REPORT TO

MACARTHUR PROJECTS

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED SHOPTOP DEVELOPMENT

AT

1105-1107 BARRENJOEY ROAD, PALM BEACH, NSW

Date: 1 December 2020

Ref: 33500Srpt

JKGeotechnics

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DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
33500Srpt	Final Report	1 December 2020

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ATTACHMENTS

1 INTRODUCTION

This report presents the results of a geotechnical investigation for a proposed shop-top development at 1105-1107 Barrenjoey Road, Palm Beach, NSW. The location of the site is shown in Figure 1.

The proposed scheme involves demolition of the existing two-storey development and excavation of a basement level over which will be a constructed a 3-storey shop-top development. Details of the development have been provided in architectural drawings by PBD Architects, Project No 1816 dated 18 September 2018. The basement will have a finished floor level (FFL) at RL 0.5m which will require excavation to depths of up to about 4m below surrounding ground levels. Structural details such as column loads were not known at the time of preparing this report so have been assumed to be typical for developments of this size.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions, and to use this as a basis for providing comments and recommendations on issues including excavation, shoring, earth pressures, foundations, groundwater and soil aggression.

This geotechnical investigation was carried out in conjunction with an environmental site assessment by our environmental division, JK Environments (JKE). Reference should be made to the separate report by JKE, Ref: E33500PHrpt, for the results of the environmental site assessment.

2 INVESTIGATION PROCEDURE

Two boreholes, BH1 and BH2, were drilled using our track-mounted JK205 and JK308 rigs to depths of 19.5m (BH1) and 16.1m (BH2). These boreholes were initially auger drilled to the water table and then extended by wash boring with a casing advancer to depths of 12.0m and 10.1m respectively. Drilling was continued by diamond coring techniques using an NMLC core barrel with water flush. As the rock in BH1 was of such poor quality, coring was stopped and wash boring continued, and then coring re-commenced when harder drilling resistance was noted.

The investigation locations, as shown on Figure 2, were set out by taped measurements from existing surface features and inferred site boundaries. The approximate surface levels, as shown on the borehole logs, were estimated by interpolation between spot levels shown on the supplied survey plan by Veris Australia (Job No. 200442, dated 22 May 2018). The datum of the levels is Australian Height Datum (AHD).

The strength and relative density of the subsurface 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. The strength of the cored sandstone was assessed with reference to Point Load Strength Index ($I_{s(50)}$) test results. The point load strength test results are summarised on the attached STS Table A and on the cored borehole logs.

Groundwater observations were made during and on completion of auger drilling. The use of water for wash boring and core drilling limited meaningful measurements of groundwater levels immediately following drilling. Standpipe piezometers were installed in BH1 and BH2 to allow long-term monitoring of groundwater levels, and we revisited site about 1 week following installation to measure water levels in the standpipes.

Our geotechnical engineers set out the borehole locations, nominated the sampling and testing locations, and prepared logs of the strata encountered. The borehole logs, which include field test results and groundwater observations, 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 samples were returned to Soil Test Services Pty Ltd (STS) and Envirolab Services Pty Ltd, both NATA registered laboratories, for testing to determine point load strength index, pH, sulphate content, chloride content and resistivity. The results of the laboratory testing are summarised in the attached STS Table A and Envirolab Report No. 146253.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located at the junction of Barrenjoey Road and Iluka Road on a relatively level, low-lying estuarine terrace. About 40m to the east, on the opposite side of Barrenjoey Road, a steep wooded hillslope rises to the ridgeline of the Palm Beach peninsula.

The adjacent roads form the northern, eastern and western boundaries to the site. To the south the neighbouring site is occupied by a three-storey mixed use development which has what appears to be a single basement level but the floor level could not be ascertained.

The site was occupied by a two-storey mixed-use development with some attic rooms forming a third level. Based on a cursory external inspection the buildings on the subject and adjoining site appear to be in good condition.

3.2 Subsurface Conditions

The Sydney Geological Sheet shows the site to be underlain by Quaternary alluvium which can be interpreted to overlie strata of the Newport Formation of the Narrabeen Group. The Newport Formation commonly includes interbedded shales and sandstones.

The investigation has confirmed the expected subsurface profile, a summary of the borehole findings is presented below but for details reference should be made to the attached borehole logs and test results.

Pavements and Fill

Both boreholes encountered brick pavers over a bedding course of sand with traces of gravel.

Alluvial Soils

An alluvial soil profile was found in both boreholes extending to depths of 12m in BH1 and 10.1m in BH2. The soil profiles varied between the boreholes showing that stratification is of limited lateral extent. In BH1 the soils comprised mainly grey sand with traces of fine to medium grained gravel and shell fragments throughout. The sand ranged from loose to medium dense throughout. In BH2 the similar sand layer extended to 5.7m depth where sandy silty clay was present, becoming silty clay from 6.8m depth to rock level at 10.12m depth. The upper sand layer was loose to medium dense whilst the sandy and silty clays were initially stiff, becoming very stiff then very stiff to hard with depth.

Bedrock

The bedrock which occurred below the depths of 12.0m and 10.1m respectively in boreholes BH1 and BH2 was extremely weathered and hard/dense soil strength, interbedded with bands of hard silty clay. Several substantial zones where no core was recovered are expected to comprise similar materials eroded away by the drill flush water. There was no material change in rock quality in BH1 to the termination depth of 19.5m. In BH2 however, bands of low to medium strength sandstone were interbedded with hard silty clay below about 12m depth.

Groundwater

Groundwater seepage and borehole collapse were encountered at depths of 3.0m in BH1 and 3.6m in BH2 during auger drilling, which prompted a change to wash boring. Groundwater levels recorded in the standpipe piezometers on 18 November 2020, about 6 days after drilling, were at depths of 3.3m in BH1 (RL0.7m) and 3.5m in BH2 (RL0.6m).

3.3 Laboratory Test Results

The point load strength index test results presented in Table A confirm the relatively low strength of the sandstone bands in the deeper part of BH2. The samples from the remainder of BH2 and from BH1 were too weak to test.

The results of the soil chemistry tests presented in Envirolab Report No 255670 show the following:

		BH1 0.5-0.95m	BH2 5.7-6.2m	BH2 7.2-7.7m
pH 1:5 soil:water	pH Units	9.5	8.6	5.6
Chloride, Cl 1:5 soil:water	mg/kg	<10	24	53
Sulphate SO4 1:5 soil:water	mg/kg	<10	75	230
Resistivity in soil*	ohm m	220	77	58

Based on the above the soil pH varies from moderately acidic to strongly alkaline, chloride and sulphate contents are low and resistivity is quite high. Based upon these values the soil conditions are considered mildly aggressive to concrete piles and non-aggressive to steel piles, in accordance with the classification in the piling code, AS 2159-2009.

