REPORT TO

HANNAS CONTRACTING SERVICES PTY LTD

ON

PRELMINARY GEOTECHNICAL INVESTIGATION

FOR

PROPOSED INDUSTRIAL DEVELOPMENT

AT

101-105 OLD PITTWATER ROAD,
BROOKVALE, NSW

Date: 28 February 2022
Ref: 34695PHrpt

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ATTACHMENTS

1 INTRODUCTION

This report presents the results of a preliminary geotechnical investigation for the proposed industrial development at 101-105 Old Pittwater Road, Brookvale, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by a Hannas Contracting Services Pty Ltd Purchase Order Number 569-09, dated 12 January 2022. The commission was on the basis of our fee proposal, Ref. P55630PH, dated 10 December 2021.

From the provided architectural design proposal drawings (Issue P4, dated 18 February 2022) by Rothelowman (which were received after completion of our fieldwork), we understand that following demolition of the existing structures on site, a two level industrial unit building suspended over a single basement level will be constructed. The finished floor level (FFL) of the proposed basement will be at RL9.6m, requiring excavation to a depth of about 6.5m along its western end, tapering to about 3.4m along its eastern end. The outline of the proposed basement is shown on Figure 2.

We have not been provided with any structural loads, but expect they could be relatively high noting the above ground portion of the proposed building will be suspended over the proposed basement.

The purpose of the investigation was to assess the subsurface conditions at three investigation locations, and based on the information obtained, to provide comments and recommendations on an additional geotechnical investigation and geotechnical constraints, shoring design, dewatering, excavation, footing design, soil aggression and the basement floor slab.

This geotechnical investigation was carried out in conjunction with a Preliminary (Stage 1) Site Investigation (PSI) by our environmental division, JK Environments (JKE). Reference should be made to the separate report by JKE, Ref: 34695PRrpt, for the results of the PSI.

2 INVESTIGATION PROCEDURE

The fieldwork for the investigation was carried out on 22 January 2022 and comprised the following scope of work.

1. The completion of three Cone Penetrometer Tests (CPT) to refusal depths of 12.67m (CPT1), 13.12m (CPT2) and 9.03m (CPT3) below the existing surface levels using our CPT rig. Due to the presence of fill, the upper 0.8m of CPT1 and 0.4m of CPT2 was probed using a 'dummy' cone; and
2. At each CPT location, the auger drilling of three boreholes (BH1, BH2 and BH3) using our Eziprobe drilling rig to depths of 2.0m, 3.0m, and 2.0m, respectively, for the purpose of recovering soil samples for subsequent laboratory testing.

The investigation locations, as shown on Figure 2, were set out using a tape measure from existing surface features. The approximate surface RLs shown on the attached borehole logs and CPT results were interpolated between spot levels and surface contour lines shown on the supplied survey plan prepared by

Land Partners (Plan Number. SY075462.000.4, Revision 1, dated 3 February 2022). The supplied survey plan laid over a recent Nearmap aerial image forms the basis of Figure 2. The survey datum is the Australian Height Datum (AHD).

The pavements were cored with a diamond tipped thin walled tube with water flush. CPT testing involves continuously pushing a probe with a 35mm diameter conical tip into the soil profile using the hydraulic rams of the CPT rig. Measurements of the end resistance of the conical tip and the frictional resistance of a separate 134mm long sleeve located directly behind the cone are made during the testing. We note that CPT testing does not provide sample recovery and as such the subsurface material identification (including material strength/density) is by interpretation of the test results using empirical correlations and correlation with the boreholes.

Further details of the techniques and procedures employed in the investigation are presented in the attached Report Explanation Notes, which also define the logging terms and symbols used.

Groundwater observations were made in the boreholes during and on completion of drilling. Groundwater observations were also made in the CPT holes, on completion of testing. No long term ground level monitoring was carried out.

Our geotechnical engineer (Bryan Zhang) was present full time during the fieldwork to set out the investigation locations, nominate sampling, and prepare the attached CPT results and borehole logs. The CPT results were interpreted by our Senior Associate Geotechnical Engineer.

Selected soil samples were returned to a NATA accredited laboratory (Envirolab Services Pty Ltd) for soil pH, sulfate, chloride and resistivity testing. The test results are summarised in the attached Envirolab Services 'Certificate of Analysis 287344'.

3 RESULTS OF THE INVESTIGATION

3.1 Site Description

The site is located in relatively flat topography, about 170m to the south-west of Brookvale Creek. The site itself had an overall slope of about 2° down to the east, though a concrete driveway running down the centre of the site (in an east-west orientation) at its western and eastern ends, sloped down to the east at up to about 8°. The central portion of the site was relatively flat. The site has an approximate 42m wide western frontage onto Old Pittwater Road.

At the time of the fieldwork, the site contained (apart from the concrete driveway in the central portion of the site), several storey brick, concrete tilt-up panel and metal clad warehouse buildings.

The neighbouring property to the north of the site (97-99 Old Pittwater Road) contained a three storey concrete building, which abutted the common boundary. It was not clear from our observations from Old Pittwater Road whether there was a basement below the building.

The neighbouring property to the south of the site (107 Old Pittwater Road) contained several single storey brick buildings. The eastern most building abutted the common boundary, with the other two buildings in the central and western portions of the site set back about 3m and 7m, respectively, from the common boundary. An on-grade concrete driveway ran along the common boundary for most of its length. A concrete block fence ran along the majority of the common boundary. Ground surface levels across the boundary were up to about 0.5m higher within the neighbouring property to the south across the western and central portions of the wall, but were up to about 0.4m lower across the eastern portion of the wall.

The above described neighbouring buildings appeared to be in generally good external condition based on a cursory inspection from within the site and Old Pittwater Road, but noting our observations into the neighbouring properties were limited by the existing buildings on the subject site.

It was not possible to observe the features across the eastern site boundary due to the existing buildings on site. However, reference to the survey plan and a recent Nearmap aerial image of the site, the area to the east appears to contains an on-grade car park and a single storey metal warehouse building.

The obtained Sydney Water Dial Before You Dig (DBYD) plan shows a 225mm diameter Cast Iron Cement Lined (CICL) sewer main running below the eastern site boundary. The invert of the maintenance hole located off the north-eastern corner of the subject site is shown to be at 1.3m. The sewer returns along the eastern half of the northern site boundary at a similar depth. The DBYD plan also shows a 375mm CICL and 750mm diameter Steel Cement Lined Internal Bitumen Lined (SCL IBL) water mains running below Old Pittwater Road, just beyond the western site boundary. The invert depths of the water mains are not shown, but are assumed to be relatively shallow.

3.2 Subsurface Conditions

The 1:100,000 geological map of Sydney (Geological Survey of NSW, Geological Series Sheet 9130) indicates the site to be underlain by Hawkesbury Sandstone, close to the contact with the overlying Quaternary alluvial sands, silts and clays, associated with Brookvale Creek. We note the boundary between the different stratigraphical units on the geological map is approximate only.

