SPECIALIST ADVICE REPORT TO

IGGI ABERASTURI

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED RESIDENTIAL DEVELOPMENT

AT

26 TUPIA STREET, BOTANY, NSW

6 February 2023

Ref: 32491SNrpt rev2

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DOCUMENT REVISION RECORD

Report Reference	Report Status	Report Date
32491SNrpt	Final Report	4 July 2019
32491SNrpt Rev1	Updated to address architectural changes	2 November 2022
32491SNrpt Rev2	Updated to address architectural changes	6 February 2023

This report presents specialist geotechnical advice to inform the structural design of the proposed development. This report is not intended to present a design in accordance with the Design and Building Practitioners Act 2020.

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Borehole Logs 1 to 3 Inclusive

Electronic Friction Cone Penetrometer Test Results (EFCP 1 to 4 Inclusive)

Figure 1: Site Location Plan

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Figure 3: Graphical Borehole Summary

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1 INTRODUCTION

This report presents the results of a previously completed preliminary geotechnical investigation for a proposed residential development at 26 Tupia Street Botany, NSW. The report supersedes our previous report Ref: 21914Wrpt Rev1 dated 23 December 2011, which was prepared based on a previous architectural design. The location of the site is shown in Figure 1. This report was commissioned by Mr Iggi Aberasturi by returned Acceptance of Proposal dated 19 June 2019, on the basis of our proposal Ref: P49590PN dated 27 May 2019.

From the supplied concept architectural drawings (Project No. 6641, Dwg Nos. SK0101 to SK0104, SK0201, SK0202, SK0203, SK1001, SK2002, SK2003, SK2004, SK2005, SK2001, SK2801, 2802, 2803, and 3101 all Issue 01, and SK2806 issue P1) prepared by CotteeParker, we understand the proposed development will comprise three separate 4 storey residential apartment buildings over 2 levels of common basement carparking. The finished floor level of the lowest basement will be at Reduced Level (RL) -1.8m, and excavation to depths between about 4.5m and 7m below existing surface levels is expected to be required.

As no structural loads have been supplied, typical loads have been assumed.

The purpose of the investigation was to obtain geotechnical information on the subsurface conditions as a basis for comments and recommendations on hydrogeology, excavation conditions, shoring options, retaining wall design, and footing design.

An acid sulfate soil investigation has been completed by our specialist environmental consulting division, JK Environments (JKE). Reference should be made to the JKE report for the result of their investigation.

2 INVESTIGATION PROCEDURE

Three boreholes (BH1 to BH3) were drilled to depths between 6.375m and 6.45m using spiral augering techniques with our truck mounted JK550 drill rig. The relative density of the sandy soils was assessed by Standard Penetration Test (SPT) 'N' values. PVC standpipes were installed in each of the boreholes for subsequent groundwater level monitoring using data loggers.

Four Electronic Friction Cone Penetrometer (EFCP) tests (now known as Cone Penetrometer Tests [CPTs]), one adjacent to each borehole and one at an additional location, were conducted to depths between 12.6m and 15.3m. EFCP testing involves continuously pushing a testing probe with a 44mm diameter conical tip into the soil using the hydraulic rams of our ballasted truck mounted EFCP rig. Measurements are made during testing of the end resistance of the cone tip and the frictional resistance of a separate 164mm long sleeve located directly behind the cone. At each test location, a dummy probe was used to probe the first 0.5m depth. Dummy probing involves pushing a blank steel probe into the ground which is used to penetrate through asphaltic concrete surfacing and to check for obstructions where fill is suspected to be present. No data is recorded when using the dummy probe.

EFCP testing does not provide sample recovery. The subsurface material identification, including material strength/relative density, is by interpretation of the test results based on available borehole logs, past experience and empirical correlations. The material identification is 'approximate' and may be subject to site specific correlation with samples obtained from boreholes.

Groundwater observations were made both during and on completion of augering, and during the subsequent installation and removal of the groundwater level data loggers. The groundwater level monitors (Odyssey pressure and temperature type data loggers) were installed in each of the standpipes for a minimum period of 2 weeks recording at 5 minute intervals. The data logger in BH3 was installed for a shorter period than those in BH1 and BH2 due to the malfunctioning of the original data logger. The results from the data loggers have been presented on the attached Figures 5 to 7 inclusive as ground water level against time. Also shown on these figures is the daily rainfall for Sydney Airport, as supplied by the Bureau of Metrology, to the end of March 2007.

Our geotechnical engineers, Mr J Chaghouri and Mr J Kanaan, set out the borehole and EFCP test locations, nominated the sampling and testing locations, and prepared logs of the strata encountered. The borehole logs and EFCP test results sheets, which include field test results and groundwater observations, are attached to this report together with a set of explanatory notes, which describe the investigation techniques, and their limitations, and define the logging terms and symbols used.

The borehole and EFCP test locations, as shown on the attached Investigation Location Plan (Figure 2), were set out by taped measurements from existing surface features shown. The approximate surface level of the boreholes and EFCP tests was estimated by interpolation between the spot levels on the supplied unreferenced survey plan which forms the basis for Figure 1. The datum of the levels is Australian Height Datum (AHD).

Selected samples were tested by Soil Test Services Pty Ltd (STS), a NATA registered laboratory, to determine percentage fines values. The results of the laboratory testing are summarised in Table A. Contamination testing of the site soils was outside the scope of this investigation.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The site report below was prepared based on our inspections in 2007. Review of available Nearmap aerial imagery indicates there has been no significant changes in site development in the intervening period.

The site is located within gently sloping terrain on the northern side of Botany Bay. The site itself graded down to the south west at around 1º to 2º. The site is generally L-Shaped and is accessed from Tupia Street in the north east corner of the site (Figure 1). The site is about 130m wide (east – west) by about 38m deep on the eastern side increasing up to about 95m (north – south) in the main site area.

The site is bounded to the north by a Sydney Water easement which we understand contains a concrete sewer culvert (the main sewerage carrier).

At the time of the 2007 fieldwork, the site was occupied by three separate one and two storey brick and concrete warehouse buildings. The buildings appeared to be in poor condition based on a cursory inspection with cracks up to 5mm wide within the brickwork. Surrounding the buildings were asphaltic concrete surfaced driveway and parking areas which appeared to be in poor condition. Along the site boundaries were numerous trees up to around 15m high.

To the west and south of the site is a grassed and landscaped park area.

