REPORT TO

MELISSA ZHOU

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED RESIDENTIAL DEVELOPMENT

AT

3 BERITH STREET, WHEELER HEIGHTS, NSW

Date: 26 March 2021

Ref: 32859SFrptRev1

JKGeotechnics

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32859SFrpt	Final Report	20 January 2020
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ATTACHMENTS

1 INTRODUCTION

This report presents the results of a geotechnical investigation for a proposed residential development at 3 Berith Street, Wheeler Heights, NSW. The investigation was commissioned by Mr Barry Rush of Barry Rush & Associates Pty Ltd on behalf of Ms Melissa Zhou (client) by email dated 3 January 2020. The investigation was carried out in accordance with our proposal, Ref: P50954YF, dated 2 January 2020. The location of the site is shown in Figure 1. This report has been prepared based on the latest architectural drawings accompanying the modification application and supersedes our previous geotechnical report dated 20 January 2020.

We were supplied with the following relevant documents:

- Architectural drawings by Barry Rush & Associates Pty Ltd (Job No. 1801, Version S4.55, Dwg. A01 to A08);
- Landscape plan by Greenland Design (Ref.: 3 Berith Street, Wheeler Heights, NSW, Issue A, Drawing No. 1921.GD.01 to 02, dated September 2018);
- Survey plan by Donovan & Associates (Ref.: 1297/147760, survey dated 16 may 2017).
- Geotechnical Investigation report prepared by White Geotechnical Group (WGG), Ref: J2436, dated 17 October 2019.
- Reference should also be made to our previous geotechnical letter, Ref: 32859YFlet, dated 13 December 2019 which contains our peer review of the report by WGG and also our slope risk assessment for the site.

From the provided documents above, we understand that it is proposed to demolish the existing site structures and construct a two storey residential development over one basement level. The basement level has a proposed Finished Floor Level (FFL) between RL72.65m and RL73.10m resulting in excavations of approximately 2.6m in the north-west corner and 5.3m in the south-east corner of the subject site, although some locally deeper excavations will occur, such as for the lift pit. This is based upon an estimated Bulk Excavation Level (BEL) between RL72.4m and RL72.9m which allows for the assumed drainage blanket and basement slab. The proposed basement will be set back from the common boundaries by approximately 2.3m, 4.0m, 2.0m and 9.6m from the northern, eastern, southern and western boundaries, respectively.

The purpose of the investigation was to obtain geotechnical information on subsurface conditions and to use this as a basis for comments and recommendations on excavation conditions, retention systems, hydrogeological considerations, footings, subgrade preparation and basement slabs.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation comprised drilling of two boreholes (BH101 and BH102) initially drilled using hand auger to 0.5m to 1.0m below existing surface. The boreholes then extended to depths ranging from 8.83m and 6.64m below surface level in BH101 and BH102, respectively, using a TT56 double tube barrel fitted with a diamond coring bit and water flush.

The boreholes were augmented by two Dynamic Cone Penetrometer (DCP) tests (DCP101 and DCP102) extending to refusal depths of 0.5m and 1.0m below existing surface level. The purpose of the DCP tests was to interpret the degree of compaction/relative density of the soil profile, with the hand augering used to identify the soils present.

Where bedrock was diamond cored, the recovered core was returned to our NATA registered laboratory (Soil Test Services (STS)) for photographing and Point Load Strength Index (Is_{50}) testing. Using established correlations the Unconfined Compressive Strength (UCS) of the bedrock was then calculated from the Is_{50} results. Copies of the colour photographs are provided with the borehole logs.

The fieldwork was completed in the full-time presence of our geotechnical engineer who set out the borehole locations, nominated the testing and sampling, and prepared the attached borehole logs and DCP sheet. The borehole locations are shown on the attached Figure 2, and these were set out by taped measurements from assumed site boundaries as shown on survey plan. The relative levels shown on the attached logs were interpolated from spot heights shown on the survey plan and are therefore approximate. The height datum used is the Australian Height Datum (AHD). For more details of the investigation procedures and their limitations, reference should be made to the attached Report Explanation Notes.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site location is shown in Figure 1. The site is located within hilly topography on the Collaroy Plateau and is situated on a north-west facing hill that slopes down at about 4°. The site itself is rectangular in plan measuring about 63m (east-west) by 20.1m (north-south) with surface levels ranging from RL78.7m at the south-eastern corner to RL74m at the north-western corner, which is an overall fall of about 4°.

The site contains a two-storey brick residence with tile roof and a nearly oval shape inground pool centrally positioned within the site. In front of the residence there is a concrete driveway along the northern boundary. In the backyard there is a detached timber shed located in the south-eastern corner of the site. Apart from these structures mentioned above, the external areas of the site predominantly comprise of grassed areas with patches of shrubs and medium to large sized trees. The building, driveway and pool are generally in good external condition; however, the timber shed is in a poor condition.

Two thirds of the northern site boundary (timber paling fence in moderate condition) is bounded by the long-axis side of 1 Berith Street while the remainder is bounded by the short-axis sides of 25 and 27 Rose Avenue. 1 Berith Street contains a single level rendered brick residence with a setback of about 1.2m from site boundary. The timber shed in 27 Rose Avenue backyard is setback by about 2m from site boundary. The rear of 25 Rose Avenue is a garden area with some large trees. Landform and ground levels remain similar across the site boundaries when viewed within our site.

To the south of the site is 5 Berith Street which contains a centrally positioned single storey brick residence and a detached brick garage. The buildings appear in good external condition. There is a swimming pool in the back yard. Along the southern site boundary there is low 0.5m high brick retaining wall which covers the western one-third portion of the boundary. The retaining wall appears to lean towards the subject site at 2° to 3°. The retaining wall is then followed to the east by timber paling fence in moderate condition and extend to the eastern end of the site. Ground levels remain similar across the site boundary when viewed within our site.

The site is bounded by a basketball court to the east. Ground levels remain similar across the site boundary (timber paling fence) when viewed within our site.

The site also has a western frontage to Berith Street, which is a two lanes asphaltic concrete paved road in moderate to good condition. The pavement falls towards north at about 3°.

3.2 Subsurface Conditions

3.2.1 Current Investigation

The 1:100,000 Geological Map of Sydney indicates the site to be underlain by Hawkesbury Sandstone of the Wianamatta Group comprising medium to coarse grained quartz sandstone with very minor siltstone lenses and laminite lenses.

In summary, both boreholes disclosed a similar subsurface profile generally comprising shallow sand fill and residual clayey sand overlying relatively shallow sandstone bedrock. Reference should be made to the borehole logs for detailed descriptions of the subsurface conditions encountered. A summary of the encountered subsurface conditions is provided below.

Fill

Silty sandy fill was encountered in both boreholes from the surface to 0.2m below existing surface level. The fill in BH101 contained root inclusions. The fill was generally assessed to be poorly compacted based on DCP test results.

Residual Clayey Sand

Residual clayey sand was encountered below the fill in both boreholes. Based on the results from DCP tests, the sandy soils were assessed to be dense. The residual soils contained varying proportions of extremely weathered sandstone bands and sandstone gravels. The soils were assessed to be dry.

Sandstone Bedrock

Initially in both boreholes, sections of "no core" were encountered at the commencement of core drilling at 0.50m (≈RL77.0m) in BH101 and 1.0m (≈RL74.2m) in BH102. These "no core" sections are usually the result of poor quality extremely weathered shale or sandstone with soil properties or bands of clayey sands which have been washed away by the drill flush water.

Following the "no core" sections, sandstone of medium strength or better was then encountered at depths of 0.9m, or about RL76.6m, in BH101 and 1.8m, or about RL73.4m, in BH102 indicating the bedrock is sloping down similar to the natural hillside. Notwithstanding, a number of no core sections were also encountered in both boreholes ranging in thickness of 0.1m to 1.0m.

