SPECIALIST ADVICE TO

ALLEN GROUP DEVELOPMENTS PTY LTD

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED RESIDENTIAL DEVELOPMENT

AT

92 NORTH STEYNE, MANLY, NSW

Date: 4 March 2025

Ref: 37307Srpt

JKGeotechnics

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ATTACHMENTS

1 INTRODUCTION

This specialist advice report presents the results of a geotechnical investigation for the proposed residential development at 92 North Steyne, Manly. The location of the site is shown in Figure 1. The investigation was commissioned by email dated 23 January 2025 from Lighthouse Project Group on behalf of Allen Group Developments Pty Ltd. The commission was on the basis of our proposal Ref: P70505S dated 30 October 2024.

With reference to the provided preliminary architectural drawings prepared by Platform Architects (Project No. NSM2, undated), we understand that the proposed development will include:

- Demolition of the existing structures on site
- Excavation for the purpose of one basement level to accommodate car parking spaces, with additional car parking, building services and residential storage at ground floor level.
- Construction of a 5-storey residential building.

The basement will have a finished floor level at Reduced Level (RL) 2.67m, and bulk excavation to a maximum depth of about 3.0 to 3.5m below existing levels is expected to be required, with locally deeper excavations for lift pits and services. As no structural loads have been provided, typical loads for this type of development have been assumed.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions, and to use this as a basis for providing comments and preliminary recommendations on hydrogeological conditions, excavation conditions, shoring options, retaining wall design parameters, footing design, on-grade floor slabs, and additional geotechnical input required.

This report provides specialist advice for use by the structural and civil designers in preparing their preliminary designs and no part of this report is considered a regulated design in accordance with the Design and Building Practitioners Act 2020.

An acid-sulphate soil investigation was carried out as part of the works by our environmental division, JK Environments. Their report is presented under separate cover as Report No E37307BW dated February 2025

2 INVESTIGATION PROCEDURE

One Cone Penetration Test (CPT101) and one Dilatometer test (DMT 101) were completed to depths of 30m and 20m respectively, using our 22 tonne, truck mounted rig. The CPT involves continuously pushing a calibrated probe with a 44mm diameter conical tip into the soil. Measurements are taken during testing of the end resistance of the cone tip and the frictional resistance of a separate 134mm long sleeve located directly behind the cone. The CPT probing provides a continuous plot of soil type and strength with depth, but does not provide soil samples. The identification of subsurface material and strength is by interpretation of the CPT results based on empirical correlations, nearby borehole information, and our experience. All of the CPT's were completed using a piezocone, which also measures pore pressures within the soils.

The DMT test was undertaken and extended to a depth of 20m below existing surface levels. The dilatometer testing involves pushing a stainless steel blade with a flat circular membrane behind a horizontal piston into the ground. At regular intervals of 0.10 or 0.20m the membrane is inflated and the pressure required to inflate the membrane is recorded against the movement of the piston. The soil material properties (including elastic soil modulus) are determined from established correlations and are shown on the attached log sheets.

Three boreholes (BH1 to BH3) were drilled to depths of 6.5m (BH1), 6m (BH2) and 20m (BH3). BH1 was drilled closest to CPT101. BH1 and BH2 were drilled using spiral augering techniques with our truck mounted JK400 drill rig. BH3 was commenced with spiral augers but below 3m depth drilling was continued by wash boring with a casing advancer. The compaction of the fill and relative density of the sands were assessed from Standard Penetration Test (SPT) 'N' values and correlation from the results of the CPT.

Groundwater observations were made in the boreholes during and on completion of drilling. On completion of drilling, monitoring wells were installed in all of the boreholes to allow for future groundwater level monitoring. We revisited site at intervals of 4 to 7 days following the completion of drilling to measure standing water levels in the boreholes and to carry out infiltration tests.

Our geotechnical engineers were present full-time during the fieldwork to set out the investigation locations, nominate testing and sampling, progressively compile the CPT test results, direct the monitoring well installation, carry out infiltration testing and prepare the attached borehole logs. The interpretation of the attached CPT and DMT results was carried out by a Geotechnical Engineer. For details of the investigation techniques adopted, their limitations, and a glossary of logging terms and symbols used, reference should be made to the attached Report Explanation Notes.

The borehole, DMT and CPT locations, as shown on the attached Test Location Plan (Figure 2), were set out by taped measurements from existing surface features. Figure 2 is based on a site survey carried out by Bee and Lethbridge Pty Ltd, Ref 23086 dated 6 December 2023. The reduced surface levels at test locations were assessed by interpolation between spot heights shown on the survey drawing

Selected soil samples were returned to Envirolab Services Pty Ltd, a NATA accredited laboratory, for pH, sulphate content, chloride content and resistivity testing. The results are presented in the attached Envirolab Services 'Certificate of Analysis' 372936. Testing for possible soil or groundwater contamination was outside the agreed scope of the investigation.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site is located within relatively level coastal topography with the site itself having a slight 1° to 2° westwards slope to the rear of the property. The site is located west of Manly Beach and is bounded by North Steyne to the east.

At the time of the fieldwork the site was occupied by a three-storey brick apartment building. The site had a concrete surface surrounding the building with the exception of a grassed parking area on the eastern side of the building. A brick retaining wall extended around the southern, western and northern boundaries and was up to 0.7m high.

To the south of the site (No. 91 North Steyne) was a four-storey rendered apartment building. The apartment building abutted the common boundary in the western portion, and setback about 1.0 to 1.8m in other areas. There is a vehicular ramp on the North Steyne frontage that suggests there is probably at least one basement level below the building.

To the north of site (No. 93 – 95 North Steyne) was a rendered multi-storey apartment with shops on the ground floor level. The apartment building abutted the common boundary of the site. The site includes a battle axe section that extends to Whistler Street at the rear and there appears to be a vehicular ramp leading to a basement carpark of unknown depth.

3.2 Subsurface Conditions

The NSW seamless geology shows the site is underlain by coastal Quaternery deposits. The investigation has shown that the site is underlain by sandy soils that extend to depths in excess of 30m. Some characteristic features of the site are presented below but for details reference should be made to the attached borehole and CPT logs.

Pavements and Fill

Concrete a maximum of 110mm thick (BH2) and concrete over brick pavers 200mm thick (BH3) were penetrated from the surface. There was no pavement at the test locations at the front of the site (BH1 and CPT101) as the tests were located between concrete strip driveways.

Fill, predominantly comprising sandy soils with some igneous and sandstone gavel was encountered beneath the pavements or from surface level to depths generally between 0.3m and 0.8m. The fill was predominantly assessed to be poorly compacted.

Marine Sands

Marine sands were encountered beneath the fill in all of the CPTs and boreholes. There were occasional layers interpreted as gravelly sand at depth in CPT101.

The sands ranged from very loose to very dense relative density, and the correlation between the different test locations is not well defined.

The soils encountered in the CPT test from 29.5m to refusal at 30.51m have been interpreted as very dense sand but may well be extremely weathered bedrock and the refusal may have occurred on less weathered bedrock but no sampling was possible to confirm this.

Groundwater

Groundwater seepage was encountered in all of the boreholes during drilling, at depths between 4m and 5m. At intervals of 4 days to 7 days after drilling the groundwater levels were recorded at depths of 4.1m to 4.7m below the ground surface level (RL 0.3m to 0.9m). Groundwater was interpreted at a similar depth from the CPT pore pressure readings. No longer term groundwater monitoring was completed.

3.3 Laboratory Test Results

The following table summarises the results from Envirolab Services. Reference should be made to the attached Certificate of Analysis No. 372936 for further details.

Sample	Soil Type	pH	Chloride mg/kg	Sulphate mg/kg	Resistivity ohm.cm
BH1 1.5-1.95m	SAND	7.2	<10	<10	31,000
BH2 3.0-3.45m	SAND	9.2	<10	<10	20,000
BH3 9.0-9.45m	SAND	9.3	10	20	14,000

These results correlate with exposure classifications of 'Mild' and Non-aggressive' for buried concrete and steel elements respectively in accordance with Tables 6.4.2(C) and 6.5.2(C) of AS2159-2009 respectively.

4 COMMENTS AND RECOMMENDATIONS

4.1 Geotechnical Issues

Prior to any design work commencing it is essential that details of the buildings on the sites to the north and south be determined, in particular the depth of any basement levels and the type of footings used (ie raft slabs, piles or piled rafts). Until this information is known the effect of the proposed construction on these structures and, vice-versa, the effect of the structures on the proposed construction cannot be determined. For the purpose of our preliminary considerations, we have assumed the adjoining buildings have a single basement level that is at a similar level to that proposed on the subject site.

The proposed basement level will require excavation to about RL 2.4m assuming a slab-on-grade of nominal thickness is used; if a raft slab is used to support the building it will probably be in the order of 0.6m thickness and the excavation will extend to about RL 2.0m. given that the water table has been recorded at levels between RL 0.3m and RL 0.9m the excavations should be above the water table and no dewatering will be required. In the long term the water table may rise significantly and it will be necessary to ensure the basement is watertight as well as having the capacity to pump excess water away to the stormwater system for short periods.