4 COMMENTS AND RECOMMENDATIONS

4.1 Geotechnical Issues

The main geotechnical issues associated with the proposed development are the high water table and the weak soil and bedrock profile which provide somewhat limited support potential for footing systems. As a result, dewatering will be required during construction, the basement will probably have to be tanked and designed to resist uplift forces, an impermeable shoring system such as a secant pile wall will be required and piled footings will have limited capacity. These and other issues are discussed in more detail below.

4.2 Dilapidation Surveys

Prior to demolition, a dilapidation report should be completed, both externally and internally, on the adjoining property located to the south of the site as well as the surrounding roads. The owners should be asked to confirm that the dilapidation reports represent a fair record of actual conditions. The dilapidation reports may then be used as a benchmark against which to assess possible future claims for damage resulting from the works. Detailed surveys should also be carried out of the buried services and note made of any that are likely to be sensitive to displacement.

4.3 Excavation and Shoring

Prior to excavation commencing a shoring system must be installed around the perimeter of the proposed basement. The shoring will need to be impermeable and therefore, for a site of this size, a secant pile wall is expected to be the most appropriate. Other wall types such as a cutter soil mix wall require large plant that is probably not economic but would be satisfactory from a technical perspective. Sheet piles are not suitable due to the vibration issues associated with installation. It must be decided at an early stage whether the shoring is to be load bearing as that will have a major influence on design of the shoring piles and the foundation system.

The shoring will need to provide a cut-off to groundwater flow as the groundwater level, which is currently just above the proposed basement floor level, must be drawn down to about 1m below bulk excavation level to permit construction of footings and services; the drawdown level will therefore be around RL-1m.

Once the shoring wall is installed excavation can commence and will require only the usual excavation plant to remove sandy soils. Care should be taken when operating plant to avoid sudden stop-start movements which can generate damaging levels of vibration in sandy soils. Where the shoring is anchored or propped, as it probably will have to be, it is important to avoid over-excavation at the levels where anchors or props are to be installed. Excavation works should be complemented by reference to the Code of Practice 'Excavation Work' prepared by the NSW Government and dated January 2020.

4.4 Dewatering

Groundwater is expected to be a significant issue for this development due to the shallow water table across the site. Dewatering will thus be required for basement excavation and for trafficability of construction equipment. Since an impermeable shoring will be constructed around the basement perimeter, we expect temporary dewatering should not cause excessive drawdown outside the site provided the cut-off is properly designed and constructed. Nevertheless, dewatering must be carefully controlled and monitored to reduce the risk of excessive drawdown outside of the basement causing settlement of adjoining buildings supported on shallow footings.

Detailed hydrogeological analysis of the dewatering will be required to assess the effect of dewatering on neighbouring properties and optimise the depth of shoring cut-off. In the northern area it should be possible to embed the cut-off wall into the clay/clayey sand layers which occur below about 6m depth to keep flows below acceptable levels. No such clay layer was encountered in BH1 and a deeper cut-off may be required in that area. Further more detailed investigation of this and other geotechnical conditions is required, as soon as access is feasible.

Permanent dewatering systems are not likely to be approved, therefore the basement will need to be tanked and designed to take the hydrostatic lateral and uplift pressures into account. However, if it can be shown the adjacent basement is at the same level as that proposed and the existing basement is dewatered permanently, then it may be possible to mount a case for similar treatment at the subject site.

Dewatering may be carried out using vacuum spear point wells, though that may not be successful where there is a clay layer at shallow depth such as at BH2 and in that case larger wells with submersible pumps may be more effective. The power supply and noise generation of dewatering systems needs to be carefully planned to minimise disturbance.

Water quality must be determined to assess the need for treatment prior to discharge.

4.5 Shoring Design

To reduce the effects of dewatering on the neighbouring property, the retention system must be installed to a minimum depth which satisfies stability and dewatering considerations. It must also be decided whether the shoring system is to support structural loads as founding the shoring wall in the soil layers will result in a low bearing capacity and it may become necessary to found the wall on rock if this becomes an issue.

Another key issue to determine at an early stage is the exact location of the shoring wall of the adjoining basement. If the wall is on the boundary then it will probably simplify the proposed design as there will be little support required to the adjoining property.

Lateral restraint in the form of soil anchors will probably be required to reduce deflections, and these must be installed progressively as excavation proceeds. If anchors are to be installed, they will extend beyond the site boundaries, and permission of the owners and authorities must be obtained before installation. If approval is not forthcoming then walls will need to be laterally supported by alternative methods, such as berms or props which would cause difficulties in construction of the footings and slab. We note the sands are of low to medium density and this will somewhat limit the capacity of anchors.

For preliminary design of propped or anchored walls, we recommend the use of a rectangular envelope of lateral pressure of 6H (kPa), where H is the retained height in metres. In areas that are sensitive to adjacent movements, such as where structures or movement sensitive services are located within 2H of the wall, a higher earth pressure distribution of 8H kPa should be used. Design using more sophisticated software, such as Wallap and Plaxis, is likely to result in more economical design.

The lateral toe resistance of shoring walls can be calculated using a passive earth pressure coefficient, K_p, of 3.0 for stiff clay and medium dense sand. A factor of safety of 2 should be applied to the calculated resistance due to the large strains necessary to generate the full pressure.

All surcharge loads and hydrostatic pressures should be allowed for in the shoring design. The design must also take into account the groundwater situation where there will be differential water levels on the active and passive sides of the wall as well as allow for higher groundwater levels which could occur due to climatic reasons, e.g. rainfall or flooding.

As a guide soil anchors bonded into the sands may be designed (if required) based on an effective friction angle of 28° for sands of very loose to loose relative density, 30^0 for loose to medium dense and 32° for medium dense sands. Uncased anchor holes within the sands will collapse and temporary casing of these holes will be required. Anchors with penetrations through the wall below the water table would be subject to inflows of both soil and water which would cause subsidence outside the excavation and would be very difficult to control. In this case it should only be necessary to use a single row of anchors with heads above the water table.

Only expert contractors should be used for this type of anchor construction as poor techniques can result in damage to adjoining properties. Anchor bond lengths should be proof-tested to 1.3 times the working load under the direction of an experienced engineer of construction superintendent, independent of the anchor contractor. Lift-off tests should be carried out on 10% of anchors after 72 hours from initial tensioning to check that the anchors are holding their loads.

It is normal good practice for anchors to be a specialist design and construct sub-contract to avoid disputes if anchors fail to hold their test load.