The investigation has disclosed a generalised subsurface profile comprising concrete pavements and fill covering a relatively deep interlayered alluvial sand and clay profile of variable strengths and densities over inferred sandstone bedrock. Reference should be made to the attached borehole logs and the CPT results for specific details of the subsurface conditions at each location. Some of the characteristic features of the subsurface conditions encountered in the boreholes, and indicated by the CPT results, is provided below.

Pavements

Concrete pavements with no observed reinforcement were encountered at each borehole/CPT location and were 180mm, 200mm and 240mm thick, respectively.

Fill

Sand fill with inclusions of igneous and sandstone gravel, fibre cement fragments, slag and ash was encountered below the pavements in each borehole to depths of 0.7m (BH1), 0.6m (BH2) and 0.8m (BH3).

Alluvial Soils

Alluvial soils comprising interlayered sands and clays were indicated/encountered below the fill in each borehole/CPT. The sands ranged from loose to dense, whilst the clays ranged from stiff to hard strength.

Sandstone Bedrock

The CPTs were inferred to have refused on inferred sandstone bedrock at depths of 12.67m (CPT1), 13.12m (CPT2) and 9.03m (CPT3), based upon a previous investigation at 113 Old Pittwater Road showing bedrock at depths in the order of 14-16m.

Based on the CPT refusal depths, the bedrock surface appears to be stepping up to the west, with a possible buried cliff line in the order of about 4.1m high between CPT2 and CPT3.

Groundwater

The boreholes were 'dry' during and on completion of drilling. However, on completion of the CPTs, groundwater was measured in the CPT holes at depths of 4.0m (CPT1), 3.9m (CPT2) and 4.0m (CPT3), relating to reduced levels of about 10.3m-10.6n AHD.

3.3 Laboratory Test Results

The soil aggression test results indicated slightly acidic to slightly alkaline (pH 5.8 to 8.4) conditions, low sulphate and chloride contents (maximum 90mg/kg) and relatively high resistivity values (10,000 ohm.cm to 18,000 ohm.cm).

4 COMMENTS AND RECOMMENDATIONS

4.1 Additional Geotechnical Investigation and Geotechnical Constraints

The comments and recommendations provided in this report are considered preliminary and based mostly on three CPTs positioned within the central portion of the site.

The natural soil profile comprises interlayered sands and clays, with bands of relatively low strength (stiff clay and loose sand), and is not considered a suitable founding material based on the anticipated relatively high structural loads. The proposed building should therefore be uniformly founded within the bedrock.

We strongly recommend that following demolition of the existing buildings, an additional geotechnical investigation comprising the drilling of at least six cored boreholes to assess the depth to, and quality of, the underlying sandstone bedrock be caried out. This will also allow for the optimisation of the bearing pressures.

This investigation has also shown that groundwater is present with the depth of the proposed basement. Therefore, unless the proposed basement FFL is raised significantly, dewatering will be required to construct the proposed basement in the 'dry' and the basement will need to be designed as a 'tanked' structure. This will require the construction of an impermeable shoring system, such as a diaphragm wall or secant pile wall, where the walls/piles are socketed into bedrock to form a 'cut off', as the sands may extend down to the bedrock surface, and it is unlikely that it can be shown the clays form a continuous layer which would cut-off water flows.

Where dewatering is required, it is almost certain in our recent experience that Council will refer the project to WaterNSW who will require a temporary dewatering licence in order to allow the dewatering to proceed. The WaterNSW temporary dewatering licence application will need to be accompanied by a Dewatering Management Plan, evidence of groundwater level monitoring for at least three months from at least three groundwater monitoring wells, as well as permeability testing, groundwater sampling and groundwater quality testing. It will also be necessary to undertake analysis of the seepage volumes into the excavation. The additional geotechnical investigation must address the WaterNSW requirements, which are outlined in the 'Minimum Requirements for Building Site Groundwater Investigation and Reporting' [NSW Department of Planning, Industry and Environment (DPIE), Ref. PUB20/940, January 2021].

Another issue for the construction of the proposed basement will be the lateral restraint of the shoring system. As there are neighbouring buildings abutting the northern and southern site boundaries, and noting excavation will extend to a depth of about 6.5m and the soils are poor, the shoring will need to be anchored or internally propped, rather than cantilevered. We note that the design and installation of temporary anchors will not be trivial due to the poor nature of the soils and presence of groundwater, and the builders and excavation constractors are usually resistant to the use of internal bracing.

4.2 Sydney Water

The supplied Sydney Water DBYD plan shows a sewer at the rear (eastern end) of the site, and two water mains below Old Pittwater Road, directly opposite the site.

Prior to any demolition and excavation, the structural drawings for the proposed development should be forwarded to Sydney Water for their review and approval.

In our recent experience, Sydney Water will almost certainly require a Specialist Engineering Assessment (SEA) of the potential impact the excavation and construction of the proposed basement will have on the sewer and water mains below Old Pittwater Road, opposite the site. The SEA is to be prepared by the structural engineer, or a water services co-ordinator (WSC), and should include finite element analysis (FEA) of the adjacent sewer, water mains and proposed development; we can assist with the FEA, if requested.

The SEA can take significant time for its preparation and approval by Sydney Water, and so the SEA should be completed at early stage.

4.3 Dilapidation Surveys

Prior to the commencement of any demolition and excavation, dilapidation surveys should be completed on the neighbouring buildings to the north (97 Old Pittwater Road), south (107 Old Pittwater Road) and east, including any boundary walls which are to be retained.

The dilapidation survey reports can be used as a benchmark against which to set vibration limits, and for assessing possible future claims for damage arising from the works.

The respective owners of the adjoining properties should be asked to confirm in writing that the dilapidation survey report on their property presents a fair assessment of the existing conditions. As dilapidation survey reports are relied upon for the assessment of potential future damage claims, they must be carried out thoroughly with all defects rigorously described (ie. defect type, defect location, crack width, crack length etc) and defects photographed where practical.

4.4 Shoring Design

We strongly recommend early in the design process, that 'as constructed' drawings be obtained for the neighbouring building to the north and south, so that details of the structures, including any basement levels be confirmed, so the shoring can be designed appropriately.

4.4.1 Shoring System

Due to the presence of collapsible sand and groundwater, we recommend a diaphragm wall or secant pile wall comprising continuous flight auger (CFA) piles or double rotary/cased CFA piles be installed to support the sides of the basement excavation. Options such as cutter soil mix walls could also be considered, however, the use of sheet piles should be discounted as they would be unlikely to penetrate the bedrock surface, and vibrations from their installation would have a high likelihood of damaging the neighbouring buildings and pavements to the north and south.

The walls/piles of the shoring system must be founded below bulk excavation level and into sandstone bedrock to form a 'cut off' for groundwater inflows into the excavations. It will be necessary for the shoring system to be either anchored or internally propped to reduce wall deflections as the excavation proceeds. Careful control of pile verticality and the construction sequence will be required to reduce seepage through the walls and potential wall movements.

Due to the presence of sand and groundwater, the installation of the shoring system may cause ground surface movements, due to potential vibrations associated with pile drilling. There is also a potential for soil mining during pile drilling causing subsidence of the ground around the piles. Care must therefore be taken by the piling contractor and builder during the piling works by monitoring the ground surface around the piles with regular checks by the piling supervisor and builder. The volume of soil must be monitored, and if the volume of spoil is excessive compared with the volume of the pile, it is likely that soil mining is occurring. If there are any signs of ground surface movement/subsidence and/or excessive spoil removal, then the piling operations must immediately cease and further geotechnical advice sought. Such issues could be largely overcome by using double rotary/cased CFA piles and reducing the socket length of the piles. Further advice would be provided on this issue following the additional investigation.

4.4.2 Shoring Design Parameters

The major consideration in the selection of earth pressures for the design of the shoring system is the need to limit deformations occurring outside the excavation.

For anchored or internally propped walls, a uniform rectangular earth pressure distribution and a lateral earth pressure of 8H (kPa) should be adopted for the soil profile, where H is the retained height in metres. The retained height must be measured from the base of any nearby detailed excavations (eg. footings, service trenches, etc).

Any surcharge affecting the walls (eg. nearby footings, pavement loads, inclined retained surfaces, etc.) should be allowed for in the design using an 'at rest' earth pressure coefficient, K_0, of 0.55.

A bulk unit weight of 20kN/m^3 should be adopted for the soil profile above groundwater and 10kN/m^3 for below groundwater.

Hydrostatic pressures must be considered in the wall designs and these are in addition to the earth pressure recommendations above. The shoring system must be designed to withstand hydrostatic lateral and uplift pressures with a design head of water equivalent to 2m above the current groundwater levels, or to the design flood level, if the site is affected by flooding. There will also be differential water pressures between the inside and outside of the shoring dewatering and these must also be considered.

Lateral toe restraint may be achieved by the resistance of the ground in front of the walls. For embedment depth design, a triangular lateral earth pressure distribution should be assumed with a 'passive' lateral earth pressure coefficient, K_P, of 2.7. We note that significant movement is required in order to mobilise the full passive pressure of the soil and so a factor of safety of 2 should be adopted in order to reduce such movement. All localised excavations in front of the wall, such as for buried services, footings, etc, must be taken into account in the embedment depth design. The upper 0.5m depth of the embedment below bulk excavation level should not be taken into account to allow for tolerance effects and possible disturbance during excavation.

To act as a 'cut off' for groundwater, we recommend the shoring system be embedded about 0.3m into the bedrock to reduce the potential for soil mining, otherwise double rotary/cased CFA piles would be required for deeper sockets, or multi-level restraint.

Where temporary anchors extend beyond the site boundaries, then permission must be sought from the respective neighbouring property owner, prior to installation. Our experience has shown that this process can take time and therefore should be completed early in the construction process. Soil anchors bonded into the soil profile may be designed for an effective angle of internal friction, ϕ', of 28°. All anchors must be proof-loaded to at least 1.3 times the design working load before being locked off at about 85% of the working load, all under the direction of a geotechnical engineer independent of the anchoring contractor. It may be preferable to pressure grout the bond length of the anchors to improve their load capacity. The construction of anchors in such conditions is specialised, and so we recommend that only experienced 'top tier' contractors be considered for the anchor installation, as excess sand can be removed during drilling which weakens the anchor bond, as well as potentially causing settlement below adjoining properties. Such contactors must be engaged early in the planning process to confirm whether these anchors are even feasible in these variable and sometimes poor conditions.

As an alternative to installing temporary anchors, the shoring walls may be internally propped with props that can be hydraulically stressed to limit deflections, and we consider this would be a better alternative in this instance.

We have assumed that permanent lateral support of the shoring will be provided by bracing from the proposed structure, after which time the anchors can be de-stressed or props removed.

4.4.3 Underpinning

The proposed basement will require excavation to a maximum depth of about 6.5m through variable soils including loose sand and stiff clay, and will extend below groundwater. The neighbouring buildings to the north, south and east abut the common boundaries and are within the 'zone of influence' of the excavation. The 'zone of influence' for the subsurface conditions encountered is considered to be about twice the excavation depth away from the excavation. Therefore, the neighbouring buildings to the north, south and east are likely to be susceptible to damage, if ground subsidence occurs due to deflection of the shoring system.

The proposed shoring system will undergo some deflection due to the proposed basement excavation, even if the system is designed to be 'stiff' and comprises a well constructed anchored or propped wall as recommended in this report. The wall deflections could manifest as settlement of the ground directly behind the retaining wall. The structural engineer will need to assess the magnitude of those settlements which will be a function of the wall deflection, and whether such settlements can be safely accommodated by the neighbouring structures. If the settlements cannot be safely accommodated, then underpinning of the neighbouring structures and pavements would be warranted. FEA of the shoring system may assist the structural engineer in this decision, and we can assist by completing the FEA if requested to do so.

4.5 Dewatering

Unless the proposed basement FFL is raised significantly such that it will be well above groundwater, in order to maintain a 'dry' excavation during construction, internal dewatering will be required. We expect the dewatering could be carried out using a spear point system or well system installed inside the shoring walls, though the efficiency will be impacted by the layered nature of the soils.

Provided the shoring piles are sufficiently embedded into the underlying bedrock below bulk excavation level, we do not expect any notable drawdown of groundwater to occur outside the excavation. This should be confirmed, or otherwise, by the installation of at least three groundwater monitoring standpipes just outside the footprint of the proposed basement excavation, such as along the western side of the proposed basement and within the adjoining on-grade car park to the east, which will require permission from the respective property owner. The groundwater levels should be monitored daily by the builder during dewatering to confirm that groundwater levels are within about 0.3m of the groundwater level measured prior to the commencement of dewatering. If, however, groundwater levels during dewatering are found to have dropped by more than about 0.3m from the pre-dewatering levels, then it may be necessary to reinject groundwater outside the excavation to maintain the groundwater level, so that the groundwater is not drawdown to a level that may cause settlement of the ground surface which is especially important noting the neighbouring structures along the northern and southern site boundaries.

We recommend the proposed dewatering methodology be reviewed by the geotechnical engineer, prior to implementation to confirm its suitability.

If any groundwater is to be discharged to the stormwater system for disposal, then approval will be required from Council.

4.6 Footing Design

On completion of excavation, a variable soil profile will be exposed at bulk excavation level. Much of the soils are of relatively low strength (loose sands and stiff clays) and are variable and interlayered, and as such are not a suitable founding material for the expected relatively high column loads. Therefore, we recommend that the proposed structures be uniformly founded within the bedrock.

As a guide and subject to confirmation by the additional geotechnical investigation, double rotary/cased CFA piles or CFA piles socketed at least 0.3m into sandstone bedrock of at least low strength may be designed for an allowable end bearing pressure of 1,000kPa. Rock sockets formed below the nominal 0.3m length requirement may be tentatively designed for maximum allowable shaft adhesion values of 100kPa (in compression) and 50kPa (in tension). We note that higher bearing pressures, in the order of 2,000kPa to 3,500kPa, are likely to be feasible following completion of the additional geotechnical investigation. If socketing into the bedrock by more than 0.3m, then we recommend double rotary/cased CFA piles, as we would expect that there will be some load on the perimeter shoring system.

If piles are required to assist with resisting the uplift pressures from the groundwater, then the mass of the concrete within the piles could also be taken into consideration.

We do not recommend using steel screw piles, as the bedrock may be of at least low strength from first contact, as even the small diameter cone used for this investigation could not get any penetration. If that is the case, then during installation of screw piles on this site, the plain steel tip of the screw pile is expected to refuse on the bedrock surface, with the load bearing helix 'hung up' in the soil profile. If the screw pile is then rotated without penetration to attempt to gain embedment, the soil under the helix would be disturbed such that it would have no contribution to the load capacity. In that case the entire load would be transferred through the centre shaft of the pile, resulting in extremely high and unacceptable pressures on the rock.

We recommend the initial stages of pile drilling be witnessed by a geotechnical engineer to check whether it is reasonable to assume the piles have been founded in the bedrock.

The piling contractor must be required to certify the load capacity of the piles.

A Hazard Factor (Z) of 0.08 and a Site Subsoil Class C_e should be adopted for earthquake design in accordance with AS1170.4-2007 ('Structural Design Actions, Part 4: Earthquake Actions in Australia', including Amendment Nos 1 & 2).

4.7 Piling Rig Working Platform

The piling contractor may require a working platform to be constructed, prior to commencement of the piling works. The design of such a platform depends on the loading from the piling rig and material used for the platform. We can complete the design of such a platform when such information is available, if requested to do so.

4.8 Soil Aggression

Based on the soil aggression test results, concrete and steel elements in contact with the soil and groundwater should be designed for a 'Non-aggressive' exposure classification, in accordance with AS2159-2009 'Piling-Design and Installation'.

4.9 Basement Floor Slab

The basement floor slab will need to be designed as a 'tanked' structure to resist hydrostatic uplift pressures, with a design head of water equivalent to 2m above the current groundwater levels, or to the design flood level if the site is affected by flooding. As it will not be possible to tolerate any differential movement between the slab and the structure, as this would allow leakage, the slab should be suspended from piles founded in the underlying bedrock.

Care must be taken with the detailing and construction of waterproofing at the interface between the floor slab and basement walls, as well as any penetrations through the floor slab.

4.10 Further Geotechnical Input

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

1. Additional geotechnical investigation including at least six cored boreholes to confirm the depth to, and quality of, the underlying sandstone bedrock.
2. Unless the basement level is raised significantly such that dewatering is not required, groundwater level monitoring, permeability testing, seepage analysis and preparation of a Dewatering Management Plan, as required by WaterNSW.
3. FEA to address expected Sydney Water requirements.
4. FEA of the shoring system which may assist the structural engineer in determining whether underpinning of adjacent structures is required.
5. Working platform design, if requested.
6. Witnessing installation and proof testing of anchors.
7. Witnessing of pile installations.

5 GENERAL COMMENTS

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

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

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

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

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

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CERTIFICATE OF ANALYSIS 287344

Client Details

Client	JK Geotechnics
Attention	Bryan Zheng
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**34695PH, 99-105 Old Pittwater Rd, Brookvale, NSW**
Number of Samples	3 Soil
Date samples received	25/01/2022
Date completed instructions received	25/01/2022

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	02/02/2022
Date of Issue	28/01/2022

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Results Approved By
Nick Sarlamis, Assistant Operation Manager

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil				
Our Reference		287344-1	287344-2	287344-3
Your Reference	UNITS	1	2	3
Depth		0.2-0.7	1.3-1.5	0.4-0.6
Date Sampled		22/01/2022	22/01/2022	22/01/2022
Type of sample		Soil	Soil	Soil
Date prepared	-	27/01/2022	27/01/2022	27/01/2022
Date analysed	-	27/01/2022	27/01/2022	27/01/2022
pH 1:5 soil:water	pH Units	8.4	5.8	6.7
Chloride, Cl 1:5 soil:water	mg/kg	<10	<10	<10
Sulphate, SO4 1:5 soil:water	mg/kg	79	90	75
Resistivity in soil*	ohm m	100	180	160

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	-			27/01/2022	[NT]	[NT]	[NT]	[NT]	27/01/2022	[NT]
Date analysed	-			27/01/2022	[NT]	[NT]	[NT]	[NT]	27/01/2022	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	101	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	93	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	94	[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. **1**

1/1

Client: HANNAS CONTRACTING SERVICES PTY LTD

Project: PROPOSED INDUSTRIAL DEVELOPMENT

Location: 10/105 OLD PITTWATER ROAD, BROOKVALE, NSW

Job No.: 34695PH **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 14.3m

Date: 22/1/22 **Datum:** AHD

Plant Type: EZIPROBE **Logged/Checked by:** B.Z./A.J.H.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION						0			CONCRETE: 180mm.t				NO OBSERVED REINFORCEMENT
						0.5		-	FILL: Silty sand, fine to medium grained, dark brown and grey, with fine to medium grained igneous gravel, trace of fibre cement fragments, slag and ash.	M			
								SM	Silty SAND: fine to medium grained, light grey.	M			ALLUVIAL
						1 1.5		SP	SAND: fine to medium grained, brown, with clay and silt, trace of cemented nodules.				
						2 2.5 3 3.5			END OF BOREHOLE AT 2.0m				

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BOREHOLE LOG

Borehole No. **2**

Client: HANNAS CONTRACTING SERVICES PTY LTD

Project: PROPOSED INDUSTRIAL DEVELOPMENT

Location: 10/105 OLD PITTWATER ROAD, BROOKVALE, NSW

Job No.: 34695PH **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 14.5m

Date: 22/1/22 **Datum:** AHD

Plant Type: EZIPROBE **Logged/Checked by:** B.Z./A.J.H.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLET-ION						0			CONCRETE: 200mm.t				NO OBSERVED REINFORCEMENT
								-	FILL: Silty sand, fine to medium grained, dark brown, trace of fine to medium grained igneous gravel, slag and ash.	M			
						0.5		SP	SAND: fine to medium grained, with clay and silt, trace of fine to medium grained cemented nodules.	M			ALLUVIAL
						1							
						1.5							
						2							
						2.5							
						3			END OF BOREHOLE AT 3.0m				
						3.5							

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BOREHOLE LOG

Borehole No. 3

1/1

Client: HANNAS CONTRACTING SERVICES PTY LTD
Project: PROPOSED INDUSTRIAL DEVELOPMENT
Location: 10/105 OLD PITTWATER ROAD, BROOKVALE, NSW

Job No.: 34695PH **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 14.5m
Date: 22/1/22 **Datum:** AHD
Plant Type: EZIPROBE **Logged/Checked by:** B.Z./A.J.H.

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLET-ION						0			CONCRETE: 240mm.t				NO OBSERVED REINFORCEMENT
								-	FILL: Gravelly sand, fine to medium grained, dark brown, fine to medium grained angular igneous and sandstone gravel, with ash and slag.	M			
						0.5			FILL: Sand, fine to medium grained, dark brown, trace of fine to medium grained sandstone gravel, and ash.				
						1		SP	SAND: fine to medium grained, brown.	M			ALLUVIAL
						1.5							
						2			END OF BOREHOLE AT 2.0m				
						2.5							
						3							
						3.5							

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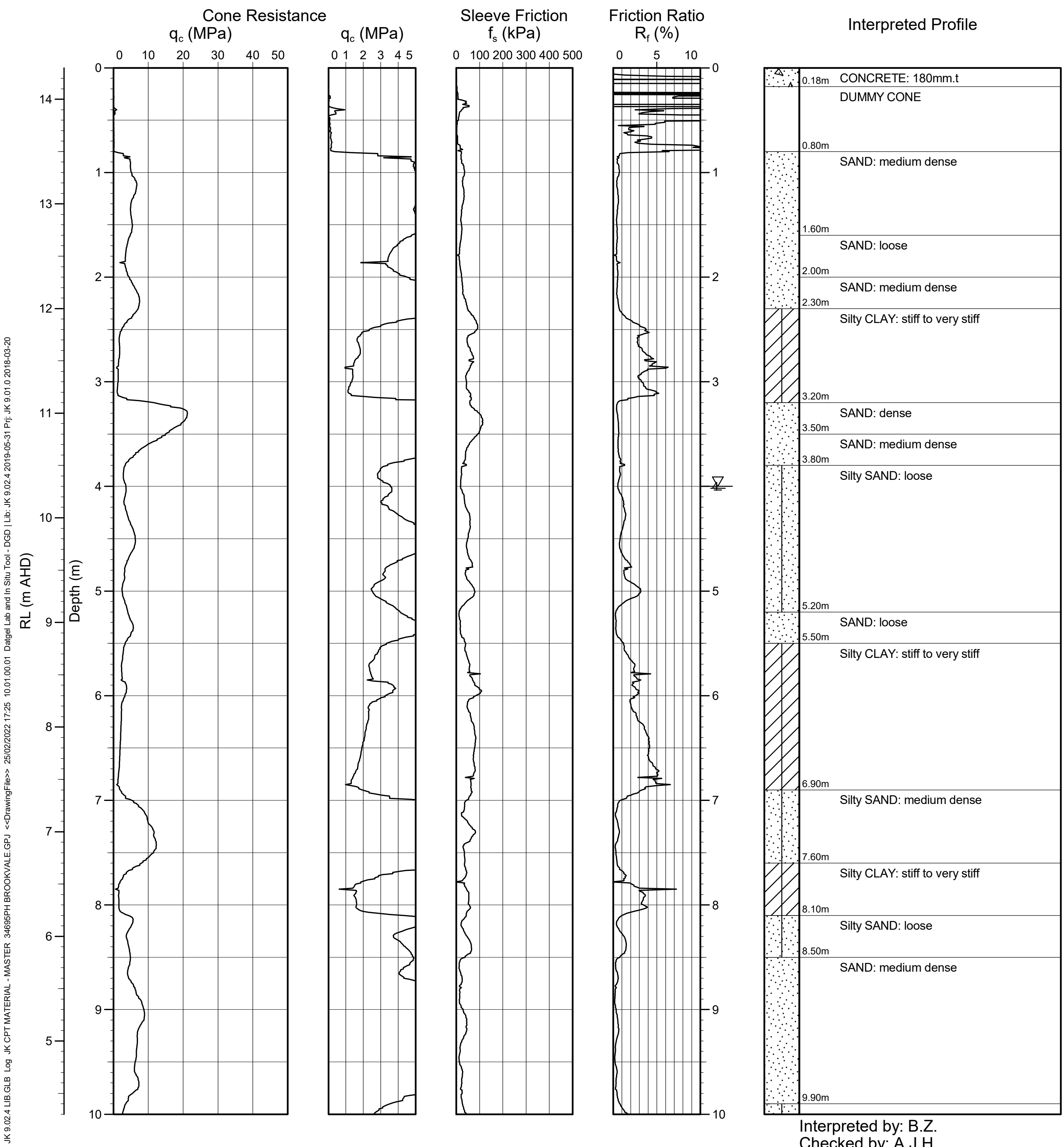
CONE PENETROMETER TEST RESULTS

CPT No.
1
1 / 2

Client: HANNAS CONTRACTING SERVICES PTY LTD
Project: PROPOSED INDUSTRIAL DEVELOPMENT
Location: 101-105 OLD PITWATER ROAD, BROOKVALE, NSW

Job No.: 34695PH	**R.L. Surface:** ~14.3 m	**Data File:** 34695PH Brookvale
Date: 22/1/22	**Datum:** AHD	**Operator:** B.J.



Interpreted by: B.Z.
Checked by: A.J.H.

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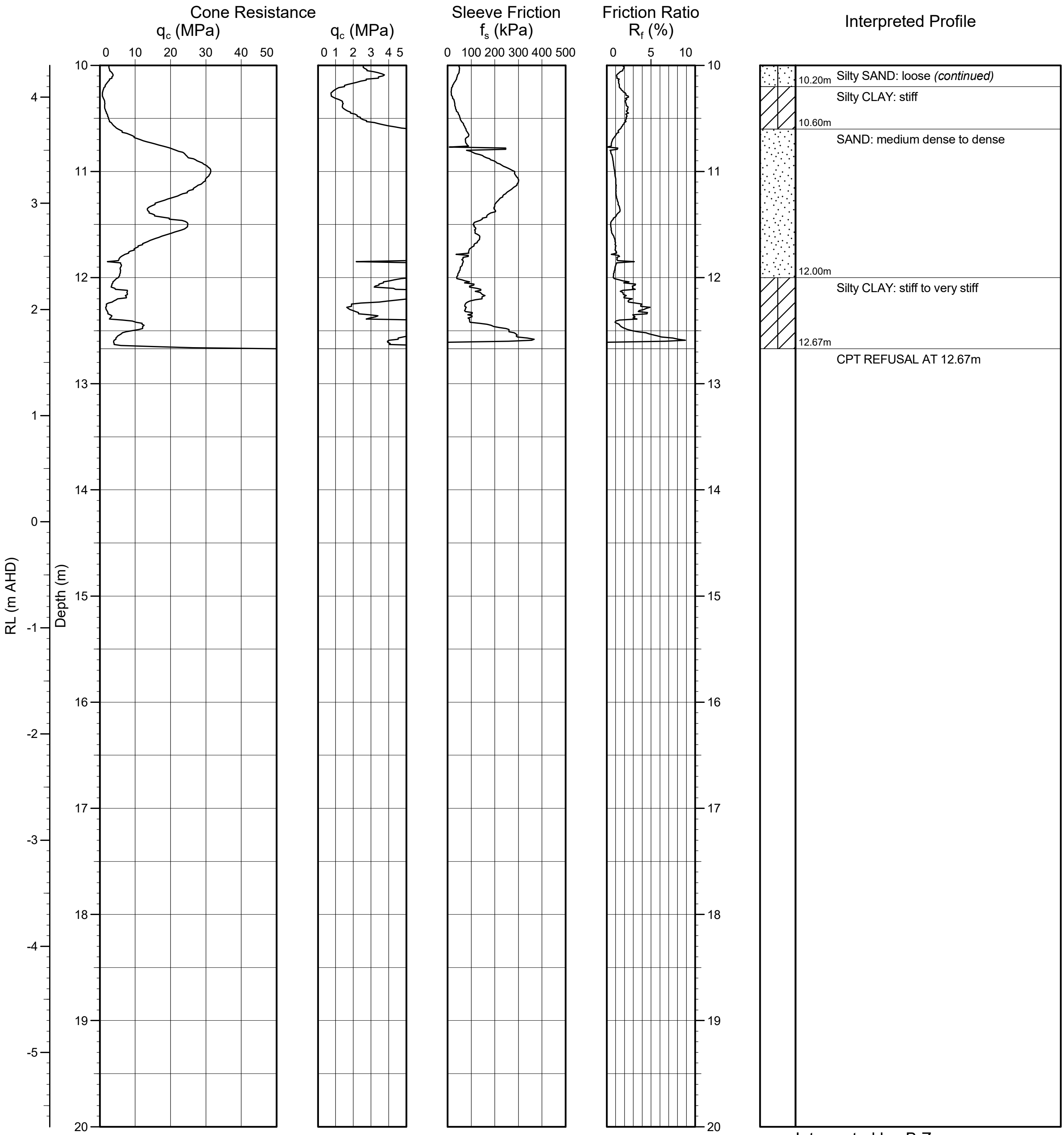
CONE PENETROMETER TEST RESULTS

Client: HANNAS CONTRACTING SERVICES PTY LTD
Project: PROPOSED INDUSTRIAL DEVELOPMENT
Location: 101-105 OLD PITWATER ROAD, BROOKVALE, NSW

Job No.: 34695PH	**R.L. Surface:** ~14.3 m	**Data File:** 34695PH Brookvale
Date: 22/1/22	**Datum:** AHD	**Operator:** B.J.



Interpreted by: B.Z.
Checked by: A.J.H.

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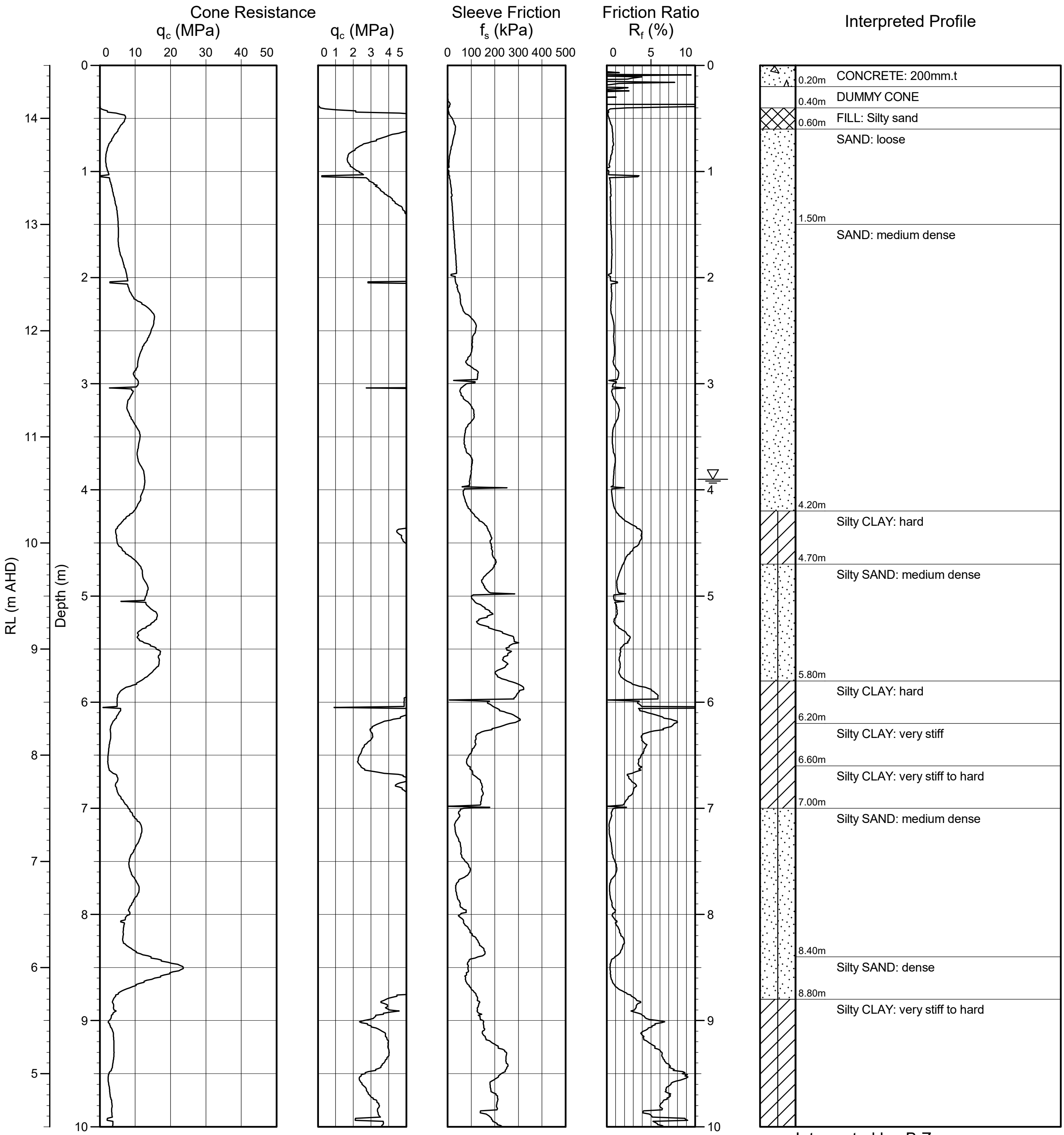
CONE PENETROMETER TEST RESULTS

CPT No.
2
1 / 2

Client: HANNAS CONTRACTING SERVICES PTY LTD
Project: PROPOSED INDUSTRIAL DEVELOPMENT
Location: 101-105 OLD PITWATER ROAD, BROOKVALE, NSW

Job No.: 34695PH | **R.L. Surface:** ~14.5 m | **Data File:** 34695PH Brookvale
Date: 22/1/22 | **Datum:** AHD | **Operator:** B.J.



Interpreted by: B.Z.
Checked by: A.J.H.

JK 9.02.4 LIB.GLB Log JK CPT MATERIAL - MASTER 34695PH BROOKVALE.GPJ <<DrawingFile>> 25/02/2022 17:25 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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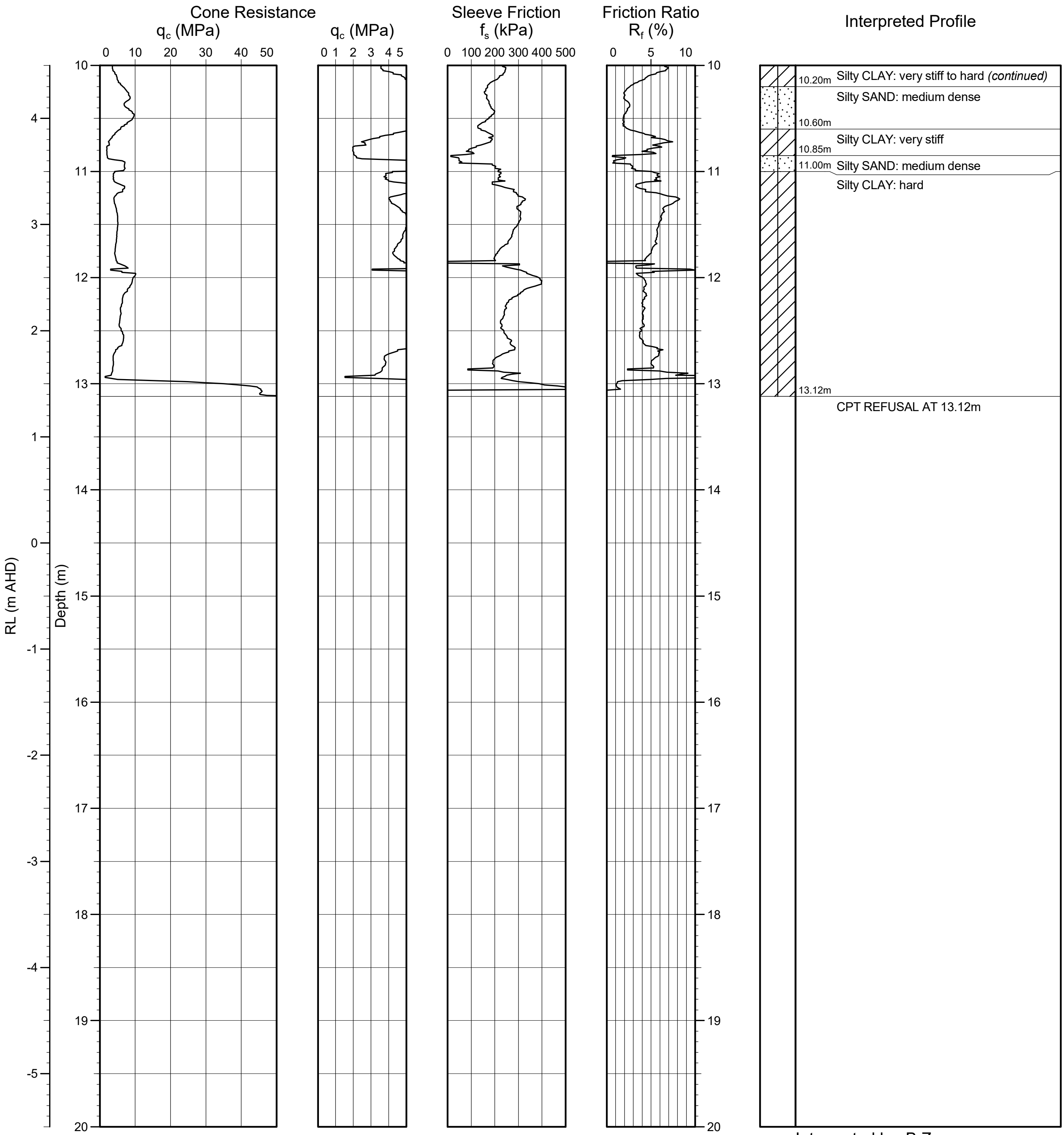
CONE PENETROMETER TEST RESULTS

CPT No.
2
2 / 2

Client:	HANNAS CONTRACTING SERVICES PTY LTD
Project:	PROPOSED INDUSTRIAL DEVELOPMENT
Location:	101-105 OLD PITWATER ROAD, BROOKVALE, NSW

Job No.: 34695PH	**R.L. Surface:** ~14.5 m	**Data File:** 34695PH Brookvale
Date: 22/1/22	**Datum:** AHD	**Operator:** B.J.



Interpreted by: B.Z.
Checked by: A.J.H.

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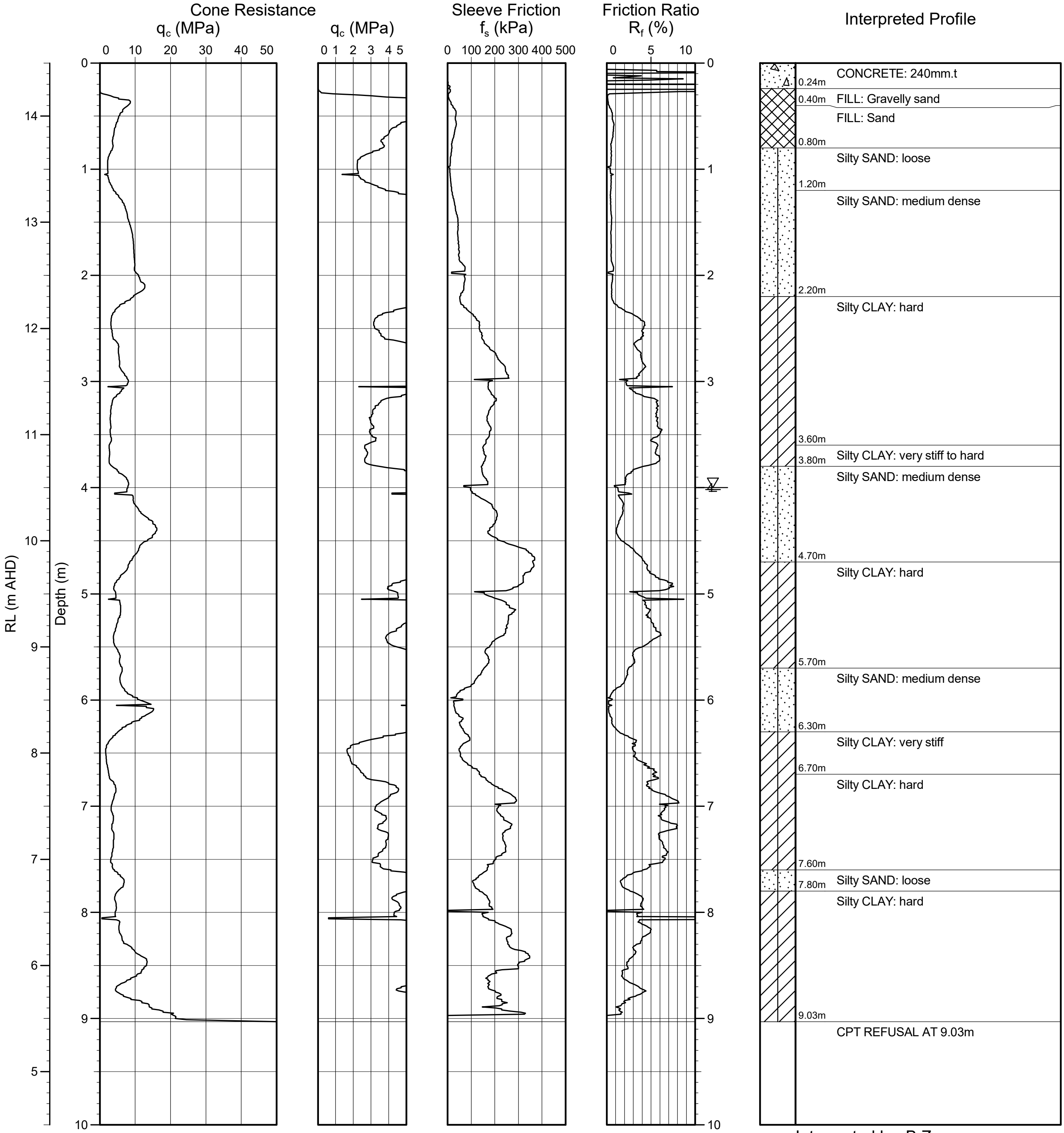
CONE PENETROMETER TEST RESULTS