To the east of the site was a single storey brick building located around 1m from the site boundary. The building appeared to be in good condition, based on a cursory inspection from within the subject site. An asphaltic concrete surfaced driveway and parking area were also present.

3.2 Subsurface Conditions

The 1:100,000 Geological Map of Sydney indicates that the site is underlain by recent sediments of the Botany Basin which typically comprise marine sands and transgressive dune sand. However, some lenses or layers of clay and/or peat have been encountered on other sites nearby.

The boreholes and EFCP tests disclosed a subsurface profile consisting of pavements and fill overlying sands and silty sands with clay bands. Reference should be made to the attached borehole logs and EFCP test results sheets for detailed descriptions of the subsurface conditions. Summary profiles are presented on the attached Figures 3 and 4. The more pertinent details of the encountered subsurface profile are discussed below:

Pavements and Fill

Asphaltic concrete between 80mm and 180mm in thickness was penetrated from the surface in each of the test locations. Fill consisting of sand, gravelly sand and silty sand was encountered to depths between 0.2m (BH3) and 2.3m (BH1). Based on the SPT and EFCP test results, the fill was assessed as being either poorly or moderately compacted.

Marine Sands

Sands were encountered beneath the fill in all of the boreholes and EFCP tests. However, the sands have been subdivided into two units based on the relative density and experience nearby.

The Upper Sands are predominantly very loose to loose and medium dense and are inferred to have been deposited since the last ice ages. At EFCP3 and EFCP4, very loose and loose sands were encountered to depths of 4.1m and 4.0m respectively. At EFCP1 and EFCP2 the sands beneath the fill were found to be medium dense to dense.

The Lower Sands are predominantly dense to very dense but include interbedded bands or layers inferred to be of clay to sandy clay. The Lower Sands were encountered from about RL-1.5mAHD (EFCP1 and EFCP2), and from beneath the very loose to loose and medium dense sands at EFCP3 and EFCP4 at about RL-3mAHD to RL-4mAHD. These Lower Sands extended to termination of the EFCP tests.

The interbedded bands/layers of clay, silty clay and sandy clay were encountered within the sand profile between about 8.0m and 13.0m depth (about RL-5mAHD to RL-11mAHD) at all of the EFCP test locations. The clayey bands were between about 0.2m and 0.65m thick and are inferred to be of very stiff to hard strengths. Also included within this “banded profile” were some bands inferred to be loose sand to silty and clayey sand up to about 0.6m thick. We have shown on Figure 3 some layers based on correlation between the EFCP test locations. However, the layers may not be continuous and there are bands which could not be correlated between EFCP tests.

We note that EFCP3 and EFCP4 refused in very dense sands at depths of 14.06m and 12.63m respectively. EFCP2 terminated at 15.28m in very stiff sandy clay, which may be another band or may be the commencement of clay strata which are known to generally underlie the sands in this area.

Bedrock in this area is likely to be about 20m to 30m depth based on general mapping of bedrock levels from past investigations.

Groundwater

Groundwater seepages were encountered in all of the boreholes at depths of 2.5m (BH1), 0.8m (BH2) and 0.9m (BH3). On completion of drilling, groundwater was present at depths of 2.1m (BH1), 1.5m (BH2) and 0.9m (BH3).

The data logger results from the standpipes at BH1, BH2 and BH3 (Figures 4 to 6 respectively) show the groundwater level to be slowly dropping over the monitoring period. The rainfall data shows the period of monitoring was relatively ‘dry’ weather with only about 33mm of rainfall over March 2008. Immediately prior to the data logger installation there was a wet period having a total rainfall of 51mm over three days. Earlier in February there had been about 150mm rainfall over 15 days.

The groundwater levels measured by the data loggers dropped from around RL1.4mAHD to between RL1.2mAHD (BH2) and RL1.0mAHD (BH3). The ground water levels indicated a gradient from roughly north to south which is consistent with the regional ground water flow regime to Botany Bay to the south.

3.3 Laboratory Test Results

The percentage fines tests on the disturbed soil samples recovered from the boreholes gave results of either 1% fines (BH1, 3.0m to 3.45m and BH3, 3.0m to 3.45m) or 2% (BH2, 1.5m to 1.95m), indicating that the sandy soils are ‘clean’ with almost no fines.

4 COMMENTS AND RECOMMENDATIONS

The comments and recommendations in the following sections of the report and preliminary nature. As noted in Section 4.1 below, supplementary investigation will be required post demolition to further assess variability in the subsurface profile across the site. Following the supplementary investigation, the comments and recommendations in the following sections of the report must be reviewed and updated as required.

4.1 Geotechnical Design Issues

From the site conditions encountered, a number of geotechnical design issues are apparent and will impact on the design and construction of the proposed development. These issues are outlined briefly below:

- The high ground water table will require that temporary dewatering be conducted during construction and that a tanked basement design be adopted as permanent dewatering will almost certainly not be permitted. The basement wall can be utilised as a cut off to control ground water flow with respect to flow gradients and possibly flow volumes. Additional investigation work will be required to confirm the required depth of cut off wall and/or to determine whether a groundwater 'cut off' can be achieved by penetration into a consistent clay layer, and to address regulatory requirements from both Council and WaterNSW/DPIE. External water pressures will be relatively large and will have an impact on the structural design of the basement perimeter walls and the lower basement floor slab.
- Proximity of the proposed basement walls to the site boundaries and existing adjacent development, including the Sydney Water sewer to the north, will require provision of a basement retention system installed prior to commencement of excavation. Temporary support will require the use of sand anchors or support from within the basement area.
- Previous experience of comparable situations indicates that the process to obtain Sydney Water approval will be very time consuming., and will require detailed analysis to predict the impact on their assets. The analysis process will also likley require input from both civil and structural consultants.
- The expected relatively large column loads will require careful selection of the footing system due to the variable density of the sands and the presence of weak layers, predominantly clays, within the sands. Integration of the structural support system with the measures required to address excavation support and ground water control will offer some economies.
- The variable nature of the sandy soils will require additional investigation following demolition to determine the extent of the clay layers and loose bands to allow for detailed analysis of the footing system and for detailed groundwater modelling.

Solutions to these issues are readily available but will require careful design and construction. The above design issues are discussed in more detail in the following sections.

