REPORT TO

PYMBLE LADIES COLLEGE

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED SCHOOL BUILDING

AT

20 AVON ROAD, PYMBLE, NSW

Date: 8 February 2021

Ref: 33775SCrpt

JKGeotechnics

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For and on behalf of
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DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
33775SCrpt	Final Report	8 February 2021

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1 INTRODUCTION

This report presents the results of a geotechnical investigation for a proposed school building within Pymble Ladies College, Avon Road, Pymble, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Malcolm Boyes of Pymble Ladies College and carried out in accordance with our proposal (Ref: P53314PH) dated 8 January 2021.

We understand the development is at an early stage and the location of the proposed structures, building levels, proposed earthworks and structural loads were unavailable at the time of investigation and preparation of this report. However, based on discussions with Mr Malcolm Boyes of Pymble Ladies College, we understand that a new school building is proposed for this part of the site and is likely to comprise a five storey building potentially with a lower ground floor or basement level. Due to the sloping nature of the site we expect excavations to a maximum depth of about 3m may be required.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions as a basis for comments and recommendations on excavation, earthworks, retention and footing design.

2 INVESTIGATION PROCEDURE

The fieldwork was carried out on 14 and 15 January 2021 and comprised the drilling of four boreholes (BH201 to BH204 inclusive). Boreholes were drilled using our track mounted JK305 drilling rig to total depths ranging from 6m to 10.47m below existing ground surface levels.

- BH201 and BH202 were initially auger drilled to depths of 2.2m and 4.33m and were then continued by diamond coring techniques using an NMLC core barrel with water flush to total depths of 9.72m and 10.47m, respectively.
- BH203 and BH204 were auger drilled using a Tungsten Carbide (TC) bit to depths of 6m.

The borehole locations, as shown on the attached Figure 2, were set out by taped measurements from existing surface features. The approximate surface levels, as shown on the borehole logs, were estimated by interpolation between spot levels and contours shown on the supplied survey plan by LTS Lockley (Drawing No. 15263 00 DT, dated 8 October 2020, Issue H) The datum of the levels is Australian Height Datum (AHD).

The apparent compaction of the fill and the strength of the residual soils were assessed from Standard Penetration Test (SPT) 'N' values, augmented by hand penetrometer test results on cohesive samples recovered by the SPT split tube sampler. Within the augered portions of the boreholes, the strength of the underlying weathered bedrock was assessed from observation of the resistance to penetration of the TC bit attached to the augers, together with inspection of the recovered rock chip samples and subsequent correlations with laboratory moisture content test results. The strength of the cored siltstone and sandstone was assessed from inspection of the recovered core and subsequent laboratory Point Load Strength Index ($I_{S(50)}$) test results. The point load strength index test results are summarised on the cored borehole logs.

Groundwater observations were made during and on completion of auger drilling. Thereafter, the use of water for core drilling limited further meaningful measurements of groundwater levels. No longer term monitoring of groundwater levels was carried out.

Our Geotechnical Engineer, Mr Ben Smith, set out the borehole locations, nominated the sampling and testing locations, and prepared logs of the strata encountered. The borehole logs, including colour photographs of the recovered core, are attached to this report together with a set of explanatory notes, which describe the investigation techniques, and their limitations, and define the logging terms and symbols used.

Selected soil samples were returned to Soil Test Services Pty Ltd (STS) and Envirolab Services Pty Ltd, both NATA accredited laboratories, for testing to determine moisture contents, Atterberg limits, linear shrinkages, point load strength index values, pH, sulphate contents, chloride contents and resistivity. The results of the laboratory testing are presented in the attached STS Tables A and B and Envirolab Certificate of Analysis 259686.

3 RESULTS OF INVESTIGATION

3.1 Site Description

For the purpose of this site description, the 'site' shall be regarded as the general area where the new school building is proposed, as shown on the attached Figure 1.

The site is located within the school grounds of Pymble Ladies College, which is situated within the upper reaches of a hill within undulating topography. The site itself was positioned on a north-easterly facing hillside which sloped at about 10° to 15° and contained lawns, footpaths, marquees and demountable buildings. The demountable buildings were single storey and the marquee constructed from canvas and steel framing; both structures appeared to be in good external condition based on a cursory inspection. The footpaths were concrete surfaced and appeared to be in generally good condition, however some areas of paving showed signs of distress and cracking. The vegetation on site comprised sloping lawns, planter beds and medium to large sized trees.

To the north of the site was a two-storey brick building which had been built into the hillside. On the southern side of the building was a rendered block wall ranging in height from about 1m to 2m, which retained the hillside slope. The building and retaining wall both appeared to be in good condition based on cursory inspection. To the north-east of the site the sloping lawns continued down to the north-east.

On the eastern site boundary were a series of residential properties with swimming pools and yard areas abutting the common boundary. The setback distances and ground surface levels across this boundary could not be observed, however due to the overall direction and sloping nature of the site, the neighbouring ground levels are likely to be lower than the subject site.

To the west of the site was a two-storey brick school building with similar surface levels to the subject site. The brick building appeared to be in good external condition based on cursory inspection.

To the south of the site was the sports complex and synthetic sports fields which were approximately 2m higher than the subject site being supported by brick retaining walls to facilitate the difference in surface levels.

3.2 Subsurface Conditions

The 1:100,000 geological map of Penrith (Geological Survey of NSW, Geological Series Sheet 9030) indicates the site to be underlain by Ashfield Shale of the Wianamatta Group. Generally, the boreholes encountered fill overlying residual soil, then weathered siltstone and sandstone at depth. A summary of subsurface conditions is presented below but reference should be made to the attached borehole logs for details at each specific location.

Fill
Fill comprising silty clay of low to medium plasticity, was encountered to depths ranging from 0.2m to 0.8m. Based on SPT 'N' values the fill was assessed to be moderately compacted.

Residual Silty Clay
Residual silty clay typically of medium to high plasticity was encountered in all boreholes and was assessed to be of at least very stiff to hard strength with varying fractions of ironstone gravel inclusions.

Weathered Bedrock
Weathered siltstone bedrock (Ashfield Shale) was encountered at depths ranging from 1.2m (BH201) to 2.5m (BH202). The reduced level of the top of rock reduced towards the east of the site. On first rock contact the weathered siltstone was typically either extremely weathered and hard (soil strength) to distinctly weathered and very low strength, increasing to low to medium strength with depth.

The cored portions of BH201 and BH202 were very different, in BH201 the cored siltstone was initially assessed to be highly weathered to moderately weathered and of very low strength but improving to low to medium strength then medium to high strength below a depth of 5m. However, the cored portion of BH202 encountered extremely weathered and highly weathered siltstone of hard (soil strength) to very low strength to a depth of 10.2m, at which medium to high strength sandstone was encountered. BH203 and BH204 were similar to BH201.