Apart from the no core sections, most defects contained within the rock core are sub-horizontal bedding partings and a number of extremely weathered seams. Three joints with inclination ranging from 50° to 90° were encountered in the boreholes at 3.5m, 8.36m (BH101) and 6.06m (BH102).

3.2.2 Previous Investigation by WGG

The previous investigation undertaken by White Geotechnical Group was limited to handheld equipment comprising of one hand auger borehole augmented by six DCP tests. The auger borehole extended to inferred bedrock at a refusal depth of 1.0m below surface level, or about RL75.0m. The DCP tests extended to refusal depths between 0.6m and 1.5m below existing surface level, or between RL77.4m in the south-east corner of the site to RL73.6m in the north-west corner of the site. No groundwater was encountered during the investigation. As the investigation was limited to handheld equipment, no proving of the bedrock was carried out. We note, the results of the WGG investigation correlate well with the results from our investigation.

3.3 Laboratory Test Results

The results of the of the Point Load Strength Index test correlated well with the field logging assessments of the rock strength. The correlated Unconfined Compressive Strength (UCS) tests on the intact rock core indicated the bedrock ranges from 6MPa to 36MPa.

4 COMMENTS AND RECOMMENDATIONS

4.1 Excavation Conditions

All excavation recommendations should be complemented by reference to the latest version of Safe Work Australia's '*Excavation Work Code of Practice*' and by reference to AS3798-2007 *'Guidelines on Earthworks for Commercial and Residential Developments'*.

4.1.1 Dilapidation Surveys

Prior to the commencement of excavation, we recommend that dilapidation surveys be completed on the neighbouring buildings to the north at 1 Berith Street and the south at 5 Berith Street given the subsurface conditions and the proposed excavation depth.

The dilapidation surveys should include internal and external inspections of the buildings, where all defects including defect location, type, length and width are described and photographed. The respective owners of the neighbouring buildings should be asked to confirm that the dilapidation survey reports present a fair record of existing conditions. The dilapidation survey reports may be used as a benchmark against which to assess possible future claims for damage arising from the works and protect against spurious claims for existing damage.

4.1.2 Excavation Methods

We expect an excavation depth between 2.6m and 5.3m below existing surface level will be required in order to achieve the Bulk Excavation Level between about RL72.4m and RL72.9m. Based on the results of the investigation, the basement excavation will encounter pavements, fill, natural clayey sands and sandstone bedrock. An assessment of the excavation characteristics of the various strata is presented below.

Excavation of the soils and up to very low strength sandstone will be achievable using conventional excavation equipment, such as the buckets of hydraulic excavators. Excavation of the sandstone bedrock of low strength or better will require the use of rock excavation equipment, such as hydraulic rock hammers, rotary grinders, ripping hooks and rock saws. Sandstone of high strength is likely to be encountered and will represent 'hard rock' excavation conditions. The excavator contractor should be made aware of this by being supplied with all geotechnical information; low productivity and increased equipment wear should be expected due to the rock strength.

4.1.3 Potential Vibration Risk

If rock hammers are to be used, we recommend that the initial excavation in rock be commenced away from likely critical areas, with electronic vibration monitoring undertaken. Trial excavations can then be undertaken together with vibration monitoring to assess how close the hammer can operate to any critical boundary, while maintaining transmitted vibrations within acceptable levels. Guideline levels of vibration velocity for evaluating the effects of vibration in structures are given in the attached Vibration Emission Design Goals sheet. We recommend that the acceptable limit for transmitted vibrations be set at quite a low peak particle velocity of 5mm/s for frequencies of less than 10Hz at foundation level.

To fall within these limits, we recommend that the size of rock hammers initially used during the trial not exceed medium sized rock hammer say 900kg such as Krupp 580. If it is found that transmitted vibrations are unacceptable, then it would be necessary to change to a smaller excavator with a smaller rock hammer, or to a rotary grinder, rock saws, or jackhammers. Should this monitoring indicate that transmitted vibrations are trending towards the predetermined limits provided in the attached 'Vibration Emission Design Goals' then continuous vibration monitoring should be carried out for the remainder of all percussive rock excavation.

The use of a rotary rock grinders or grid rock sawing in conjunction with ripping and/or hammering also provides relatively low vibration excavation techniques. When using a rock saw or rotary rock grinder, the resulting dust must be suppressed by spraying with water.

Only excavation contractors with experience in similar work using a competent supervisor who is aware of vibration damage risks and rock face instability issues, etc. should be used. The contractor should have all appropriate statutory and public liability insurances.

4.2 Retention Systems

Given the proposed basement set back from the common boundaries and the expected depth to top of bedrock, we expect that generally temporary batters and then vertical excavation of the sandstone bedrock is feasible. Based on the expected soils, temporary batters should be formed no steeper than 1 Vertical (V) to 1.5 Horizontal (H). If more sandy soils are encountered, then we recommend the batters are flattened to 1V:2H. At the north-western corner of the proposed basement where the basement approaches closer to the boundary, temporary batters may not be feasible due to space constraints, but this will be dependent on the depth to the good quality sandstone bedrock. If temporary batters are not feasible, then a shotcrete and soil nail option may be viable but further advice can be provided once the bedrock level is confirmed at this location.

The sandstone bedrock of low strength or better can be cut vertically, but must be progressively inspected by a geotechnical engineer at no more than 1.5m depth increments to assess the need for temporary support (e.g. rock bolts, dowels, shotcrete etc.) of potentially unstable rock wedges or extremely weathered bands. We note bedrock potentially suitable for vertical excavation was encountered below approximately RL76.6m in BH101 and RL73.1m in BH102.

The boreholes indicated frequent seams of extremely weathered and/or clay bands up to 1.0m thick that will require support. If exposed rock faces are proposed in the car park (with dish drains accessible for cleaning then) then we recommend installing reinforced shotcrete dowelled into the stronger material, including strip drains to prevent the build-up of any hydrostatic pressures behind the shotcrete face. Thin seams may simply be grubbed out to a depth equal to the height of the seam and non-shrink grout applied to prevent fretting of the material in the long-term (with weep holes at regular intervals to allow hydrostatic pressures to dissipate). A provision should be made in the contract documents (budget and program) for the above inspections and stabilisation measures, noting that a substantial amount of shotcreting should be anticipated unless full height retaining walls are adopted.

Alternatively to the above treatment, full height retaining walls could be completed when the excavation is completed, with only essential short-term stabilisation works carried out during excavation. This will reduce (possibly greatly) the excavation time and the permanent walls will present a better, lower maintenance, finish in the basement. Suitable wall types include reinforced blockwork, Dincel and CSR Rediwall. Provided no major instability is present due to adverse jointing, the walls could be designed for an earth pressure of 10kPa. Walls should be backfilled with 'blue metal' or no-fines concrete with sub-soil drains around the base.

Note that in the (unlikely) event that a major wedge instability is present then whichever means of excavation support is adopted, the design would have to be adjusted to provide the required support in the long-term to replace temporary rock bolts, etc.

4.3 Hydrogeological Considerations

Groundwater was not encountered during the investigation however we expect some groundwater inflow into the excavation will occur as local seepage flows at the soil/rock interface, as well as through joints and bedding partings within the bedrock profile, particularly during and immediately following periods of rainfall.

In the event that groundwater is encountered during construction we expect the inflows could be controlled by conventional sump and pump drainage. In the long term, drainage should be provided around the final excavation perimeter and below the floor slabs. The completed excavation should be inspected by the hydraulic engineer to confirm that the designed drainage is sufficient for the actual seepage flows. If rock faces are left exposed then access to clean out dish drains should be provided. We caution against placing dry walls in front of the rock face as removal of the debris that inevitably frets from the rock faces into drains becomes problematic and will lead to dampness. A possible solution would be to apply shotcrete to all rock faces (with appropriate drainage provisions), noting that the fractured nature of the rock and significant thicknesses of weathered seams will require substantial areas be shotcreted anyway, unless full height retaining walls are adopted.