The requirements of a shoring system are not fully defined at present as the details of the inferred basements to the north and south are not known. If those basements extend up to the site boundaries then it may not be necessary to install a shoring system in those areas, though shoring at the eastern and western ends of the basement will certainly be required. For the purposes of this report we have assumed that shoring on all four sides of the excavation will be required. Appropriate shoring systems include contiguous continuous flight auger (CFA) piles and cutter soil mix (CSM) walls. We do not recommend sheet piled systems due to the risk of vibration damage to the immediately adjacent structures.

In terms of footing systems, it seems from the information available to date, that options for a raft slab, a piled raft slab and conventional piled footings exist.

The investigation data recorded to date has shown some discrepancies that must be resolved by further testing after demolition when access to the whole site is possible for the CPT/DMT rig.

4.2 Dilapidation Reports

Prior to any demolition and excavation commencing, we recommend that detailed dilapidation reports be prepared for the existing buildings to the north, south and west of the site. As the buildings to the north and south are large unit buildings, we suggest limiting the reports to the units closest to the excavation and perhaps including a few other units as reference points. The dilapidation surveys should comprise detailed inspections both externally and internally, with all defects rigorously described, e.g. defect location, defect type, crack width, crack length, etc. The respective building owners should be provided with a copy of the dilapidation reports and be asked to confirm that they present a fair representation of existing conditions. Such reports can then be used as a baseline against which to assess possible future claims for damage arising from the works. We note that Council may also require that dilapidation reports be prepared for their nearby assets.

4.3 Excavation, Seepage, and Shoring

4.3.1 Excavation

Prior to any excavation commencing we recommend that reference be made to the Safe Work Australia 'Excavation Work Code of Practice' dated July 2015.

Bulk excavation to a maximum depth of about 3.5m is expected to be required for the proposed development, and is expected to encounter the soil profile only. Excavation of such materials will be readily achievable using hydraulic excavators, following installation of the shoring (where required). Reference should be made to Section 5 below for discussion on the offsite disposal of excavated materials.

Given the proposed basement extends up to or within close proximity to the site boundaries in most areas, temporary batters are not considered feasible, apart from short term batters within the shoring walls during the excavation. We therefore consider that a shoring system must be installed prior to the commencement of excavation unless the shoring systems on the sites to the north and south extend right up to the boundary in which case shoring would be redundant.

Construction plant will cause some ground vibrations, however, we expect these can be properly managed by adopting appropriate work procedures and equipment. Nevertheless, considerable caution should be taken during construction works as the poorly compacted fill and loose sands are likely to extend across the site boundaries. Therefore, all construction activities, particularly demolition and excavation, should be carefully controlled to avoid ground vibration damage, or vibration induced settlement of the sand foundation materials leading to loss of support to the neighbouring buildings and structures. For example, stop-start operations, movement and manoeuvring of large tracked plant, or piling, or vibratory rollers will cause ground vibrations and these should be carefully monitored and controlled (lighter plant and good expertise will need to be used). Considerable caution should be taken as ground vibrations may be transmitted to nearby structures. Vibrations, measured as Peak Particle Velocity (PPV), can be limited to no higher than 5mm/sec for the neighbouring buildings, subject to the structural engineers review of the dilapidation survey reports and confirmation of this limit. If higher vibrations are recorded, additional advice must be sought before work proceeds. The structural engineer must also assess the neighbouring structures to advise whether underpinning will be required to protect those structures.

4.3.2 Hydrogeological Conditions and Seepage

Based on the results of our investigation, groundwater will be below bulk excavation level, and as such, no dewatering is expected to be required for the development.

Should any groundwater seepage be observed during excavation works, excavation must cease and further geotechnical advice be sought without delay before works recommence.

4.3.3 Shoring Options

Given the encountered subsurface profile, we consider a contiguous pile wall to be a suitable shoring option for the proposed development. We note that where a contiguous pile wall is adopted, care must be taken that any gaps between piles are promptly filled with ram packed mortar as they are exposed to protect against the uncontrolled flow of sand through gaps between piles. Due to the collapsing nature of the soils, and as the piles would need to found below groundwater level, continuous flight auger (CFA) piling techniques would be required for a contiguous pile wall.

A secant pile wall or cutter soil mix (CSM) wall could also be considered.

The effect of ground movements on any structures and services that lie within the influence zone of the excavation must also be taken into account. The influence zone of the excavation may be defined as a horizontal distance of at least 2H (where 'H' is the depth of the excavation in metres) behind the wall.

We note that for the above wall options, a cast in situ guide wall will be required to facilitate construction, and the excavation for the guide wall may undermine the footings of nearby structures. The structural engineer and shoring consultant must consider this as well as the potential for settlement of the sands below footings due to vibration or decompression of sands around the auger when assessing the need to underpin any nearby structures.

Steel sheet pile walls should not be considered due to the possibility of vibration induced damage to nearby structures during installation.

For all of the proposed retention options, the toe of the walls must be embedded to a suitable depth below bulk excavation level to provide adequate wall stability. To reduce excavation induced movements along site boundaries, the shoring system must be braced as the excavation progresses, or alternatively, a top-down construction process adopted. If anchors in the soil profile are to be considered further, additional advice must be sought from specialist anchoring contractors who can design and install these, possibly using pressure grouting techniques to increase the load capacity.

The construction of the shoring wall must be of high quality so as to reduce as much as possible the potential for any 'gaps' in the wall. If any gaps occur, uncontrolled flow of soil could occur, particularly during/after rainfall, and remediation will likely be very difficult to achieve. If the presence of any such gaps is suspected, grouting behind the shoring wall in advance of the excavation would likely be the only option to seal such gaps. The contractor responsible for the shoring system must also be required to provide a method statement detailing how such sealing and rectification works will be carried out, as it will not be safe to allow a flow of soil through the shoring wall whilst details are sorted.

We assume that permanent lateral support of the shoring system will be provided by the basement structure.

We recommend carrying out computer modelling (e.g. WALLAP and/or Plaxis) of the proposed retaining wall system to analyse loads on the walls and potential movement of the walls. We can complete such modelling if commissioned to do so.

4.3.4 Shoring Design

For the preliminary design of propped walls for the proposed basement, we recommend that a uniform rectangular lateral earth pressure distribution of magnitude 8HkPa, be used for the soil profile (where 'H' is the depth of the excavation in metres). A reduced magnitude of 6HkPa could be considered where only landscaped areas are within the zone of influence of the wall.

Any surcharge (including pavements, traffic, etc.) affecting the walls should be allowed in the design using an average at rest earth pressure coefficient (K_0) of 0.5. If inclined backfill surfaces are proposed, then they should be treated as a surcharge. Hydrostatic pressures must also be incorporated into the design of the shoring walls as appropriate.

We emphasise that some movement may occur within the zone of influence even with an anchored or propped wall that is designed for the lateral pressures detailed above. Adoption of an appropriate support and excavation sequence and construction of high quality is essential to reduce the risk of damage. The wall designer must decide whether the adjoining buildings require underpinning, depending on the potential shoring movements, and sensitivity of the structures.

Lateral toe restraint of the walls may be achieved by embedment of the toe of the walls into the soil profile below bulk excavation level, including allowance for any footing or service excavations. Such restraint could be calculated using a triangular lateral earth pressure distribution and an average 'passive' earth pressure coefficient (K_p) of 2.5 for the fill or loose sand, 3.0 for medium dense sand, or 3.5 for dense sand. Due to the large movements required to mobilise full passive pressures, a Factor of Safety of at least 2.0 should be adopted when assessing the passive resistance to limit these deflections. The design must make a suitable allowance for potential over excavation and disturbance effects. We note that for soils below excavation level where groundwater will be close to, or above, excavation level, the assessment of passive resistance must consider the buoyant unit weight of the soils.

Design of the shoring walls should consider possible elevated external water levels which could occur during flooding, and we recommend that 1 in 100 year flood levels in the surrounding area be considered for such an assessment. Council should be contacted for advice regarding design flood levels.

The geotechnical material properties below are recommended for the various soil strata.

Material	Bulk Density (kN/m^{3})	Drained Cohesion – c' (kPa)	Undrained Cohesion – c_u (kPa)	Undrained Friction Angle – Φ' (°)	Poisson's Ratio	Elastic Modulus (MPa)
Fill and Very Loose/Loose Sand	18	0	0	28	0.3	10
Medium Dense Sand	19	0	0	32	0.3	50
Dense and Very Dense Sand	21	0	0	37	0.3	200

The properties tabulated above could also be adopted where thin loose bands are present in the sands (i.e. the thin loose bands ignored for the purpose of analysing the shoring walls).

4.4 Footings

4.4.1 Piles

From the results of the investigation medium dense sand is likely to be present at/below BEL, though there are some discrepancies in the relative density of the sands interpreted from the boreholes as against those from the CPT/DMT. It is not possible at this stage to state with certainty that the looser sand conditions revealed in the boreholes at varying depths are due to inaccuracy of the SPT tests (which can be affected by the base of the borehole 'blowing' when the drill rods/augers are withdrawn) or whether it is a real variation in sand density around the site. It is considered most likely that the boreholes have shown looser conditions than a CPT/DMT test would and the results of the latter are considered the more reliable on this issue. Thus, the final design may be dependent upon further testing after demolition allows additional CPT testing of the whole site area.