4.6 Footings

There are a number of potential options for the footings for the proposed structure. These comprise piles founded in soil, piles to rock, a stiffened raft slab and a piled raft slab. The design of the footing system should take into account the silty clay and clayey sand layers encountered within the sandy profile. The footing options are discussed in more detail below but a final decision on the best system will require further investigation when access to the whole site for drilling and CPT rigs is possible. In the meantime, it may be sufficient to have a conservative design based on piles to rock as the fall-back option.

4.6.1 Piles to Rock

The proposed structure may be supported using piled footings founded in the underlying shale and sandstone though we note that the rock is moderately deep and of very low strength. The rock appears to dip slightly towards the south and a similar rock level was recoded in a borehole log we have access to that was drilled on the site to the south. Variations in the buried topography could result in irregular depths to rock and to date there has been only limited investigation of the site due to lack of access.

All piles should be uniformly founded within the underlying sandstone and shale bedrock. Where an ultimate limit state approach is adopted the following ultimate base resistance and shaft frictions may be used. For piles founded within the rock, skin friction within the upper 0.3m of rock socket should be ignored. A geotechnical strength reduction factor (ϕ_g) of 0.60 for the wall (ie high redundancy) and 0.52 for individual piles with low redundancy should be adopted where a limit state design approach is used in accordance with AS2159-2009, subject to further investigation to complete coverage of the site to the required standard. The following parameters may be adopted:

Rock Class	Ultimate End Bearing Pressure (MPa)	Serviceability End Bearing Pressure (MPa)	Ultimate Shaft Adhesion (kPa)	Serviceability Shaft Adhesion (kPa)
Hard Clay	2	0.6	75	30
V	3	0.8	150	60

Classification in accordance with Foundations on Sandstone and Shale in the Sydney Region, Pells, Mostyn and Walker, Australian Geomechanics, Dec 1998 and 2019.

Substantially more investigation will be necessary to confirm the rock quality and whether there is better quality rock at greater depth that may be more economical for pile design. Based upon the current boreholes the hard clay stratum probably starts at about 13.5m in BH1 and 10.2m in BH2. The Class V shale (and interbedded sandstone) starts at 16.8m in BH1 and 12.1m in BH2.

4.6.2 Raft Slabs

As loose to medium dense sands appear to be present at the bulk excavation level, the use of a raft slab may be considered; a piled raft slab is also possible.

The design of a raft slab would need to take into consideration the potential for settlements due to loose sand layers and the stiff clays (found in BH2). The existing information is suitable only to carry out a feasibility design and when access is available it would be necessary to complete Cone Penetrometer Testing (CPT) and preferably dilatometer testing (DMT) as the latter provides direct measurements of sol modulus used in design.

As discussed in Section 4.5, the shoring system may terminate in the medium dense clayey sand stratum, and may not be suitable for bearing substantial loads. Therefore, column and line loads around the perimeter of raft may be high, which is difficult to accommodate economically in raft design. The alternative would be to push the shoring wall even deeper but this would be costly.

Detailed analysis of a raft would be required to estimate the settlements and the contact pressures below the raft. The preliminary design of the raft may be completed using the elastic parameter model provided below with both upper and lower bound parameters being checked so as to estimate differential settlements due to horizontal variations in soil properties beneath the raft. We must emphasise that the properties of the deep soil layers have not been clearly established to date and further work in this regard is necessary.

RECOMMENDED ELASTIC PARAMETERS FOR DESIGN			
Unit	**Parameters**	**Lower Bound**	**Upper Bound**
Loose to Medium Dense Sand	Depth (m)/ Elastic Modulus (MPa)/ Poisson's Ratio	BH1 3m – 12m/10/0.3 BH2 3m -5.5m/10/0.3	20/0.3 20/0.3
Sandy silty clay	Depth (m)/ Elastic Modulus (MPa)/ Poisson's Ratio	BH2 5.5m–10.5m/8/0.3	40/0.3
Class V shale and sandstone	Depth (m)/ Elastic Modulus (MPa)/ Poisson's Ratio	BH1 12m-19.5m/30/0.3 BH2 10.5m – 12m/30/0.3	100/0.25 100/0.25
Class V sandstone	Depth (m)/ Elastic Modulus (MPa)/ Poisson's Ratio	BH2 12m – 16m/75/0.3	150/0.25

The design of heavily loaded raft footings is complex and requires complex analysis procedures for soil/structure interaction. Therefore, we expect that the design of the raft will be an iterative procedure with both the geotechnical and structural engineers having input to the process. The first pass of the analysis will demonstrate the potential of the concept and identify the parameters critical to the design. The parameters will then need refinement and may require further investigation and testing to justify the key assumptions and enable the design to be refined. Further geotechnical investigations involving a close grid of Cone Penetrometer (CPT) testing together with Marchetti Dilatometer (DMT) testing may be needed to obtain a continuous subsurface profile and assess the extent of any weaker subsurface conditions. The latter tool, the dilatometer, is particularly useful as it provides a direct measurement of the soil stiffness characteristics (elastic modulus). A potential drawback of any indirect testing such as the CPT is that the mixed soils (not clearly clay or sand) are difficult to interpret and some direct sampling may be necessary.

We can assist with the detailed geotechnical analysis of the raft using our finite element analysis software, once the initial raft details are supplied by the structural engineer.

4.7 Basement Slab

For a tanked basement, the basement floor slab or raft slab must be designed for uplift forces due to hydrostatic pressure, with normal groundwater levels assumed at about RL1m but with peak levels at, say, 1.5m above "normal" level. If flood studies show the area may be inundated then peak groundwater level should be taken as surface level. Peak levels can be limited by use of pressure relief drains if necessary, but only if the basement is protected by dam doors or similar. Waterproof construction systems are required for external walls. An assessment of groundwater seepage rates during construction can be assessed by computer modelling as discussed in Section 4.4 above. Data could be improved by completing pump out tests within the monitoring wells installed in BH1 and BH2.

As a minimum, following dewatering and bulk excavation, the exposed subgrade should be proof-rolled with a 5 tonne deadweight, smooth drum vibratory roller. The proof-rolling should be carried out under the direction of an experienced earthworks superintendent to assist in the detection of unstable areas which were not disclosed by this investigation and to be sure that vibrations do not affect adjoining properties. Tolerable vibration thresholds are provided in the attached Vibration Emission Design Goals explanatory note. Any unstable areas identified during proof-rolling should be locally excavated down to a competent base and replaced with engineered fill. If a raft slab footing is adopted a more stringent specification will be needed which will include a testing regime to demonstrate that the subgrade matches the design assumptions for the raft.