Over the long term the proposed provision of effective drainage of all sub-structures will allow 'through-flow' of groundwater with no build-up of groundwater levels to the extent that neighbouring properties will be adversely affected. In view of the above, the proposed development will have negligible effects on the groundwater regime above and below the site and the neighbouring buildings and structures. However, we recommend that the proposed excavations be monitored to confirm groundwater conditions. We also note that effective control of surface run-off will be required both during and after construction.

4.4 Footings

Based on the results of the investigation, it appears sandstone of medium to high strength will be encountered at the bulk excavation level. However, the investigation results also indicate the presence of extremely weathered and/or clay seams that will likely fall within the footings zone of influence. As such, we recommend the footing bearing pressures are limited to an Allowable Bearing Pressure of 1,000kPa. Given the presence of the weathered and/or clay seams, we recommend all footings are inspected by a geotechnical engineer prior to pouring concrete.

4.5 Basement Slab

Based on the investigation results, the proposed basement floor slab will directly overlie sandstone bedrock. We therefore recommend that underfloor drainage blanket be provided. The underfloor drainage should comprise a strong, durable, single-sized washed aggregate such as 'blue metal' gravel. The underfloor drainage should connect with the perimeter drains and lead groundwater seepage to a sump for pumped disposal to the stormwater system.

Joints in the basement concrete on-grade floor slabs should be designed to accommodate shear forces but not bending moments by using dowelled or keyed joints.

4.6 Earthquake Design Factors

For earthquake design for the proposed residential development in accordance with AS1170.4-2007 'Structural Design Actions, Part 4: Earthquake Actions in Australia', the following design parameters should be adopted, provided there is no more than 3m of soil above bedrock:

- Hazard Factor (Z) = 0.08
- Site Subsoil Class = Class B_e

4.7 Further Geotechnical Input

The following is a summary of the further geotechnical input which is required and which has been detailed in the preceding sections of this report:

- Dilapidation surveys for the neighbouring structures, especially as rock hammers are almost certain to be used.
- Monitoring of groundwater seepage into the excavation to confirm drainage requirements.
- At least initial Vibration Monitoring during bulk excavation.
- Progressive inspection of excavations and cut faces to confirm if additional support or treatment is required.
- Footing inspections and testing

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

Occasionally, the subsurface conditions between the completed boreholes may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

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

A waste classification is required for any soil and/or bedrock excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), Excavated Natural Material (ENM), General Solid, Restricted Solid or Hazardous Waste. Analysis can take up to seven to ten working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) could be expected. We strongly recommend that this requirement is addressed prior to the commencement of excavation on site.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. If there is any change in the proposed development described in this report then all recommendations should be reviewed. Copyright in this report is the property of JK Geotechnics. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

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SOIL TEST SERVICES

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	JK Geotechnics	**Ref No:**	32859YF
Project:	Proposed Residential Development	**Report:**	A
Location:	3 Berith Street, Wheeler Heights, NSW	**Report Date:**	8/01/2020
		Page 1 of 1	

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
101	1.03 - 1.06	1.6	32
	1.60 - 1.64	0.5	10
	2.44 - 2.47	1.7	34
	2.87 - 2.91	0.8	16
	3.14 - 3.18	0.7	14
	3.70 - 3.74	0.4	8
	5.24 - 5.28	0.5	10
	5.71 - 5.75	0.8	16
	6.59 - 6.63	0.5	10
	7.08 - 7.12	0.7	14
	7.68 - 7.72	1.6	32
	8.4 - 8.48	1.0	20
102	2.03 - 2.06	1.8	36
	2.80 - 2.84	1.8	36
	3.36 - 3.40	4.6	92
	4.65 - 4.68	1.3	26
	5.33 - 5.36	0.7	14
	5.79 - 5.83	0.3	6
	6.12 - 6.16	0.6	12
	6.39 - 6.44	0.4	8

NOTES:

1. In the above table testing was completed in the Axial direction.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RMS T223.
4. For reporting purposes, the $I_{S(50)}$ has been rounded to the nearest 0.1MPa, or to one significant figure if less than 0.1MPa
5. The Estimated Unconfined Compressive Strength was calculated from the Point Load Strength Index by the following approximate relationship and rounded off to the nearest whole number :
   U.C.S. = 20 $I_{S\,(50)}$

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BOREHOLE LOG

Client: MELISSA ZHOU
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 3 BERITH STREET, WHEELER HEIGHTS, NSW

Job No.: 32859YF **Method:** HAND AUGER **R.L. Surface:** ~77.5 m
Date: 6/1/20 **Datum:** AHD
Plant Type: **Logged/Checked By:** B.Z./O.F.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING					REFER TO DCP TEST RESULTS		0			FILL: Silty sand, fine to medium grained, dark brown, with roots.	D			GRASS COVERED
				■					SC	Clayey SAND: medium to coarse grained, yellow brown, low to medium plasticity clay, with extremely weathered sandstone bands.	D	D		APPEARS POORLY COMPACTED RESIDUAL
						77				REFER TO CORED BOREHOLE LOG				HAND AUGER REFUSAL
							1							
						76								
							2							
						75								
							3							
						74								
							4							
						73								
							5							
						72								
							6							
						71								

Client: MELISSA ZHOU
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 3 BERITH STREET, WHEELER HEIGHTS, NSW

Job No.: 32859YF | **Core Size:** TT56 | **R.L. Surface:** ~77.5 m
Date: 6/1/20 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: MELVELLE | **Bearing:** N/A | **Logged/Checked By:** B.Z./O.F.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS: DESCRIPTION (Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness) Specific / General	Formation
					START CORING AT 0.50m					
100% RETURN		77			NO CORE 0.45m					
		76	1		SANDSTONE: medium to coarse grained, yellow brown and red brown, with iron staining, indistinctly thinly cross bedded, inclined up to 15°.	MW	M - H	1.6, 0.50	(1.32m) Be, 5°, Un, R, Cn	Hawkesbury Sandstone
					as above, but light grey.				(1.77m) XWS, 0°, 20 mm.t, CLAYEY SAND; (1.83m) J, 75°, Un, R, FILLED ROOTS	
			2		Extremely Weathered sandstone: clayey SAND, fine to medium grained, light grey, low to medium plasticity clay.	XW	(D)		(1.95-2.17m) XWS, 0°, 220 mm.t, CLAYEY SAND	
					NO CORE 0.10m					
		75, 74	3, 4		SANDSTONE: medium to coarse grained, yellow brown, with iron staining, with medium to coarse grained sub angular quartz clasts inclusion, distinctly very thinly cross bedded, inclined up to 15°.	MW	M - H / M	1.7, 0.80, 0.70, 0.40	(2.37m) Be, 0°, Un, R, Qz FILLED; (2.61m) Be, 6°, Un, R, Qz FILLED; (2.75m) Be, 7°, Un, R, Qz FILLED; (3.31m) Be, 13°, Un, R, FILLED, SAND, 2mm.t; (3.50m) Ji, 90°, Un, Cn; (3.95m) Be, 0°, Un, R, Fe Sn	Hawkesbury Sandstone
		73	5		NO CORE 1.00m					
		72			SANDSTONE: medium to coarse grained, yellow brown and light brown, with medium to coarse grained sub angular quartz clasts inclusion, indistinctly very thinly cross bedded, inclined up to 15°.	MW - SW	M	0.50, 0.80	(5.43m) Be, 0°, Un, R, FILLED, SAND, 2mm.t; (5.80m) Be, 15°, Un, R, Fe Sn	Hawkesbury Sandstone
			6		NO CORE 0.40m					
		71			SANDSTONE: medium to coarse grained, light grey mottled brown and red brown, distinctly very thinly cross bedded, inclined up to 20°, with iron staining.	MW - SW	M	0.50	(6.51m) Be, 0°, Un, R, Fe Sn; (6.96m) Be, 3°, P, R, FILLED, SAND, 2mm.t	

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CORED BOREHOLE LOG

Borehole No.
101

Client: MELISSA ZHOU
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 3 BERITH STREET, WHEELER HEIGHTS, NSW