Based on the above, we consider piled footings, founded in the medium dense sands below 14m depth, will likely be one of the appropriate foundation solutions for the proposed development. For piled footings, CFA techniques would be required, as steel screw piles into dense and very dense sands are typically not feasible due to their inability to penetrate into such materials.

Allowable bearing pressures for piles founded in sandy soils are dependent on both the diameter of the piles and the depth of embedment. Indicatively, for 0.6m diameter (or larger) piles, founded in medium dense sands at about 16m below current levels, (and not less than 7m below bulk excavation level) and not less than 2m above any clay layers or loose sands, an allowable end bearing pressure of 1,000kPa can be adopted. There would be some contribution of shaft friction for piles of such length which can be approximated to 8kPa below 4m depth, though that will be subject to more detailed analysis when further subsurface data is available. For such piles, elastic settlements of 10mm or less would be expected. Following the finalisation of the structural design, more detailed advice for the design of piles can be provided.

If pile groups in the sandy soils are proposed, additional analysis will be required to assess interaction effects between such closely spaced piles, which would be expected to result in larger settlements than for individual piles.

CFA piles in sands for developments such as that proposed are typically completed on a design and construct basis by the piling contractor, with certification also typically provided by the piling contractor. We note that a higher end bearing pressure as well as shaft adhesions for the piles may be able to be calculated using the results of the CPTs, particularly if a limit state design approach is adopted, and data files with the CPT results can be provided on request.

We recommend the piling design be reviewed by a geotechnical engineer prior to construction commencing to confirm appropriate parameters have been adopted. We further recommend the piling records be reviewed by a geotechnical engineer to confirm the pile depths etc. are consistent with the design.

4.4.2 Raft Slab/Piled Raft

As the building loads and load distributions are not known at present it is only possible to give very generalised advice. It appears that at the nominal founding level of about RL 2.0m there is probably a layer of medium dense, possibly very dense sand present to about RL -1m, below which the sand is probably loose, though possibly very loose if the SPT results from BH3 prove to be accurate. Assuming the conditions revealed in the CPT test are more representative of the ground conditions, a raft slab to support moderately heavy loads is likely to be feasible. The bearing capacity of the sands is not usually the critical parameter for raft design and the design will be more controlled by the settlements and allowable deflections in the raft itself. Such issues are best resolved by detailed analysis using 3D finite element software such as Plaxis, and we would be able to complete such work at a more advanced stage of design when the structural loads are known and the raft can be modelled with input from the structural engineer. In the event that a raft bearing only upon the sands at subgrade level does not work within reasonable tolerances, piles can be placed strategically at the most heavily loaded points to control settlements to acceptable limits. It should be noted that raft slabs can affect structures outside the site boundaries due to the load spreading effect. If adjacent structures are founded above or below the proposed basement level special precautions including piling of the proposed structure maybe necessary.

4.5 Basement Floor Slabs

Sandy materials will be present at subgrade level, and we consider the adoption of a slab-on-grade will be feasible.

For a piled building, a working platform will likely be required for installation of the piles. Based on the presence of medium dense sands at subgrade level, such a working platform would be expected to comprise an approximate 0.5m thickness of compacted high strength granular material (e.g. crushed concrete) over a well compacted, proof rolled subgrade. Such a subgrade would be suitable for support of a floor slab, provided that the slab is not connected to piled elements of the structure and relative movement is possible or cracking may occur due to differential movement of the structure. Design of a piling working platform can only be completed once piling rig details are provided by the contractor. If a raft slab is adopted then similar subgrade preparation will be necessary, though the thickness of the granular capping layer will be reduced.

4.6 Groundwater

Given the measured groundwater levels in the monitoring wells ranged between RL 0.3m and RL 0.9m it appears that the proposed basement excavation with a FFL at RL 2.67m should be comfortably above the groundwater level in most conditions. We are not aware what flood levels (if any) are predicted for this area and recommend that council be approached to determine the design flood level.

The drainage system below the slab should be capable of pumping well in excess of nominal seepage/stormwater flows into the garage due to heavy rainfall as an insurance against unpredictable peak water level fluctuations. Waterproofing of basement walls is also a consideration, though as the basement is not a habitable area some seepage would not be an issue as long as it is properly managed with surface and subsurface drainage measures.

4.7 Further Geotechnical Input

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

- Review of recommendations when the level and design of adjoining basements/footing systems are known.
- Further investigation when demolition has been completed.
- Detailed analysis if a raft or piled raft footing system is proposed.
- Detailed analysis of the proposed basement retaining walls.
- Geotechnical review of pile design and pile construction records.

- Geotechnical review of drainage system design.
- Working platform design.

We consider that a meeting of the design team would be fruitful to discuss potential issues and solutions prior to detail design work commencing.

5 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the design and construction phases of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. In the event that any of the advice presented in this report is not implemented, the general recommendations may become inapplicable and JK Geotechnics accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

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

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

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

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

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

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CERTIFICATE OF ANALYSIS 372936

Client Details

Client	JK Geotechnics
Attention	Tom Foster
Address	PO Box 976, North Ryde BC, NSW, 1670

Sample Details

Your Reference	**37307S 92 North Steyne, Manly, NSW**
Number of Samples	3 Soil
Date samples received	13/02/2025
Date completed instructions received	13/02/2025

Analysis Details

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

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

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

Report Details

Date results requested by	20/02/2025
Date of Issue	19/02/2025

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Results Approved By
Jenny He, Inorganic Team Leader

Authorised By
Nancy Zhang, Laboratory Manager

Misc Inorg - Soil				
Our Reference		372936-1	372936-2	372936-3
Your Reference	UNITS	BH1	BH2	BH3
Depth		1.5-1.95	3.0-3.45	9.0-9.45
Date Sampled		13/02/2025	13/02/2025	13/02/2025
Type of sample		Soil	Soil	Soil
Date prepared	-	17/02/2025	17/02/2025	17/02/2025
Date analysed	-	17/02/2025	17/02/2025	17/02/2025
pH 1:5 soil:water	pH Units	7.2	9.2	9.3
Chloride, Cl 1:5 soil:water	mg/kg	<10	<10	10
Sulphate, SO4 1:5 soil:water	mg/kg	<10	<10	20
Resistivity in soil*	ohm m	310	200	140

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

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			17/02/2025	[NT]	[NT]	[NT]	[NT]	17/02/2025	[NT]
Date analysed	-			17/02/2025	[NT]	[NT]	[NT]	[NT]	17/02/2025	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	[NT]	[NT]	[NT]	[NT]	102	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	106	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	[NT]	[NT]	[NT]	[NT]	99	[NT]
Resistivity in soil*	ohm m	1	Inorg-002	<1	[NT]	[NT]	[NT]	[NT]	[NT]	[NT]

Result Definitions

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

Quality Control Definitions

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

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

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

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

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

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

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

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

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

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

Where matrix spike recoveries fall below the lower limit of the acceptance criteria (e.g. for non-labile or standard Organics <60%), positive result(s) in the parent sample will subsequently have a higher than typical estimated uncertainty (MU estimates supplied on request) and in these circumstances the sample result is likely biased significantly low.

Measurement Uncertainty estimates are available for most tests upon request.

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

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

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BOREHOLE LOG

Borehole No. 1

1/1

Client: ALLEN GROUP DEVELOPMENTS PTY LTD

Project: PROPOSED RESIDENTIAL DEVELOPMENT

Location: 92 NORTH STEYNE, MANLY, NSW

Job No.: 37307S **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 5.5m

Date: 7/2/25 **Datum:** AHD

Plant Type: JK400 **Logged/Checked by:** T.F./P.S.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0		FILL: Silty sand, fine to medium grained, grey, with fine to coarse grained igneous and sandstone gravel, trace of brick fragments and root fibres.	D			GRASS COVER APPEARS POORLY COMPACTED
	N = 8 2,4,4							
		1	SP	SAND: fine to medium grained, yellow brown.	M	L		MARINE
	N = 5 2,3,2	2						
	N = 8 3,3,5	3						
		4						GROUNDWATER MONITORING WELL INSTALLED TO 6.0m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 6.0m TO 3.0m. CASING 3.0m TO 0m. 2mm SAND FILTER PACK 6.0m TO 3.0m. BENTONITE SEAL 3.0m TO 1.0m. BACKFILLED WITH CUTTINGS TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.
ON 13/2/25	N = 25 5,9,16					MD		
					W			
		5		as above, but fine to coarse grained, trace of shell fragments.				
	N = 17 2,8,9	6						
				END OF BOREHOLE AT 6.45m				
		7						