The materials recommended for use as engineered fill are well-graded granular materials, such as ripped and/or crushed sandstone, free of deleterious substances, contaminants and having a maximum particle size of 75mm. The sandy soils excavated from the site would also be suitable for reuse as engineered fill. Engineered fill should generally be placed in loose layers not exceeding 150mm and compacted to at least 98% of Standard Maximum Dry Density (SMDD). In-situ density tests will be required at close frequency to confirm the target density has been achieved.

A gravel working platform would be necessary to support the large piling rigs likely to be needed within the excavation if piled footings are used; such a layer would also be useful as a construction platform. This working platform can be a significant cost factor which must be considered early in the design. Large rigs even on medium dense sand may need platforms 600mm or more in thickness and the cost of exporting the over-excavated material for the platform is also likely to be high.

4.8 Further Work

Although the investigation to date has provided a good basic understanding of the geotechnical conditions at the site, design and construction of the proposed development will require significant further geotechnical work once the design concepts are better known. We envisage some or all of the following being necessary:

- Additional CPT and/or dilatometer testing to assess soil parameters for raft slab design.
- Additional cored boreholes to assess rock properties for pile design.
- Groundwater quality testing.

- Wallap/Plaxis analysis of shoring walls.
- Plaxis analysis of a raft slab
- Seepage analysis to assess likely volumes of groundwater inflows during construction and drawdown effect on water table outside the excavation.
- Calculation of working platform thickness for construction plant.

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.

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.

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.

TABLE A
POINT LOAD STRENGTH INDEX TEST REPORT

Client: Macarthur Projects
Ref No: 33500S

Project: Proposed Mixed Use Development
Report: A

Location: 1105-1107 Barrenjoey Road, Palm Beach, NSW
Report Date: 12/11/20

BOREHOLE NUMBER	DEPTH (m)	$I_{S\ (50)}$ (MPa)	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)	TEST DIRECTION
2	12.20 - 12.24	0.2	4	A
	13.66 - 13.70	0.3	6	A
	14.40 - 14.44	0.3	6	A
	15.06 - 15.09	0.3	6	A
	16.02 - 16.06	0.2	4	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).

Envirolab Services Pty Ltd
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12 Ashley St Chatswood NSW 2067
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www.envirolab.com.au

CERTIFICATE OF ANALYSIS 255670

Client Details	
Client	JK Geotechnics
Attention	Warren Smith
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details	
Your Reference	**33500S, Palm Beach**
Number of Samples	3 Soil
Date samples received	13/11/2020
Date completed instructions received	13/11/2020

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	20/11/2020
Date of Issue	20/11/2020

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Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil				
Our Reference		255670-1	255670-2	255670-3
Your Reference	UNITS	BH1	BH2	BH2
Depth		0.5-0.95	5.7-6.15	7.2-7.65
Date Sampled		11/11/2020	11/11/2020	11/11/2020
Type of sample		Soil	Soil	Soil
Date prepared	-	16/11/2020	16/11/2020	16/11/2020
Date analysed	-	16/11/2020	16/11/2020	16/11/2020
pH 1:5 soil:water	pH Units	9.5	8.6	5.6
Chloride, Cl 1:5 soil:water	mg/kg	<10	24	53
Sulphate, SO4 1:5 soil:water	mg/kg	<10	75	230
Resistivity in soil*	ohm m	220	77	58

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	-			16/11/2020	1	16/11/2020	16/11/2020		16/11/2020	[NT]
Date analysed	-			16/11/2020	1	16/11/2020	16/11/2020		16/11/2020	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	1	9.5	9.5	0	102	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	<10	<10	0	92	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	<10	<10	0	94	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	1	220	210	5	[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.
1

Client: MACARTHUR PROJECTS
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 1105-1107 BARRENJOEY ROAD, PALM BEACH, NSW

Job No.: 33500S | **Method:** SPIRAL AUGER AND WASHBORE | **R.L. Surface:** ~4.0 m
Date: 12/11/20 | **Datum:** AHD
Plant Type: JK205 | **Logged/Checked By:** W.S./P.S.

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			4	0		-	BRICK PAVERS: 50mm.t				
							FILL: fine coarse grained sand, orange brown, trace of igneous gravel.	M			
				0.25		SP	SAND: fine to medium grained, light yellow brown.	M	MD		MARINE
		N = 11 5,5,6									
			3	1					L		
		N = 8 3,4,4									
			2	2					MD		
			1	3				W			
ON 18/11/20		N = 16 3,5,11									
ON COMPETION OF CORING			0	4							COMMENCE WASHBORE DRILLING
							as above, light grey, with trace of clay.				
		N = 11 3,4,7									
			-1	5					L		
			-2	6							
		N = 6 2,2,4									

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BOREHOLE LOG

Borehole No. 1

Client: MACARTHUR PROJECTS

Project: PROPOSED MIXED USE DEVELOPMENT

Location: 1105-1107 BARRENJOEY ROAD, PALM BEACH, NSW

Job No.: 33500S | **Method:** SPIRAL AUGER AND WASHBORE | **R.L. Surface:** ~4.0 m

Date: 12/11/20 | **Datum:** AHD

Plant Type: JK205 | **Logged/Checked By:** W.S./P.S.

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
						SP	SAND: fine to medium grained, grey, trace of fine to coarse grained sandstone gravel, and shell fragments.	W	L - MD		
	DS	N = 10 1,3,7									
			-4	8							
			-5	9							
	DS	N = 11 3,5,6									
			-6	10			SAND: fine to coarse grained, grey, trace of fine to medium grained igneous gravel.				
	DS	N = 8 3,4,4									
			-7	11							
											LOW 'TC' BIT RESISTANCE
		N=SPT 2/ 0mm REFUSAL	-8	12			REFER TO CORED BOREHOLE LOG				NO SPT RETURN GROUNDWATER MONITORING WELL INSTALLED TO 5.9m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 0.1m TO 5.9m. 2mm SAND FILTER PACK 0.4m TO 5.9m. BENTONITE SEAL 0.1m TO 0.4m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.
			-9	13							

JKGeotechnics

CORED BOREHOLE LOG

Borehole No.
1
3 / 4

Client: MACARTHUR PROJECTS
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 1105-1107 BARRENJOEY ROAD, PALM BEACH, NSW

Job No.: 33500S	**Core Size:** NMLC	**R.L. Surface:** ~4.0 m
Date: 12/11/20	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK205	**Bearing:** N/A	**Logged/Checked By:** W.S./P.S.