Defects within the cored siltstone and sandstone comprised extremely weathered seams of generally less than 100mm, sub-horizontal bedding partings, and joints inclined at up to 90°.

Groundwater
All boreholes were 'dry' during and for a short period after completion of drilling. No longer term groundwater monitoring was carried out.

3.3 Laboratory Test Results

Based on the Atterberg limits and linear shrinkage test results, the silty clays tested are of medium plasticity and are assessed to have a moderate to high potential for shrink/swell movements with changes in moisture content.

The moisture content and point load strength index test results showed reasonably good correlation with our field assessment of rock strength. The Unconfined Compressive Strength (UCS) of the rock core, estimated from the point load strength index test results, generally ranged from 6MPa to 66MPa in BH201 however BH202 resulted in much lower values of generally between 1MPa and 6MPa, with a higher value of 84MPa for the sandstone.

The pH values on samples of the clayey fill, residual silty clay and weathered siltstone ranged from 5.2 to 7.5, indicating slightly acidic to neutral soil conditions. The sulphate contents ranged from 30mg/kg to 370mg/kg, the chloride contents ranging from <10mg/kg to 81mg/kg, and the resistivity ranged from 5,600ohm.cm to 53,000ohm.cm. Based on these results, the clayey fill, residual silty clay and weathered siltstone would classify as 'mild' exposure classification for concrete piles in accordance with Table 6.4.2(C) of AS2159-2009 'Piling – Design and Installation' and 'non-aggressive' exposure classification for steel piles in accordance with Table 6.5.2(C) of AS2159-2009.

4 COMMENTS AND RECOMMENDATIONS

4.1 Excavation and Groundwater

Due to the sloping nature of the site we envisage excavations of up to about 3m depth will be required to form the building platform. Excavation to such depths will encounter clayey fill, residual soils and weathered siltstone.

Excavation of the soils and upper rock of up to very low strength should be achievable using conventional excavation equipment, such as the buckets of hydraulic excavators. Some ripping of higher strength bands may be necessary if they are encountered within the weaker rock.

Excavation of bedrock of low strength or higher strength will require assistance with rock excavation equipment. Such equipment may comprise hydraulic rock hammers, ripping hooks, rotary grinders or rock saws. Hydraulic rock hammers must be used with care due to the risk of damage to the neighbouring buildings. If hydraulic rock hammers are to be used the vibrations transmitted to the buildings should be quantitatively monitored at least at the start of rock hammer operation to confirm that the transmitted vibrations are within acceptable limits. If during the initial monitoring the transmitted vibrations are close to acceptable limits full time monitoring may then be warranted. Reference should be made to the attached Vibration Emission Design Goals sheet for acceptable limits of transmitted vibrations. Where the transmitted vibrations are excessive it would be necessary to change to alternative excavation equipment, such as a smaller rock hammer, ripping hooks, rotary grinders or rock saws.

No groundwater seepage was encountered during auger drilling of the boreholes. As such we do not consider that groundwater will be a significant issue for the proposed development. Nevertheless, some seepage may occur into the excavation and this would likely tend to occur along the soil/rock interface and through joints and bedding partings within the rock, particularly during and following rainfall. Any such seepage that does occur should be able to be controlled during construction using gravity drainage and conventional sump and pump techniques. In the long term, drainage should be provided behind all retaining walls and possibly below the lowest floor slab. The completed excavation should be inspected by the hydraulic consultant to confirm that the designed drainage system is adequate for the actual seepage flows.

4.2 Subgrade Preparation and Filling

The boreholes encountered limited fill across the site and we expect that the fill will be excavated and removed as part of the proposed bulk excavation. However, where floor slabs are proposed we recommend that where the fill is not excavated as part of the bulk excavations that it be removed and replaced with controlled, engineered fill. Alternatively, if the fill is left in place the ground floor slab should be designed as a fully suspended slab supported on the piled footing system. For the proposed pavements the fill may be left in place provided it is treated as required following proof rolling.

Within areas where floor slabs are proposed all existing fill should be fully stripped to expose the residual silty clay or weathered siltstone. Within pavement areas the vegetation and root affected soils should be stripped, but the fill below may be left in place. This root affected fill is not suitable to reuse as engineered fill, but may be reused within landscaped areas.

Following stripping, the exposed subgrade should be proof rolled with at least 7 passes of a minimum 8 tonne dead weight, smooth drum, vibratory roller. The final pass of the proof rolling should be carried out without vibration and in the presence of a geotechnical engineer to detect any weak subgrades areas. Care must be taken during rolling due to the risk of damage to adjoining structures from the vibrations generated by the roller. If vibrations are of concern the rolling may need to be carried out with a static roller only.

Any weak or unstable areas detected during proof rolling should be locally excavated to a sound base and the excavated material replaced with controlled, engineered fill, or as directed by the geotechnical engineer during proof rolling. Some weak subgrade areas may be experienced where the existing fill is left in place or where the clays are allowed to soften due to water ponding. Following treatment of weak areas, engineered fill should be placed in thin layers as recommended in Section 4.2.1 below.

In view of the high reactivity potential of some of the residual clays, particular attention should be given to providing adequate drainage both during construction and for long term site maintenance. The principal aim of the drainage should be to promote run-off and reduce ponding. Placement of a blinding layer of durable granular fill or subbase material to provide a trafficable surface during construction may be necessary or desirable. The earthworks should be carefully planned and scheduled to maintain cross-falls during construction. If the clay is exposed to prolonged periods of rainfall, softening will result and site trafficability will be poor. If soil softening occurs, the subgrade should be over-excavated to below the depth of moisture softening and the excavated material replaced with engineered fill.

4.2.1 Engineered Fill and Compaction Control

Engineered fill should preferably comprise well graded granular materials, such as ripped rock or crushed sandstone, free of deleterious substances and having a maximum particle size not exceeding 75mm. Such fill should be compacted in horizontal layers of not greater than 200mm loose thickness, to a density of at least 98% of Standard Maximum Dry Density (SMDD). For backfilling confined excavations such as service trenches, a similar compaction to engineered fill should be adhered to, but if light compaction equipment is used then the layer thickness should be limited to 100mm loose thickness.

The excavated material may be reused as engineered fill, provided it is free of deleterious materials and particles greater than 75mm in size. All excavated material should be inspected and approved by a geotechnical engineer prior to reuse. Any clay fill should be compacted in maximum 200mm loose thickness layers to a density strictly between 98% and 102% of SMDD and at moisture contents within 2% of Standard Optimum Moisture Content (SOMC).

Density tests should be regularly carried out on the fill to confirm the above specifications are achieved. The frequency of density testing should be at least one test per layer per 500m^2 or three tests per visit, whichever requires the most tests. Where fill is to support footing loads it should be placed under Level 1 control as defined by AS3798-2007. Preferably the geotechnical testing authority should be engaged directly on behalf of the client and not by the earthworks subcontractor.