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BOREHOLE LOG

Borehole No. 2

1/1

Client: ALLEN GROUP DEVELOPMENTS PTY LTD

Project: PROPOSED RESIDENTIAL DEVELOPMENT

Location: 92 NORTH STEYNE, MANLY, NSW

Job No.: 37307S **Method:** SPIRAL AUGER **R.L. Surface:** ≈ 5.0m

Date: 3/2/25 **Datum:** AHD

Plant Type: JK400 **Logged/Checked by:** T.F./P.S.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
			0		CONCRETE: 110mm.t				7mm DIA. REINFORCEMENT, 62mm TOP COVER
				-	FILL: Silty sand, fine to medium grained, grey and brown, with fine to coarse grained igneous gravel.	M			
		N = 9 3,4,5		SP	SAND: fine to medium grained, yellow brown.	M	L		MARINE
		N = 11 4,5,6	1 – 2				MD		GROUNDWATER MONITORING WELL INSTALLED TO 6.0m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 6.0m TO 3.0m. CASING 3.0m TO 0m. 2mm SAND FILTER PACK 6.0m TO 3.0m. BENTONITE SEAL 3.0m TO 2.0m. BACKFILLED WITH CUTTINGS TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.
		N = 15 4,6,9	3						
ON 13/2/25			4						
ON COMPLET-ION		N = 16 5,8,8				W			CASING ADVANCER TO 6.0m
			5		SAND: fine to coarse grained, yellow brown, with shell fragments.				
			6		END OF BOREHOLE AT 6.0m				
			7						

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BOREHOLE LOG

Borehole No.
3

Client: ALLEN GROUP DEVELOPMENTS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 92 NORTH STEYNE, MANLY, NSW

Job No.: 37307S **Method:** SPIRAL AUGER / WASHBORE **R.L. Surface:** ≈ 5.0m
Date: 3/2/25 **Datum:** AHD
Plant Type: JK400 **Logged/Checked by:** T.F./P.S.

Groundwater Record	Field Tests	Depth (m)	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
		0		CONCRETE: 65mm.t OVER, BRICKS and CONCRETE: 135mm.t				7mm DIA. REINFORCEMENT, 40mm TOP COVER
			-	FILL: Silty sand, fine to medium grained, grey and brown, trace of fine to coarse grained igneous gravel.	W			APPEARS POORLY COMPACTED
	N = 9 4,4,5		SP	SAND: fine to medium grained, yellow brown.	M	L		MARINE
		1						
	N = 10 3,5,5							
		2						
	N = 8 3,4,4	3						COMMENCE CASING ADVANCER AT 3.0m AFTER SPT
		4						
ON 7/2/25 ▼	N = 16 6,8,8				W	MD		
		5						
				SAND: fine to coarse grained, yellow brown, with shell fragments.				
	N = 2 1,1,1	6				VL		
		7						

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BOREHOLE LOG

Borehole No.
3

Client: ALLEN GROUP DEVELOPMENTS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 92 NORTH STEYNE, MANLY, NSW

Job No.: 37307S **Method:** SPIRAL AUGER / WASHBORE **R.L. Surface:** ≈ 5.0m
Date: 3/2/25 **Datum:** AHD
Plant Type: JK400 **Logged/Checked by:** T.F./P.S.

Groundwater Record	Samples (ES / U50 / DB / DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
						SAND: fine to coarse grained, yellow brown, with shell fragments.	W	VL		
	DS	N = 3 1,1,2								
			8							
			9					MD		
	DS	N = 12 4,5,7								
			10							
			11							
			12					VL		NO SPT RETURN
		N = 1 2,1,0								
			13		SM	Silty SAND: fine to medium grained, dark grey.				COLOUR CHANGE IN RETURN TO GREY
	DS	N = 5 1,2,3						L		
			14							

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BOREHOLE LOG

Borehole No.
3

Client: ALLEN GROUP DEVELOPMENTS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 92 NORTH STEYNE, MANLY, NSW

Job No.: 37307S **Method:** SPIRAL AUGER / WASHBORE **R.L. Surface:** ≈ 5.0m
Date: 3/2/25 **Datum:** AHD
Plant Type: JK400 **Logged/Checked by:** T.F./P.S.

Groundwater Record	SAMPLES (ES, U50, DB, DS)	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
						Silty SAND: fine to medium grained, dark grey.	W	L		
			15							
			16							
	DS	N = 22 6,10,12						MD		
			17			as above, but grey and light grey.				
			18							GROUNDWATER MONITORING WELL INSTALLED TO 9.0m. CLASS 18 MACHINE SLOTTED 50mm DIA. PVC STANDPIPE 9.0m TO 6.0m. CASING 6.0m TO 0.1m. 2mm SAND FILTER PACK 9.0m TO 5.9m. BENTONITE SEAL 5.9m TO 2.0m. BACKFILLED WITH CUTTINGS TO THE SURFACE. COMPLETED WITH A CONCRETED GATIC COVER.
			19							
	DS	N > 6 14,6/40mm REFUSAL								
			20			END OF BOREHOLE AT 20.0m				

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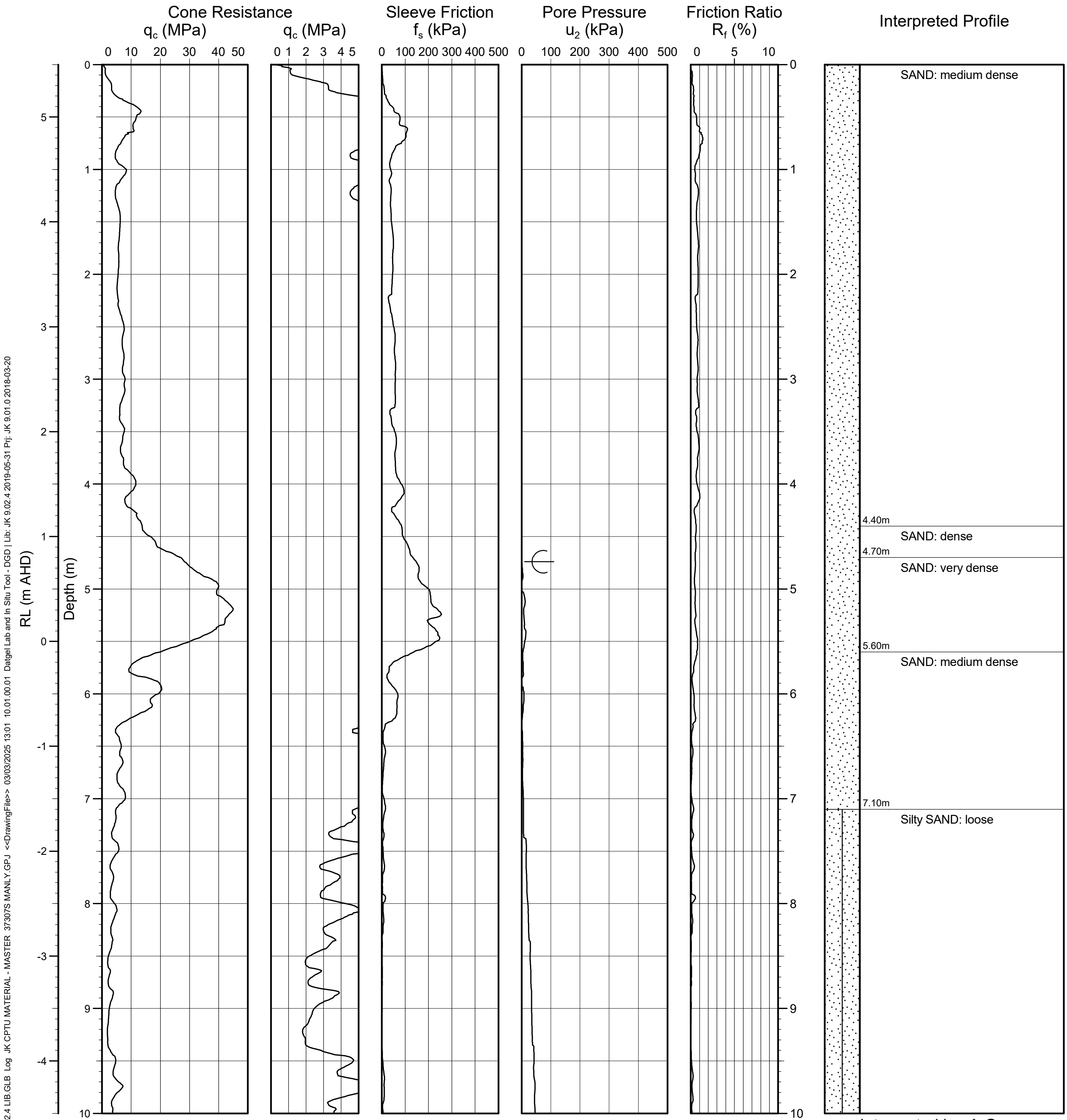
CONE PENETROMETER TEST RESULTS

CPT No.
CPT01

Client: ALLEN GROUP DEVELOPMENTS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 92 NORTH STEYNE, MANLY, NSW

Job No.: 37307S	**R.L. Surface:** ~5.5 m	**Data File:** 37307S Manly
Date: 3/2/25	**Datum:** AHD	**Operator:** B.J.



Interpreted by: A.G.
Checked by: P.S.