Water Loss\Level	Barrel Lift	RL (m AHD)	Depth (m)	Graphic Log	CORE DESCRIPTION: Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ (VL -0.1, L -0.3, M -1, H -3, VH -10, EH)	DEFECT DETAILS: SPACING (mm) (600, 200, 60, 20)	DEFECT DETAILS: DESCRIPTION (Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness) Specific / General	Formation
					START CORING AT 12.00m						
		-8	12		SANDSTONE: fine to medium grained, light grey mottled with orange brown.	MW	H				
					NO CORE 0.60m						
		-9	13		Silty sandy CLAY: medium plasticity, light grey and red brown, with iron indurated bands.	w>PL	St			(12.80m) HP: 440 kPa (12.90m) HP: 140 kPa (13.20m) HP: 250 kPa (13.60m) HP: >600 kPa	Newport Formation
							Hd				
		-10	14		WASHBORING						
		-11	15								
		-12	16		NO CORE 0.71m						
		-13	17		Extremely Weathered sandstone: silty sandy CLAY, grey, with iron indurated bands.	XW	Hd			(16.90m) HP: >600 kPa (17.50m) HP: 400 kPa (17.90m) HP: 500kPa	Newport Formation

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 33500S PALMBEACH.GPJ <<DrawingFile>> 01/12/2020 09:46 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

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CORED BOREHOLE LOG

Borehole No.
1
4 / 4

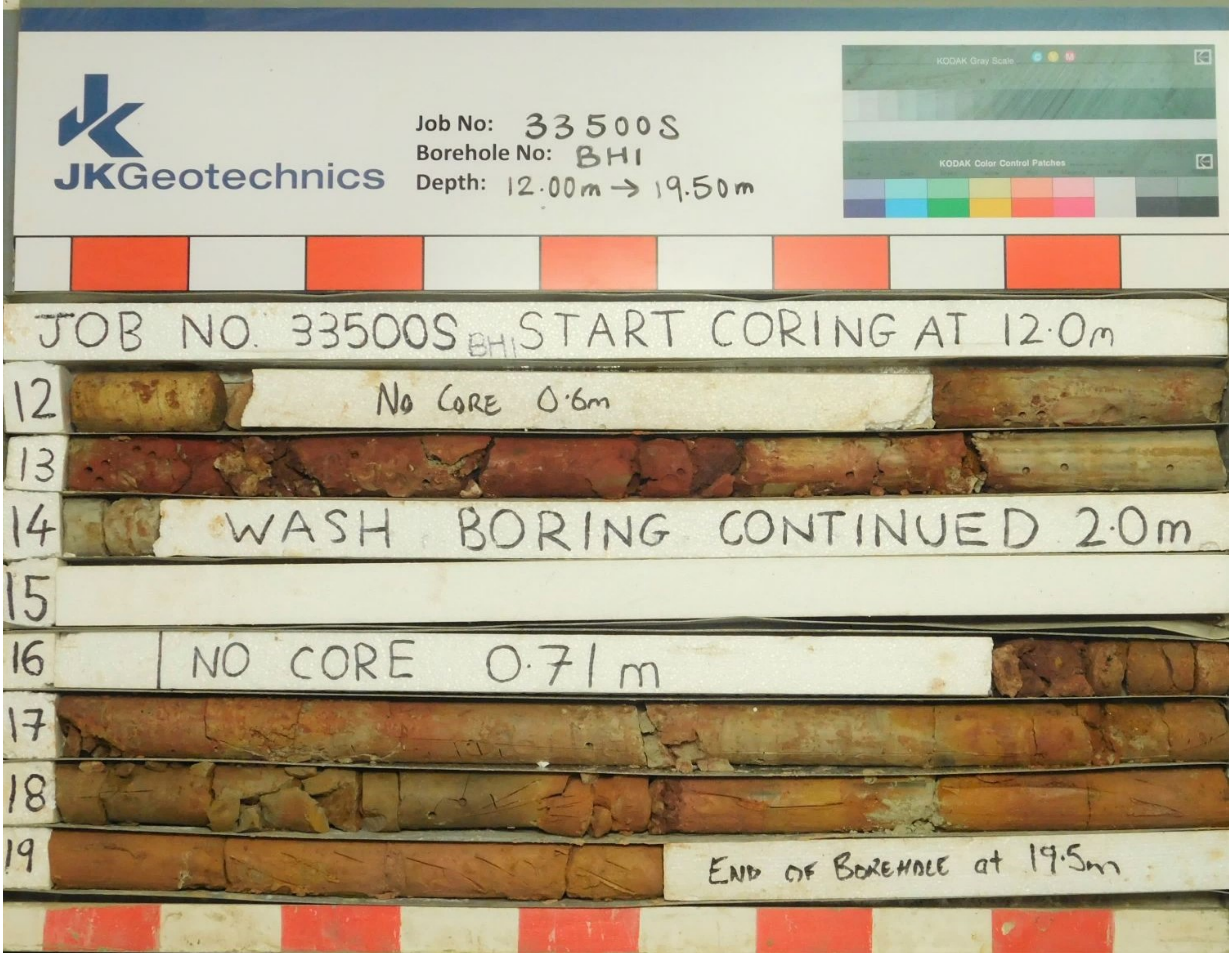
Client: MACARTHUR PROJECTS
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 1105-1107 BARRENJOEY ROAD, PALM BEACH, NSW

Job No.: 33500S	**Core Size:** NMLC	**R.L. Surface:** ~4.0 m
Date: 12/11/20	**Inclination:** VERTICAL	**Datum:** AHD
Plant Type: JK205	**Bearing:** N/A	**Logged/Checked By:** W.S./P.S.

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)	SPACING (mm) (600, 200, 60, 20)	DEFECT DETAILS DESCRIPTION: Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness — Specific / General	Formation
					Extremely Weathered siltstone: silty CLAY, dark grey, with iron indurated bands.	XW	Hd			(18.30m) HP: >600 kPa	Newport Formation
		-15	19		Extremely Weathered sandstone: silty clayey SAND, fine to medium grained, light grey and orange brown.		D				
					END OF BOREHOLE AT 19.50 m						
		-16	20								
		-17	21								
		-18	22								
		-19	23								
		-20	24								

JK 9.02.4 LIB.GLB Log JK CORED BOREHOLE - MASTER 33500S PALMBEACH.GPJ <<DrawingFile>> 01/12/2020 09:46 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

FRACTURES NOT MARKED ARE CONSIDERED TO BE DRILLING AND HANDLING BREAKS


JKGeotechnics
Job No: 33500S
Borehole No: BH1
Depth: 12.00m → 19.50m
KODAK Gray Scale
KODAK Color Control Patches
JOB NO. 33500S BH1 START CORING AT 12.0m
12
NO CORE 0.6m
13
14
WASH BORING CONTINUED 2.0m
15
16
NO CORE 0.71m
17
18
19
END OF BOREHOLE at 19.5m

JKGeotechnics

BOREHOLE LOG

Borehole No.
2
1 / 3

Client: MACARTHUR PROJECTS
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 1105-1107 BARRENJOEY ROAD, PALM BEACH, NSW

Job No.: 33500S **Method:** SPIRAL AUGER AND WASHBORE **R.L. Surface:** ~4.1 m
Date: 11/11/20 **Datum:** AHD
Plant Type: JK308 **Logged/Checked By:** S.D./P.S.