Job No.: 32859YF | **Core Size:** TT56 | **R.L. Surface:** ~77.5 m
Date: 6/1/20 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: MELVELLE | **Bearing:** N/A | **Logged/Checked By:** B.Z./O.F.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS: SPACING (mm)	DEFECT DETAILS: DESCRIPTION (Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness) Specific / General	Formation
100% RETURN		70			SANDSTONE: medium to coarse grained, light grey mottled brown and red brown, distinctly very thinly cross bedded, inclined up to 20°, with iron staining. *(continued)*	MW - SW	M	0.70 1.6			Hawkesbury Sandstone
			8		NO CORE 0.43m						Hawkesbury Sandstone
		69			SANDSTONE: medium to coarse grained, light grey and brown.	MW	M	1.0		(8.36m) J, 65°, Un, R, Cn (8.55m) XWS, 0°, 25 mm.t, FILLED SANDY CLAY	Hawkesbury Sandstone
					END OF BOREHOLE AT 8.63 m						
		68	9								
		67	10								
		66	11								
		65	12								
		64	13								

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JK Geotechnics
32859YF, BH101, CORING STARTS AT: 0.5m
0
0.5m →
CORE LOSS 0.45m
1
2
CORE LOSS 0.10m
3
4
CORE LOSS 1.00m
5
6
CORE LOSS 0.40m
7
CORE LOSS 0.44m
8
← 8.63m END

JKGeotechnics

BOREHOLE LOG

Borehole No.
102
1 / 2

Client: MELISSA ZHOU
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 3 BERITH STREET, WHEELER HEIGHTS, NSW

Job No.: 32859YF **Method:** HAND AUGER **R.L. Surface:** ~75.2 m
Date: 6/1/20 **Datum:** AHD
Plant Type: **Logged/Checked By:** B.Z./O.F.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING					REFER TO DCP TEST RESULTS	75				FILL: Silty sand, fine to medium grained, dark brown.	D			APPEARS POORLY COMPACTED
									SC	Clayey SAND: fine to medium grained, yellow brown, trace fine to medium grained, sub angular sandstone gravel.	D	D		RESIDUAL
						74	1			REFER TO CORED BOREHOLE LOG				HAND AUGER REFUSAL
						73	2							
						72	3							
						71	4							
						70	5							
						69	6							

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CORED BOREHOLE LOG

Borehole No. 102

Client: MELISSA ZHOU
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 3 BERITH STREET, WHEELER HEIGHTS, NSW

Job No.: 32859YF | **Core Size:** TT56 | **R.L. Surface:** ~75.2 m
Date: 6/1/20 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: MELVELLE | **Bearing:** N/A | **Logged/Checked By:** B.Z./O.F.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (VL -0.1, L -0.3, M -1, H -3, VH -10, EH)	DEFECT DETAILS: SPACING (mm) (600, 200, 60, 20)	DEFECT DETAILS: DESCRIPTION (Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness) Specific / General	Formation
			0–1.00		START CORING AT 1.00m						
100% RETURN		74	1.00–1.80		NO CORE 0.80m						
		73	1.80–3.55		SANDSTONE: medium to coarse grained, brown, with medium to coarse grained, sub angular quartz gravel, distinctly very thinly cross bedded, inclined up to 20°.	MW	M - H	1.8, 1.8, 4.6		(2.28m) Be, 10°, Un, R, Fe Vn (2.35m) Be, 90°, Un, R, Fe Vn (2.41m) Be, 0°, Un, R, Fe Vn (2.44m) Be, 12°, Un, R, Fe Sn (2.57m) Be, 0°, Un, R, Fe Sn (2.68m) Be, 3°, Un, Vr, Fe Sn (2.74m) Be, 10°, Un, Vr, Fe Sn (2.95m) Be, 3°, Un, Vr, Fe Sn (3.44m) Be, 22°, Un, Vr, Fe Vn (3.52m) Be, 0°, Un, R, Fe Vn	Hawkesbury Sandstone
		72, 71	3.55–4.40		NO CORE 0.85m						
			4.40–4.65		SANDSTONE: medium to coarse grained, brown mottled red brown with fine to medium grained quartz clasts.	MW	M - H	1.3		(4.60m) Be, 5°, Un, R, Fe Vn	
		70	4.65–5.18		NO CORE 0.53m						
			5.18–5.70		SANDSTONE: medium to coarse grained, brown, distinctly very thinly plane bedded, inclined up to 10°.	MW	M	0.70		(5.45m) Be, 0°, Un, R, Fe Sn (5.64m) Be, 0°, Un, R, Qz FILLED	Hawkesbury Sandstone
		69	5.70–6.64		SANDSTONE: medium to coarse grained, light grey, massive	FR		0.30, 0.60, 0.40		(6.06m) J, 50°, C, R, Cn (6.36m) Be, 3°, Un, Vr, Cn	
		68	6.64–		END OF BOREHOLE AT 6.64 m						

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


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JOB No. 32859YF BH102 CORE START AT: 1.0m
1 CORE LOSS 0.80m
2
3 CORE LOSS. 0.85m
4 CORE LOSS 0.53m
5
6 ←6.64m END

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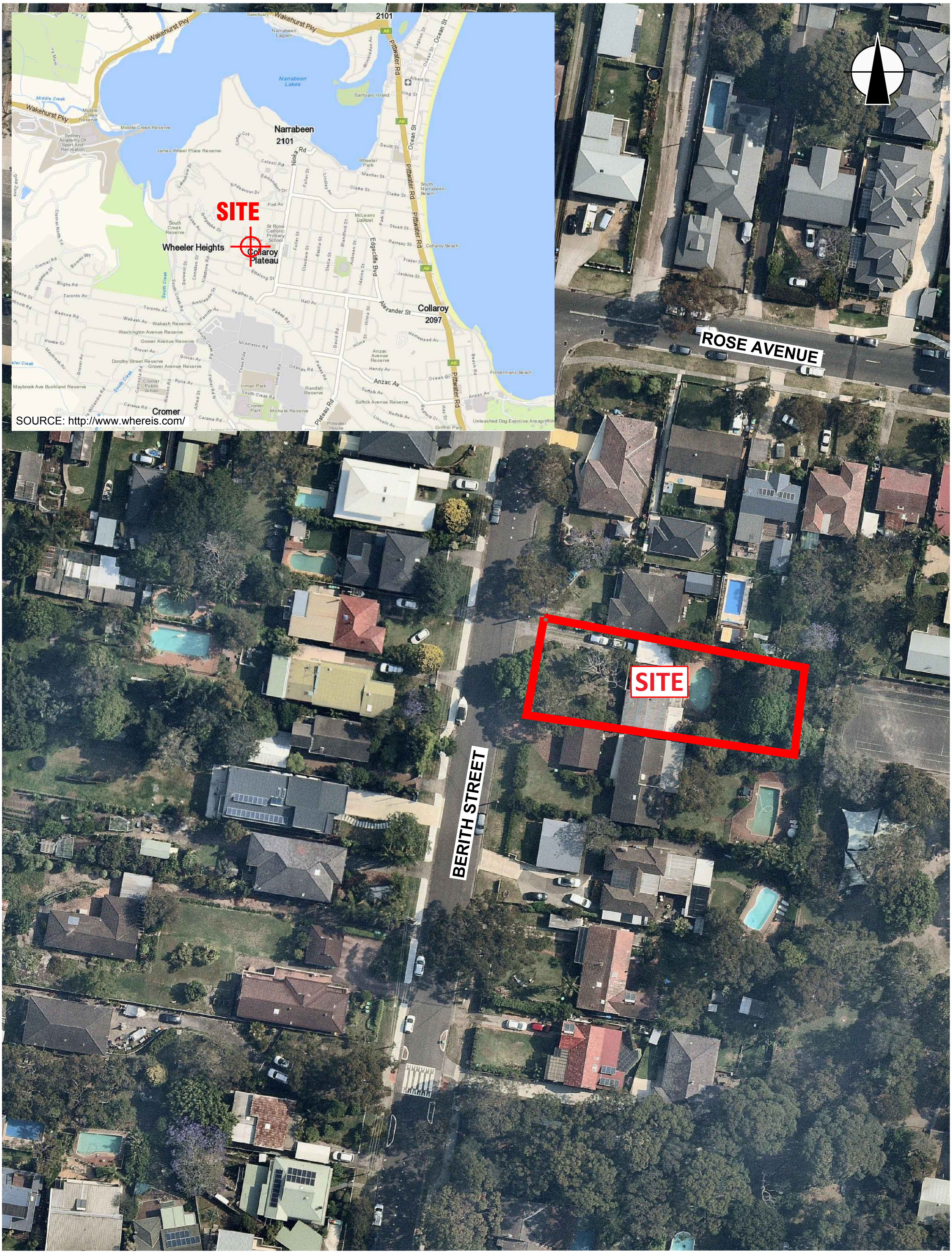
DYNAMIC CONE PENETRATION TEST RESULTS