4.2 Dilapidation Surveys

Prior to demolition commencing, detailed dilapidation reports should be compiled on neighbouring buildings and structures which fall within the zone of influence of the excavation. The zone of influence is generally defined by a distance back from the excavation perimeter of twice the depth of the excavation. The respective owners should be asked to confirm that the reports represent a fair record of actual conditions as they may then be used as a benchmark against which future claims for damage arising from the works. Depending on the final layout of the basement, this will likely include the building to the east and the Sydney Water sewer pipeline. It may be difficult to obtain detailed information on the condition of the sewer carrier; the use of remote controlled cameras will probably be necessary.

4.3 Hydrogeological Considerations

From the data loggers, the groundwater table varied during the period of monitoring from around RL1.4mAHD down to about RL1.0mAHD. The groundwater levels are considered likely to rise above these values following extended wet periods. A rise of up to about 1.5m to 2.0m is considered possible following heavy or prolonged rain periods which would bring the groundwater table to about existing ground level. We note that the lower level of around 1.0mAHD approximately corresponds with high tide level and the groundwater levels are not expected to fall much below this level. The site is too far removed from Botany Bay for any tidal influence as evidenced by the data logger results. Research into water bore levels and basements in the surrounding area may provide further information in relation to groundwater table fluctuations, seepage volumes etc.

Given the above, and considering WaterNSW policy, it is recommended that the basement be tanked and designed to resist hydrostatic uplift pressures corresponding to a maximum water level at ground level. We note that tanked basements require careful design and construction of water stops to achieve a reasonable seal, but that construction limitations mean that some ground water leakage in to the basement is possible.

Based on the investigation results, temporary dewatering will be required from within the proposed basement excavation. The extent of dewatering required will be dependent on the continuity of the clay bands, or otherwise the lack of continuity, across the site and the toe level of the retention system/cut off wall. Application to WaterNSW for a temporary dewatering permit will be required. Consideration will have to be given to water quality (including possible contaminants) before offsite disposal will be permitted. In this regard, we note that the site is within the Botany Groundwater Extraction Exclusion Area, and additional constrains on dewatering may apply.

The basement retaining wall will need to extend to sufficient depth below the basement excavation for the de-watering to be practical. If the wall is not embedded to a sufficient depth, heaving and/or 'boiling' of the sands inside the excavation may occur due to the upward flow of ground water around the cut off wall into the excavation area. As a worst case, if the clay bands are ignored, or are not continuous, then for preliminary design a cut off depth of about 5m below the internal dewatered level will be required to prevent 'boiling' or quick sand conditions. Stability considerations for the wall would also have to be taken into account in determining the required toe level as discussed in Sections 4.6 and 4.7 below.

Significant temporary dewatering would also be required to achieve the required internal dewatering for a stable subgrade and construction access. The temporary dewatering would likely require a combination of spear points (also known as well points) and sumps or deeper dewatering wells. The location of the dewatering points would have to be integrated into the basement design. The dewatering would be required until there is sufficient dead load in the overlying structure to resist the uplift forces. At that time the dewatering points would be permanently capped off. As the uplift pressures will be in the order of 60kPa, there should be sufficient dead weight in the tower buildings to compensate for the uplift. However, there are gaps between the proposed towers where dead weight will not be large at all, the use of anchor piles or similar may need to be considered.

From Figure 3 it can be seen that a cut-off toe level of RL-9mAHD will intersect the clay bands which may form continuous clay layers across the site. If such a layer were continuous then 'boiling' conditions would be unlikely and seepage inflow volumes would be significantly reduced. However, ground water pressures would be high beneath such continuous clay layers such that hydraulic uplift or base heave would be of concern. Assuming an indicative ground level, and hence maximum ground water level, of RL+3mAHD and a basement bulk excavation level of RL-3mAHD, then uplift will not be of concern if the continuous clay layer were deeper than about RL-9mAHD to RL-11mAHD. If the continuous clay layer were above this level, then hydraulic uplift conditions would be addressed by provision of temporary pressure relief wells installed through the clay layer at regular spacing over the basement area. The required spacing is a function of the well size and level of the clay layer.

Additional investigation work will be required to confirm the cut off depth required. In particular, if the clay bands at around 8m to 10m depth (about RL-5mAHD to RL-11mAHD) can be confirmed to be continuous across the site, then design may utilise this layer as part of the cut off system. If the clay bands are not continuous, the wall will need to be embedded to greater depth and the dewatering system would be required to handle higher water volumes due to seepage flows below the toe of the wall.

Additional investigation of groundwater table fluctuations and detailed groundwater modelling would be required to determine possible drawdown effects of temporary dewatering. Further advice should also be provided as to whether precautions are required to reduce any possible adverse effects on any surrounding buildings and structures which may arise from drawdown due to the temporary dewatering. To this end, further details of footing systems for adjacent structure would assist in evaluation of requirements and should be sought.

Detailed modelling of groundwater flows with and without the basement would be required to determine the likely effects of the basement cut-off wall on groundwater flows in the area. Such a cut off can lead to mounding, or an increase in ground water levels on the up gradient side of the basement wall (in relation to the regional flow direction). Such an increase in ground water level may have an adverse effect on adjacent structures or properties, such as by causing surface seepage or localised settlements. Provision of subsurface interceptor drains and by-pass pipes through or around the basement can overcome such problems. We can complete this modelling if commissioned to do so.

4.4 Excavation

We understand the basement excavation is to be between about 4m and 7m deep below existing ground surface levels. The excavation is expected to encounter fill and marine sands.

Excavation of the fill and sands should be readily achievable using conventional means such as the buckets of hydraulic excavators. Due to the presence of very loose and loose sands and poorly compacted fill, which probably extend beyond the site boundaries, we recommend that tracking of hydraulic excavators and other plant be carried out with caution. Sudden stop/start movements may result in vibration damage to close by neighbouring buildings and structures, say within about 20m from the vibration source.

Where space permits, temporary batter slopes of 1.5H:1V are recommended in the short term above the water table, provided no surcharge loads, including construction loads are placed at the top of the batters. Batters would not be feasible below the water table due to the collapsing nature of the sands resulting in relative flat batters. Given the combination of high ground water level and proximity of most of the basement walls to the site boundary and existing trees (which we expect are to be retained), we would expect that temporary batters will not be an option.