Groundwater Record	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		4	0	-	BRICK PAVER: 50mm.t	M			
					FILL: Sand, fine to medium grained, brown, with silt, trace of fine to medium grained igneous and sandstone gravel.				
	N = 8 3,4,4			SP	SAND: fine to medium grained, light brown and brown.	M	L		MARINE
		3	1						
	N = 8 2,3,5				as above, but light brown.				
		2	2						
	N = 8 3,4,4	1	3						
(3.3m) ON COMPLETION OF CORING									
(3.48m) ON 18/11/20									
DURING DRILLING						W			
		0	4						
	N = 25 11,13,12						MD		
									COMMENCED WASHBORE DRILLING
		-1	5						
	N = 6 2,2,4			CI	Sandy silty CLAY: medium plasticity, light grey, with fine to medium grained sand.	w>PL	St	170 190 110	RESIDUAL
		-2	6						
				CH			VSt		

JKGeotechnics

BOREHOLE LOG

Borehole No.
2

Client: MACARTHUR PROJECTS
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 1105-1107 BARRENJOEY ROAD, PALM BEACH, NSW

Job No.: 33500S | **Method:** SPIRAL AUGER AND WASHBORE | **R.L. Surface:** ~4.1 m
Date: 11/11/20 | **Datum:** AHD
Plant Type: JK308 | **Logged/Checked By:** S.D./P.S.

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
	DS	N = 17 2,7,10	-3			CH	Silty CLAY: high plasticity, light grey, trace of fine to medium grained ironstone gravel. *(continued)*	w>PL	VSt	310 290 320	
			-4	8							
	DS	N = 17 4,9,8	-5	9			as above, but without ironstone gravel.		VSt - Hd	360 420 290	
			-6	10			REFER TO CORED BOREHOLE LOG				GROUNDWATER MONITORING WELL INSTALLED TO 5.76m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 2.76m TO 5.76m. CASING 0.11m TO 2.76m. 2mm SAND FILTER PACK 0.6m TO 5.76m. BENTONITE SEAL 0.3m TO 0.6m. BACKFILLED WITH SAND TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.
			-7	11							
			-8	12							
			-9	13							

JK 9.02.4 LIB.GLB Log JK AUGERHOLE - MASTER 33500S PALMBEACH.GPJ <<DrawingFile>> 01/12/2020 09:46 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

CORED BOREHOLE LOG

Client: MACARTHUR PROJECTS
Project: PROPOSED MIXED USE DEVELOPMENT
Location: 1105-1107 BARRENJOEY ROAD, PALM BEACH, NSW

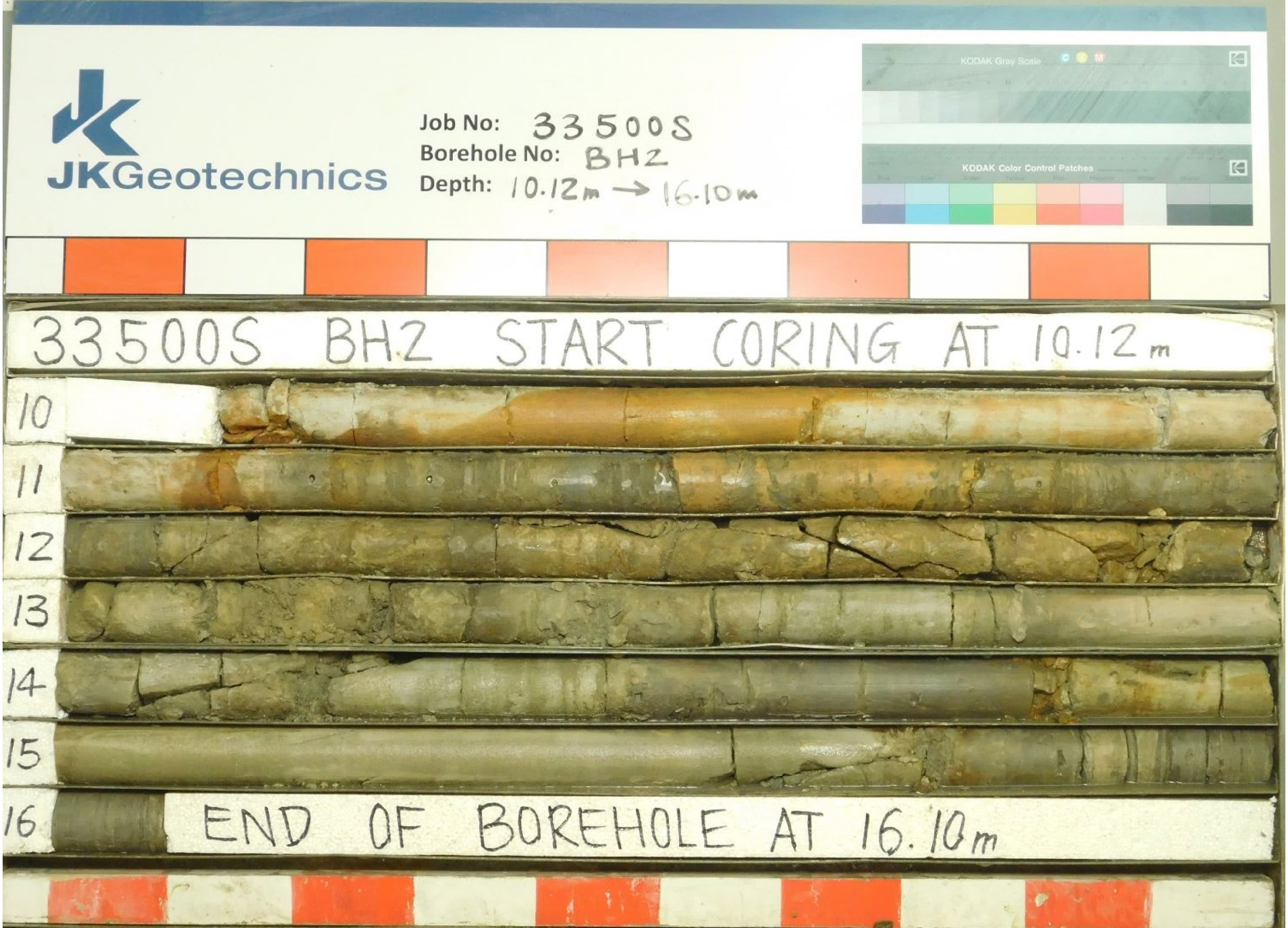
Job No.: 33500S | **Core Size:** NMLC | **R.L. Surface:** ~4.1 m
Date: 11/11/20 | **Inclination:** VERTICAL | **Datum:** AHD
Plant Type: JK308 | **Bearing:** N/A | **Logged/Checked By:** S.D./P.S.