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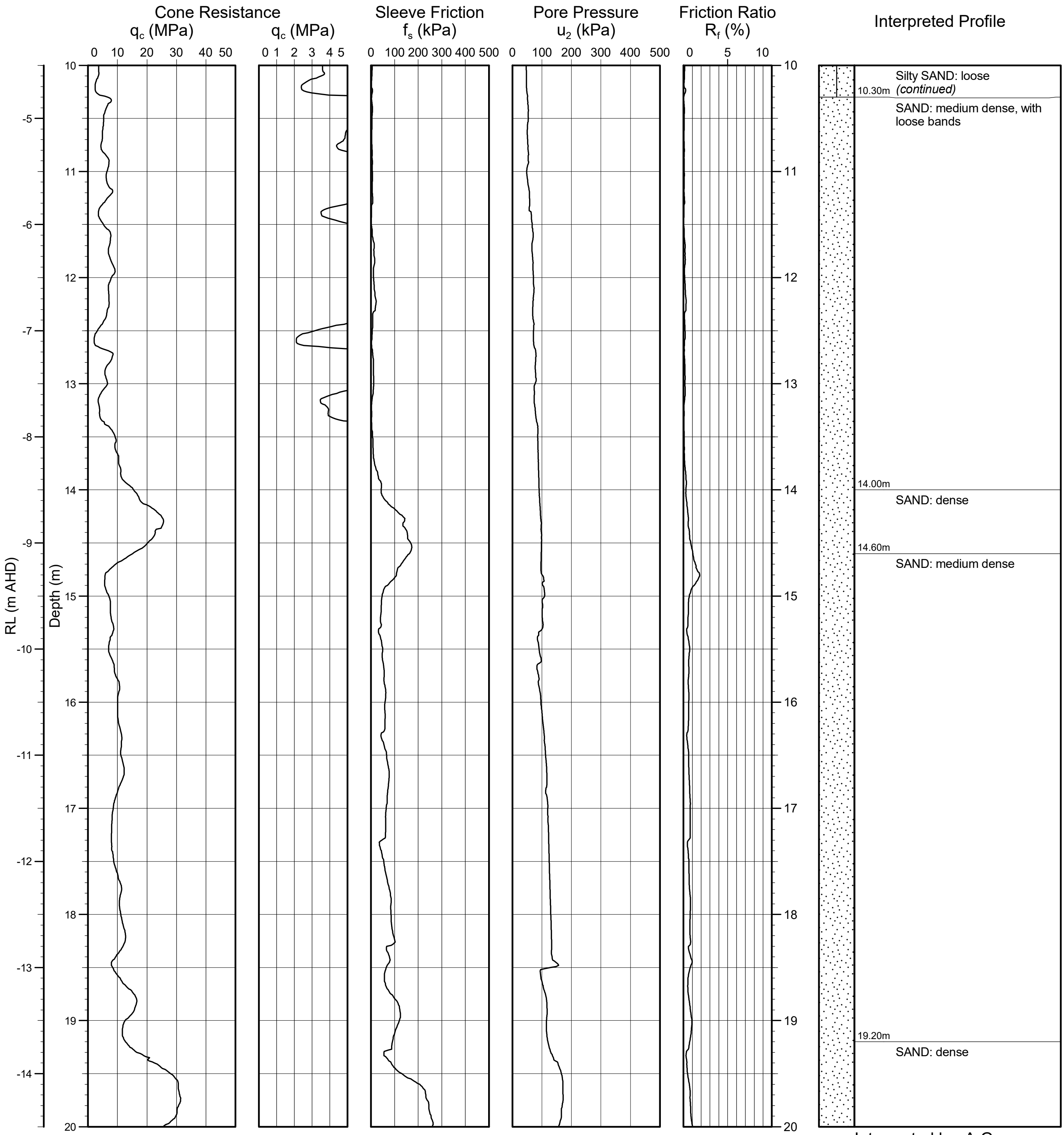
CONE PENETROMETER TEST RESULTS

CPT No.
CPT01

Client:	ALLEN GROUP DEVELOPMENTS PTY LTD
Project:	PROPOSED RESIDENTIAL DEVELOPMENT
Location:	92 NORTH STEYNE, MANLY, NSW

Job No.: 37307S	**R.L. Surface:** ~5.5 m	**Data File:** 37307S Manly
Date: 3/2/25	**Datum:** AHD	**Operator:** B.J.



Interpreted by: A.G.
Checked by: P.S.

JK 9.02.4 LIB.GLB Log JK CPTU MATERIAL - MASTER 37307S MANLY.GPJ <<DrawingFile>> 03/03/2025 13:01 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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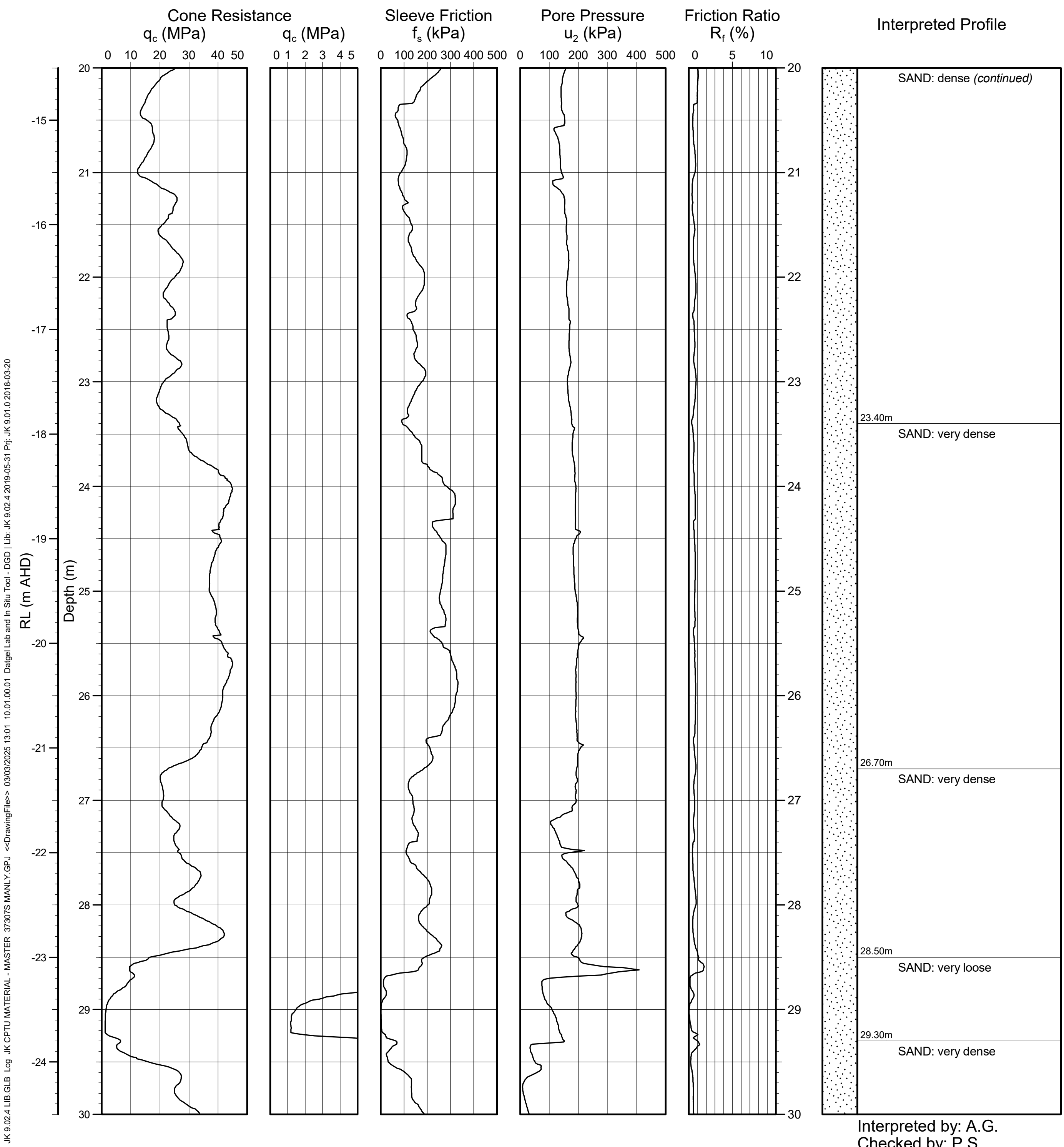
CONE PENETROMETER TEST RESULTS

CPT No.
CPT01

Client: ALLEN GROUP DEVELOPMENTS PTY LTD
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 92 NORTH STEYNE, MANLY, NSW

Job No.: 37307S	**R.L. Surface:** ~5.5 m	**Data File:** 37307S Manly
Date: 3/2/25	**Datum:** AHD	**Operator:** B.J.



Interpreted by: A.G.
Checked by: P.S.