4.3 Batters and Retaining Wall

Suitable retention systems will depend on the proposed basement depth and set-back distances from adjoining structures and properties. For basements which extend up to or close to the site boundaries, full depth retention systems will need to be installed prior to the start of excavation.

Where space permits, temporary batters through the clayey soils and poor-quality siltstone bedrock may be formed at no steeper than 1 Vertical (V): 1 Horizontal (H). Where adopted all surcharge loads such as stockpiles, traffic loads etc must be kept well clear of the crest of the batters. Where permanent batters are adopted, they should be formed at no steeper than 1 Vertical (V): 2 Horizontal (H) and should be protected from erosion by vegetation, shotcrete and mesh or similar. For maintenance purposes it may be more practical to from permanent batters at no stepper than 1V:3H or 4H.

Where space does not allow for the formation of batters and excavation will extend below adjoining buildings a retention system will need to be installed prior to the commencement of excavation. Such a retention system may comprise soldier pile walls with shotcrete infill panels. From experience the construction of such shoring systems has become very cost effective and we do not expect that creation of temporary batters, stockpiling of materials for use as back fill, export of surplus materials to tip, import of expensive drainage gravel and construction of “conventional” retaining walls will necessarily be the most economical option.

Bored piers would be appropriate for the piled walls, but some groundwater seepage may be encountered requiring the use of pumps and tremie concreting techniques. The piers should be founded at least 1m below the base of the excavation, including excavations for footings and services, but more as required for stability design.

Piles supporting cuts up to 3m can probably be designed as cantilevers unless the surcharge loads of adjacent footings are high.

If required, temporary lateral restraint of the retention system could be provided by external anchors or internal props, with each restraining point progressively installed as it is exposed during excavation. Long term lateral support would be provided by the floor slabs within the excavation and the toe sockets of the piles. If anchors are to locally extend below neighbouring properties, permission would need to be obtained from the owners of the adjoining properties before the installation of the anchors below their properties. Such permission can take some time to obtain, which should be allowed for within the project program. The use of anchors will need to take into account the neighbouring site levels and location of any basements and services within the adjoining buildings so that these can be avoided. However, this will be subject to the final building layout plan and proposed ground floor/basement levels.

Cantilever walls can be designed using an active earth pressure coefficient , K_a of 0.35 where there are no structures or services adjacent, but increase to 0.6, where movements are to be kept low.

Propped or anchored retaining walls may be designed based on a trapezoidal earth pressure distribution of magnitude 6H kPa, where H is the retained height in metres, where structures or movement sensitive services are located beyond a horizontal distance of 2H from the wall. Where structures or movement sensitive services are located within 2H of the wall, a trapezoidal earth pressure distribution of 8H kPa should be used. These pressures should be constant over the central 50% of the trapezoidal pressure distribution. In addition to these pressures, the retention wall design should be checked and designed to accommodate a wedge formed by a joint inclined at 45° intersecting the excavation face at the base of the cut.

The above pressures assume horizontal backfill behind the walls and any inclined backfill should be taken as a surcharge load. All surcharge loads should be allowed for in the design, plus full hydrostatic pressures, unless measures are undertaken to provide complete and permanent drainage behind the wall.

Anchors should have their bond formed within rock outside a line drawn up at 45° from the base of the excavation, with a minimum bond length of 3m and a minimum free length of 3m. Provisional design of the anchors may be based on a bond stress of 100kPa for rock of at least very low to low strength and 200kPa for low or higher strength rock. All anchors should be proof loaded to at least 1.3 times their design working load before locking off at about 80% of their working load. Lift-off tests should be carried out on at least 10% of the anchors 24 to 48 hours following locking off to confirm that the anchors are holding their load. Generally, anchors are installed on a design and construct basis so that optimisation of the bond stresses does not become a contractual issue in the event of anchors failing to hold their test loads.

Passive toe resistance of the retention system below the base of the bulk excavation may be estimated based on an allowable lateral resistance of 200kPa for rock of at least very low to low strength. The passive resistance should be ignored for at least 0.5m below the base of the excavation, including footing and service excavations.

4.4 Footings

Weathered siltstone was encountered at levels ranging from about RL112m to RL119m, therefore the bulk excavation depth is greater than about 3m, weathered siltstone is likely to be exposed at bulk excavation level, however this will depend on the final building layout and floor levels. Notwithstanding, we recommend that the building is supported on the underlying siltstone or sandstone bedrock to provide uniform support and reduce the risk of differential movements.

We expect that pad/strip footings founded within the siltstone and sandstone would be appropriate. Where above ground portions of the buildings extend outside the ground floor/basement excavation the use of piles may be required so that the footings are founded within bedrock below the zone of influence of the basement excavation.

Due to the variable bedrock quality encountered within the boreholes, an appropriate allowable bearing pressure for footings founded within the weathered siltstone would commence at 700kPa for siltstone of at least very low strength, but higher bearing pressures are expected to be possible if medium or high strength siltstone and sandstone is encountered, which will depend partly on the depth of excavation. Bored piles bearing on low strength siltstone can be designed for an allowable bearing pressure of 1000kPa. Subject to additional cored boreholes confirming the quality of the medium strength siltstone and clearing up the uncertainty around BH202, allowable bearing pressure of 3500kPa should be achievable at depths of about 5m.

Where piles are used, allowable shaft adhesions equivalent to 10% of the allowable end bearing pressure may be used for the design of piles in compression, below a nominal 0.3m socket and provided socket roughness and cleanliness is maintained.

The footing excavations should be inspected by a geotechnical engineer to confirm that the appropriate foundation material has been encountered.

4.5 Floor Slabs

The subgrade at bulk excavation level will likely comprise weathered siltstone. As recommended above, drainage will need to be provided below the basement slab either as a closely spaced grid of subsoil drains or a gravel blanket. The drainage will need to be connected to a permanent fail-safe pump out system, which is fitted with automatic level controls to avoid flooding unless gravity drainage can be provided.

The basement slab should be designed with a subbase layer of at least 100mm thickness of crushed rock to RMS QA specification 3051 (2013) unbound base material (or other approved good quality and durable fine crushed rock), which is compacted to at least 100% of Standard Maximum Dry Density (SMDD) if a continuous drainage blanket is not adopted. This subbase layer will provide a separation between the siltstone subgrade and the slab and provide a uniform base for the slab.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. 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.

The long term successful performance of floor slabs and pavements is dependent on the satisfactory completion of the earthworks. In order to achieve this, the quality assurance program should not be limited to routine compaction density testing only. Other critical factors associated with the earthworks may include subgrade preparation, selection of fill materials, control of moisture content and drainage, etc. The satisfactory control and assessment of these items may require judgment from an experienced engineer. Such judgment often cannot be made by a technician who may not have formal engineering qualifications and experience. In order to identify potential problems, we recommend that a pre-construction meeting be held so that all parties involved understand the earthworks requirements and potential difficulties. This meeting should clearly define the lines of communication and responsibility.