Client:	MELISSA ZHOU		
Project:	PROPOSED RESIDENTIAL DEVELOPMENT		
Location:	3 BERITH STREET, WHEELER HEIGHTS, NSW		
Job No.	32859YF	Hammer Weight & Drop: 9kg/510mm	
Date:	6-1-20	Rod Diameter: 16mm	
Tested By:	B.Z	Point Diameter: 20mm	

Test Location	**101**	**102**					
Surface RL	≈77.5m	≈75.2m					
Depth (mm)	Number of Blows per 100mm Penetration						
0 - 100	2	4					
100 - 200	4	13					
200 - 300	9	11					
300 - 400	18	13					
400 - 500	19	17					
500 - 600	REFUSAL	13					
600 - 700		11					
700 - 800		11					
800 - 900		12					
900 - 1000		20					
1000 - 1100		REFUSAL					
1100 - 1200							
1200 - 1300							
1300 - 1400							
1400 - 1500							
1500 - 1600							
1600 - 1700							
1700 - 1800							
1800 - 1900							
1900 - 2000							
2000 - 2100							
2100 - 2200							
2200 - 2300							
2300 - 2400							
2400 - 2500							
2500 - 2600							
2600 - 2700							
2700 - 2800							
2800 - 2900							
2900 - 3000							

Remarks:
1. The procedure used for this test is described in AS1289.6.3.2-1997 (R2013)
2. Usually 8 blows per 20mm is taken as refusal
3. Datum of levels is AHD

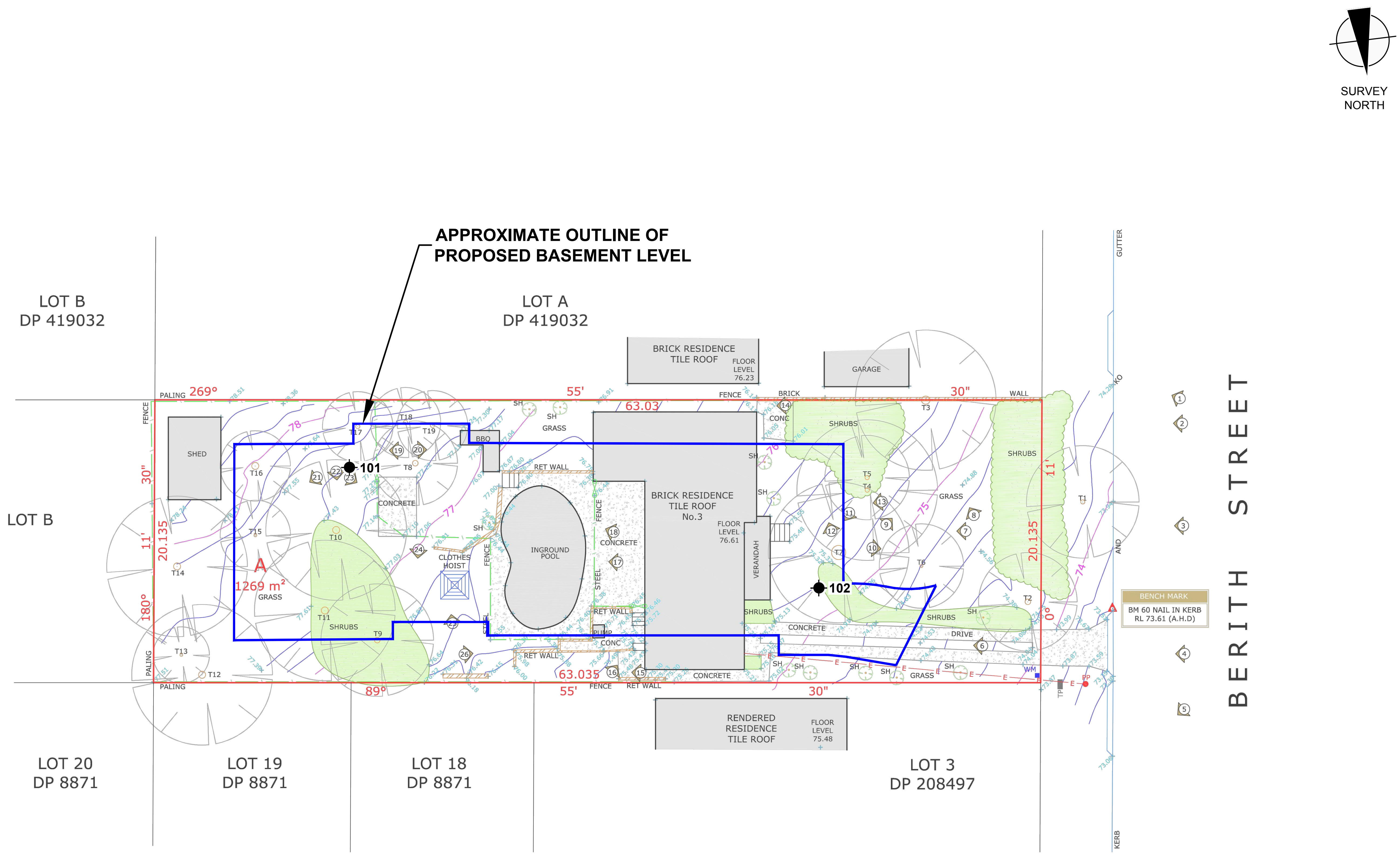


AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:	SITE LOCATION PLAN
Location:	3 BERITH STREET, WHEELER HEIGHTS, NSW
Report No: 32859YF	Figure No: 1
	JKGeotechnics

PLOT DATE: 21/01/2020 4:28:25 PM DWG FILE: X:\32859YF WHEELER HEIGHTS\CAD\32859YF.DWG



LEGEND

- BOREHOLE AND DCP TEST

Title:	**BOREHOLE LOCATION PLAN**
Location:	3 BERITH STREET, WHEELER HEIGHTS, NSW
Report No:	32859YF
Figure No:	2

SCALE 0 2 4 6 8 10 1:200 @A3 METRES

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

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PLOT DATE: 26/03/2021 10:50:07 AM DWG FILE: Y:\32000'S\32859YF WHEELER HEIGHTS\CAD\32859YF.DWG

VIBRATION EMISSION DESIGN GOALS

German Standard DIN 4150 – Part 3: 1999 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally recognised to be conservative.

The DIN 4150 values (maximum levels measured in any direction at the foundation, OR, maximum levels measured in (x) or (y) horizontal directions, in the plane of the uppermost floor), are summarised in Table 1 below.

It should be noted that peak vibration velocities higher than the minimum figures in Table 1 for low frequencies may be quite 'safe', depending on the frequency content of the vibration and the actual condition of the structure.

It should also be noted that these levels are 'safe limits', up to which no damage due to vibration effects has been observed for the particular class of building. 'Damage' is defined by DIN 4150 to include even minor non-structural effects such as superficial cracking in cement render, the enlargement of cracks already present, and the separation of partitions or intermediate walls from load bearing walls. Should damage be observed at vibration levels lower than the 'safe limits', then it may be attributed to other causes. DIN 4150 also states that when vibration levels higher than the 'safe limits' are present, it does not necessarily follow that damage will occur. Values given are only a broad guide.