START CORING AT 10.12m

Water Loss\Level	Depth (m)	CORE DESCRIPTION (Rock Type, grain characteristics, colour, texture and fabric, features, inclusions and minor components)	Weathering	Strength	DEFECT DETAILS – DESCRIPTION (Type, orientation, defect shape and roughness, defect coatings and seams, openness and thickness) – Specific	Formation
100% RETURN	10.12–11.0	Extremely Weathered sandstone: silty clayey SAND, fine to medium grained, light grey, with iron indurated bands.	XW	(D)		Newport Formation
	11.0–12.08	Extremely Weathered siltstone: silty CLAY, medium to high plasticity, light grey, grey and brown.	XW	Hd	(11.20m) HP: 500 kPa (11.40m) HP: >600 kPa (11.60m) HP: >600 kPa (11.80m) HP: >600 kPa	
	12.08–13.25	SANDSTONE: fine to medium grained, light grey and brown, bedded at 0-5°.	HW	VL	(12.10m) J, 65°, P, R, Cn (12.32m) CS, 0° (12.42m) J, 70 - 90°, Un, R, Cn (12.73m) J, 60°, P, R, Cn (12.82m) J, 60°, P, R, Cn (12.87m) J, 80°, Un, R, Cn (13.11m) XWS, 0°	
	13.25–13.5	Extremely Weathered siltstone: silty CLAY, medium to high plasticity, grey.	XW	Hd	(13.40m) HP: >600 kPa	
	13.5–13.78	SANDSTONE: fine to medium grained, light grey, bedded at 0-5°.	SW	L - M	(13.75m) J, 90°, P, R, Cn	
	13.78–14.07	Extremely Weathered siltstone: silty CLAY, medium to high plasticity, grey.	XW	Hd	(13.90m) HP: >600 kPa	
	14.07–14.48	SANDSTONE: fine to medium grained, light grey, bedded at 0-5°.	SW	L - M	(14.21m) J, 70°, P, R, Cn	
	14.48–14.82	Extremely Weathered siltstone: silty CLAY, high plasticity, grey and brown.	XW	Hd	(14.60m) HP: >600 kPa (14.70m) HP: >600 kPa	
	14.82–16.10	SANDSTONE: fine to medium grained, light grey, bedded at 0-5°.	SW	L - M	(15.26m) Jh, 75°, Un (15.60m) J, 80°, Un, R, Cn (15.79m) CS, 0° (15.92m) CS, 0°	
	16.10	END OF BOREHOLE AT 16.10 m				

RL (m AHD): -6 to -12 over depth 10.12–16.10 m. Point load strength index $I_s(50)$ 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: 33500S
Borehole No: BH2
Depth: 10.12m → 16.10m
KODAK Gray Scale
KODAK Color Control Patches
33500S BH2 START CORING AT 10.12m
10
11
12
13
14
15
16
END OF BOREHOLE AT 16.10m

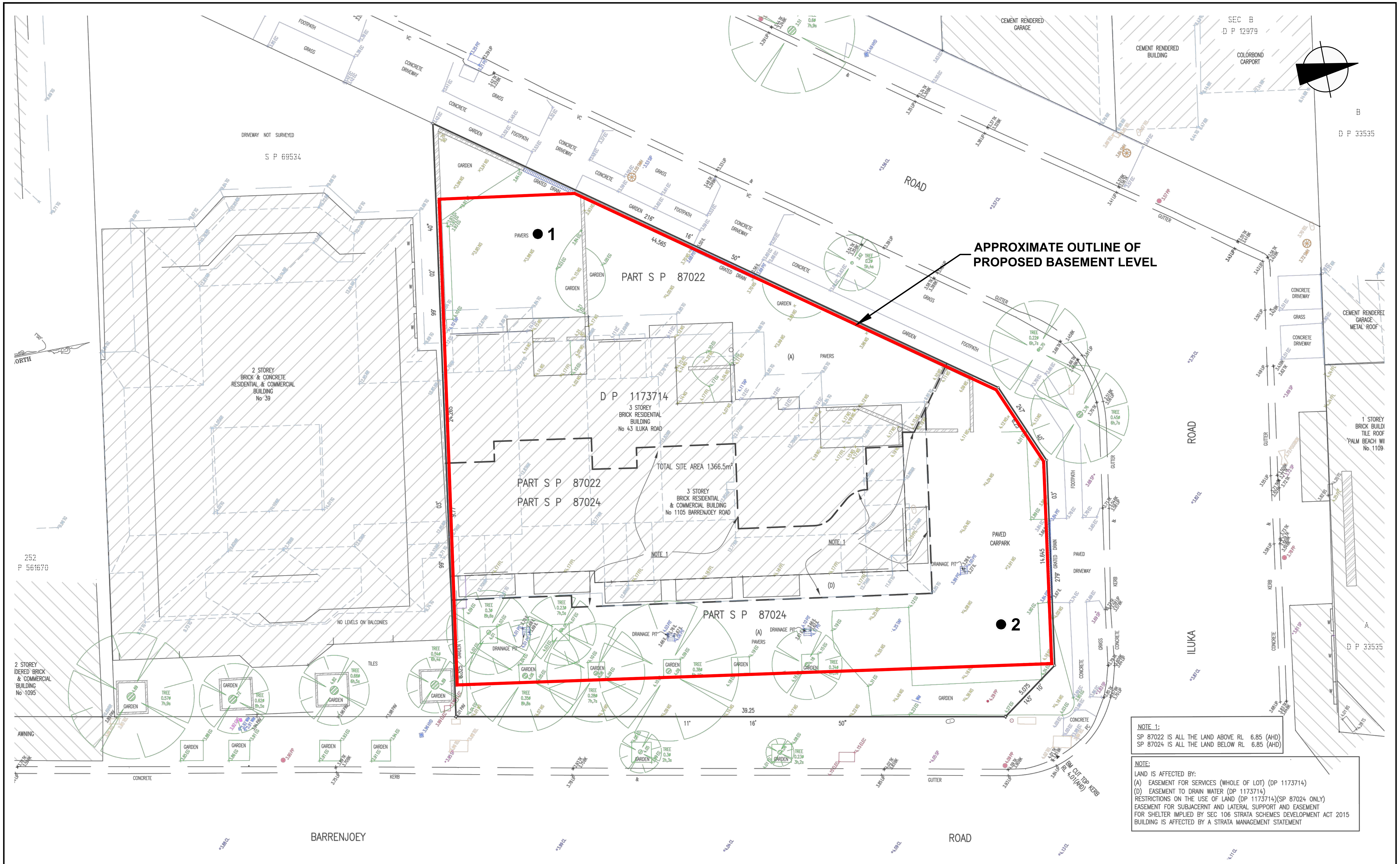
PLOT DATE: 17/11/2020 3:31:17 PM DWG FILE: Z:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\33000'S\33500S PALM BEACH\CAD\33500S.DWG