4.5 Retention Options

In lieu of temporary batter slopes, the excavation sides will need to be supported by a properly designed temporary and/or permanent shoring system, installed prior to the start of excavation. The effect of ground movement on any buildings or structures which 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 2H (where H is the excavation depth in metres) behind the wall. This shoring system could be incorporated into the permanent basement retention system and ground water cut off.

Suitable shoring systems would include secant pile walls, vibratory installed steel sheet piles, diaphragm wall, and a soil mixed/jet grouted wall. A contiguous pile wall would not be suitable due to the inevitable gaps which will result in ground water inflow. Conventional bored piles are unsuitable due to the collapsing nature of the sands.

Secant pile walls involve the drilling of ‘soft’ piles (which are concrete with a strength gain retardant) at around 1.5 times pile diameter spacing with ‘hard’ piles then drilled between the 'soft' piles cutting into the 'soft' piles to either side. If delays are encountered when installing the 'hard' piles, then the 'hard' piles may not fully intersect the 'soft' piles over the required full depth and the wall may not be watertight. Pile misalignment, which is common below depths of about 6m, will result in gaps which may not be visible but will allow much higher water inflows that would otherwise be expected. Additionally, should there be excessive movement of the wall during construction, separation of the 'hard' and 'soft' piles may occur which would result in the wall not being watertight. Due to the collapsing nature of the sandy soils encountered on site, bored piles would not be suitable, and a continuous flight auger (CFA) pile system will be required if a secant pile wall system is to be adopted. Cased CFA (double rotary) piling techniques provide better pile alignment and hence interlock when compared to conventional CFA techniques.

A steel sheet pile wall involves installing interlocking steel sheets using a vibrating motor. As the steel sheets piles are not suitable in the long term due to corrosion of the steel, a permanent wall would need to be constructed in front of the sheet piles during construction. The use of steel sheet piles should be approached with caution due to the potential for vibration induced damage to any nearby structures during pile installation. Use of special amplitude modulated vibrators (rather than frequency modulated vibrators) would likely be possible, but would require a site trial to confirm the resulting vibration levels at the critical structures are acceptable.

A diaphragm wall involves the cutting of a narrow vertical trench, typically of about 0.5m to 0.6m wide and in short sections, say 5m to 10m long, to the required depth. The trench walls are stabilised by keeping the trench full of bentonite slurry. Reinforcing steel is then placed in the slurry filled trench and concrete poured using tremie techniques which displaces the slurry. A waterproofing seal is placed between the panels of the wall. The shallow depth to the groundwater table may preclude the use of this system as there may not be sufficient slurry pressure above the groundwater table to maintain stability of the trench walls. Provision of temporary bunds to raise the slurry level would be possible. The advantages of the diaphragm wall are the high structural capacity and stiffness possible, together with relatively planar exposed face and good water tight integrity. Cost and the large amounts of site space required for the necessary plant are usual disadvantages. This site would appear to have ample site space for the plant.

Jet grouting involves the injection and mixing of cement into the insitu soils at high pressures using a specialised drilling rig. Insitu Cutter Soil Mixing (CSM) is similar but mixes cement into the soil by physically cutting and mixing the soil using a specialist rig. The process is carried out around the basement perimeter to form a continuous wall/line. To achieve the required structural design capacity, reinforcing steel or structural steel columns/beams have to be inserted into the mixed columns, or CSM panels. Alternatively jet grouted columns can be formed between CFA piles which form the structural support element. These systems avoid the possible problems of separation associated with drilling of secant piles. However, for jet grouting, the mixing has be carefully controlled to achieve the required treated diameter for continuity. In addition, the exposed face has to be trimmed to the required line as part of the excavation procedure. CSM panels form relatively regular 'flat' faces, but treatment is usually required, e.g. shotcrete facing, in the permanent case.

Selection of the most suitable basement wall system will depend on how the wall is integrated into the structural support system, construction times, construction constraints and cost. Experience has shown that CSM walls are usually adopted on sites similar to this one.

To reduce excavation induced movements, especially adjacent to site boundaries, the shoring system must be provided with adequate lateral support, such as by bracing or anchoring, as the excavation progresses.

We assume that permanent lateral support of all retaining walls will be provided by the proposed concrete floor slabs.

The retaining walls should be socketed below the base of the excavation for an appropriate design depth to maintain wall stability, taking into account unintended over excavation, local footings, lift pits and service excavations. In addition the ground water cut off requirements must be taken into account (as discussed in Section 4.3 above).

Caution should be exercised during retaining wall installation to the potential for disturbance of the upper loose and very loose sands and poorly compacted fill, and hence, the possible settlement of any shallow footings in the immediate vicinity of the works. For this development, both the site to the south-east and Sewer Main to the north may be a direct constraint in this regard. Pre-treatment of the ground surface by grouting may be required.

4.6 Lateral Earth Pressures

For the preliminary design of anchored or propped walls, where wall deflection is not critical, we recommend that a rectangular lateral earth pressure distribution of magnitude 4H kPa be used (where H is the depth in metres of the excavation), provided that there are no movement sensitive structures within a lateral distance of 2H from the top of the excavation.

For areas that are sensitive to lateral movement, such as due to the presence of adjacent buildings and/or services, a rectangular lateral earth pressure distribution of magnitude 8H kPa should be used for both the temporary and permanent cases to limit possible ground movements.

All appropriate surcharge loads, including from adjacent footings, should be incorporated in the design of the retaining walls. Full hydrostatic pressures to the higher levels specified in Section 4.3 should be incorporated into the design of the retaining walls.

We emphasise that some ground movements (settlement and/or lateral displacement) will occur within the zone of influence even with an anchored or propped wall that is designed for the higher lateral pressures. The amount of movement is a result of the combined effects of structural stiffness, sequence of construction and quality of construction. Further detailed design studies would be required to quantify likely movements. We note also that detailed design using specialist retaining wall programs, such as WALLAP or finite element programs, such as PLAXIS, may enable alternative design earth pressure distributions to be considered. Such designs are able to take into account specific construction sequence, wall and support stiffness, and give predicted movements and bending moments/shear forces for structural design. We can complete such design studies if so commissioned.

4.7 Lateral Restraint

Support for the basement wall can be achieved by provision of earth anchors (external to the basement), or propping and strutting from within the basement area.

Toe support for the wall embedded into the sandy soils below the depth of any excavation, including footings or trench excavations, can be designed using a passive earth pressure coefficient (K_p) of 3.0 but with a factor of safety of 2 to limit the large deformations which are required to develop full passive pressures. A bulk unit weight of $18kN/m^3$ should be used for the natural sandy soils above the water table. The internal ground water level must be taken into account as it reduces the available lateral resistance.