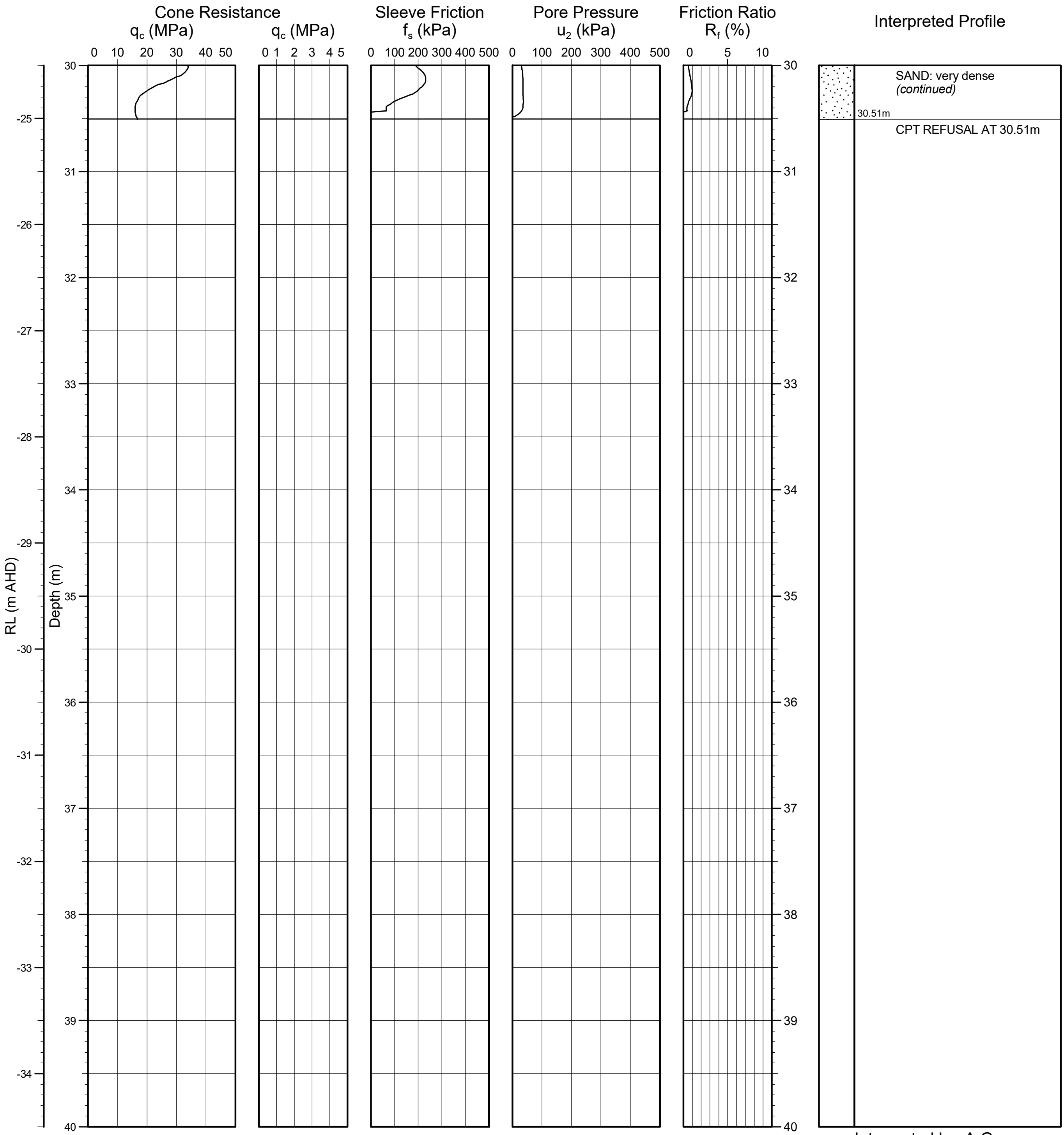
JKGeotechnics

CONE PENETROMETER TEST RESULTS

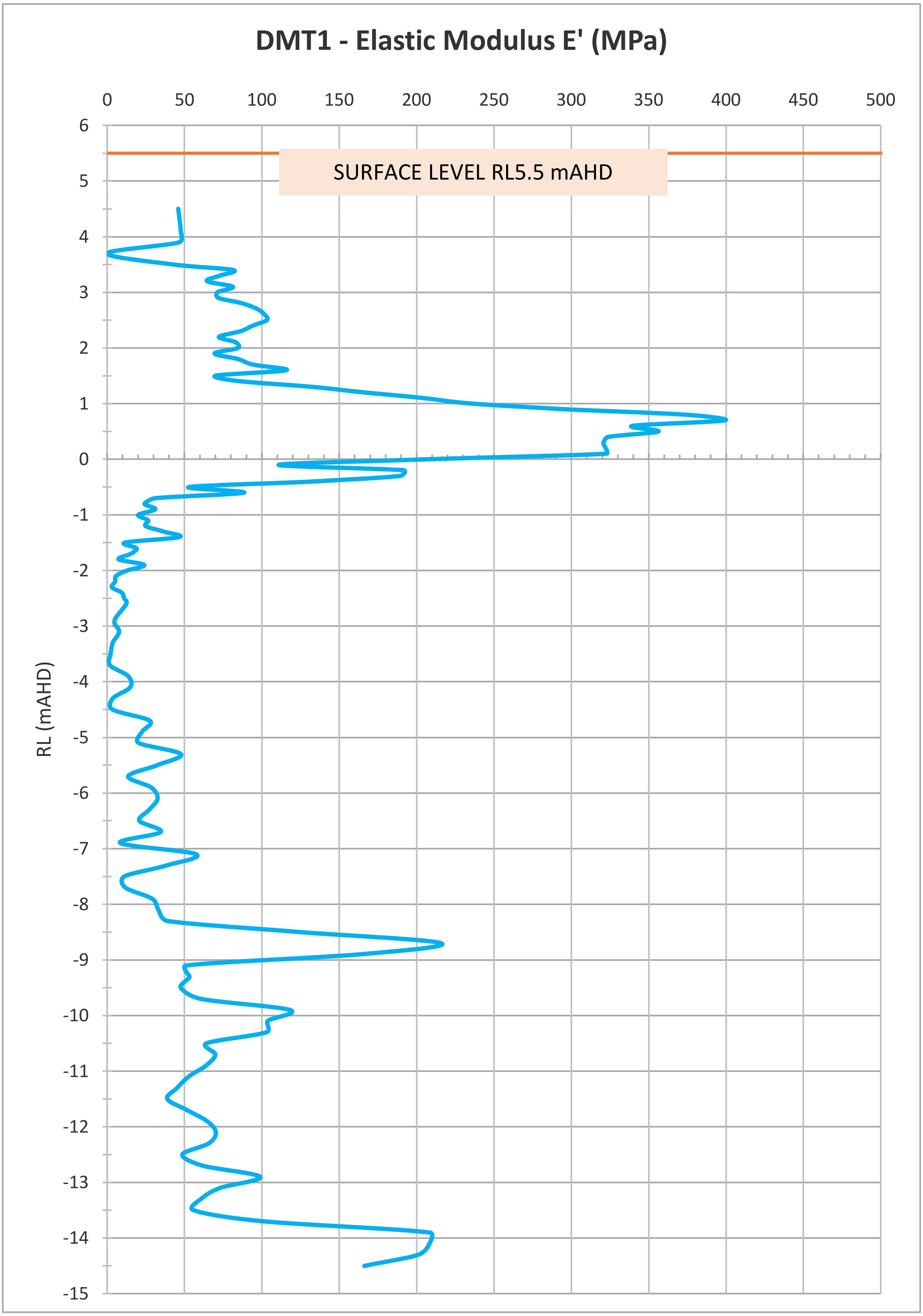
CPT No.
CPT01

Client:	ALLEN GROUP DEVELOPMENTS PTY LTD
Project:	PROPOSED RESIDENTIAL DEVELOPMENT
Location:	92 NORTH STEYNE, MANLY, NSW

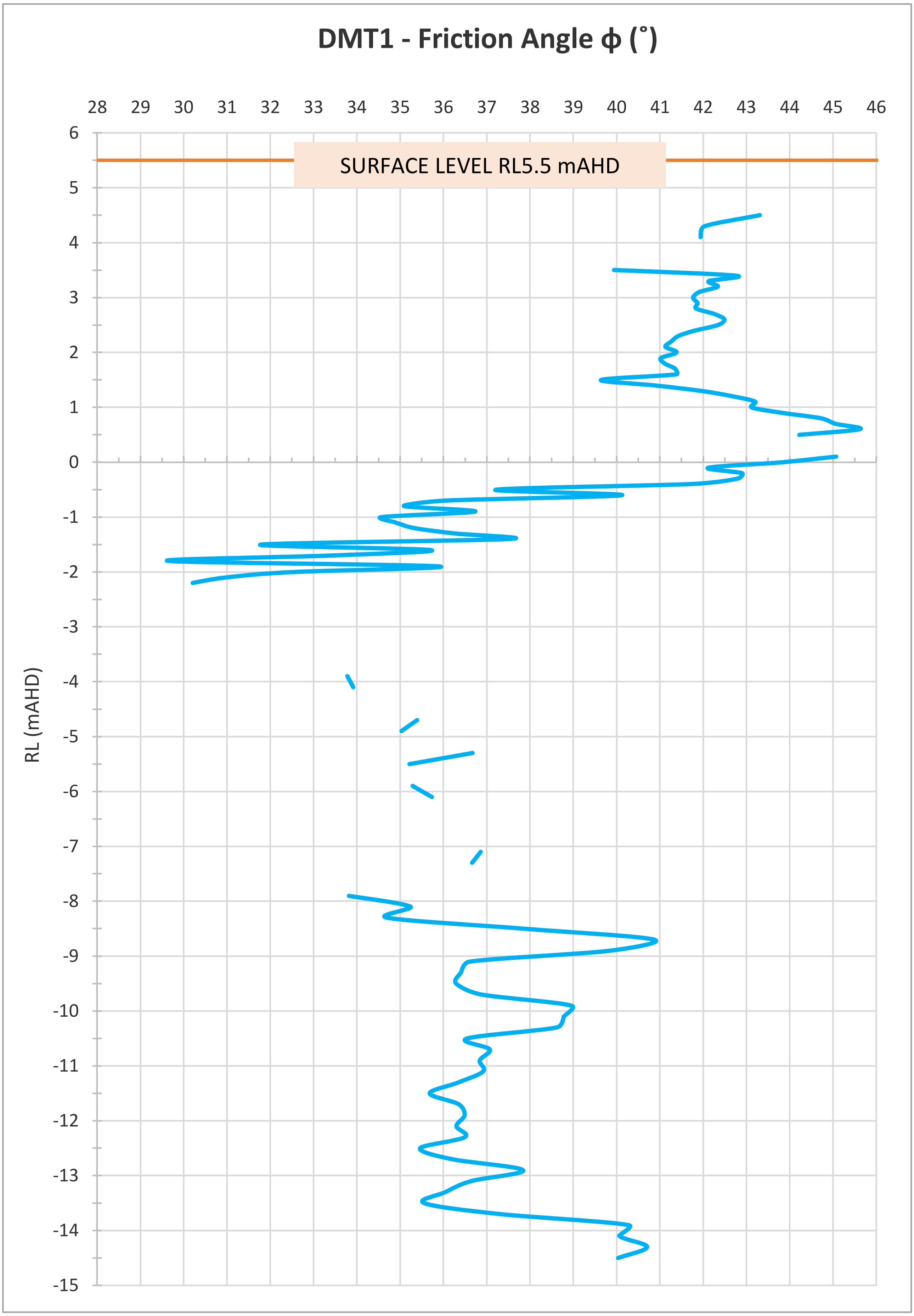
Job No.: 37307S	**R.L. Surface:** ~5.5 m	**Data File:** 37307S Manly
