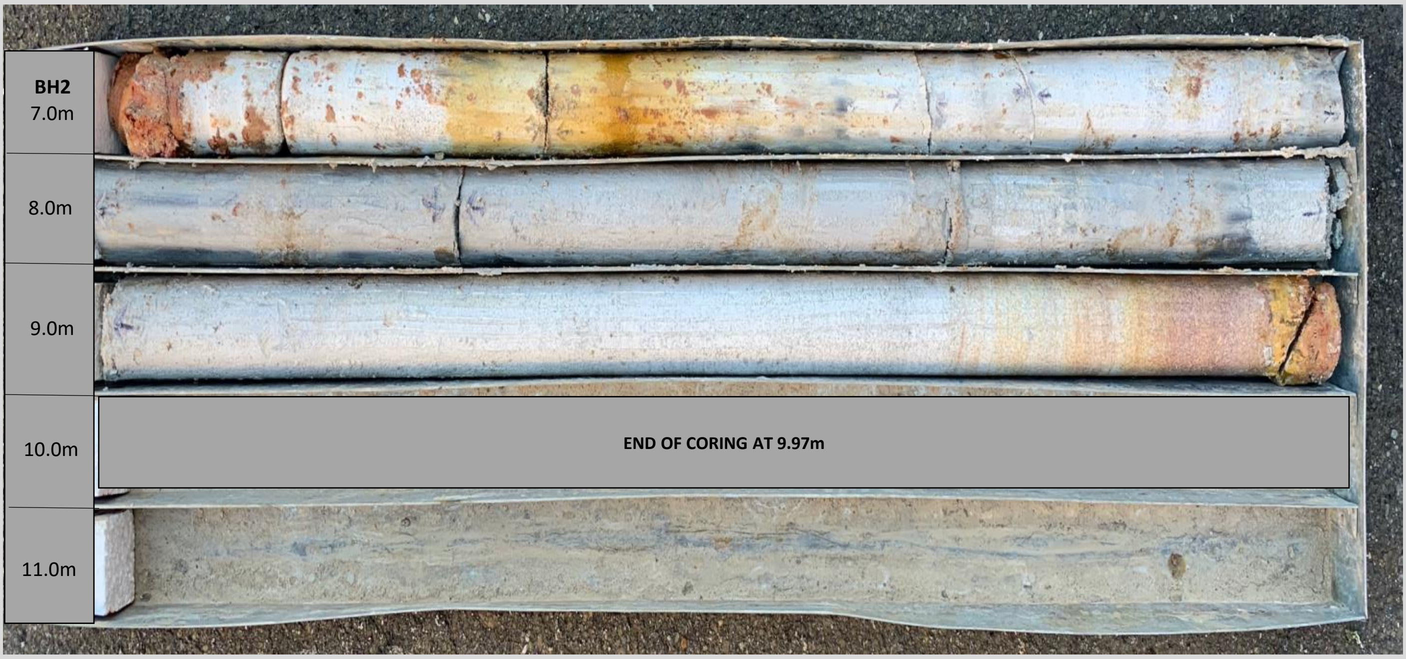
CHECKED BY: JC

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

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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

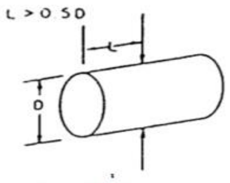
(MA) Massive
(BE) Bedded
(IB) Interbedded
(LA) Laminated
(CR) Crystalline

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)

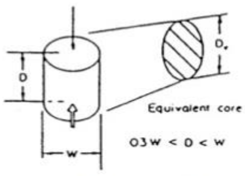
Test Type

D: Diametral Test



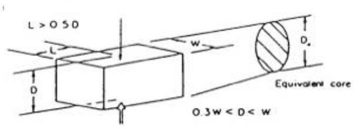
L > 0.5 D

A: Axial Test



0.3W < D < W

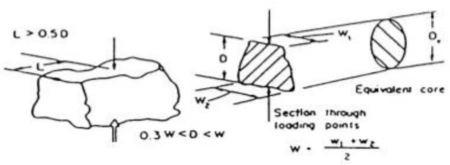
B: Block Test



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:	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

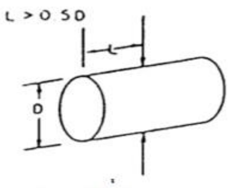
(MA)	Massive
(BE)	Bedded
(IB)	Interbedded
(LA)	Laminated
(CR)	Crystalline

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)