Anchors with their bond length in medium dense to dense sand can be designed based on an effective friction angle (ϕ') of 35º subject to the following:

- Anchor bond length of at least 3m behind the 'active' zone of the excavation, taken as above a 45º line from the base of the excavation.
- Overall stability, including anchor group interaction, is satisfied.
- Design of the anchor bond zone takes into account possible higher ground water levels as outlined in Section 4.3.
- Anchor installation uses appropriate techniques to minimise ground loss and ground disturbance taking into account the difficulties of drilling within sands below the ground water table. Provision should be made for a seal at the temporary anchor penetration through the wall to control ground water inflow and possible inflow of sands with ground water. To confirm these issues have been adequately addressed, a work method statement detailing the equipment, materials and step by step procedures proposed by the anchoring contractor should be provided for review prior to commencement.
- All anchors are proof loaded to at least 1.3 times the design working load before locking off at the working load. All proof stressing should be witnessed by an experienced engineer independent of the anchoring contractor.
- Suitable provision is made for long term water proofing of anchor penetrations through the basement wall once anchors are de-stressed.

Consideration could be given to specialised mechanical anchors, such a screw plate piles or "platypus" type anchors, as an alternative to conventional drilled and grouted anchors. However, most of the above design and construction requirements would still have to be addressed.

It should be noted that the approval of neighbouring landowners would be required if anchors are to extend below their land. Permanent anchors, if required, would require appropriate corrosion protection.

Internal strutting and propping may be possible provided due consideration is given to the construction sequence and resulting impacts of the necessary support berms and struts on construction access. The dewatering system requirements can also affect the design and construction of such systems. Such a system would be more difficult to design and install where potential wall movements are a design constraint, as such systems are usually more flexible.

A variation on the internal support would be to consider 'top down construction' where the permanent floor slabs are utilised for both temporary and permanent support. Floor slabs are cast on grade and then the sand removed from under the slab for the next basement level. Detailed consideration is required for the dewatering requirements, construction sequence and access constraints, including ventilation requirements when excavating below the slab above.

4.8 Footing Design

Selection of a suitable footing system should be integrated with the basement wall types, wall support and dewatering requirements.

Provision of a tanked basement will require the basement floor slab to be designed for relatively high uplift pressures of about 60kPa depending on the slab soffit level and ground level. As a result, a relatively thick lower basement floor slab is likely to be required to distribute the uplift loads. Hence, consideration could be given to utilising this slab as part of the footing system such as a stiffened raft slab, or piled raft to distribute the column loads. These footings would be feasible given the dense sand conditions encountered at or just below bulk excavation level. In addition, the structural loads are, in effect, reduced by the buoyancy effect of the high ground water levels. Design would have to take into account the possible fluctuations in levels during both the temporary and permanent stages. A piled raft is likely to be the most economic combination of basement slab and structural support for the development. However, additional structural and loading information would be required for the analysis of these options which has not been allowed at this stage. However, we can complete this analysis if commissioned to do so. Additional investigation works would also be required to confirm similar subsurface conditions between the completed test locations.

Due to the high anticipated column loads and the need for a tanked slab, shallow footings founded within the natural sands at bulk excavation level are not considered to be suitable for the development.

Alternatively, a piled system may be used. However, details of the building loads would be required to evaluate pile diameters, founding depths and settlements. We note that the variable relative density profiles found by the EFCP tests, and in particular the presence of clay bands and loose zones interbedded in the sand profile, will be a constraint for maximising pile capacity.

As a preliminary guide, drilled piles 0.6m in diameter founded in the medium dense, or better, sands and to at least eight pile diameters below bulk excavation level, with no loose sand bands or clay bands within five pile diameters of the base of the pile may be designed for an allowable end bearing pressure of 1,200kPa based on serviceability. Higher pressures may be possible at specific locations and levels provided continuity of the founding stratum is verified. Preferred pile types would include continuous flight auger (CFA) piles, barrettes constructed with CSM columns, and possibly steel screw piles. The total expected settlement for single piles under this load would be about 5mm with expected differential settlements up to about 3mm. Further consideration would be required for pile group interaction effects. If steel screw piles are to be used, provision would need to be made for the long term, corrosion protection of the piles.

Given the caving conditions, conventional bored piles would not be suitable. Consideration could be given to use of casing and/or bentonite support but this would most likely only be economic if a diaphragm wall system is being used.

Driven or vibratory installed piles may not be suitable due to the potential for damage to nearby buildings and services. However, given the size of the site and distance to nearby structures, these pile types may be feasible over much of the site. Further advice would be required from specialist piling contractors in relation their system and resultant expected vibrations. Such piles would likely have a higher load capacity than drilled piles.

Consideration should be given to subsequent investigations extending to greater depths to confirm whether alternative founding strata can be identified below the current investigation depth.

4.9 Basement On-Grade Slab and External Pavements

Although the basement slab will act as a garage pavement, it is likely to be quite thick due to hydrostatic uplift, and hence traffic loadings will not be critical. Consideration should be given to placing a granular sub-base type material to assist with site trafficability over the dewatered lower basement subgrade. All basement slab joints will need to be provided with appropriate water stops and structural connection to walls and columns.

Any external pavements may be design based on a preliminary CBR of 10% and should have a subbase layer of at least 100mm thickness of crushed rock to RTA QA specification 3051 unbound base material. The subbase should be compacted to at least 98% of Modified Maximum Dry Density (MMDD). Concrete pavements should be designed with an effective shear transmission at all joints by way of either dowels or keyed joints.

All external pavements should have the subgrade prepared by compacting to at least 98% SMDD or minimum density index of 70% after proof rolling using at least five passes of a minimum 10 tonne dead weight drum roller. The proof rolling should be witnessed by an experienced earthworks foreman or engineer. The purpose of the proof rolling is to identify possible soft spots which would not otherwise be obvious and may have an adverse impact on the pavement construction and performance. Over excavation and replacement may be required for soft spots so identified.