Date: 3/2/25	**Datum:** AHD	**Operator:** B.J.



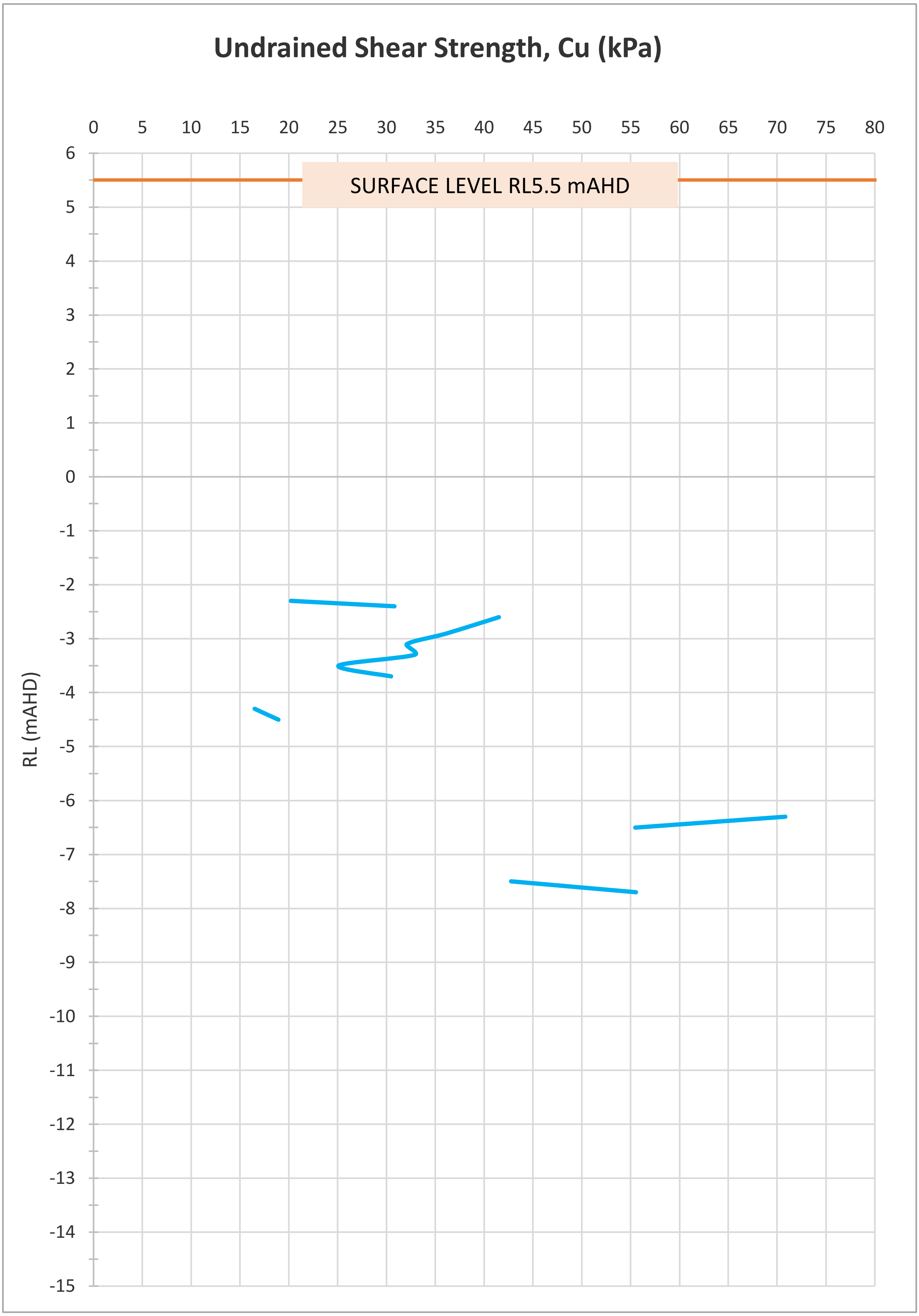
Interpreted by: A.G.
Checked by: P.S.

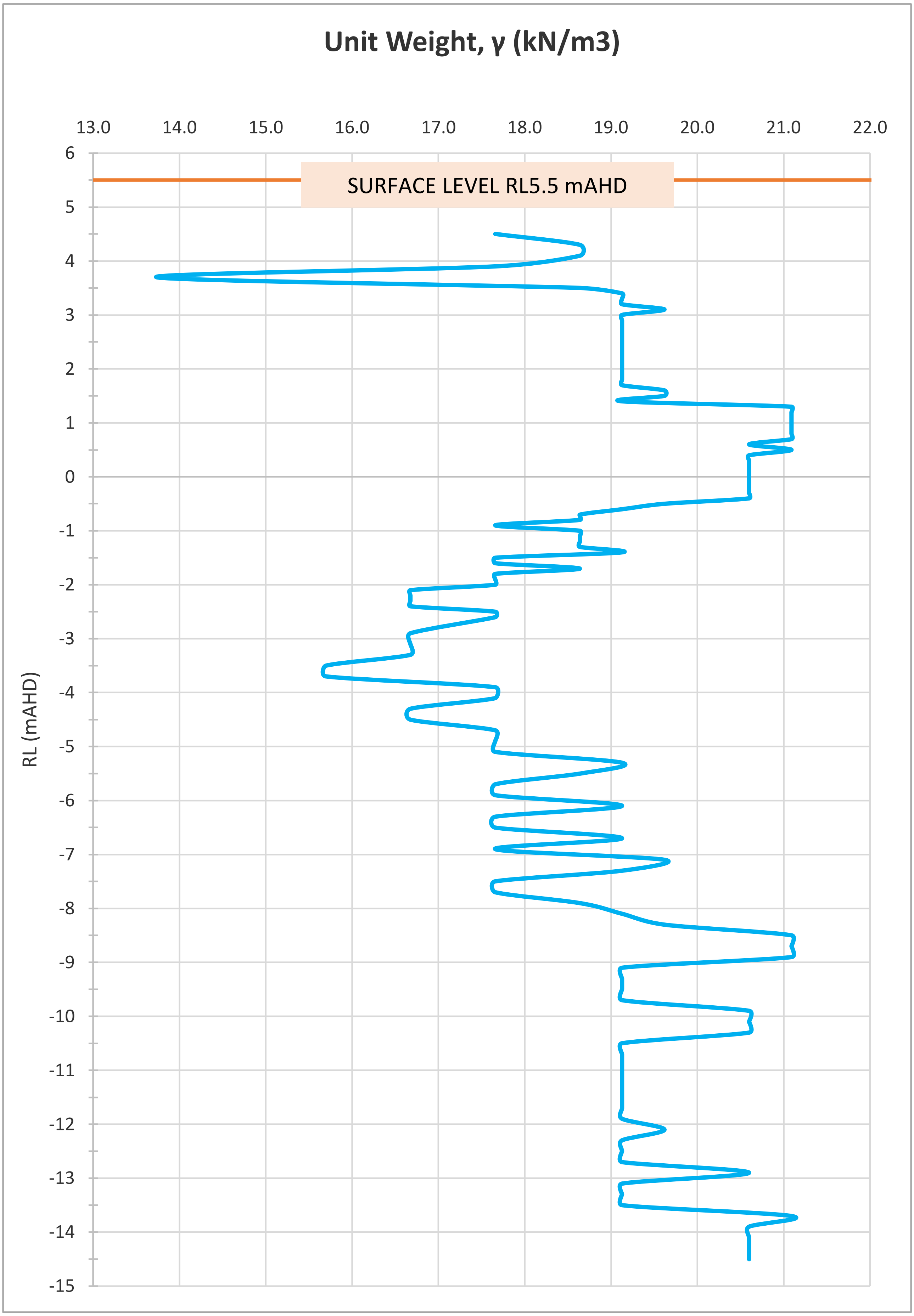


DMT1 - Elastic Modulus E' (MPa)
0
50
100
150
200
250
300
350
400
450
500
6
5
4
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
-11
-12
-13
-14
-15
RL (mAHD)
SURFACE LEVEL RL5.5 mAHD