Table 1: DIN 4150 – Structural Damage – Safe Limits for Building Vibration

Group	Type of Structure	Peak Vibration Velocity in mm/s			
		At Foundation Level at a Frequency of:			Plane of Floor of Uppermost Storey
		Less than 10Hz	10Hz to 50Hz	50Hz to 100Hz	All Frequencies
1	Buildings used for commercial purposes, industrial buildings and buildings of similar design.	20	20 to 40	40 to 50	40
2	Dwellings and buildings of similar design and/or use.	5	5 to 15	15 to 20	15
3	Structures that because of their particular sensitivity to vibration, do not correspond to those listed in Group 1 and 2 and have intrinsic value (eg. buildings that are under a preservation order).	3	3 to 8	8 to 10	8

Note: For frequencies above 100Hz, the higher values in the 50Hz to 100Hz column should be used.

REPORT EXPLANATION NOTES

INTRODUCTION

These notes have been provided to amplify the geotechnical report in regard to classification methods, field procedures and certain matters relating to the Comments and Recommendations section. Not all notes are necessarily relevant to all reports.

The ground is a product of continuing natural and man-made processes and therefore exhibits a variety of characteristics and properties which vary from place to place and can change with time. Geotechnical engineering involves gathering and assimilating limited facts about these characteristics and properties in order to understand or predict the behaviour of the ground on a particular site under certain conditions. This report may contain such facts obtained by inspection, excavation, probing, sampling, testing or other means of investigation. If so, they are directly relevant only to the ground at the place where and time when the investigation was carried out.

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726:2017 *'Geotechnical Site Investigations'*. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the predominating particle size and behaviour as set out in the attached soil classification table qualified by the grading of other particles present (eg. sandy clay) as set out below:

Soil Classification	Particle Size
Clay	< 0.002mm
Silt	0.002 to 0.075mm
Sand	0.075 to 2.36mm
Gravel	2.36 to 63mm
Cobbles	63 to 200mm
Boulders	> 200mm

Non-cohesive soils are classified on the basis of relative density, generally from the results of Standard Penetration Test (SPT) as below:

Relative Density	SPT 'N' Value (blows/300mm)
Very loose (VL)	< 4
Loose (L)	4 to 10
Medium dense (MD)	10 to 30
Dense (D)	30 to 50
Very Dense (VD)	> 50

Cohesive soils are classified on the basis of strength (consistency) either by use of a hand penetrometer, vane shear, laboratory testing and/or tactile engineering examination. The strength terms are defined as follows.

Classification	Unconfined Compressive Strength (kPa)	Indicative Undrained Shear Strength (kPa)
Very Soft (VS)	≤ 25	≤ 12
Soft (S)	> 25 and ≤ 50	> 12 and ≤ 25
Firm (F)	> 50 and ≤ 100	> 25 and ≤ 50
Stiff (St)	> 100 and ≤ 200	> 50 and ≤ 100
Very Stiff (VSt)	> 200 and ≤ 400	> 100 and ≤ 200
Hard (Hd)	> 400	> 200
Friable (Fr)	Strength not attainable – soil crumbles	

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, 'shale' is used to describe fissile mudstone, with a weakness parallel to bedding. Rocks with alternating inter-laminations of different grain size (eg. siltstone/claystone and siltstone/fine grained sandstone) is referred to as 'laminite'.

SAMPLING

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

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure. Bulk samples are similar but of greater volume required for some test procedures.

Undisturbed samples are taken by pushing a thin-walled sample tube, usually 50mm diameter (known as a U50), into the soil and withdrawing it with a sample of the soil contained in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shrink-swell behaviour, strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling used are given on the attached logs.

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by the Company and some comments on their use and application. All methods except test pits, hand auger drilling and portable Dynamic Cone Penetrometers require the use of a mechanical rig which is commonly mounted on a truck chassis or track base.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils and 'weaker' bedrock if it is safe to descend into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for a large excavator. Limitations of test pits are the problems associated with disturbance and difficulty of reinstatement and the consequent effects on close-by structures. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction or to design and construct the structure so as not to be adversely affected by poorly compacted backfill at the test pit location.

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Refusal of the hand auger can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Continuous Spiral Flight Augers: The borehole is advanced using 75mm to 115mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling and insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface by the flights or may be collected after withdrawal of the auger flights, but they can be very disturbed and layers may become mixed. Information from the auger sampling (as distinct from specific sampling by SPTs or undisturbed samples) is of limited reliability due to mixing or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table is of even lesser reliability than augering above the water table.

Rock Augering: Use can be made of a Tungsten Carbide (TC) bit for auger drilling into rock to indicate rock quality and continuity by variation in drilling resistance and from examination of recovered rock cuttings. This method of investigation is quick and relatively inexpensive but provides only an indication of the likely rock strength and predicted values may be in error by a strength order. Where rock strengths may have a significant impact on construction feasibility or costs, then further investigation by means of cored boreholes may be warranted.

Wash Boring: The borehole is usually advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be assessed from the cuttings, together with some information from "feel" and rate of penetration.

Mud Stabilised Drilling: Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilise the borehole. The term 'mud' encompasses a range of products ranging from bentonite to polymers. The mud tends to mask the cuttings and reliable identification is only possible from intermittent intact sampling (eg. from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, NMLC or HQ triple tube core barrels, which give a core of about 50mm and 61mm diameter, respectively, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as NO CORE. The location of NO CORE recovery is determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the bottom of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils, as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289.6.3.1–2004 (R2016) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – Standard Penetration Test (SPT)'*.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63.5kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

- In the case where full penetration is obtained with successive blow counts for each 150mm of, say, 4, 6 and 7 blows, as

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

The results of the test can be related empirically to the engineering properties of the soil.

A modification to the SPT is where the same driving system is used with a solid 60° tipped steel cone of the same diameter as the SPT hollow sampler. The solid cone can be continuously driven for some distance in soft clays or loose sands, or may be used where damage would otherwise occur to the SPT. The results of this Solid Cone Penetration Test (SCPT) are shown as 'N_c' on the borehole logs, together with the number of blows per 150mm penetration.

Cone Penetrometer Testing (CPT) and Interpretation: The cone penetrometer is sometimes referred to as a Dutch Cone. The test is described in Australian Standard 1289.6.5.1–1999 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Static Cone Penetration Resistance of a Soil – Field Test using a Mechanical and Electrical Cone or Friction-Cone Penetrometer'*.

In the tests, a 35mm or 44mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with a hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm or 165mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck. The CPT does not provide soil sample recovery.

As penetration occurs (at a rate of approximately 20mm per second), the information is output as incremental digital records every 10mm. The results given in this report have been plotted from the digital data.

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa. There are two scales presented for the cone resistance. The lower scale has a range of 0 to 5MPa and the main scale has a range of 0 to 50MPa. For cone resistance values less than 5MPa, the plot will appear on both scales.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed as a percentage.

  The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and occasionally very soft clays, rising to 4% to 10% in stiff clays and peats. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

Correlations between CPT and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of CPT values can be made to empirically derive modulus or compressibility values to allow calculation of foundation settlements.

Stratification can be inferred from the cone and friction traces and from experience and information from nearby boreholes etc. Where shown, this information is presented for general guidance, but must be regarded as interpretive. The test method provides a continuous profile of engineering properties but, where precise information on soil classification is required, direct drilling and sampling may be preferable.

There are limitations when using the CPT in that it may not penetrate obstructions within any fill, thick layers of hard clay and very dense sand, gravel and weathered bedrock. Normally a 'dummy' cone is pushed through fill to protect the equipment. No information is recorded by the 'dummy' probe.

Flat Dilatometer Test: The flat dilatometer (DMT), also known as the Marchetti Dilometer comprises a stainless steel blade having a flat, circular steel membrane mounted flush on one side.