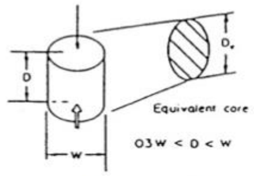
Test Type

D: Diametral Test



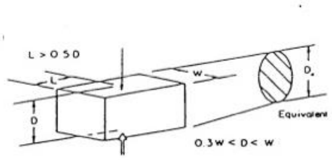
L > 0.5 D

A: Axial Test



0.3W < D < W

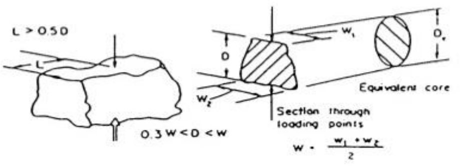
B: Block Test



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: THE GEORGE GROUP PTY LTD

Project: GEOTECHNICAL SITE INVESTIGATION REPORT FOR PROPOSED NEW MIXED-USE DEVELOPMENT

Location: 28 FISHER ROAD & 9 FRANCIS STREET, DEE WHY NSW 2099

Date: 5/05/2020

Project No.: SRE/524/DW/19

Page: 1 of 1

Date Started: 21/04/2020

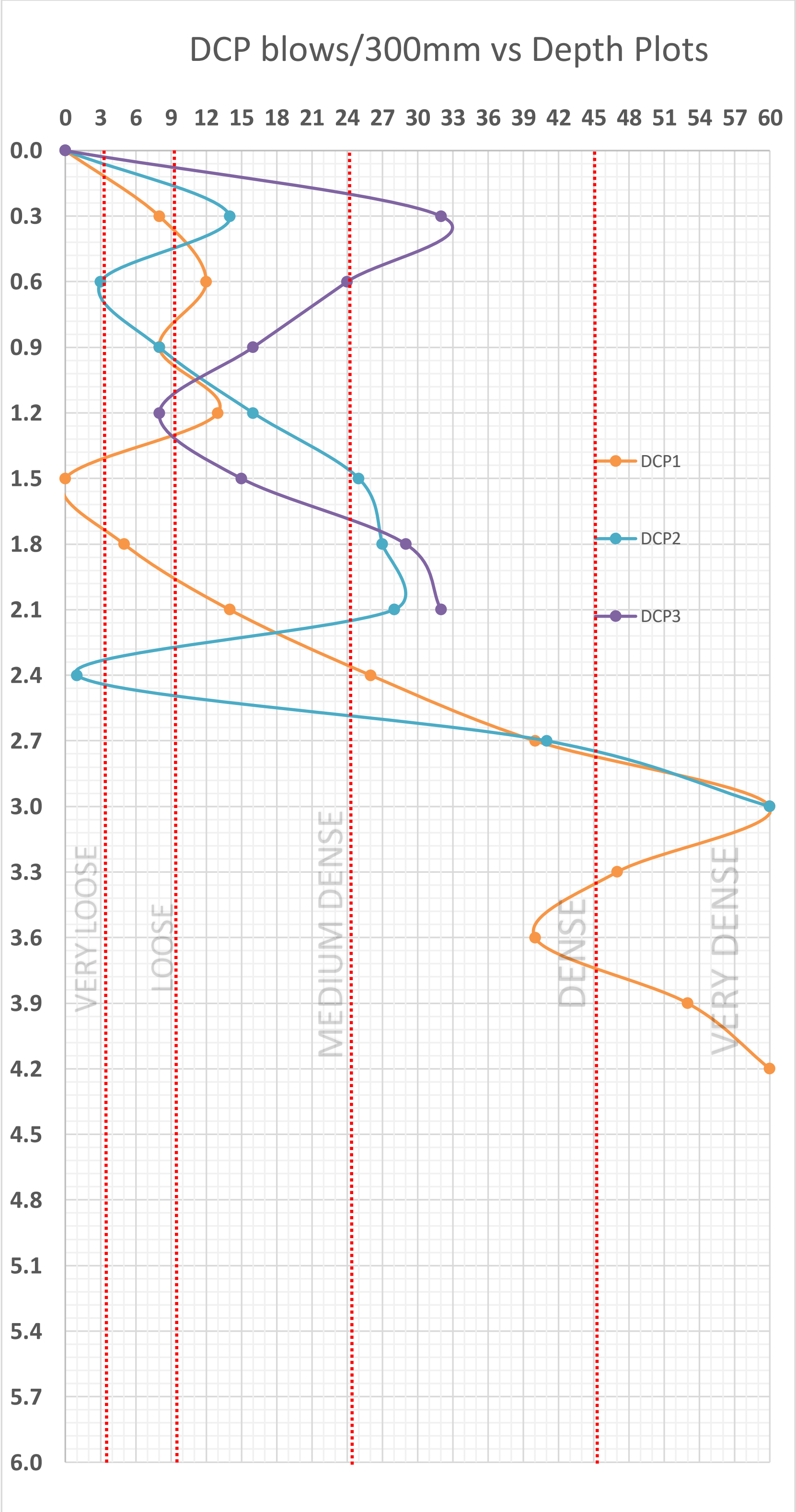
Date Completed: 21/04/2020

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	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

PROJECT: GEOTECHNICAL SITE INVESTIGATION REPORT FOR PROPOSED NEW MIXED-USE BUILDING DEVELOPMENT

LOCATION: 28 FISHER ROAD & 9 FRANCIS STREET, DEE WHY NSW 2099

DATE: 6/05/2020

PROJECT NO.: SRE/524/DW/19

PAGE: 1 of 1

DATE RECORD: 21/04/2020

LOGGED BY: AT

CHECKED BY: JC

SITE PHOTOGRAPHS

Photo 1 - West view to BH1 test location.

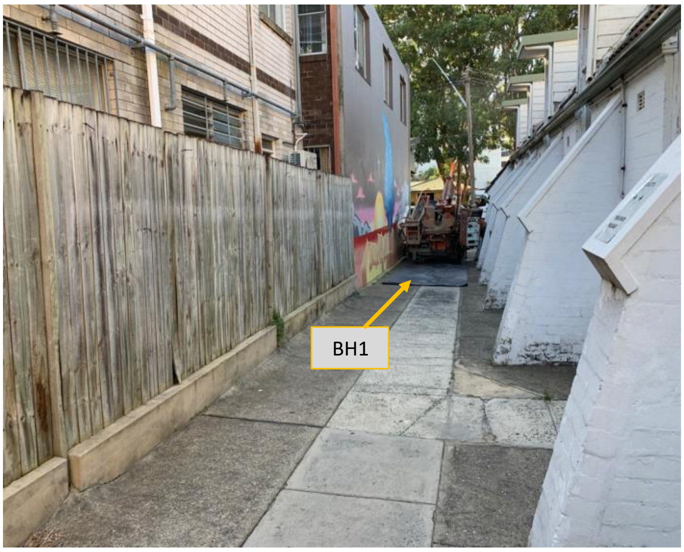


Photo 2 - East view to BH1 test location.

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.