4.10 Further Geotechnical Input

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

- Dilapidation reports on neighbouring properties and structures that fall within the movement influence zone of the basement excavation.
- We believe that a meeting of the design team would be fruitful, once the DA design is approved and during design and construction documentation stage, in order to discuss geotechnical issues and solutions in more detail.
- Council may require a geotechnical inspection and test plan be prepared once the design is complete.
- Additional investigation and analysis to determine the following;
  - Continuity across the site of the clay bands located at around RL-5mAHD to RL-10mAHD.
  - Confirm subsurface conditions between the completed test locations.
  - Possible alternative deeper founding strata.

- Long term groundwater conditions and fluctuations.
- Groundwater modelling to confirm the dewatering, pressure relief, long term ground water effects and to address authority requirements.
- Analysis of the perimeter retention system and construction sequence.
- Analysis and design of a stiffened raft or piled raft footing system, if required.
- Analysis of load and settlement characteristics of piles and pile groups relative to design loads.

5 GENERAL COMMENTS

The proposed development is considered to be feasible for this site. However, the geotechnical issues associated with the subsurface conditions require careful design and integration with the structural solutions and with construction methods and sequence. The construction works below ground level should not be regarded as a conventional basement and footing construction project. The geotechnical issues require the application of "civil engineering" methods and will require an experienced builder and a well integrated design. Exploration of various design and construction options is likely to result in a well designed, economic solution to all the issues.

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

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

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

A waste classification will need to be assigned to any soil excavated from the site prior to offsite disposal. Subject to the appropriate testing, material can be classified as Virgin Excavated Natural Material (VENM), General Solid, Restricted Solid or Hazardous Waste. Analysis takes seven to 10 working days to complete, therefore, an adequate allowance should be included in the construction program unless testing is completed prior to construction. If contamination is encountered, then substantial further testing (and associated delays) should be expected. We strongly recommend that this issue is addressed prior to the commencement of excavation on site.

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

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Macquarie Park, NSW 2113
PO Box 976
North Ryde, Bc 1670
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

Ref No:21914W

TABLE A
SUMMARY OF PERCENTAGE FINES TEST RESULTS

AS 1289	TEST METHOD	3.6.1
BOREHOLE NUMBER	**DEPTH**	**PERCENTAGE FINER THAN 0.075mm**
	m	**%**
1	3.00-3.45	1
2	1.50-1.95	2
3	3.00-3.45	1

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Borehole No.
1
1/1

BOREHOLE LOG

Client: IGNACIO ABERASTURI
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 26 TUPIA STREET, BOTANY, NSW

Job No. 21914W
Date: 27-2-08
Method: SPIRAL AUGER JK550
Logged/Checked by: J.C./
R.L. Surface: ≈ 3.3m
Datum: AHD

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
			0			ASPHALTIC CONCRETE: 180mm.t				
	DS	N = 11 6,6,5			-	FILL: Sand, fine to medium grained, with fine to coarse grained igneous gravel.	D	-	-	APPEARS MODERATELY COMPACTED
ON COMPLETION	DS	N = 5 3,2,3	1			FILL: Silty sand, fine to medium grained, dark brown, with shell fragments, root fibres and sandstone gravel.	M			APPEARS POORLY COMPACTED
	DS	N = 13 5,5,8	2 3		SP	SAND: fine to medium grained, light brown and dark brown.	W	MD	-	
	DS	N = 30 6,12,18	4 5			SAND: fine to medium grained, light brown.		D		50mm DIA. PVC STANDPIPE INSTALLED TO 6m, SLOTTED BETWEEN 2m AND 6m. CAST IRON GATIC COVER AT SURFACE
	DS	N > 46 7,32, 14/75mm REFUSAL	6					VD		
			7			END OF BOREHOLE AT 6.375m				

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BOREHOLE LOG

Borehole No.
2
1/1

Client: IGNACIO ABERASTURI
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 26 TUPIA STREET, BOTANY, NSW

Job No. 21914W
Date: 27-2-08
Method: SPIRAL AUGER JK550
Logged/Checked by: J.C./
R.L. Surface: ≈ 2.8m
Datum: AHD

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
			0		-	ASPHALTIC CONCRETE: 80mm.t				
						FILL: Gravelly sand, fine to coarse grained, grey, fine grained igneous gravel.	M	-	-	
		N = 13 6,7,6			SP	SAND: fine to medium grained, brown, with root fibres.	M	M	-	
			1			SAND: fine to medium grained, light brown.				
ON COMPLETION		N = 10 2,5,5					W			
			2			SAND: fine to medium grained, light brown, with sandstone gravel.		L		
		N = 7 2,3,4	3							
			4					MD		
		N = 18 5,8,10	5							50mm DIA. PVC STANDPIPE INSTALLED TO 6m, SLOTTED BETWEEN 2m AND 6m. CAST IRON GATIC COVER AT SURFACE
		N = 54 9,24,30	6					VD		
						END OF BOREHOLE AT 6.45m				
			7							

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BOREHOLE LOG

Borehole No. **3**
1/1

Client: IGNACIO ABERASTURI
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 26 TUPIA STREET, BOTANY, NSW

Job No. 21914W
Date: 27-2-08
Method: SPIRAL AUGER JK550
Logged/Checked by: J.C./
R.L. Surface: ≈ 1.7m
Datum: AHD

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
ON COMPLETION			0			ASPHALTIC CONCRETE: 120mm.t				
					-	FILL: Gravelly sand, fine to coarse grained, grey, fine to medium grained igneous gravel.	M	-	-	
		N = 2 2,1,1			SP	SAND: fine to medium grained, light brown.	-	VL	-	
			1			SAND: fine to medium grained, dark grey, with shell fragments.	W			
		N = 9 3,4,5						L		
			2			SAND: fine to medium grained, light brown and dark brown.				
		N = 2 SUNK, 2	3					VL		
			4			SAND: fine to medium grained, dark brown.				
		N = 45 9,20,25				SAND: fine to medium grained, dark brown and light brown.		D		50mm DIA. PVC STANDPIPE INSTALLED TO 5.45m, SLOTTED BETWEEN 1.45m AND 5.45m. CAST IRON GATIC COVER AT SURFACE
			5			SAND: fine to medium grained, light brown.				
		N = 32 4,11,21	6							
						END OF BOREHOLE AT 6.45m				
			7							