The blade is connected to a control unit at ground surface by a pneumatic-electrical tube running through the insertion rods. A gas tank, connected to the control unit by a pneumatic cable, supplies the gas pressure required to expand the membrane. The control unit is equipped with a pressure regulator, pressure gauges, an audio-visual signal and vent valves.

The blade is advanced into the ground using our CPT rig or one of our drilling rigs, and can be driven into the ground using an SPT hammer. As soon as the blade is in place, the membrane is inflated, and the pressure required to lift the membrane (approximately 0.1mm) is recorded. The pressure then required to lift the centre of the membrane by an additional 1mm is recorded. The membrane is then deflated before pushing to the next depth increment, usually 200mm down. The pressure readings are corrected for membrane stiffness.

The DMT is used to measure material index (I_D), horizontal stress index (K_D), and dilatometer modulus (E_D). Using established correlations, the DMT results can also be used to assess the 'at rest' earth pressure coefficient (K_o), over-consolidation ratio (OCR), undrained shear strength (C_u), friction angle (ϕ), coefficient of consolidation (C_h), coefficient of permeability (K_h), unit weight (γ), and vertical drained constrained modulus (M).

The seismic dilatometer (SDMT) is the combination of the DMT with an add-on seismic module for the measurement of shear wave velocity (V_s). Using established correlations, the SDMT results can also be used to assess the small strain modulus (G_o).

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a 16mm diameter rod with a 20mm diameter cone end with a 9kg hammer dropping 510mm. The test is described in Australian Standard 1289.6.3.2–1997 (R2013) *'Methods of Testing Soils for Engineering Purposes, Soil Strength and Consolidation Tests – Determination of the Penetration Resistance of a Soil – 9kg Dynamic Cone Penetrometer Test'*.

The results are used to assess the relative compaction of fill, the relative density of granular soils, and the strength of cohesive soils. Using established correlations, the DCP test results can also be used to assess California Bearing Ratio (CBR).

Refusal of the DCP can occur on a variety of materials such as obstructions within any fill, tree roots, hard clay, gravel or ironstone, cobbles and boulders, and does not necessarily indicate rock level.

Vane Shear Test: The vane shear test is used to measure the undrained shear strength (C_u) of typically very soft to firm fine grained cohesive soils. The vane shear is normally performed in the bottom of a borehole, but can be completed from surface level, the bottom and sides of test pits, and on recovered undisturbed tube samples (when using a hand vane).

The vane comprises four rectangular blades arranged in the form of a cross on the end of a thin rod, which is coupled to the bottom of a drill rod string when used in a borehole. The size of the vane is dependent on the strength of the fine grained cohesive soils; that is, larger vanes are normally used for very low strength soils. For borehole testing, the size of the vane can be limited by the size of the casing that is used.

For testing inside a borehole, a device is used at the top of the casing, which suspends the vane and rods so that they do not sink under self-weight into the 'soft' soils beyond the depth at which the test is to be carried out. A calibrated torque head is used to rotate the rods and vane and to measure the resistance of the vane to rotation.

With the vane in position, torque is applied to cause rotation of the vane at a constant rate. A rate of 6° per minute is the common rotation rate. Rotation is continued until the soil is sheared and the maximum torque has been recorded. This value is then used to calculate the undrained shear strength. The vane is then rotated rapidly a number of times and the operation repeated until a constant torque reading is obtained. This torque value is used to calculate the remoulded shear strength. Where appropriate, friction on the vane rods is measured and taken into account in the shear strength calculation.

LOGS

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

The terms and symbols used in preparation of the logs are defined in the following pages.

Interpretation of the information shown on the logs, and its application to design and construction, should therefore take into account the spacing of boreholes or test pits, the method of drilling or excavation, the frequency of sampling and testing and the possibility of other than 'straight line' variations between the boreholes or test pits. Subsurface conditions between boreholes or test pits may vary significantly from conditions encountered at the borehole or test pit locations.

GROUNDWATER

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

- Although groundwater may be present, in low permeability soils it may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes and may not be the same at the time of construction.
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must be washed out of the hole or 'reverted' chemically if reliable water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after the groundwater level has stabilised at intervals ranging from several days to perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from perched water tables or surface water.

FILL

The presence of fill materials can often be determined only by the inclusion of foreign objects (eg. bricks, steel, etc) or by distinctly unusual colour, texture or fabric. Identification of the extent of fill materials will also depend on investigation methods and frequency. Where natural soils similar to those at the site are used for fill, it may be difficult with limited testing and sampling to reliably assess the extent of the fill.

The presence of fill materials is usually regarded with caution as the possible variation in density, strength and material type is much greater than with natural soil deposits. Consequently, there is an increased risk of adverse engineering characteristics or behaviour. If the volume and quality of fill is of importance to a project, then frequent test pit excavations are preferable to boreholes.

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 *'Methods of Testing Soils for Engineering Purposes'* or appropriate NSW Government Roads & Maritime Services (RMS) test methods. Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg. a three storey building) the information and interpretation may not be relevant if the design proposal is changed (eg. to a twenty storey building). If this happens, the Company will be pleased to review the report and the sufficiency of the investigation work.

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

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.
- Details of the development that the Company could not reasonably be expected to anticipate.

If these occur, the Company will be pleased to assist with investigation or advice to resolve any problems occurring.

SITE ANOMALIES

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the Company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The Company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Copyright in all documents (such as drawings, borehole or test pit logs, reports and specifications) provided by the Company shall remain the property of Jeffery and Katauskas Pty Ltd. Subject to the payment of all fees due, the Client alone shall have a licence to use the documents provided for the sole purpose of completing the project to which they relate. Licence to use the documents may be revoked without notice if the Client is in breach of any obligation to make a payment to us.

REVIEW OF DESIGN

Where major civil or structural developments are proposed or where only a limited investigation has been completed or where the geotechnical conditions/constraints are quite complex, it is prudent to have a joint design review which involves an experienced geotechnical engineer/engineering geologist.

SITE INSPECTION

The Company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related.

Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to

ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types and appropriate footing or pile founding depths, or

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

- FILL
- TOPSOIL
- CLAY (CL, CI, CH)
- SILT (ML, MH)
- SAND (SP, SW)
- GRAVEL (GP, GW)
- SANDY CLAY (CL, CI, CH)
- SILTY CLAY (CL, CI, CH)
- CLAYEY SAND (SC)
- SILTY SAND (SM)
- GRAVELLY CLAY (CL, CI, CH)
- CLAYEY GRAVEL (GC)
- SANDY SILT (ML, MH)
- PEAT AND HIGHLY ORGANIC SOILS (Pt)

ROCK

- CONGLOMERATE
- SANDSTONE
- SHALE/MUDSTONE
- SILTSTONE
- CLAYSTONE
- COAL
- LAMINITE
- LIMESTONE
- PHYLLITE, SCHIST
- TUFF
- GRANITE, GABBRO
- DOLERITE, DIORITE
- BASALT, ANDESITE
- QUARTZITE

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

Major Divisions		Group Symbol	Typical Names	Field Classification of Sand and Gravel	Laboratory Classification	
Coarse grained soil (more than 65% of soil excluding oversize fraction is greater than 0.075mm)	GRAVEL (more than half of coarse fraction is larger than 2.36mm	GW	Gravel and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 4$ $1 < C_c < 3$
		GP	Gravel and gravel-sand mixtures, little or no fines, uniform gravels	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		GM	Gravel-silt mixtures and gravel-sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	Fines behave as silt
		GC	Gravel-clay mixtures and gravel-sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	Fines behave as clay
	SAND (more than half of coarse fraction is smaller than 2.36mm)	SW	Sand and gravel-sand mixtures, little or no fines	Wide range in grain size and substantial amounts of all intermediate sizes, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	$C_u > 6$ $1 < C_c < 3$
		SP	Sand and gravel-sand mixtures, little or no fines	Predominantly one size or range of sizes with some intermediate sizes missing, not enough fines to bind coarse grains, no dry strength	≤ 5% fines	Fails to comply with above
		SM	Sand-silt mixtures	'Dirty' materials with excess of non-plastic fines, zero to medium dry strength	≥ 12% fines, fines are silty	N/A
		SC	Sand-clay mixtures	'Dirty' materials with excess of plastic fines, medium to high dry strength	≥ 12% fines, fines are clayey	