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:	**SITE LOCATION PLAN**
Location:	1105-1107 BARRENJOEY ROAD, PALM BEACH, NSW
Report No: 33500S	Figure No: 1

JKGeotechnics



0 2.5 5 7.5 10 12.5

SCALE 1:250 @A3 METRES

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

Title: **BOREHOLE LOCATION PLAN**

Location: 1105-1107 BARRENJOEY ROAD, PALM BEACH, NSW

Report No: 33500S

Figure No: 2

JKGeotechnics

PLOT DATE: 25/11/2020 3:06:32 PM DWG FILE: Y:\33000'S\33500S PALM BEACH\CAD\33500S.DWG

VIBRATION EMISSION DESIGN GOALS

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

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

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

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

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

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

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

REPORT EXPLANATION NOTES

INTRODUCTION

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

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

DESCRIPTION AND CLASSIFICATION METHODS

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

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

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

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

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

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

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

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

SAMPLING

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

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

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

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

INVESTIGATION METHODS

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

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

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

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

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

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

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

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

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

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

The test results are reported in the following form:

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

  N = 13
  4, 6, 7

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

  N > 30
  15, 30/40mm

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

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

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

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

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

The information provided on the charts comprise:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

LOGS

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

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

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

GROUNDWATER

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

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

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

FILL

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

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

LABORATORY TESTING

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

ENGINEERING REPORTS

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

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

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

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

SITE ANOMALIES

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

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

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

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

REVIEW OF DESIGN

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

SITE INSPECTION

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

Requirements could range from:

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

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

iii) full time engineering presence on site.

SYMBOL LEGENDS

SOIL

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

ROCK

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

OTHER MATERIALS

- BRICKS OR PAVERS
- CONCRETE
- ASPHALTIC CONCRETE

CLASSIFICATION OF COARSE AND FINE GRAINED SOILS

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

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

Laboratory Classification Criteria

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

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

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

NOTES:

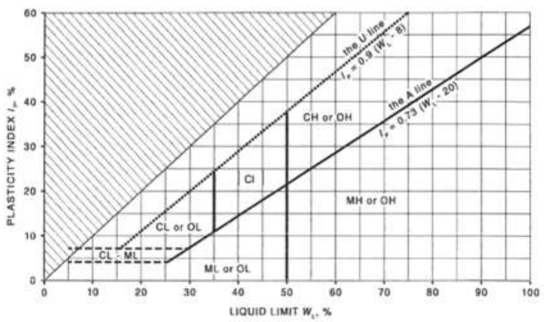
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

2 Where the grading is determined from laboratory tests, it is defined by coefficients of curvature (C_c) and uniformity (C_u) derived from the particle size distribution curve.

3 Clay soils with liquid limits > 35% and ≤ 50% may be classified as being of medium plasticity.

4 The U line on the Modified Casagrande Chart is an approximate upper bound for most natural soils.

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



LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record	▼	Standing water level. Time delay following completion of drilling/excavation may be shown.
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.
	▶	Groundwater seepage into borehole or test pit noted during drilling or excavation.
Samples	ES	Sample taken over depth indicated, for environmental analysis.
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.
	DB	Bulk disturbed sample taken over depth indicated.
	DS	Small disturbed bag sample taken over depth indicated.
	ASB	Soil sample taken over depth indicated, for asbestos analysis.
	ASS	Soil sample taken over depth indicated, for acid sulfate soil analysis.
	SAL	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	Vane shear reading in kPa of undrained shear strength.
	PID = 100	Photoionisation detector reading in ppm (soil sample headspace test).
Moisture Condition (Fine Grained Soils)	*w* > PL	Moisture content estimated to be greater than plastic limit.
	w ≈ PL	Moisture content estimated to be approximately equal to plastic limit.
	w < PL	Moisture content estimated to be less than plastic limit.
	w ≈ LL	Moisture content estimated to be near liquid limit.
	w > LL	Moisture content estimated to be wet of liquid limit.
(Coarse Grained Soils)	D	DRY – runs freely through fingers.
	M	MOIST – does not run freely but no free water visible on soil surface.
	W	WET – free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS	VERY SOFT – unconfined compressive strength ≤ 25kPa.
	S	SOFT – unconfined compressive strength > 25kPa and ≤ 50kPa.
	F	FIRM – unconfined compressive strength > 50kPa and ≤ 100kPa.
	St	STIFF – unconfined compressive strength > 100kPa and ≤ 200kPa.
	VSt	VERY STIFF – unconfined compressive strength > 200kPa and ≤ 400kPa.
	Hd	HARD – unconfined compressive strength > 400kPa.
	Fr	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 $Is_{(50)}$ (MPa)	Guide to Strength: Field Assessment
Very Low Strength	VL	0.6 to 2	0.03 to 0.1	Material crumbles under firm blows with sharp end of pick; can be peeled with knife; too hard to cut a triaxial sample by hand. Pieces up to 30mm thick can be broken by finger pressure.
Low Strength	L	2 to 6	0.1 to 0.3	Easily scored with a knife; indentations 1mm to 3mm show in the specimen with firm blows of the pick point; has dull sound under hammer. A piece of core 150mm long by 50mm diameter may be broken by hand. Sharp edges of core may be friable and break during handling.
Medium Strength	M	6 to 20	0.3 to 1	Scored with a knife; a piece of core 150mm long by 50mm diameter can be broken by hand with difficulty.
High Strength	H	20 to 60	1 to 3	A piece of core 150mm long by 50mm diameter cannot be broken by hand but can be broken by a pick with a single firm blow; rock rings under hammer.
Very High Strength	VH	60 to 200	3 to 10	Hand specimen breaks with pick after more than one blow; rock rings under hammer.
Extremely High Strength	EH	> 200	> 10	Specimen requires many blows with geological pick to break through intact material; rock rings under hammer.

Abbreviations Used in Defect Description

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