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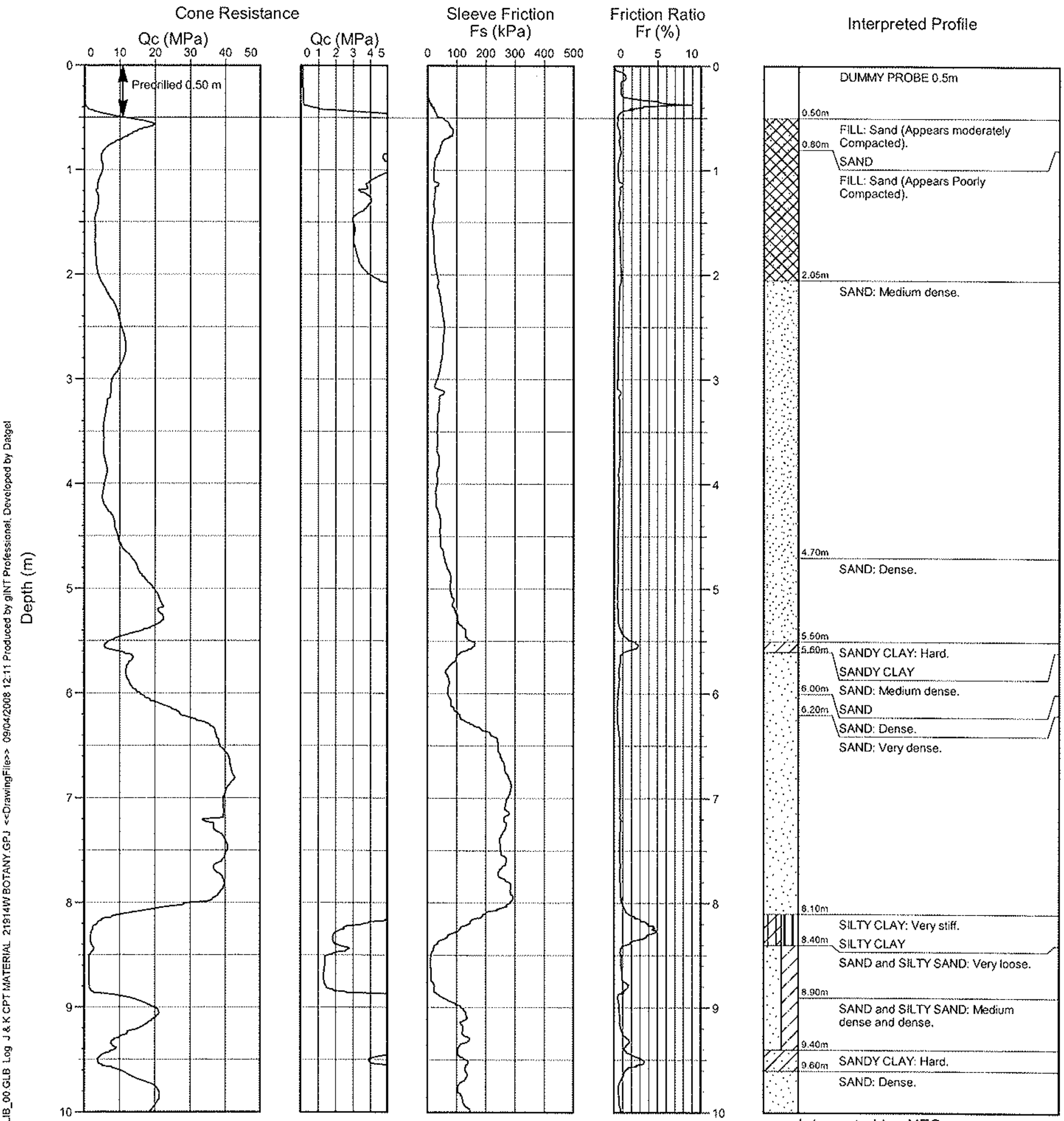
EFCP No. 1
1 / 2

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client: IGNACIO ABERASTURI
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 26 TUPIA STREET, BOTANY, NSW

Job No.: 21914W	**R.L. Surface:** 3.3 m	**Data File:** 21914W_1.GEF
Date: 27/02/08	**Datum:** AHD	**Operator:** JMK



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Checked by: [signature]

JK_LIB_00.GLB Log J & K CPT MATERIAL 21914W BOTANY.GPJ <<DrawingFile>> 09/04/2008 12:11 Produced by gINT Professional, Developed by Datgel

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EFCP No.
1
2 / 2

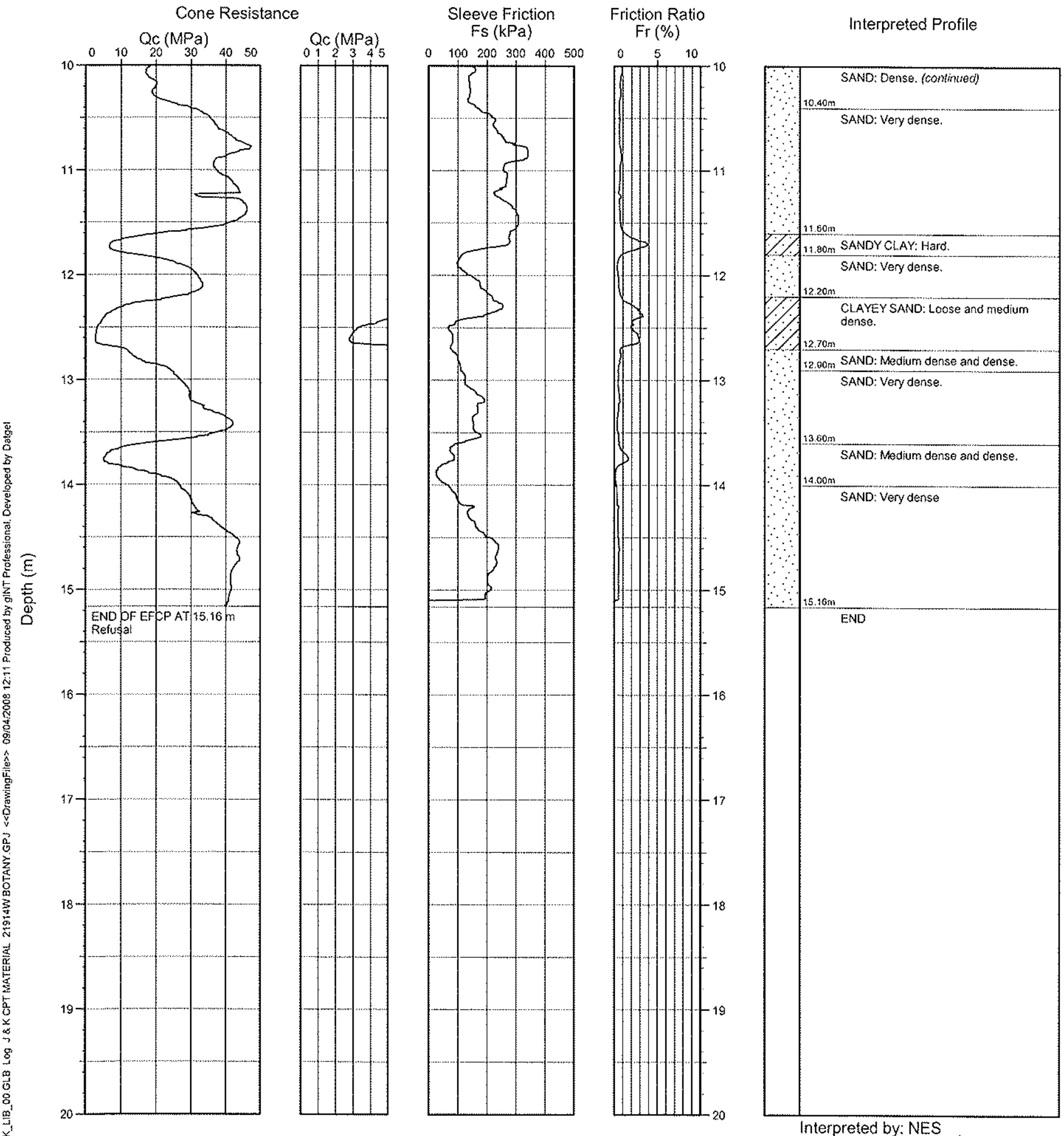
ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client: IGNACIO ABERASTURI