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 will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. Analysis takes seven to 10 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) should be expected. We strongly recommend that this issue 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

ABN 43 002 145 173

TABLE A
MOISTURE CONTENT, ATTERBERG LIMIT AND LINEAR SHRINKAGE TEST REPORT

Client: JK Geotechnics
Project: Proposed School Building
Location: 20 Avon Road, Pymble, NSW

Ref No: 33775BC
Report: A
Report Date: 22/01/2021
Page 1 of 1

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	DEPTH m	MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTICITY INDEX %	LINEAR SHRINKAGE %
201	0.50 - 0.95	21.3	48	22	26	11.5
203	2.00 - 3.00	7.5	-	-	-	-
203	3.80 - 4.20	6.6	-	-	-	-
203	5.00 - 6.00	7.4	-	-	-	-
204	2.70 - 3.00	5.7	-	-	-	-
204	3.50 - 4.50	6.4	-	-	-	-
204	5.20 - 6.00	5.9	-	-	-	-

Notes:
- The test sample for liquid and plastic limit was air-dried & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions
- Date of receipt of sample: 15/01/2021.
- Sampled and supplied by client. Samples tested as received.

NATA

NATA Accredited Laboratory Number:1327

Accredited for compliance with ISO/IEC 17025 - Testing.
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In full without approval of the laboratory. Results relate only to the items tested or sampled.

22/01/2021

Authorised Signature / Date
(D. Treweek)

TABLE B
POINT LOAD STRENGTH INDEX TEST REPORT

Client:	Pymble Ladies College	**Ref No:**	33775BC
Project:	Proposed School Building	**Report:**	B
Location:	20 Avon Road, PYMBLE, NSW	**Report Date:**	18/01/21

BOREHOLE NUMBER	DEPTH (m)	IS (50) (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
201	2.60 - 2.62	0.3	6	A
	3.18 - 3.21	0.3	6	A
	3.78 - 3.81	0.7	14	A
	4.09 - 4.12	0.6	12	A
	4.49 - 4.53	0.6	12	A
	5.21 - 5.24	1.3	26	A
	5.90 - 5.93	2	40	A
	6.32 - 6.35	2.4	48	A
	6.86 - 6.89	2.2	44	A
	7.06 - 7.08	3.3	66	A
	7.62 - 7.64	2.3	46	A
	8.27 - 8.30	1.8	36	A
	8.86 - 8.89	2.6	52	A
	9.07 - 9.09	1.5	30	A
	9.53 - 9.55	2.1	42	A
202	7.56 - 7.59	0.04	1	A
	7.76 - 7.78	0.3	6	A
	8.91 - 8.93	0.02	<1	A
	9.05 - 9.08	0.04	1	A
	9.31 - 9.33	0.6	12	A
	9.61 - 9.63	0.09	2	A
	10.00 - 10.02	0.2	4	A
	10.31 - 10.34	4.2	84	A

NOTES

1. In the above table, testing was completed in test direction A for the axial direction, D for the diametral direction, B for the block test and L for the lump test.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RMS T223.
4. For reporting purposes, the IS(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 based on the correlation provided in AS1726:2017 'Geotechnical Site Investigations' and rounded off to the nearest whole number: U.C.S. = 20 IS(50).

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CERTIFICATE OF ANALYSIS 259686

Client Details	
Client	JK Geotechnics
Attention	Ben Sheppard
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details	
Your Reference	**33775BC, Pymble**
Number of Samples	3 Soil
Date samples received	18/01/2021
Date completed instructions received	18/01/2021

Analysis Details

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

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

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

Report Details	
Date results requested by	25/01/2021
Date of Issue	22/01/2021

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Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil				
Our Reference		259686-1	259686-2	259686-3
Your Reference	UNITS	BH202	BH203	BH204
Depth		0.2-0.3	1.2-1.3	0.75-0.95
Date Sampled		14/01/2021	15/01/2021	15/01/2021
Type of sample		Soil	Soil	Soil
Date prepared	-	20/01/2021	20/01/2021	20/01/2021
Date analysed	-	20/01/2021	20/01/2021	20/01/2021
pH 1:5 soil:water	pH Units	7.5	5.9	5.2
Chloride, Cl 1:5 soil:water	mg/kg	10	<10	81
Sulphate, SO4 1:5 soil:water	mg/kg	33	30	370
Resistivity in soil*	ohm m	280	530	56

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity (non NATA). Resistivity (calculated) may not correlate with results otherwise obtained using Resistivity-Current method, depending on the nature of the soil being analysed.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			20/01/2021	[NT]	[NT]	[NT]	[NT]	20/01/2021	[NT]
Date analysed	-			20/01/2021	[NT]	[NT]	[NT]	[NT]	20/01/2021	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	102	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	118	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	109	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]

Result Definitions

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

Quality Control Definitions

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

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

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

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

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

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

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

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

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

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

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BOREHOLE LOG

Borehole No.
201

Client: PYMBLE LADIES COLLEGE
Project: PROPOSED SCHOOL BUILDING
Location: 20 AVON ROAD, PYMBLE, NSW

Job No.: 33775BC **Method:** SPIRAL AUGER **R.L. Surface:** ~121 m
Date: 14/1/21 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** B.S./T.C.

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							FILL: Silty clay, medium plasticity, dark brown, with root fibres, trace of fine grained sand, fine to medium grained ironstone gravel.	w~PL			GRASS COVER
		N = 13 5,6,7	120	1		CI-CH	Silty CLAY: medium to high plasticity, orange brown, trace of fine to medium grained ironstone gravel, ash and root fibres.	w<PL	Hd	510 >600 590	RESIDUAL
		N=SPT 11/ 50mm REFUSAL				-	Extremely Weathered siltstone: silty CLAY, high plasticity, light grey and grey, with iron indurated and very low strength bands.	XW	Hd	>600 >600 >600	ASHFIELD SHALE VERY LOW 'TC' BIT RESISTANCE
			119	2			SILTSTONE: grey and brown, with iron indurated bands.	DW	VL - L		LOW RESISTANCE
			118	3			REFER TO CORED BOREHOLE LOG				
			117	4							
			116	5							
			115	6							

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CORED BOREHOLE LOG

Borehole No.
201

Client: PYMBLE LADIES COLLEGE
Project: PROPOSED SCHOOL BUILDING
Location: 20 AVON ROAD, PYMBLE, NSW

Job No.: 33775BC	**Core Size:** NMLC	**R.L. Surface:** ~121 m
Date: 14/1/21	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK305	**Bearing:** N/A	**Logged/Checked By:** B.S./T.C.