Major Divisions		Group Symbol	Typical Names	Field Classification of Silt and Clay			Laboratory Classification
				Dry Strength	Dilatancy	Toughness	% < 0.075mm
ine grained soils (more than 35% of soil excluding oversize fraction is less than 0.075mm)	SILT and CLAY (low to medium plasticity)	ML	Inorganic silt and very fine sand, rock flour, silty or clayey fine sand or silt with low plasticity	None to low	Slow to rapid	Low	Below A line
		CL, CI	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay	Medium to high	None to slow	Medium	Above A line
		OL	Organic silt	Low to medium	Slow	Low	Below A line
	SILT and CLAY (high plasticity)	MH	Inorganic silt	Low to medium	None to slow	Low to medium	Below A line
		CH	Inorganic clay of high plasticity	High to very high	None	High	Above A line
		OH	Organic clay of medium to high plasticity, organic silt	Medium to high	None to very slow	Low to medium	Below A line
	Highly organic soil	Pt	Peat, highly organic soil	–	–	–	–

Laboratory Classification Criteria

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

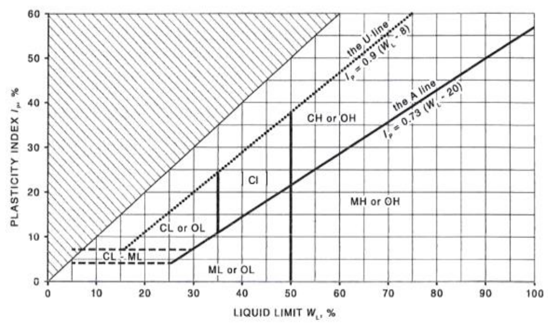
$$C_U = \frac{D_{60}}{D_{10}} \quad \text{and} \quad C_C = \frac{(D_{30})^2}{D_{10}\,D_{60}}$$

Where D_{10}, D_{30} and D_{60} are those grain sizes for which 10%, 30% and 60% of the soil grains, respectively, are smaller.

NOTES:

1. For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.
2. Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.
3. Clay soils with liquid limits > 35% and ≤ 50% may be classified as being of medium plasticity.
4. The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

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



LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record		Standing water level. Time delay following completion of drilling/excavation may be shown.
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.
		Groundwater seepage into borehole or test pit noted during drilling or excavation.
Samples	ES U50 DB DS ASB ASS SAL	Sample taken over depth indicated, for environmental analysis. Undisturbed 50mm diameter tube sample taken over depth indicated. Bulk disturbed sample taken over depth indicated. Small disturbed bag sample taken over depth indicated. Soil sample taken over depth indicated, for asbestos analysis. Soil sample taken over depth indicated, for acid sulfate soil analysis. Soil sample taken over depth indicated, for salinity analysis.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'Refusal' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60° solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25 PID = 100	Vane shear reading in kPa of undrained shear strength. Photoionisation detector reading in ppm (soil sample headspace test).
Moisture Condition (Fine Grained Soils)	w > PL $w \approx$ PL w < PL $w \approx$ LL w > LL	Moisture content estimated to be greater than plastic limit. Moisture content estimated to be approximately equal to plastic limit. Moisture content estimated to be less than plastic limit. Moisture content estimated to be near liquid limit. Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D M W	DRY – runs freely through fingers. MOIST – does not run freely but no free water visible on soil surface. WET – free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS S F St VSt Hd Fr ( )	VERY SOFT – unconfined compressive strength ≤ 25kPa. SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa. FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa. STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa. VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa. HARD – unconfined compressive strength > 400kPa. FRIABLE – strength not attainable, soil crumbles. Bracketed symbol indicates estimated consistency based on tactile examination or other assessment.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** / **SPT 'N' Value Range (Blows/300mm)**
	VL	VERY LOOSE ≤ 15 / 0 – 4
	L	LOOSE > 15 and ≤ 35 / 4 – 10
	MD	MEDIUM DENSE > 35 and ≤ 65 / 10 – 30
	D	DENSE > 65 and ≤ 85 / 30 – 50
	VD	VERY DENSE > 85 / > 50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other assessment.
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

Log Column	Symbol	Definition	
Remarks	'V' bit	Hardened steel 'V' shaped bit.	
	'TC' bit	Twin pronged tungsten carbide bit.	
	T_{60}	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.	
	Soil Origin	The geological origin of the soil can generally be described as:	
		RESIDUAL	– soil formed directly from insitu weathering of the underlying rock. No visible structure or fabric of the parent rock.
		EXTREMELY WEATHERED	– soil formed directly from insitu weathering of the underlying rock. Material is of soil strength but retains the structure and/or fabric of the parent rock.
		ALLUVIAL	– soil deposited by creeks and rivers.
		ESTUARINE	– soil deposited in coastal estuaries, including sediments caused by inflowing creeks and rivers, and tidal currents.
		MARINE	– soil deposited in a marine environment.
		AEOLIAN	– soil carried and deposited by wind.
		COLLUVIAL	– soil and rock debris transported downslope by gravity, with or without the assistance of flowing water. Colluvium is usually a thick deposit formed from a landslide. The description 'slopewash' is used for thinner surficial deposits.
		LITTORAL	– beach deposited soil.

Classification of Material Weathering

Term		Abbreviation		Definition
Residual Soil		RS		Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.
Extremely Weathered		XW		Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.
Highly Weathered	Distinctly Weathered (Note 1)	HW	DW	The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Moderately Weathered		MW		The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.
Slightly Weathered		SW		Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.
Fresh		FR		Rock shows no sign of decomposition of individual minerals or colour changes.

NOTE 1: The term 'Distinctly Weathered' is used where it is not practicable to distinguish between 'Highly Weathered' and 'Moderately Weathered' rock. 'Distinctly Weathered' is defined as follows: *'Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores'*. There is some change in rock strength.

Rock Material Strength Classification

			Guide to Strength	
Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Point Load Strength Index $I_{s(50)}$ (MPa)	Field Assessment
Very Low Strength	VL	0.6 to 2	0.03 to 0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 30mm thick can be broken by finger pressure.
Low Strength	L	2 to 6	0.1 to 0.3	Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium Strength	M	6 to 20	0.3 to 1	Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High Strength	H	20 to 60	1 to 3	A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very High Strength	VH	60 to 200	3 to 10	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
Extremely High Strength	EH	> 200	> 10	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

Abbreviations Used in Defect Description

Cored Borehole Log Column		Symbol Abbreviation	Description
Point Load Strength Index		• 0.6	Axial point load strength index test result (MPa)
		x 0.6	Diametral point load strength index test result (MPa)
Defect Details	– Type	Be	Parting – bedding or cleavage
		CS	Clay seam
		Cr	Crushed/sheared seam or zone
		J	Joint
		Jh	Healed joint
		Ji	Incipient joint
		XWS	Extremely weathered seam
	– Orientation	Degrees	Defect orientation is measured relative to normal to the core axis (ie. relative to the horizontal for a vertical borehole)
	– Shape	P	Planar
		C	Curved
		Un	Undulating
		St	Stepped
		Ir	Irregular
	– Roughness	Vr	Very rough
		R	Rough
		S	Smooth
		Po	Polished
		Sl	Slickensided
	– Infill Material	Ca	Calcite
		Cb	Carbonaceous
		Clay	Clay
		Fe	Iron
		Qz	Quartz
		Py	Pyrite
	– Coatings	Cn	Clean
		Sn	Stained – no visible coating, surface is discoloured
		Vn	Veneer – visible, too thin to measure, may be patchy
		Ct	Coating ≤ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres