SPECIALIST ADVICE TO

GOOD TIME HOLDINGS NSW

ON

PRELIMINARY GEOTECHNICAL INVESTIGATION

FOR

PROPOSED MIXED USE DEVELOPMENT

AT

277 THE GRAND PARADE, RAMSGATE, NSW

Date: 4 January 2024
Ref: 34871PHrpt Rev2

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1 INTRODUCTION

This report presents the results of a preliminary geotechnical investigation for the proposed mixed-use development at 277 The Grand Parade, Ramsgate, NSW. The location of the site is shown in Figure 1. The investigation was commissioned by Mr Richard Cridland of Bronxx Pty Ltd, on behalf of Good Time Holdings NSW, by return of a signed 'Acceptance of Proposal' form, dated 8 March 2022. The commission was on the basis of our proposal, Ref. P55661PH Rev1, dated 1 March 2022.

From the provided Development Application architectural drawings by Craft Architecture (Project No. 23.09, dated December 2023), we understand that following demolition of the existing Coles supermarket building, construction of a six storey building underlain by three basement levels is proposed. The lowest basement level (Basement 3) will be constructed at RL-6.4m requiring excavation to a depth of about 9.8m plus the slab thickness and will extend to the site boundaries, as shown on the attached Figure 2. Six lifts are also proposed.

We have not been provided with any structural loads, but expect they will be relatively high.

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

The 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: E34871PTrpt Rev1 dated 9 January 2024, for the results of the PSI.

This report provides specialist advice for use by the structural designer in preparing their design and no part of this report is intended to form a regulated design in accordance with the Design and Building Practitioners Act 2020.

2 INVESTIGATION PROCEDURE

Access for the preliminary investigation was limited to the western portion of the site which comprised an on-grade car park. To allow vehicle movements into and out of the car park during the fieldwork, the drilling and testing was limited to the central and southern portions of the car park.

The fieldwork for the preliminary investigation was carried out on 2 & 3 May 2022 and comprised the auger drilling of seven boreholes (BH1 to BH7) to depths ranging from 1.2m (BH3) to 7.0m (BH1 and BH5) below the existing car park surface levels using our truck mounted JK400 drilling rig. The boreholes were primarily drilled for the purpose of recovering soil samples for laboratory testing and installing groundwater monitoring wells. In addition, three Cone Penetrometer Tests (CPT) were completed to refusal depths of 15.40m (CPT8), 20.28m (CPT9) and 19.95m (CPT10) below the existing car park surface levels using our truck mounted CPT rig. Due to the presence of fill, the upper approximate 2.0m of each CPT was probed using a 'dummy' cone to avoid damage to the sensitive instrumental cone.

The investigation locations were set out using a tape measure from existing surface features and are shown on Figure 2. The surface levels at each investigation location were estimated by interpolation between spot levels and surface contour lines shown on the provided survey plan of the site prepared by Citisurv Pty Ltd (Plot File 12486-DET, dated 25 January 2022), and are therefore only approximate. The survey datum is Australian Height Datum (AHD). The provided survey plan forms the basis of Figure 2 which has also been laid over a recent Nearmap aerial image.

The concrete pavement at BH5 was cored with a diamond tipped thin walled tube with water flush. The relative compaction of the fill in BH1 and BH5 and relative density and strength of the natural soils were assessed from the Standard Penetration Test (SPT) results and interpretation of the CPT results.

CPT testing involves continuously pushing a probe with a conical tip into the soil using the hydraulic rams of the CPT rig. Measurements of the end resistance of the conical tip and the frictional resistance of a separate 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 and with reference to the pore pressure measurements. Groundwater monitoring wells were installed into BH1 and BH5 and comprised 50mm diameter Class 18 PVC standpipes. The annulus between the borehole and the slotted length was backfilled with 2mm filter sand. Above the sand backfill, the borehole was sealed with bentonite. A cast-iron 'Gatic' cover was concreted flush with the ground surface to protect the top of each groundwater monitoring well. The installation details are presented on the relevant attached borehole log. A return visit to measure the groundwater levels in the wells was carried out on 22 December 2023.

Our geotechnical engineers were present during the fieldwork to set out the investigation locations, nominate the sampling, and prepare the attached CPT results and borehole logs. The CPT results were interpreted by a 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 294618.

3 RESULTS OF THE INVESTIGATION

3.1 Site Description

The site is located in a flat topography, approximately 75m to the west of Botany Bay. The site was also relatively flat. The Grand Parade bounds the site to the east. An on-grade car park was located along the northern side of the site with Ramsgate Road located further to the north, about 30m from the site.

At the time of the fieldwork, as well as during our return visit on 22 December 2023, the eastern portion of the site contained a single storey brick building occupied by a Coles supermarket, which appeared to be in generally good condition based on a cursory inspection. Prior to construction of the supermarket building, we understand Ramsgate Baths which contained several swimming pools occupied the site up until 1972. With reference to a 1970 aerial image of the site provided in the Lotsearch report obtained by JKE, the former footprint of the pools appear to be within the southern portion of the site; the inferred approximate location of the former pools is shown on Figure 2. The western portion of the site contained an on-grade car park which was surfaced mostly by asphaltic concrete (AC) but with some concrete. Narrow garden beds with small shrubs and trees were located along the northern, western and southern sides of the car park.

The neighbouring property to the west (213 Ramsgate Road) contained a one and two storey rendered commercial building which abutted the common boundary. It appeared from our observations within the subject site and the adjoining car park to the north of the site, that this neighbouring building was not underlain by a basement.

The neighbouring property to the south (86-88 Alfred Street) contained several one and two storey brick townhouses set back at least 1m from the common boundary. Ground surface levels across the common boundary were obscured by a high boundary fence, however, noting the flat topography, it is likely ground surface levels across the common boundary are similar. It appeared from our observations within the subject site, Alfred Street and The Grand Parade, that these neighbouring townhouses to the south were not underlain by a basement.

The Sydney Water Dial Before You Dig (DBYD) plan shows a 225mm diameter Cast Iron Cement Lined (CICL) sewer main and 100mm diameter CICL water main running parallel to the northern site boundary. The approximate location of these mains based on the DBYD plan are shown on Figure 2. The sewer and water mains were off set about 6m and 8m, respectively, from the northern site boundary. The invert of the sewer maintenance hole located adjacent to the site (which is also shown on Figure 2) is indicated to be at 1.5m depth. The water main returns beyond the eastern site boundary, with an off set about 3m to 4m from the eastern site boundary. The invert depth of the water main is not shown, but is assumed to be relatively shallow.

3.2 Subsurface Conditions

The 1:100,000 geological map of Sydney indicates the site to be underlain by Quaternary marine sand.

The investigation disclosed a generalised subsurface profile comprising pavements and fill covering a deep marine sand profile 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 inferred by the CPT results, is provided below.

Pavements

A 30mm thick AC surfacing was encountered in BH1, BH2, BH3, BH4, BH6, BH7, CPT8 and CPT10. A 160mm thick reinforced concrete pavement was encountered in BH5 and CPT9. A thin granular roadbase layer was encountered below the AC surfacing and concrete pavement.

Fill

Sand fill with inclusions of igneous gravel and concrete and terracotta fragments was encountered below the pavements in each borehole to depths ranging from 0.3m (BH7) to 1.5m, (BH5). In the deeper boreholes, BH1 and BH5, the fill was assessed to be moderately and well compacted.

Deeper fill may be locally present where the deep ends of the former swimming pools were located.

Marine Sands

Marine sand was inferred/encountered below the fill in each borehole/CPT. The sands were mostly medium dense to dense to about 6m depth, then very loose to loose to about 10m depth then back to medium dense with dense to very dense conditions below about 11.5m to 12m depth. In CPT8, between about 7m and 9m depth, and in CPT9 and CPT10 between about 7.5m and 8.9m depth, clay bands of stiff strength were inferred, as well as sand of very loose to loose relative density.

Sandstone Bedrock

The CPTs refused on inferred sandstone bedrock at depths of 15.40m (CPT8), 20.28m (CPT9) and 19.95m (CPT10). This appears reasonable based upon previous investigations at 158-162 Ramsgate Road and 154-156 Ramsgate Road (which are opposite the site to the north) where sandstone bedrock was encountered at depths ranging from about 15m to 16.5m.

Groundwater

BH3, which was drilled to 1.2m depth, was 'dry' during and on completion of drilling. Groundwater seepage was observed during drilling in the other boreholes at depths ranging from about 1.6m to 2.0m below the existing surface levels. On completion of drilling, and up to two days following the completion of drilling, groundwater was measured in the boreholes and monitoring wells at depths ranging from 1.6m (BH4) to 1.98m (BH2). During the fieldwork, the pore pressure measurements showed groundwater to be present at a depth of about 2m. On 22 December 2023 at about 8am, groundwater was measured in the monitoring wells installed into BH1 and BH5 at depths of 2.25m and 2.42m, respectively. No other long term groundwater level monitoring has been undertaken.

3.3 Laboratory Test Results

The soil aggression test results indicated alkaline (pH 9.3 to 9.6) conditions, low sulphate and chloride contents (maximum 60mg/kg) and high resistivity values (8,400 ohm.cm to 23,000 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 on subsurface information obtained from within the western portion of the site only. We strongly recommend that following demolition of the Coles supermarket building, an additional geotechnical investigation comprising the completion of at least an additional seven CPTs and several cored boreholes to further assess the subsurface conditions and rock quality (including packer tests) with regards to groundwater cut off. The cored boreholes should be drilled at least 10m into the bedrock to assess the rock quality for the design of permanent rock anchors to tie down the building.

Following completion of the additional investigation, the recommendations in this report must be reviewed and updated as appropriate.

The investigation has shown that groundwater is present within the depth of the proposed basement excavation and we expect that groundwater levels will rise above the recorded levels during and following heavy and prolonged rainfall events and possibly in response to high tide levels given the proximity of the site to Botany Bay. Therefore, 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, where the walls are socketed into bedrock to form a ‘cut off’, as the sands extend down to the bedrock surface, and a continuous clay layer (which would cut-off water flows) below the basement has not been proven. Subject to seepage testing and modelling, it may be possible to embed the walls above bedrock, but at sufficient depth, if the seepage inflows can be managed.

Another issue for construction of the proposed basement will be the lateral restraint of the shoring system. As the excavation will extend to a depth of about 9.8m and up to the site boundaries, the shoring will need to be anchored or internally propped, rather than cantilevered. We note that the design and installation of the shoring will not be trivial, as temporary anchors will be difficult to construct within the saturated sands, and builders and excavation contractors are usually resistant to the use of internal bracing or top down construction.

4.2 WaterNSW

Construction of a basement that intersects the groundwater table is considered to be an aquifer interference activity. Such activities are subject to the Water Management Act 2000 and NSW Aquifer Interference Policy and are regulated by the Department of Planning, Housing and Infrastructure (DPHI) [formerly the Department of Planning, and Environment (DPE)], WaterNSW and Natural Resource Access Regulator (NRAR). DPHI's policy on basements is that ongoing or frequent dewatering of basements over their life is inconsistent with the principles of sustainable development and, where such dewatering is required, basements should be tanked. Dewatering during construction is permitted but is regulated through licencing which must either be obtained from WaterNSW (or NRAR for SSD developments).

The DPE document, 'Minimum Requirements for Building Site Groundwater Investigations and Reporting', dated October 2022 outlines the minimum scope of investigation required where a basement is proposed and may intersect the groundwater table. The scope broadly requires the following:

- Boreholes drilled to a minimum depth, which is defined by the proposed number of basements; at least one borehole must be drilled to 20m depth for a 3 level basement;
- The installation of a minimum of three groundwater wells installed throughout the site in a triangulated fashion;
- Permeability testing to define the coefficient of permeability of the various soil and rock layers;
- Groundwater monitoring for a minimum period of three months in the six months prior to the submission of documentation to the relevant authority;
- Groundwater seepage modelling to predict the groundwater take, groundwater drawdown behind the shoring system and potential impact on nearby buildings and other groundwater users;
- Chemical analysis of the groundwater to determine its quality.

Following the completion of the above monitoring, a Site Hydrology Report (SHR) is required which presents the results of the seepage analysis and predicted water take, and impact on surrounding buildings and water users. This report is then incorporated into a Dewatering Management Plan (DMP), which is necessary for submission when applying for the relevant licence(s).

Where dewatering is required, potentially two approvals are required from WaterNSW (or NRAR). These are:

- A Water Access Licence (WAL);
- A Water Supply Works (WSW) approval.

A WAL is a licence that provides an allocation of a certain volume of water in the aquifer to a user. However, it does not provide the right to extract this water. To extract or pump water from an aquifer, such as is required during basement dewatering, a WSW approval is required. The WAL is required where extraction of water from the aquifer exceeds 3ML/annum, where a water year coincides with a financial year. Where extraction volumes are less than this value, a WAL is not required.

We can complete the additional hydrogeological component required by DPHI as part of the additional geotechnical investigation, if commissioned to do so.

4.3 Sydney Water

There are Sydney Water assets (sewer and water mains) to the north and east of the site and these are within the zone of influence of the proposed basement.

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

4.4 Transport for NSW (TfNSW)

The Grand Parade is a TfNSW asset. For an excavation of this size, depth and proximity to the road, we expect TfNSW will require a Finite Element Analysis (FEA) of the proposed excavation sequencing and shoring system to assess the potential impact of the proposed works on The Grand Parade, including services below the footpath.

With reference to the RMS Technical Direction document, Reference: GTD2020/001, dated 2 July 2020, TfNSW will require the installation of borehole inclinometers and survey monitoring of the shoring system along The Grand Parade frontage of the site.

Once the requirements have been confirmed by TfNSW, we can prepare a proposal to assist with the geotechnical aspects, if requested.

4.5 Dilapidation Surveys

Prior to the commencement of any demolition and excavation, dilapidation surveys should be completed on the neighbouring building to the west (213 Ramsgate Road) and all neighbouring townhouses within at least 15m to the south of the site, including any boundary walls or fences which are to be retained.

The dilapidation survey reports can be used as a benchmark against which to set vibration limits for the tracking of plant, 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.

4.6 Shoring Design

We strongly recommend early in the design process, that any ‘as constructed’ drawings for the neighbouring buildings to the west and south be obtained. This is so that details of the structures, including any basement levels, can be understood so that the shoring can be designed appropriately. This work should also coincide with a site visit to inspect the neighbouring properties to confirm the details.

4.6.1 Shoring System

Due to the presence of a deep collapsible sand profile and shallow groundwater, a shoring system, such as a diaphragm wall with the panels formed under bentonite, should be installed to support the sides of the basement excavation. The use of sheet piles are not recommended as they are unlikely to penetrate the dense and very dense sand, and vibrations from their installation would have a high likelihood of damaging the neighbouring buildings, nearby buried services and pavements around the site.

The shoring walls must be founded below bulk excavation level and into sandstone bedrock to form a ‘cut off’ for groundwater inflows into the excavations, unless the seepage analysis can show a deep penetration into the sands can sufficiently reduce seepage flows during construction. It will be necessary for the shoring walls to be either anchored or internally propped to reduce wall deflections as the excavation proceeds. Careful control of wall verticality and the construction sequence will be required to reduce seepage through the walls and potential wall movements.

The installation of the shoring system may cause ground surface movements. There is also a potential for soil mining during installation of the shoring causing subsidence of the ground around the walls. Care must therefore be taken by the contractor and builder during the works by monitoring the ground surface around the shoring with regular checks by the shoring supervisor and builder. The volume of spoil must be monitored, and if the volume of spoil is excessive compared with the volume of the panel, 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 shoring operations must immediately cease and further geotechnical advice sought. Further advice would be provided on this issue following the additional investigation.

The contractor who installs the shoring system must consider the risks associated with the techniques adopted and provide appropriate protections for adjoining structures, pavements and buried services.

4.6.2 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.

As it will be important for the shoring wall to be very stiff to limit the potential for damaging nearby structures and infrastructure, we recommend the shoring wall be analysed using a soil structure interaction program, such as Wallap or Plaxis, which allows structural actions in the shoring, loads in the braces and/or anchors, and movement of the shoring wall to be estimated. The geotechnical material properties below are recommended for the various soil strata.

Material	Bulk Density (kN/m$^{3)}$	Effective Bulk Density (kN/m$^{3)}$	Drained Cohesion c' (kPa)	Effective Friction Angle Φ' (°)	Poisson's Ratio	Elastic Modulus (MPa)
Fill, Very Loose and Loose Sand	18	8	0	27	0.3	10
Medium Dense Sand	19	9	0	33	0.3	40
Dense and Very Dense Sand	20	10	0	38	0.3	100

To act as a 'cut off' for groundwater, the shoring system should only be embedded about 0.3m into the sandstone bedrock to reduce the potential for soil mining.

Where temporary anchors extend beyond the site boundaries, then permission must be sought from the respective neighbouring property owner, prior to installation. Soil anchors bonded into the soil profile may be designed using the effective friction angles in the table above. 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 only experienced 'top tier' contractors should 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 outside the site. Such contactors must be engaged early in the planning process to confirm whether anchors are feasible in these conditions and are installed under a design and construct contract.

Anchors penetrating the shoring system below groundwater require specialised techniques and will be difficult to construct; these have the potential to leak and initiate sand runs (erosion of soil from behind the shoring).

As an alternative to installing temporary anchors, the shoring walls could 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, but we acknowledge the considerable lengths of the props and thus should be checked early on with the shoring contractor.

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 and/or the props removed.

4.7 Underpinning

The neighbouring buildings to the west and south are located on the common boundary, or otherwise in very close proximity to the site, and therefore within the zone of influence of the excavation. The 'zone of influence' for an excavation in sand is considered to be about twice the excavation depth away from the excavation. Therefore, the neighbouring building to the west and the closest townhouses to the south are susceptible to damage, if ground subsidence occurs due to deflection of the shoring system or sand mining during shoring wall installation.

The structural engineer must assess the deflections associated with their shoring design, and assess the effect of those movements on the nearby structures. If their assessment is that damage could occur, then the structural engineer must decide if a stiffer wall or underpinning of the neighbouring buildings is required. Similarly, the shoring contractor must consider the possibility of sand mining or decompression in the sands, and implement appropriate controls and contingency plans.

4.8 Dewatering

In order to maintain a 'dry' excavation during construction, internal dewatering will be required. Due to the deep sandy soil profile, we expect the dewatering could be carried out using a spear point system installed inside the shoring walls.

Provided the shoring piles are embedded into the sandstone bedrock below bulk excavation level, we do not expect any notable drawdown of groundwater to occur outside the excavation. This should be confirmed by the installation of at least three groundwater monitoring wells just outside the footprint of the basement excavation, which will require permission from the respective neighbouring property owners. The groundwater levels should be monitored daily by the builder during dewatering to confirm that groundwater levels are within about 0.3m of the lowest groundwater level measured prior to the commencement of dewatering. If groundwater levels during dewatering are found to have dropped by more than about 0.3m from the lowest 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 drawndown to a level that may cause settlement of the ground surface which is especially important noting the neighbouring structures to the west and south are located in close proximity to the site.

The proposed dewatering methodology must 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.

An important consideration regarding the dewatering will be the duration during construction, ie. cease dewatering after the building has been constructed or until sufficient hold down capacity has been installed using permanent rock anchors. The contractor must have plans in place as to how the dewatering will be removed and resealing of the tanked basement.

4.9 Footing Design

On completion of excavation, marine sands of loose relative density will be exposed at bulk excavation level and will be present within a depth of about 2m (possibly more) below bulk excavation level. Noting the expected relatively high column loads, the loose sands are not considered a suitable founding material for pad and strip footings, or even a raft slab.

Therefore, the proposed building should be uniformly supported by piled footings founded in the underlying marine sands of at least dense relative density at a depth of at least 4m below bulk excavation level. The depth of founding to assess the target depths for the piles must be confirmed following the additional investigation.

Continuous flight auger (CFA) piles, or cased CFA piles would be suitable pile types from a geotechnical perspective. We expect the piling contractor will require a working platform to be constructed, prior to commencing the piling works. The design of such a platform, and its thickness, depends on the loading from the piling rig and material used for the platform. We can complete the design of such a platform if requested.

As a guide, CFA piles of at least 0.45m diameter founded with a 3D embedment into dense or very dense sand (where D is the pile diameter in metres), along with 3D of dense or very dense sand below the pile toe, may be designed for a maximum allowable bearing pressure of 2,500kPa. The piles should be designed in end bearing only, due to the very low sleeve friction values through the loose (and very loose) sands below bulk excavation level.

If limit state design is adopted, an ultimate bearing pressure of 7,500kPa may be used for piles founded in dense or very dense sand on condition appropriate load factors and a "Geotechnical Strength Reduction Factor" (ϕ_g), as defined in Clause 4.3.1 of AS2159-2009 ('Piling – Design and Installation') is used, along with sufficient pile load tests, as appropriate.

The load capacity of the piles must be certified by the piling contractor, as there is nothing that can be inspected geotechnically.

We have considered steel screw piles, however, screw piles are expected to refuse prematurely in the medium dense and dense sand and so are not recommended.

Consideration must be given to assessing possible differential movements between the shoring/cut off walls founded in bedrock and the internal piles founded in sand, though is unlikely to be an issue geotechnically. Based on the previous investigation results for the properties opposite the site, as a guide, the shoring walls founded in bedrock may be provisionally designed for an allowable bearing pressure of 1,000kPa, though depending on the assessed rock quality after the additional investigation has been completed could be as high as 3,500kPa.

4.10 Earthquake Design Parameters

A Hazard Factor (Z) of 0.08 and a Site Subsoil Class D_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.

4.11 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'.

4.12 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 the adjoining ground surface levels. Due to the high uplift pressures, it will be necessary to tie down the building, such as with permanent rock anchors.

The hydrostatic slab must be structurally connected to the building and piles, and designed to span between these supports to resist the hydrostatic pressure.

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.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the design and construction phase of the project, including the completion of an additional subsurface investigation. 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 beyond and below 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 should only be prepared following completion of the additional geotechnical investigation. 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.

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CERTIFICATE OF ANALYSIS 294618

Client Details

Client	JK Geotechnics
Attention	Arthur Kourtesis
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**34871PH, Coles Ramsgate, 277 Grand Parade**
Number of Samples	3 Soil
Date samples received	03/05/2022
Date completed instructions received	03/05/2022

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	10/05/2022
Date of Issue	06/05/2022

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Results Approved By
Diego Bigolin, Inorganics Supervisor

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil				
Our Reference		294618-1	294618-2	294618-3
Your Reference	UNITS	BH1	BH1	BH5
Depth		1.6-2.0	6.0-6.45	0.5-0.95
Date Sampled		02/05/2022	02/05/2022	02/05/2022
Type of sample		Soil	Soil	Soil
Date prepared	-	05/05/2022	05/05/2022	05/05/2022
Date analysed	-	05/05/2022	05/05/2022	05/05/2022
pH 1:5 soil:water	pH Units	9.4	9.6	9.3
Chloride, Cl 1:5 soil:water	mg/kg	49	<10	60
Sulphate, SO4 1:5 soil:water	mg/kg	30	<10	40
Resistivity in soil*	ohm m	110	230	84

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	294618-2
Date prepared	-			05/05/2022	1	05/05/2022	05/05/2022		05/05/2022	05/05/2022
Date analysed	-			05/05/2022	1	05/05/2022	05/05/2022		05/05/2022	05/05/2022
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	1	9.4	9.5	1	101	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	49	44	11	101	98
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	30	28	7	91	91
Resistivity in soil*	ohm m	1	Inorg-002	<1	1	110	120	9	[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.

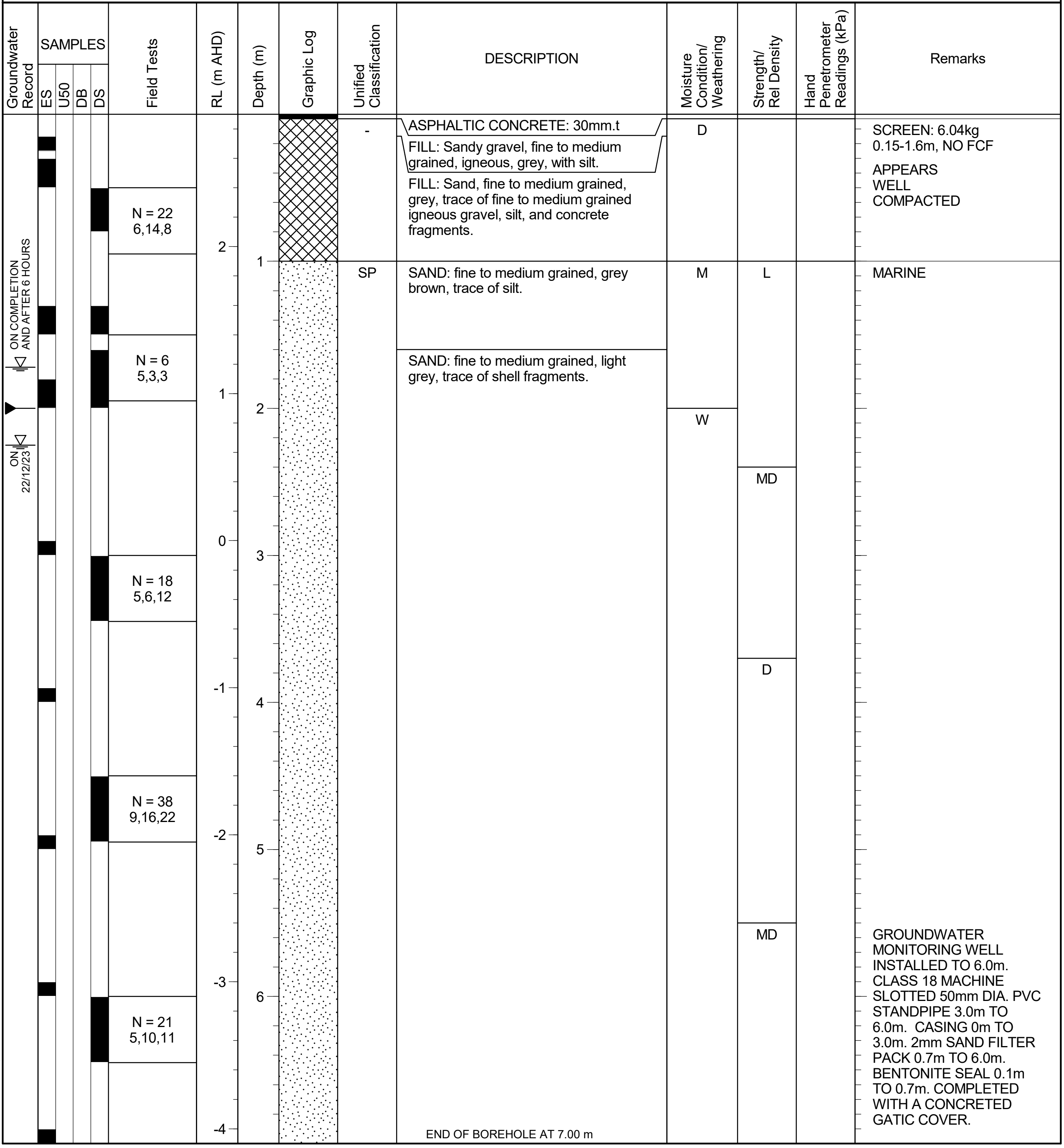
Samples for Microbiological analysis (not Amoeba forms) received outside of the 2-8°C temperature range do not meet the ideal cooling conditions as stated in AS2031-2012.

BOREHOLE LOG

Borehole No.
1
1 / 1

Client: GOOD TIME HOLDINGS NSW
Project: PROPOSE MIXED USE DEVELOPMENT
Location: 277 GRAND PARADE, RAMSGATE, NSW

Job No.: 34871PH
Method: SPIRAL AUGER
R.L. Surface: ~2.9 m
Date: 2/5/22
Datum: AHD
Plant Type: JK400
Logged/Checked By: T.F./A.J.H.



Groundwater Record	Field Tests	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
		-	ASPHALTIC CONCRETE: 30mm.t	D			SCREEN: 6.04kg 0.15-1.6m, NO FCF
			FILL: Sandy gravel, fine to medium grained, igneous, grey, with silt.				
	N = 22 6,14,8		FILL: Sand, fine to medium grained, grey, trace of fine to medium grained igneous gravel, silt, and concrete fragments.				APPEARS WELL COMPACTED
ON COMPLETION AND AFTER 6 HOURS		SP	SAND: fine to medium grained, grey brown, trace of silt.	M	L		MARINE
	N = 6 5,3,3		SAND: fine to medium grained, light grey, trace of shell fragments.				
ON 22/12/23				W			
	N = 18 5,6,12				MD		
	N = 38 9,16,22				D		
	N = 21 5,10,11				MD		GROUNDWATER MONITORING WELL INSTALLED TO 6.0m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 3.0m TO 6.0m. CASING 0m TO 3.0m. 2mm SAND FILTER PACK 0.7m TO 6.0m. BENTONITE SEAL 0.1m TO 0.7m. COMPLETED WITH A CONCRETED GATIC COVER.
			END OF BOREHOLE AT 7.00 m				

BOREHOLE LOG

Borehole No.
2
1 / 1

Client: GOOD TIME HOLDINGS NSW
Project: PROPOSE MIXED USE DEVELOPMENT
Location: 277 GRAND PARADE, RAMSGATE, NSW

Job No.: 34871PH **Method:** SPIRAL AUGER **R.L. Surface:** ~3.1 m
Date: 2/5/22 **Datum:** AHD
Plant Type: JK400 **Logged/Checked By:** T.F./A.J.H.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
						3			-	ASPHALTIC CONCRETE: 30mm.t	D			
										FILL: Sandy gravel, fine to medium grained, igneous, grey, with silt.				
										FILL: Sand, fine to medium grained, grey, trace of fine grained igneous gravel, silt, and concrete fragments.				
						2	1			FILL: Sand, fine to medium grained, grey and orange brown, trace of fine to medium grained igneous gravel and terracotta fragments.				
									SP	FILL: Sand, fine to medium grained, grey, trace of silt and fine grained igneous gravel.	M			MARINE
ON COMPLETION										SAND: fine to medium grained, light grey.				
						1	2			END OF BOREHOLE AT 2.00 m				
						0	3							
						-1	4							
						-2	5							
						-3	6							

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BOREHOLE LOG

Borehole No.
3
1 / 1

Client: GOOD TIME HOLDINGS NSW
Project: PROPOSE MIXED USE DEVELOPMENT
Location: 277 GRAND PARADE, RAMSGATE, NSW

Job No.: 34871PH
Method: SPIRAL AUGER
R.L. Surface: ~2.7 m
Date: 2/5/22
Datum: AHD
Plant Type: JK400
Logged/Checked By: T.F./A.J.H.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES 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								■	-	ASPHALTIC CONCRETE: 30mm.t	D			
	■							▨		FILL: Sandy gravel, fine to medium grained, igneous, grey.				
								▨		FILL: Sand, fine to medium grained, grey, trace of fine to medium grained igneous gravel, and silt.				
	■			■		2	1	⁘	SP	SAND: fine to medium grained, light grey.	M			MARINE
						1 0 -1 -2 -3 -4	2 3 4 5 6			END OF BOREHOLE AT 1.20 m				

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BOREHOLE LOG

Borehole No.
4
1 / 1

Client: GOOD TIME HOLDINGS NSW
Project: PROPOSE MIXED USE DEVELOPMENT
Location: 277 GRAND PARADE, RAMSGATE, NSW

Job No.: 34871PH
Method: SPIRAL AUGER
R.L. Surface: ~2.6 m
Date: 2/5/22
Datum: AHD
Plant Type: JK400
Logged/Checked By: T.F./A.J.H.

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
ON COMPLETION						-	ASPHALTIC CONCRETE: 30mm.t	M			
							FILL: Sandy gravel, fine to medium grained, igneous, grey.				
			2				FILL: Sand, fine to medium grained, grey, with silt, trace of fine to medium grained igneous gravel.				
				1			FILL: Sand, fine to medium grained, light grey brown, trace of fine to medium grained igneous gravel, and silt.				
			1			SP	SAND: fine to medium grained, light grey.	M W			MARINE
							END OF BOREHOLE AT 1.70 m				

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BOREHOLE LOG

Borehole No.
5
1 / 1

Client: GOOD TIME HOLDINGS NSW
Project: PROPOSE MIXED USE DEVELOPMENT
Location: 277 GRAND PARADE, RAMSGATE, NSW

Job No.: 34871PH
Method: SPIRAL AUGER
R.L. Surface: ~3.0 m
Date: 2/5/22
Datum: AHD
Plant Type: JK400
Logged/Checked By: T.F./A.J.H.

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
										CONCRETE: 160mm.t				7mm DIA. STEEL REINFORCEMENT, 72mm & 89mm TOP COVER
	■			■					-	FILL: Sandy gravel, fine to medium grained, igneous, grey, with silt.	M			
				■	N = 9 2,5,4	2	1			FILL: Sand, fine to medium grained, grey and light brown, trace of fine to medium grained igneous gravel, silt, and concrete fragments.				APPEARS MODERATELY COMPACTED
ON COMPLETION ON 22/12/23	■			■		1	2		SP	SAND: fine to medium grained, light grey, trace of shell fragments.	M W	L		MARINE
				■	N = 20 6,9,11	0	3					MD		
				■		-1	4					(D)		
				■		-2	5							
				■		-3	6					(MD) (L)		GROUNDWATER MONITORING WELL INSTALLED TO 6.0m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 3.0m TO 6.0m. CASING 0m TO 3.0m. 2mm SAND FILTER PACK 1.8m TO 6.0m. BENTONITE SEAL 0.1m TO 1.8m. COMPLETED WITH A CONCRETED GATIC COVER.
				■						SAND: fine to medium grained, grey and dark grey, trace of silt and shell fragments.				

END OF BOREHOLE AT 7.00 m

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BOREHOLE LOG

Borehole No.
6
1 / 1

Client: GOOD TIME HOLDINGS NSW
Project: PROPOSE MIXED USE DEVELOPMENT
Location: 277 GRAND PARADE, RAMSGATE, NSW

Job No.: 34871PH **Method:** SPIRAL AUGER **R.L. Surface:** ~3.2 m
Date: 2/5/22 **Datum:** AHD
Plant Type: JK400 **Logged/Checked By:** T.F./A.J.H.

Groundwater Record	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	-	ASPHALTIC CONCRETE: 30mm.t	D			
		FILL: Silty gravel. fine to medium grained, igneous, grey.				
		FILL: Silty sand, fine to medium grained, dark grey, with fine to medium grained igneous gravel, trace of ash.				
ON COMPLETION	SP	SAND: fine to medium grained, light grey brown, trace of silt.	M W			MARINE
		END OF BOREHOLE AT 2.00 m				

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BOREHOLE LOG

Borehole No.
7
1 / 1

Client: GOOD TIME HOLDINGS NSW
Project: PROPOSE MIXED USE DEVELOPMENT
Location: 277 GRAND PARADE, RAMSGATE, NSW

Job No.: 34871PH
Method: SPIRAL AUGER
R.L. Surface: ~2.7 m
Date: 2/5/22
Datum: AHD
Plant Type: JK400
Logged/Checked By: T.F./A.J.H.

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	RL (m AHD)	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel Density	Hand Penetrometer Readings (kPa)	Remarks
	■							▬	-	ASPHALTIC CONCRETE: 30mm.t	D			
								▩		FILL: Sandy gravel, fine to medium grained, igneous, grey.				
	■			■				▒	SP	FILL: Sand, fine to medium grained, grey brown, with silt, trace of fine to medium grained igneous gravel.	M			MARINE
	■					2	1	▒		SAND: fine to medium grained, light grey and grey brown, trace of silt.				
ON COMPLETION				■		1		▒			W			
							2			END OF BOREHOLE AT 2.00 m				
						0	3							
						-1	4							
						-2	5							
						-3	6							
						-4								

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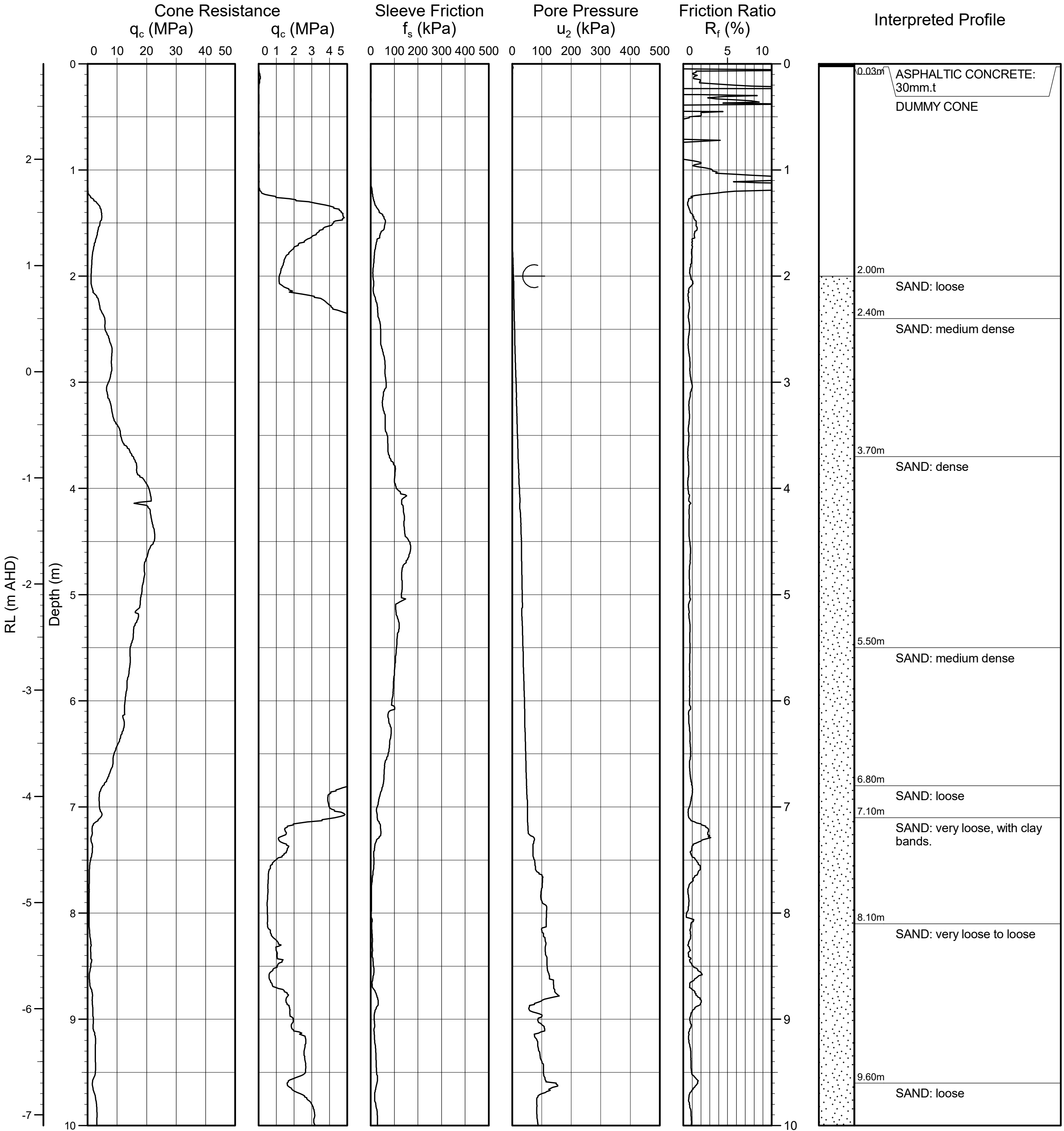
CONE PENETROMETER TEST RESULTS

CPT No.
8
1 / 2

Client: GOOD TIME HOLDINGS NSW
Project: PROPOSE MIXED USE DEVELOPMENT
Location: 277 GRAND PARADE, RAMSGATE, NSW

Job No.: 34871PH
R.L. Surface: ~2.9 m
Data File: 34871PH Ramsgate
Date: 3/5/22
Datum: AHD
Operator: B.Z./B.J.



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

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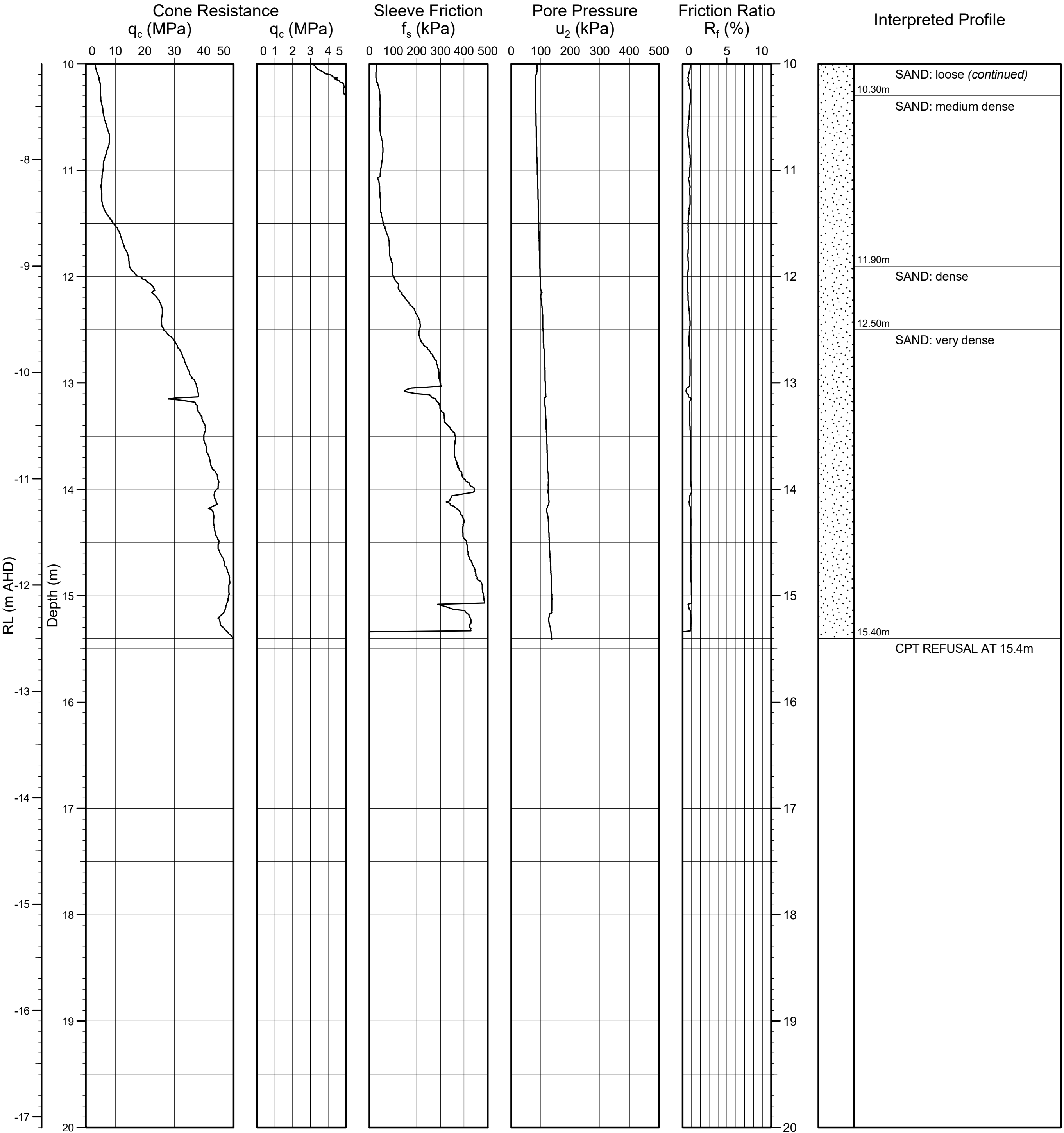
CONE PENETROMETER TEST RESULTS

CPT No.
8
2 / 2

Client: GOOD TIME HOLDINGS NSW
Project: PROPOSE MIXED USE DEVELOPMENT
Location: 277 GRAND PARADE, RAMSGATE, NSW

Job No.: 34871PH
R.L. Surface: ~2.9 m
Data File: 34871PH Ramsgate
Date: 3/5/22
Datum: AHD
Operator: B.Z./B.J.



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

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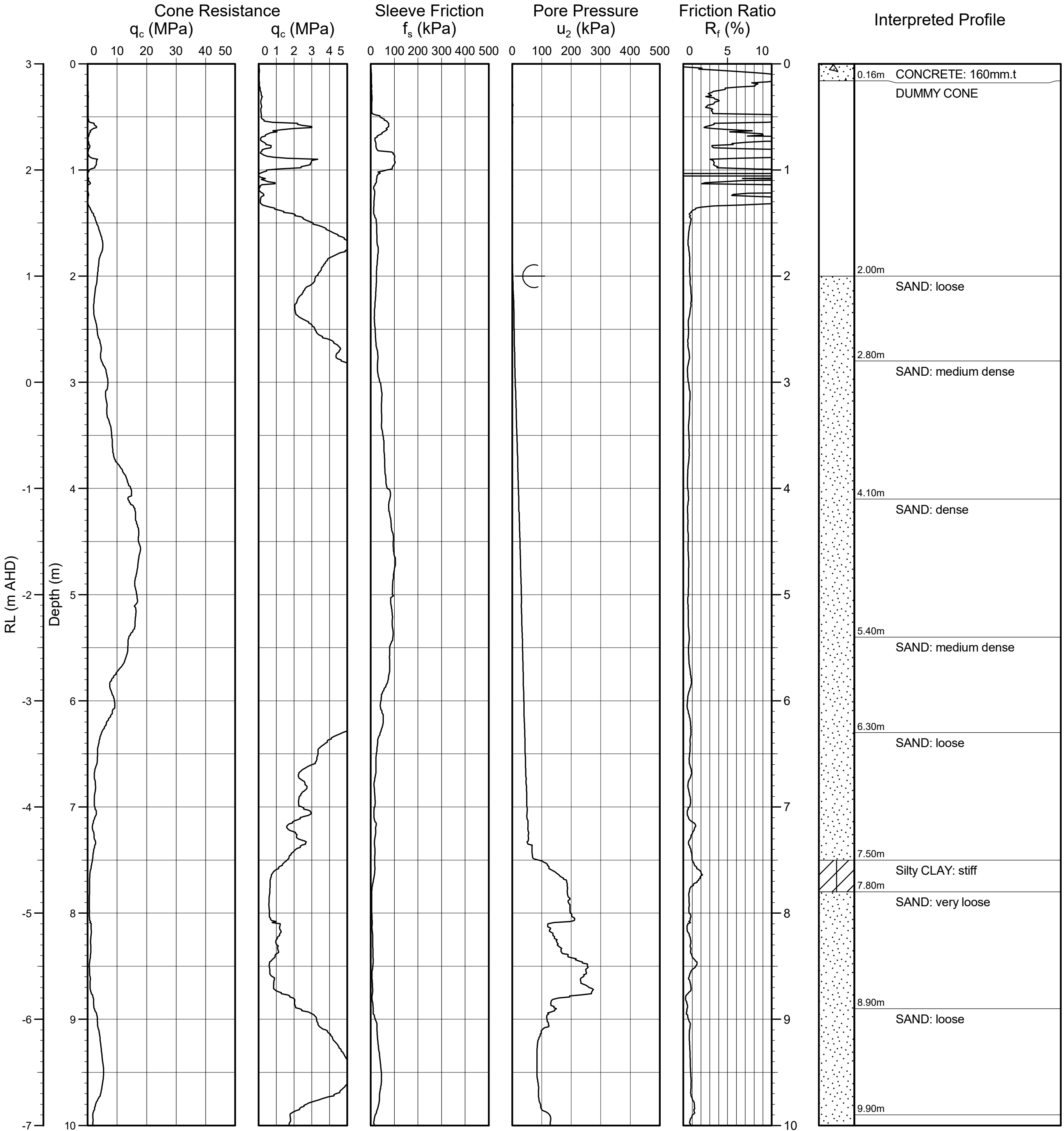
CONE PENETROMETER TEST RESULTS

CPT No.
9
1 / 3

Client: GOOD TIME HOLDINGS NSW
Project: PROPOSE MIXED USE DEVELOPMENT
Location: 277 GRAND PARADE, RAMSGATE, NSW

Job No.: 34871PH **R.L. Surface:** ~3.0 m **Data File:** 34871PH Ramsgate
Date: 3/5/22 **Datum:** AHD **Operator:** B.Z./B.J.



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

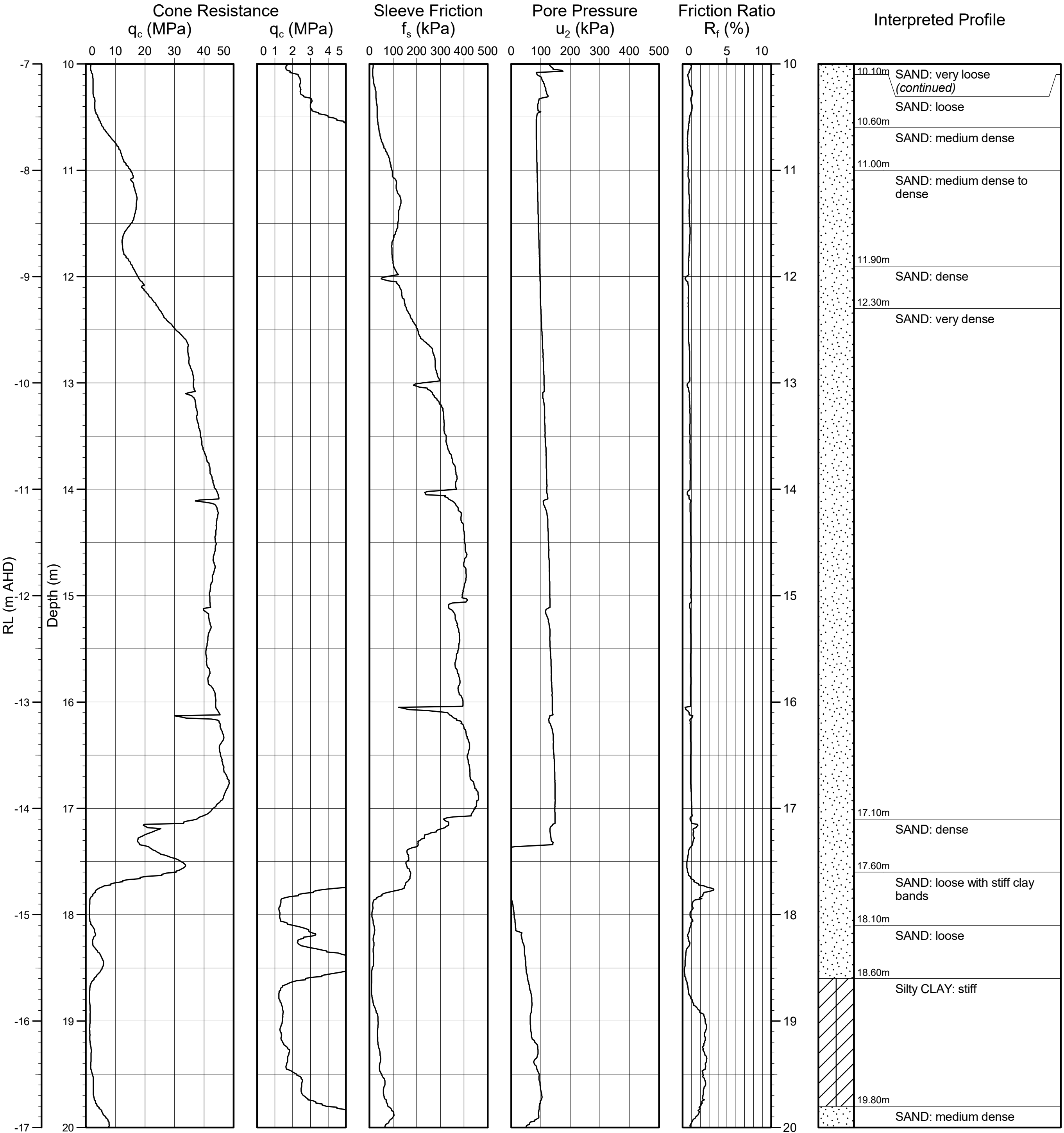
JK 9.02.4 LIB.GLB Log JK CPTU MATERIAL - MASTER 34871PH RAMSGATE.GPJ <<DrawingFile>> 04/01/2024 13:14 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

CONE PENETROMETER TEST RESULTS

CPT No.
9

Client: GOOD TIME HOLDINGS NSW
Project: PROPOSE MIXED USE DEVELOPMENT
Location: 277 GRAND PARADE, RAMSGATE, NSW

Job No.: 34871PH
R.L. Surface: ~3.0 m
Data File: 34871PH Ramsgate
Date: 3/5/22
Datum: AHD
Operator: B.Z./B.J.



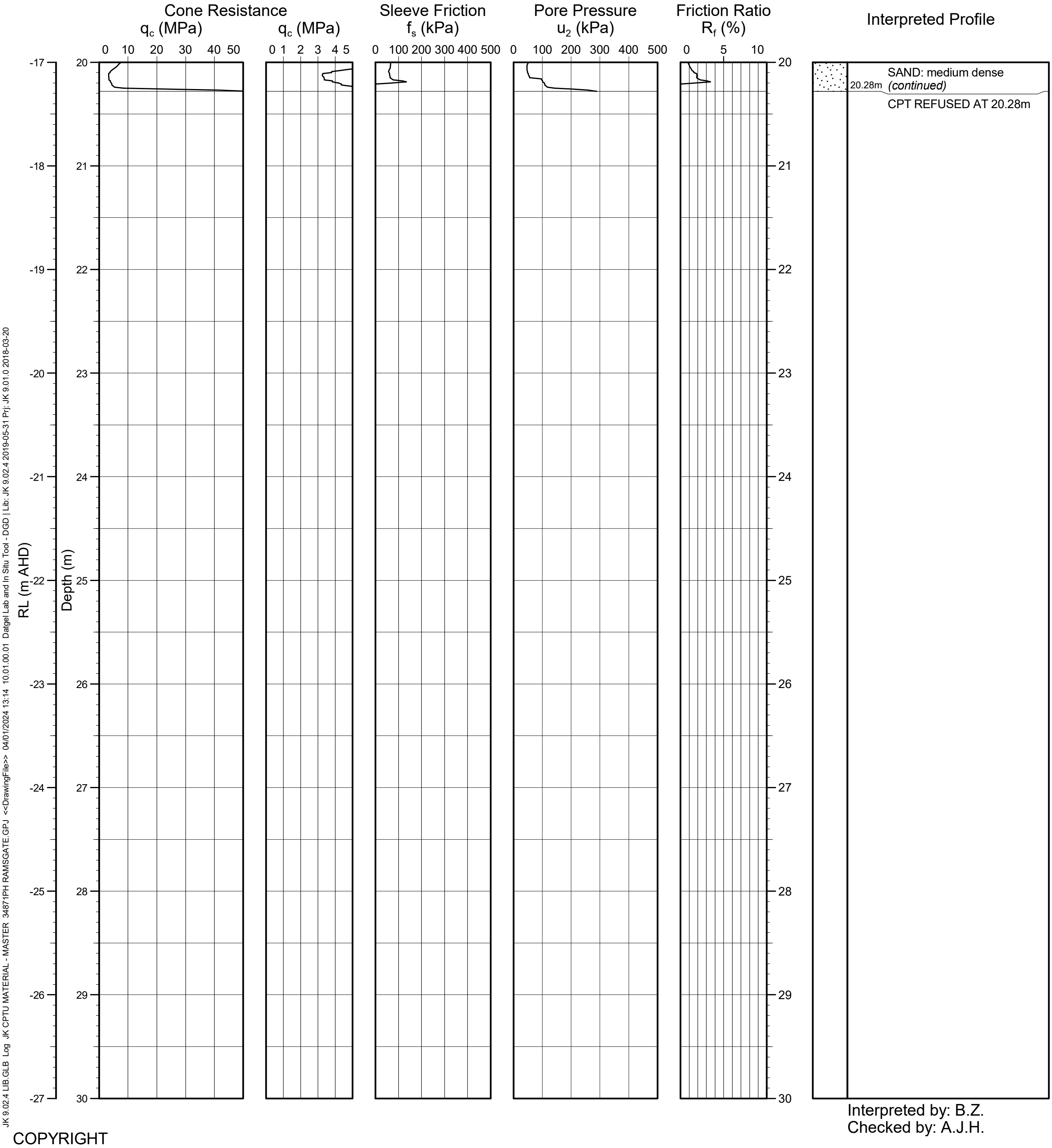
Interpreted by: B.Z.
Checked by: A.J.H.

CONE PENETROMETER TEST RESULTS

CPT No.
9
3 / 3

Client: GOOD TIME HOLDINGS NSW
Project: PROPOSE MIXED USE DEVELOPMENT
Location: 277 GRAND PARADE, RAMSGATE, NSW

Job No.: 34871PH **R.L. Surface:** ~3.0 m **Data File:** 34871PH Ramsgate
Date: 3/5/22 **Datum:** AHD **Operator:** B.Z./B.J.



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

JK 9.02.4 LIB.GLB Log JK CPTU MATERIAL - MASTER 34871PH RAMSGATE.GPJ <<DrawingFile>> 04/01/2024 13:14 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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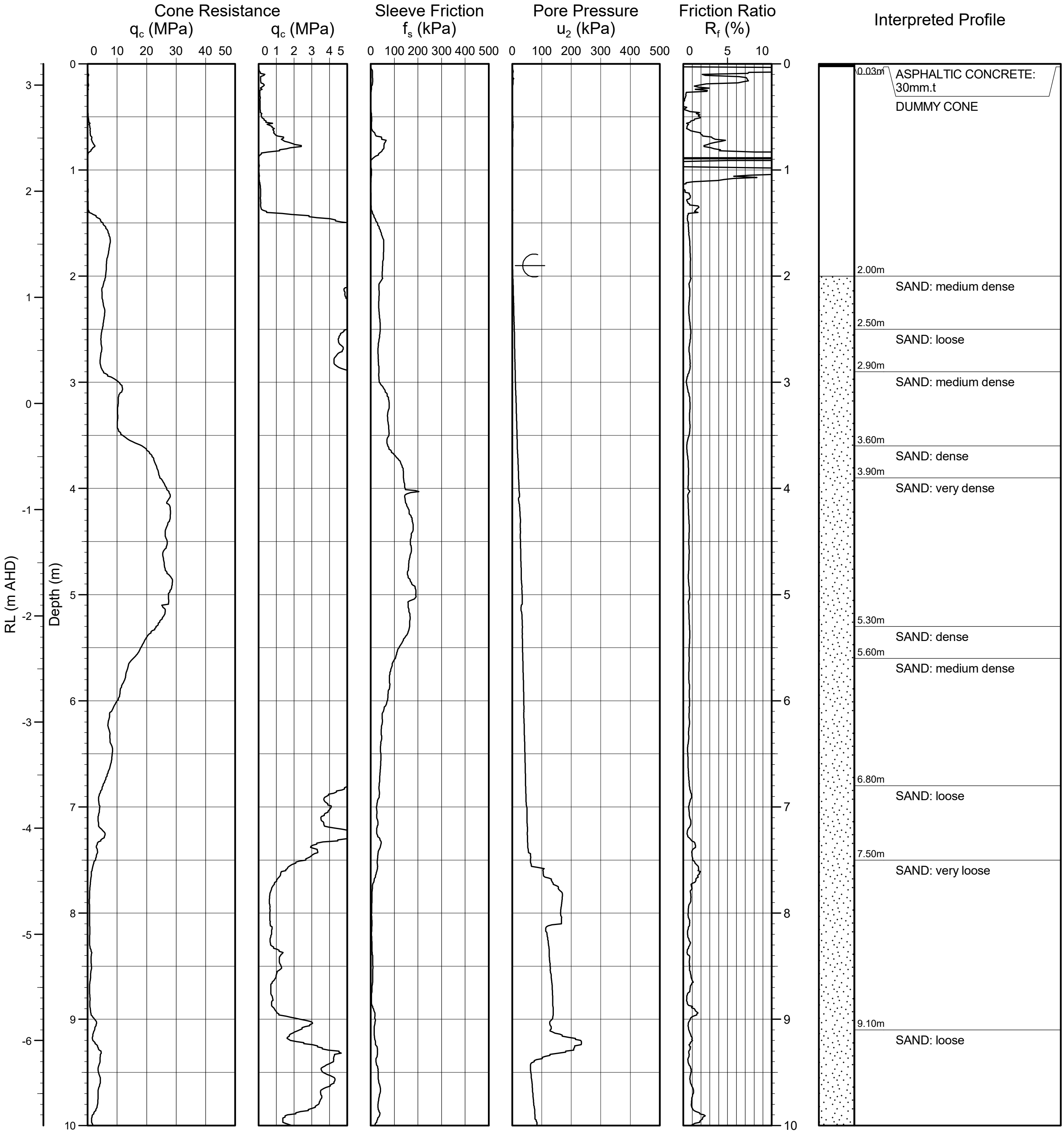
CONE PENETROMETER TEST RESULTS

CPT No.
10
1 / 2

Client: GOOD TIME HOLDINGS NSW
Project: PROPOSE MIXED USE DEVELOPMENT
Location: 277 GRAND PARADE, RAMSGATE, NSW

Job No.: 34871PH	**R.L. Surface:** ~3.2 m	**Data File:** 34871PH Ramsgate
Date: 3/5/22	**Datum:** AHD	**Operator:** B.Z./B.J.



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

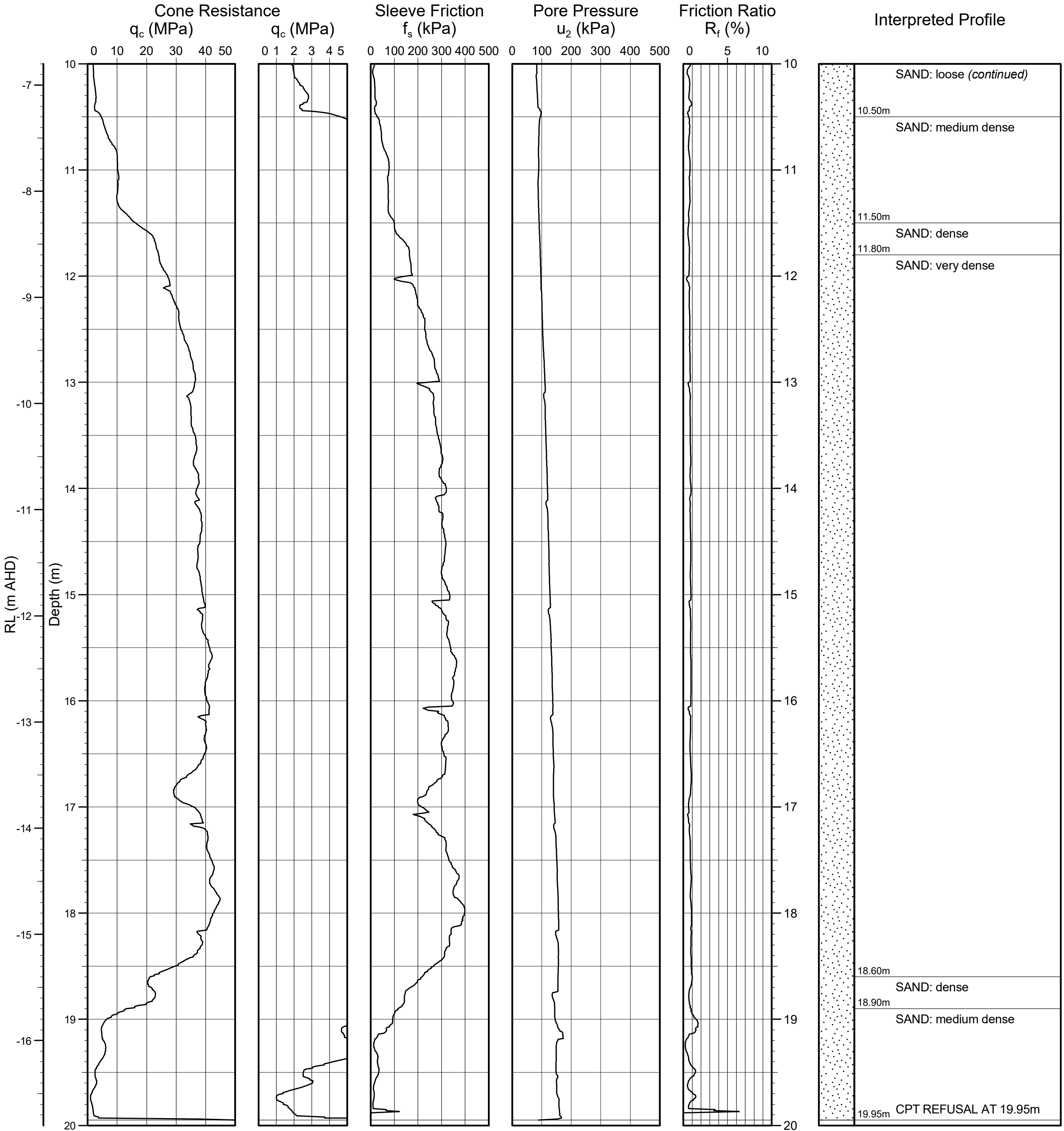
JK 9.02.4 LIB.GLB Log JK CPTU MATERIAL - MASTER 34871PH RAMSGATE.GPJ <<DrawingFile>> 04/01/2024 13:14 10.01.00.01 Datgel Lab and In Situ Tool - DGD | Lib: JK 9.02.4 2019-05-31 Prj: JK 9.01.0 2018-03-20

CONE PENETROMETER TEST RESULTS

CPT No.
10
2 / 2

Client: GOOD TIME HOLDINGS NSW
Project: PROPOSE MIXED USE DEVELOPMENT
Location: 277 GRAND PARADE, RAMSGATE, NSW

Job No.: 34871PH	**R.L. Surface:** ~3.2 m	**Data File:** 34871PH Ramsgate
Date: 3/5/22	**Datum:** AHD	**Operator:** B.Z./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: 277 THE GRAND PARADE, RAMSGATE, NSW

Report No: 34871PH

Figure No: 1

JKGeotechnics

PLOT DATE: 4/01/2024 2:38:13 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\34000'S\34871PH RAMSGATE\CAD\34871PH.DWG

LEGEND

● BOREHOLE

▼ CONE PENETROMETER TEST

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: 277 THE GRAND PARADE, RAMSGATE, NSW

Report No: 34871PH

Figure No: 2

JKGeotechnics

PLOT DATE: 4/01/2024 1:07:52 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\34000'S\34871PH RAMSGATE\CAD\34871PH.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 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	

				Field Classification of Silt and Clay			Laboratory Classification
Major Divisions		Group Symbol	Typical Names	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:

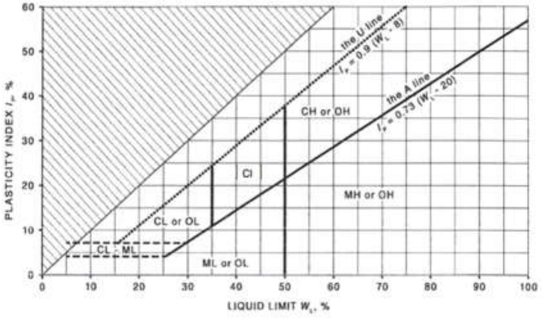
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

Abbreviations Used in Defect Description

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