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); DESCRIPTION: Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness (Specific, General) | Formation

Water Loss\Level: 50% RETURN

RL (m AHD) / Depth (m): 118 / 3; 117 / 4; 116 / 5; 115 / 6; 114 / 7; 113 / 8

START CORING AT 2.20m

Core Description	Weathering	Strength
NO CORE 0.09m		
SILTSTONE: dark grey and brown, with iron indurated bands and laminae up to 5°.	HW	VL - L
	MW	L - M
Extremely Weathered siltstone: silty CLAY, medium plasticity, with light grey laminae up to 10°.	XW	Hd
SILTSTONE: dark grey and brown, with iron indurated bands and laminae up to 5°.	MW	L - M
SILTSTONE: dark grey, with light grey laminae up to 10°.	SW	M - H
	FR	

Point Load Strength Index $I_s(50)$: 0.30, 0.30, 0.70, 0.60, 0.60, 1.3, 2.0, 2.4, 2.2, 3.3, 2.3, 1.8, 2.6

Defect Details – Description (Specific):

- (2.29-2.40m) Highly Fractured, Numerious Be, P, S, Cn
- (2.42m) XWS, 0°, 4 mm.t
- (2.45m) XWS, 0°, 3 mm.t
- (2.49m) XWS, 0°, 4 mm.t
- (2.62m) XWS, 0°, 4 mm.t
- (2.69m) J, 90°, Ir, R, Fe Sn, and Be, 0°, P R, Fe, St
- (2.71m) Be, 0°, P, R, Fe Sn
- (2.72m) XWS, 0°, 20 mm.t
- (2.78m) XWS, 0°, 7 mm.t
- (2.82m) XWS, 0°, 10 mm.t
- (2.85m) J, 70 - 90°, Ir, R, Clay Ct
- (2.95m) J x 2, 70 - 80°, P, S, XWS FILLED, and XWS, 0°, 110mm.t
- (3.14m) J, 80 - 90°, St, R, Fe Sn
- (3.30m) XWS, 0°, 5 mm.t
- (3.38m) XWS, 0°, 20 mm.t
- (3.41m) Be, 0°, P, R, Cn
- (3.58m) J, 90°, P, R, Fe Sn
- (3.60m) J, 0°, P, R, Fe Sn
- (3.66m) Be, 0°, P, R, Fe Sn, and J, 90°, P R, Cn
- (3.74m) XWS, 0°, 4 mm.t
- (3.88m) XWS, 0°, 12 mm.t
- (3.92m) Be, 5°, Un, R, Fe Sn
- (3.96m) CS, 0°, 6 mm.t
- (4.13m) Be, 5°, Un, R, Fe Sn
- (4.25m) J, 60°, P, R, XWS FILLED, 15 mm.t
- (4.70m) J, 90°, Un, R, XW Fines, 15 mm.t
- (4.84m) XWS, 0°, 35 mm.t
- (5.08m) Be, 0°, P, R, Cn
- (5.35m) J x 3, 70 - 90°, C, Cn
- (5.54m) J, 80°, P, R, Fe Sn
- (5.93m) Be, 0°, P, R, Cn
- (6.17m) J, 70°, Un, R, Fe Sn
- (6.56m) J, 70°, Un, R, Cn
- (7.11m) J, 15°, Un, R, Cn
- (7.21m) Be, 0°, Un, R, Fe Sn
- (7.27m) J, 60°, Un, R, Fe Sn

Formation: Ashfield Shale

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

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CORED BOREHOLE LOG

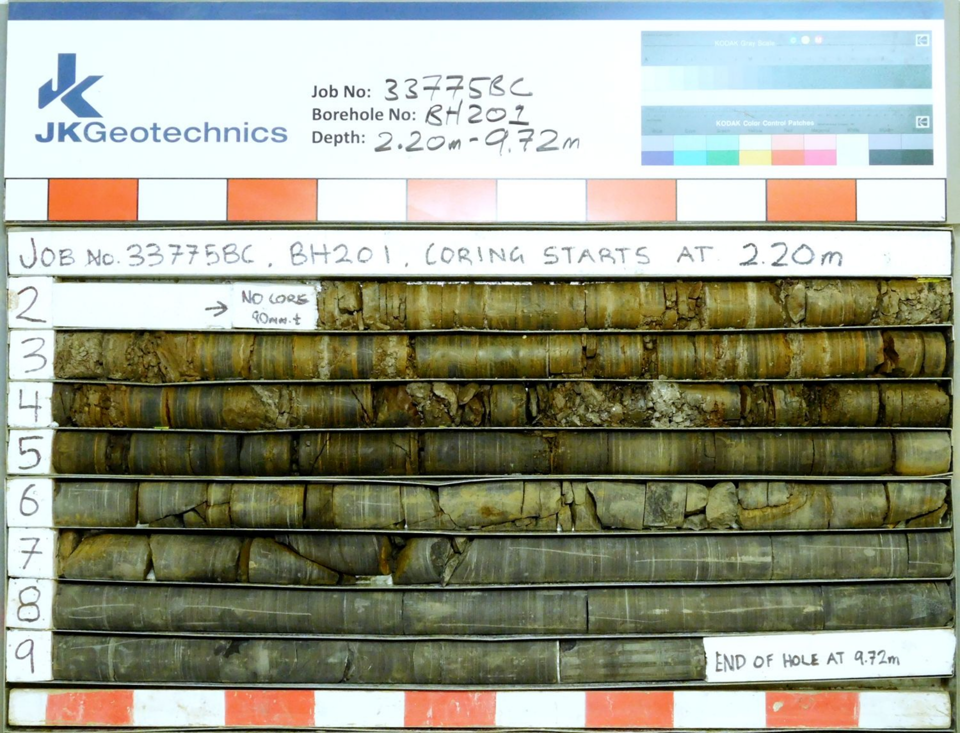
Borehole No.
201

Client: PYMBLE LADIES COLLEGE
Project: PROPOSED SCHOOL BUILDING
Location: 20 AVON ROAD, PYMBLE, NSW

Job No.: 33775BC **Core Size:** NMLC **R.L. Surface:** ~121 m
Date: 14/1/21 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK305 **Bearing:** N/A **Logged/Checked By:** B.S./T.C.

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 SPACING (mm) (600, 200, 60, 20)	DEFECT DESCRIPTION Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness — Specific / General	Formation
50% RETURN					SILTSTONE: dark grey, with light grey laminae up to 10°. *(continued)*	FR	M	•1.5 •2.1			Ashfield Shale
					END OF BOREHOLE AT 9.72 m						
		111	10								
		110	11								
		109	12								
		108	13								
		107	14								
		106	15								

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JKGeotechnics
Job No: 33775BC
Borehole No: BH201
Depth: 2.20m - 9.72m
KODAK Gray Scale
KODAK Color Control Patches
JOB No. 33775BC, BH201, CORING STARTS AT 2.20m
2
NO CORE
3
4
5
6
7
8
9
END OF HOLE AT 9.72m

JKGeotechnics

BOREHOLE LOG

Borehole No.
202

Client: PYMBLE LADIES COLLEGE
Project: PROPOSED SCHOOL BUILDING
Location: 20 AVON ROAD, PYMBLE, NSW

Job No.: 33775BC **Method:** SPIRAL AUGER **R.L. Surface:** ~114.5 m
Date: 14/1/21 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** B.S./T.C.