CPT No. 3

Client:	HANNAS CONTRACTING SERVICES PTY LTD				
Project:	PROPOSED INDUSTRIAL DEVELOPMENT				
Location:	101-105 OLD PITWATER ROAD, BROOKVALE, NSW				
Job No.:	34695PH	**R.L. Surface:**	~14.5 m	**Data File:**	34695PH Brookvale
Date:	22/1/22	**Datum:**	AHD	**Operator:**	B.J.



Interpreted by: B.Z.
Checked by: A.J.H.

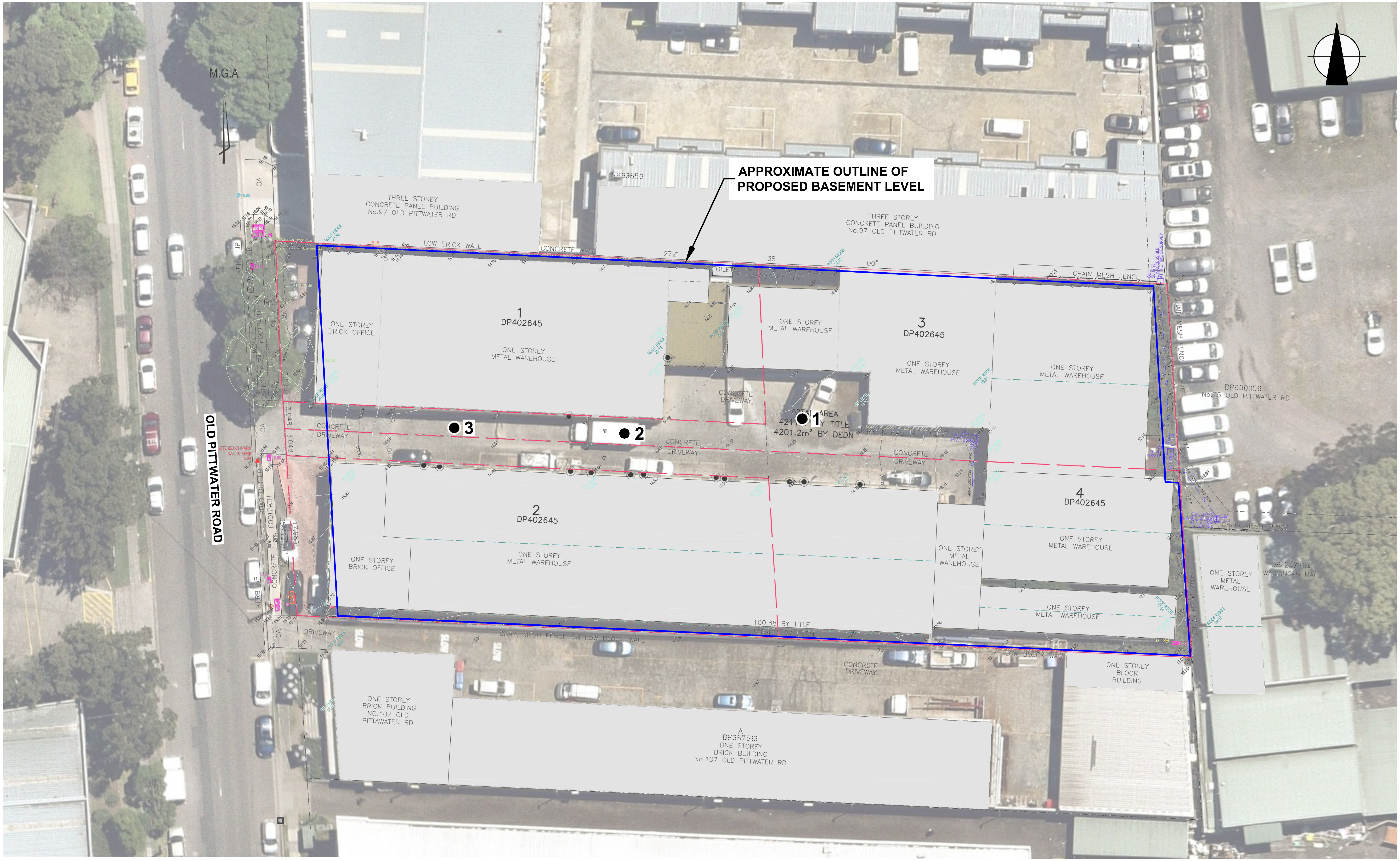
AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:	**SITE LOCATION PLAN**
Location:	101-105 OLD PITTWATER ROAD, BROOKVALE, NSW
Report No: 34695PH	Figure No: 1

JKGeotechnics

PLOT DATE: 24/02/2022 2:57:00 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\34000'S\34695PH BROOKVALE\CAD\34695PH.DWG



AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

0 4 8 12 16 20

SCALE 1:400 @A3 METRES

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

Title: **INVESTIGATION LOCATION PLAN**

Location: 101-105 OLD PITTWATER ROAD, BROOKVALE, NSW

Report No: 34695PH

Figure No: 2

JKGeotechnics

PLOT DATE: 24/02/2022 2:57:15 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\34000'S\34695PH BROOKVALE\CAD\34695PH.DWG

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:

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

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

NOTES:

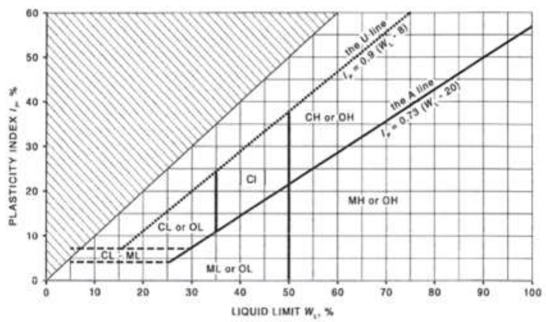
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

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

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

Classification of Material Weathering

<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)	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