DMT1 - Friction Angle ϕ (˚)
28 29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46
SURFACE LEVEL RL5.5 mAHD
RL (mAHD)
6 5 4 3 2 1 0 -1 -2 -3 -4 -5 -6 -7 -8 -9 -10 -11 -12 -13 -14 -15


Undrained Shear Strength, Cu (kPa)
0 5 10 15 20 25 30 35 40 45 50 55 60 65 70 75 80
6 5 4 3 2 1 0 -1 -2 -3 -4 -5 -6 -7 -8 -9 -10 -11 -12 -13 -14 -15
SURFACE LEVEL RL5.5 mAHD
RL (mAHD)



Unit Weight, γ (kN/m3)
13.0
14.0
15.0
16.0
17.0
18.0
19.0
20.0
21.0
22.0
SURFACE LEVEL RL5.5 mAHD
RL (mAHD)
6
5
4
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
-9
-10
-11
-12
-13
-14
-15

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

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

Title:	**SITE LOCATION PLAN**
Location:	92 NORTH STEYNE, MANLY, NSW
Report No: 37307S	Figure No: 1

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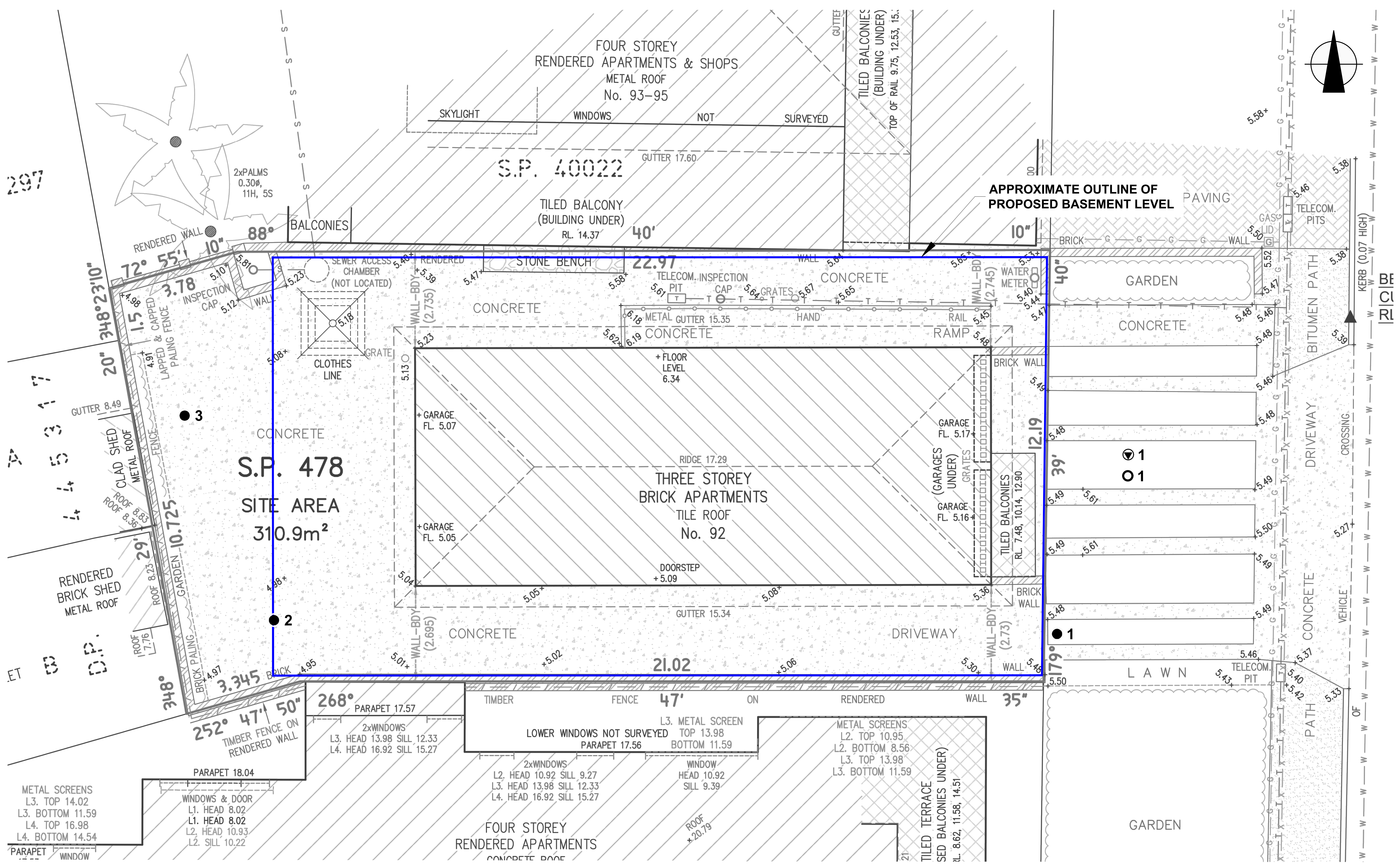
PLOT DATE: 18/02/2025 9:42:02 AM DWG FILE: J:\6F GEOTECHNICAL JOBS\37000'S\37307S MANLY\CAD\37307S.DWG



LEGEND

- ● BOREHOLE
- ▼ CONE PENETROMETER TEST
- ○ DILATOMETER TEST

SCALE 1:100 @A3 METRES

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

Title:	INVESTIGATION LOCATION PLAN
Location:	92 NORTH STEYNE, MANLY, NSW
Report No:	37307S
Figure No:	2

JKGeotechnics

PLOT DATE: 18/02/2025 9:42:12 AM DWG FILE: J:\6F GEOTECHNICAL JOBS\37000'S\37307S MANLY\CAD\37307S.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	

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

Laboratory Classification Criteria

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

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

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

NOTES:

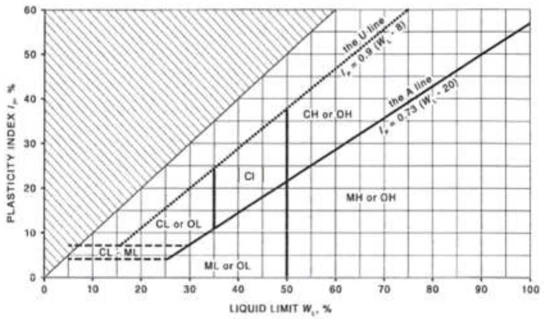
1 For a coarse grained soil with a fines content between 5% and 12%, the soil is given a dual classification comprising the two group symbols separated by a dash; for example, for a poorly graded gravel with between 5% and 12% silt fines, the classification is GP-GM.

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

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

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

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



LOG SYMBOLS

Log Column	Symbol	Definition
Groundwater Record	▼	Standing water level. Time delay following completion of drilling/excavation may be shown.
	C	Extent of borehole/test pit collapse shortly after drilling/excavation.
	▶	Groundwater seepage into borehole or test pit noted during drilling or excavation.
Samples	ES	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.

Log Column	Symbol	Definition	Density Index (I_D) Range (%)	SPT 'N' Value Range (Blows/300mm)
Density Index/ Relative Density (Cohesionless Soils)	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.		

Log Column	Symbol	Definition
Hand Penetrometer Readings	300 250	Measures reading in kPa of unconfined compressive strength. Numbers indicate individual test results on representative undisturbed material unless noted otherwise.

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

Classification of Material Weathering

<table>
<tr><th colspan="2">Term</th><th colspan="2">Abbreviation</th><th>Definition</th></tr>
<tr><td colspan="2">Residual Soil</td><td colspan="2">RS</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.</td></tr>
<tr><td colspan="2">Extremely Weathered</td><td colspan="2">XW</td><td>Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible.</td></tr>
<tr><td>Highly Weathered</td><td rowspan="2">Distinctly Weathered (Note 1)</td><td>HW</td><td rowspan="2">DW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.</td></tr>
<tr><td>Moderately Weathered</td><td>MW</td><td>The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Slightly Weathered</td><td colspan="2">SW</td><td>Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Fresh</td><td colspan="2">FR</td><td>Rock shows no sign of decomposition of individual minerals or colour changes.</td></tr>
</table>

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

Rock Material Strength Classification

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

Abbreviations Used in Defect Description

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