Groundwater Record	Samples (ES, U50, DB, DS)	Field Tests	RL (m AHD)	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION OF AUGERING						FILL: silty clay, low plasticity, dark brown, trace of fine grained sand, fine grained ironstone gravel, concrete fragments and root fibres.	w>PL			GRASS COVER APPEARS MODERATELY COMPACTED
		N = 9 5,4,5	114			as above, but without concrete fragments.				
				1	CI	Silty CLAY: medium plasticity, orange brown and brown, trace of fine grained ironstone gravel and root fibres.	w>PL	VSt	290 300 240	RESIDUAL
		N = 16 6,6,10	113	2	CI-CH	Silty CLAY: medium to high plasticity, orange brown, red brown and grey, with fine grained ironstone gravel, trace of root fibres.	w~PL	Hd	>600 >600 >600	
			112, 111	3	-	Extremely Weathered siltstone: silty CLAY, high plasticity, grey brown, with iron indurated bands and very low strength brands.	XW	(Hd)		ASHFIELD SHALE VERY LOW 'TC' BIT RESISTANCE
				4		SILTSTONE: grey and brown, with extremely weathered bands and iron indurated bands.	DW	VL		VERY LOW TO LOW BANDED RESISTANCE
			110, 109, 108	5, 6		REFER TO CORED BOREHOLE LOG				

JKGeotechnics

CORED BOREHOLE LOG

Borehole No. 202

Client: PYMBLE LADIES COLLEGE
Project: PROPOSED SCHOOL BUILDING
Location: 20 AVON ROAD, PYMBLE, NSW

Job No.: 33775BC **Core Size:** NMLC **R.L. Surface:** ~114.5 m
Date: 14/1/21 **Inclination:** VERTICAL **Datum:** AHD
Plant Type: JK305 **Bearing:** N/A **Logged/Checked By:** B.S./T.C.

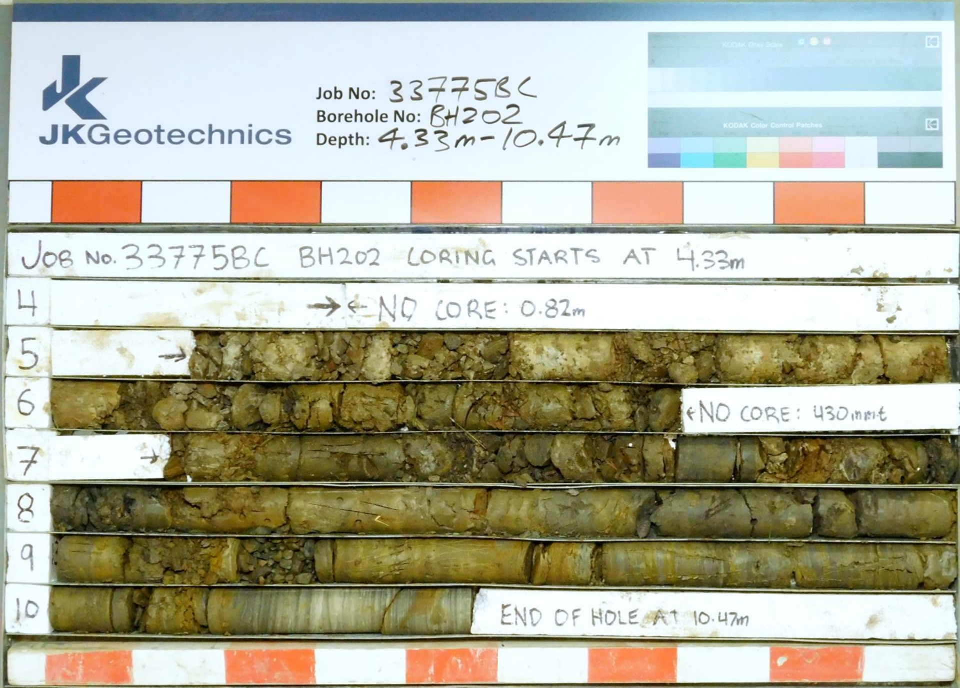
Water Loss\Level: 50% RETURN

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)	Formation
START CORING AT 4.33m					
NO CORE 0.82m					
Extremely Weathered siltstone: silty CLAY, high plasticity, grey, with high strength bands and iron indurated bands.	XW	Hd		(5.80m) HP: >600 kPa (6.08m) HP: >600 kPa	Ashfield Shale
NO CORE 0.43m					
SILTSTONE: grey and brown.	HW	VL	0.040 0.30	(7.14m) XWS, 0°, 45 mm.t (7.40m) XWS, 0°, 30 mm.t (7.43-7.62m) Numerous, Be, 0°, P, R, Fe, and J, 20-80°, P, R, Fe, St (7.78m) Be, 5°, P, R, Fe Sn	Ashfield Shale
Extremely Weathered siltstone: silty CLAY, high plasticity, with iron indurated bands.	XW	Hd		(8.24m) HP: >600 kPa (8.40m) HP: >600 kPa	Ashfield Shale
SILTSTONE: dark grey.	HW	VL	0.020 0.040	(9.10m) XWS, 0°, 100 mm.t (9.20m) J, 90°, P, R, Fe Sn	Ashfield Shale
Interbedded SILTSTONE: grey and brown and SANDSTONE: fine to medium grained, grey, with iron indurated bands.	MW	VL - L	0.60 0.090 0.20	(9.55m) XWS, 0°, 50 mm.t (10.10m) Be, 0°, Un, R, Fe Sn, and XWS, 0°, 75mm.t	Ashfield Shale
SANDSTONE: fine grained, grey, with siltstone bands.	SW	M - H	4.2	(10.37m) J, 30°, Un, R, Fe Sn (10.42m) Ji, 30°	Ashfield Shale
END OF BOREHOLE AT 10.47 m					

Depth (m): 5, 6, 7, 8, 9, 10
RL (m AHD): 110, 109, 108, 107, 106, 105, 104

Point load scale: VL -0.1, L -0.3, M -1, H -3, VH -10, EH
Spacing (mm): 600, 200, 60, 20

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JKGeotechnics
Job No: 33775BC
Borehole No: BH202
Depth: 4.33m-10.47m
KODAK Gray Scale
KODAK Color Control Patches
JOB No. 33775BC BH202 CORING STARTS AT 4.33m
4
→ ← NO CORE: 0.82m
5
6
← NO CORE: 430mm±
7
8
9
10
END OF HOLE AT 10.47m

JKGeotechnics

BOREHOLE LOG

Borehole No.
203
1 / 1

Client: PYMBLE LADIES COLLEGE
Project: PROPOSED SCHOOL BUILDING
Location: 20 AVON ROAD, PYMBLE, NSW

Job No.: 33775BC **Method:** SPIRAL AUGER **R.L. Surface:** ~117 m
Date: 15/1/21 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** B.S./T.C.

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							FILL: Silty clay, dark brown, trace of fine to medium grained sand, fine to medium grained ironstone gravel, concrete fragments and root fibres.	w~PL			GRASS COVER
		N = 14 6,6,8	116	1		CI-CH	Silty CLAY: medium to high plasticity, orange brown, trace of fine to coarse grained ironstone gravel, ash and root fibres.	w~PL	Hd	>600 >600 >600	RESIDUAL
			115	2		-	SILTSTONE: dark grey, with extremely weathered bands and iron indurated bands.	DW	L		ASHFIELD SHALE
									L		VERY LOW TO LOW 'TC' BIT RESISTANCE LOW RESISTANCE WITH MODERATE BANDS
			114	3			SILTSTONE: dark grey and grey, with extremely weathered seams.		M		MODERATE RESISTANCE
			113	4							
			112	5			as above, but iron indurated bands.		M - H		MODERATE RESISTANCE WITH HIGH BANDS
			111	6			END OF BOREHOLE AT 6.00 m				

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 33775BC PYMBLE.GPJ <<DrawingFile>> 08/02/2021 10:05 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

JKGeotechnics

BOREHOLE LOG

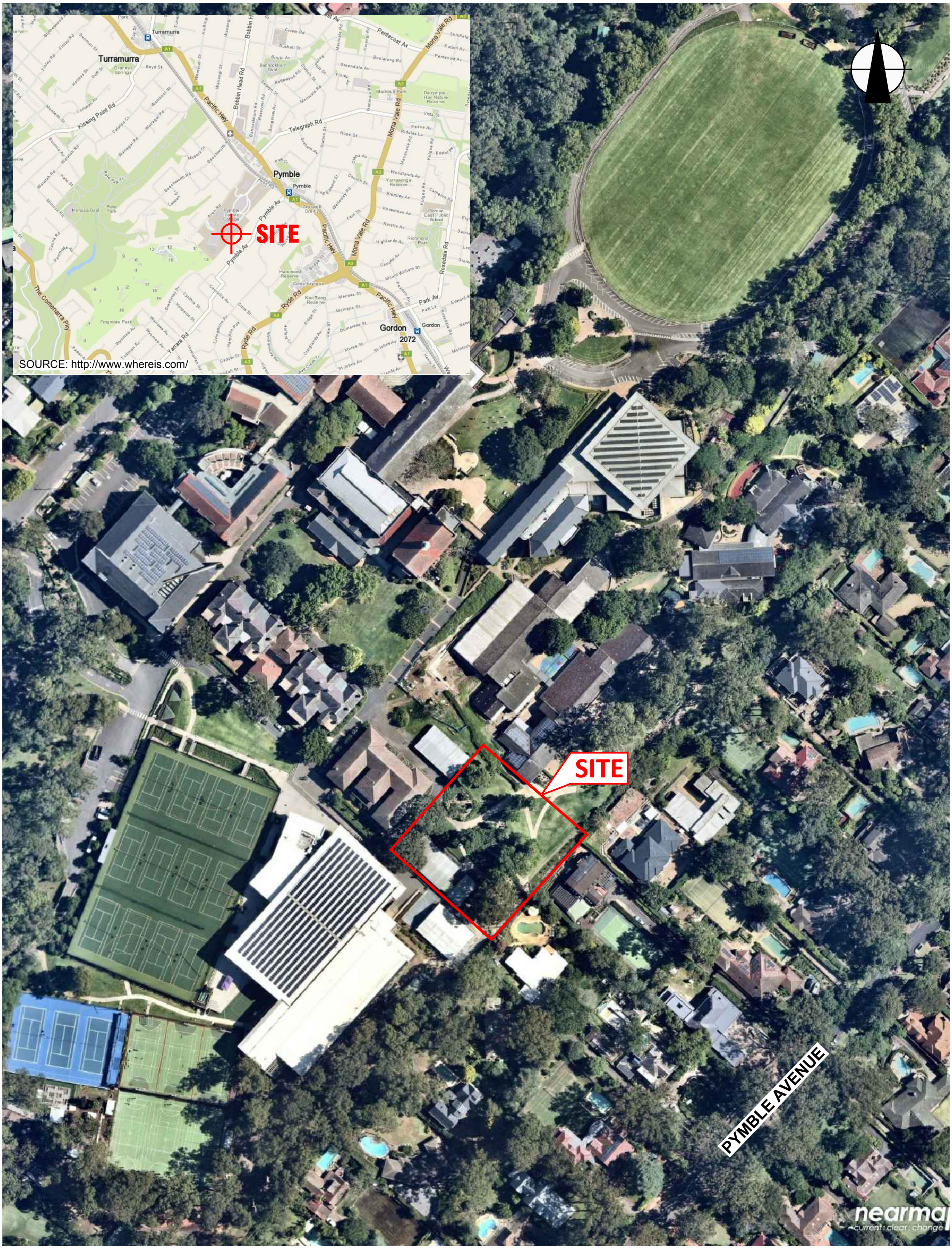
Borehole No.
204

Client: PYMBLE LADIES COLLEGE
Project: PROPOSED SCHOOL BUILDING
Location: 20 AVON ROAD, PYMBLE, NSW

Job No.: 33775BC **Method:** SPIRAL AUGER **R.L. Surface:** ~119.9 m
Date: 15/1/21 **Datum:** AHD
Plant Type: JK305 **Logged/Checked By:** B.S./T.C.

Groundwater Record	SAMPLES (ES / U50 / DB / DS)	Field Tests	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
DRY ON COMPLETION			-	CONCRETE:20mm.t	M			APPEARS MODERATELY COMPACTED
				FILL: Silty sand, fine to medium grained, dark brown, trace of concrete fragments and clay nodules.	w<PL			
		N = 11 4,6,5		FILL: Silty clay, low plasticity, brown, trace of fine to coarse grained igneous and ironstone gravel, ash and slag fragments.				
			CI	Silty CLAY: medium plasticity, orange brown, trace of fine to medium grained ironstone gavel, ash and root fibres.	w<PL	Hd	>600 >600 >600	RESIDUAL
		N = 16 7,7,9	CI-CH	Silty CLAY: medium to high plasticity, light grey and red brown, trace of fine to coarse grained ironstone gravel.	w~PL		>600 >600 >600	
			-	Extremely Weathered siltstone: silty CLAY, medium to high plasticity, grey, with iron indurated bands.	XW	Hd		ASHFIELD SHALE VERY LOW 'TC' BIT RESISTANCE
				SILTSTONE: dark grey and brown, with iron indurated bands and extremely weathered bands.	DW	L		LOW RESISTANCE WITH MODERATE BANDS
						M		MODERATE RESISTANCE WITH LOW BANDS
				as above, but without extremely weathered bands.		M - H		MODERATE RESISTANCE WITH HIGH BANDS
				END OF BOREHOLE AT 6.00 m				

RL (m AHD): 119, 118, 117, 116, 115, 114, 113
Depth (m): 1, 2, 3, 4, 5, 6



AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:

SITE LOCATION PLAN

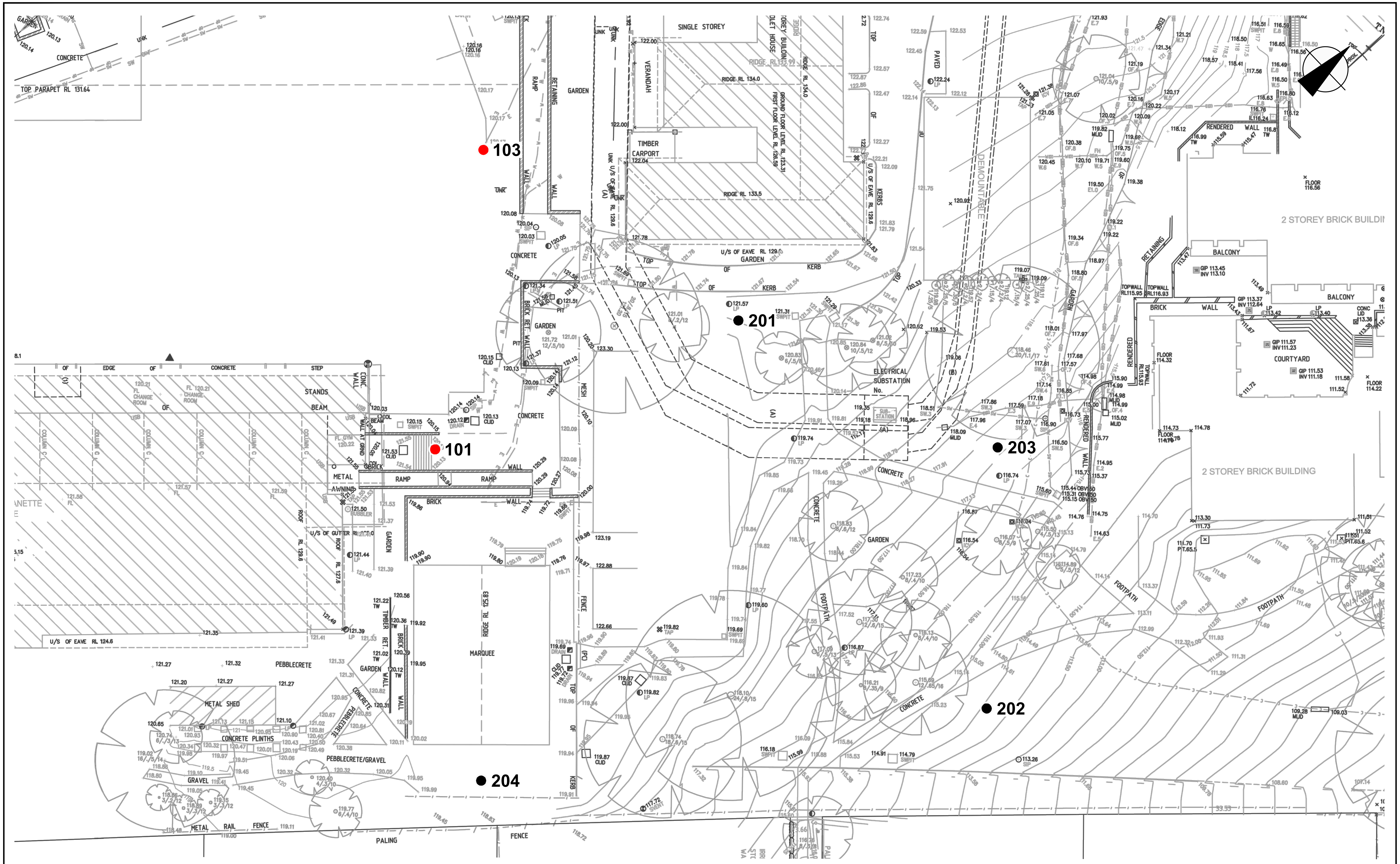
Location: 20 AVON ROAD, PYMBLE, NSW

Report No: 33775BC

Figure No: 1

JKGeotechnics

PLOT DATE: 25/01/2021 9:42:37 AM DWG FILE: Z:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\33000'S\33775BC PYMBLE\CAD\33775BC.DWG



NOTE:
BOREHOLES 101 & 103 ARE FROM OUR PREVIOUS GEOTECHNICAL INVESTIGATION.

0 4 8 12 16 20

SCALE 1:400 @A3 METRES

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

Title:	**BOREHOLE LOCATION PLAN**
Location:	20 AVON ROAD, PYMBLE, NSW
Report No: 33775BC	Figure No: 2

JKGeotechnics

PLOT DATE: 29/01/2021 4:10:32 PM DWG FILE: Z:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\33000's\33775BC PYMBLE\CAD\33775BC.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 <u>or</u> where only a limited investigation has been completed <u>or</u> 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:

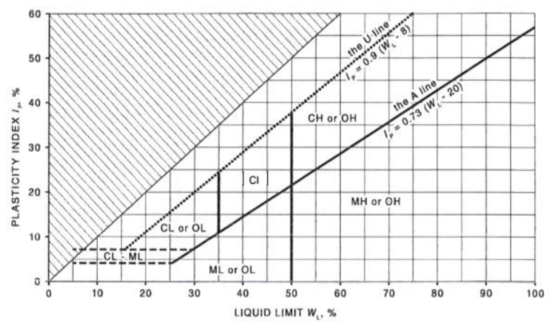
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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



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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 ≈ PL w < PL w ≈ 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

<table>
<tr><th colspan="2">Term</th><th colspan="2">Abbreviation</th><th>Definition</th></tr>
<tr><td colspan="2">Residual Soil</td><td colspan="2">RS</td><td>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.</td></tr>
<tr><td colspan="2">Extremely Weathered</td><td colspan="2">XW</td><td>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.</td></tr>
<tr><td>Highly Weathered</td><td rowspan="2">Distinctly Weathered (Note 1)</td><td>HW</td><td rowspan="2">DW</td><td>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.</td></tr>
<tr><td>Moderately Weathered</td><td>MW</td><td>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.</td></tr>
<tr><td colspan="2">Slightly Weathered</td><td colspan="2">SW</td><td>Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Fresh</td><td colspan="2">FR</td><td>Rock shows no sign of decomposition of individual minerals or colour changes.</td></tr>
</table>

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

Term	Abbreviation	Uniaxial Compressive Strength (MPa)	Guide to Strength	
			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 $\leq$ 1mm thick
		Filled	Coating > 1mm thick
	– Thickness	mm.t	Defect thickness measured in millimetres