Project: PROPOSED RESIDENTIAL DEVELOPMENT

Location: 26 TUPIA STREET, BOTANY, NSW

Job No.: 21914W	**R.L. Surface:** 3.3 m	**Data File:** 21914W_1.GEF
Date: 27/02/08	**Datum:** AHD	**Operator:** JMK



Interpreted by: NES

Checked by:

Jeffery and Katauskas Pty Ltd

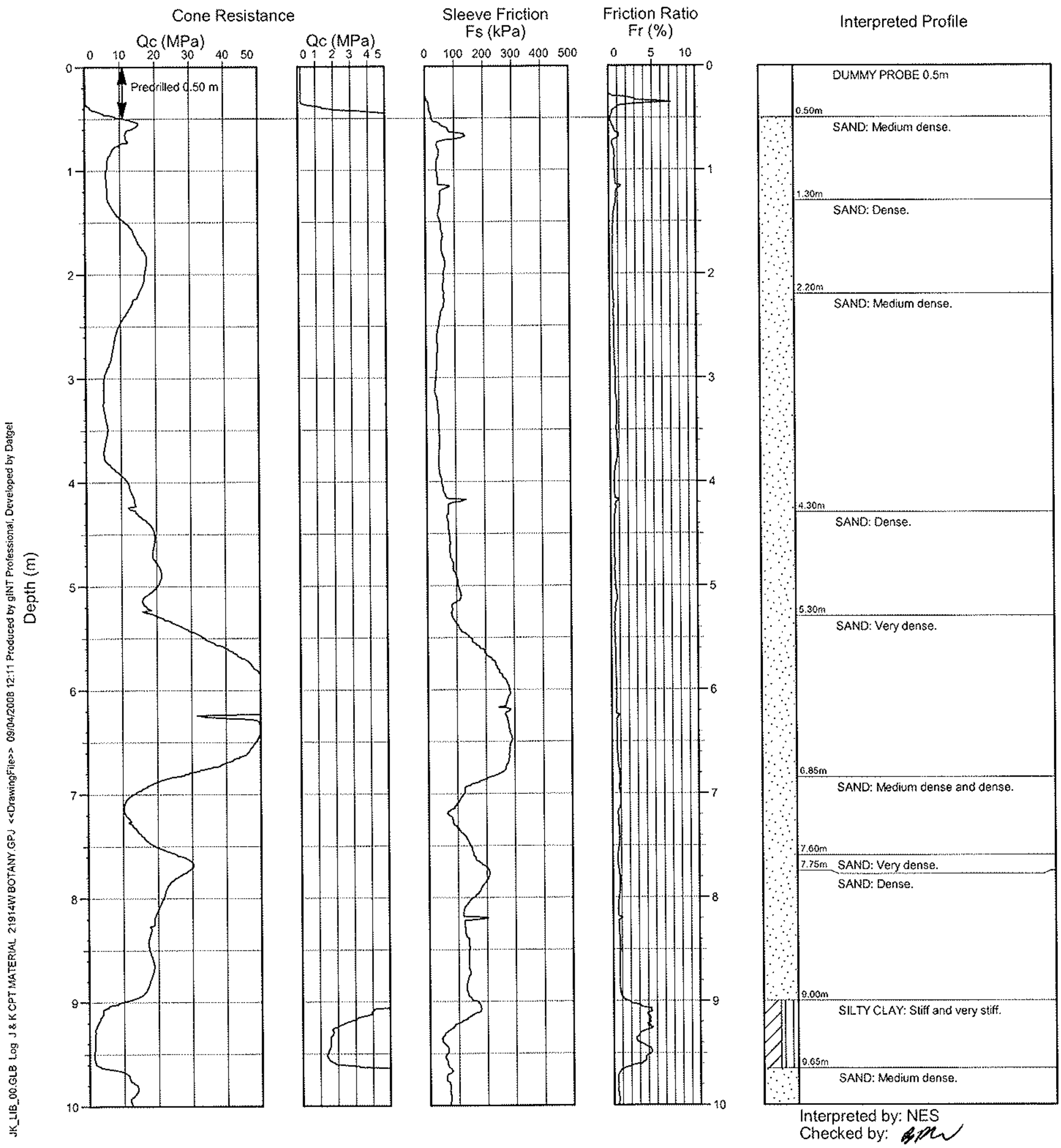
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

EFCP No.
2
1 / 2

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client: IGNACIO ABERASTURI
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 26 TUPIA STREET, BOTANY, NSW

Job No.: 21914W	**R.L. Surface:** 2.8 m	**Data File:** 21914W_2.GEF
Date: 27/02/08	**Datum:** AHD	**Operator:** JMK



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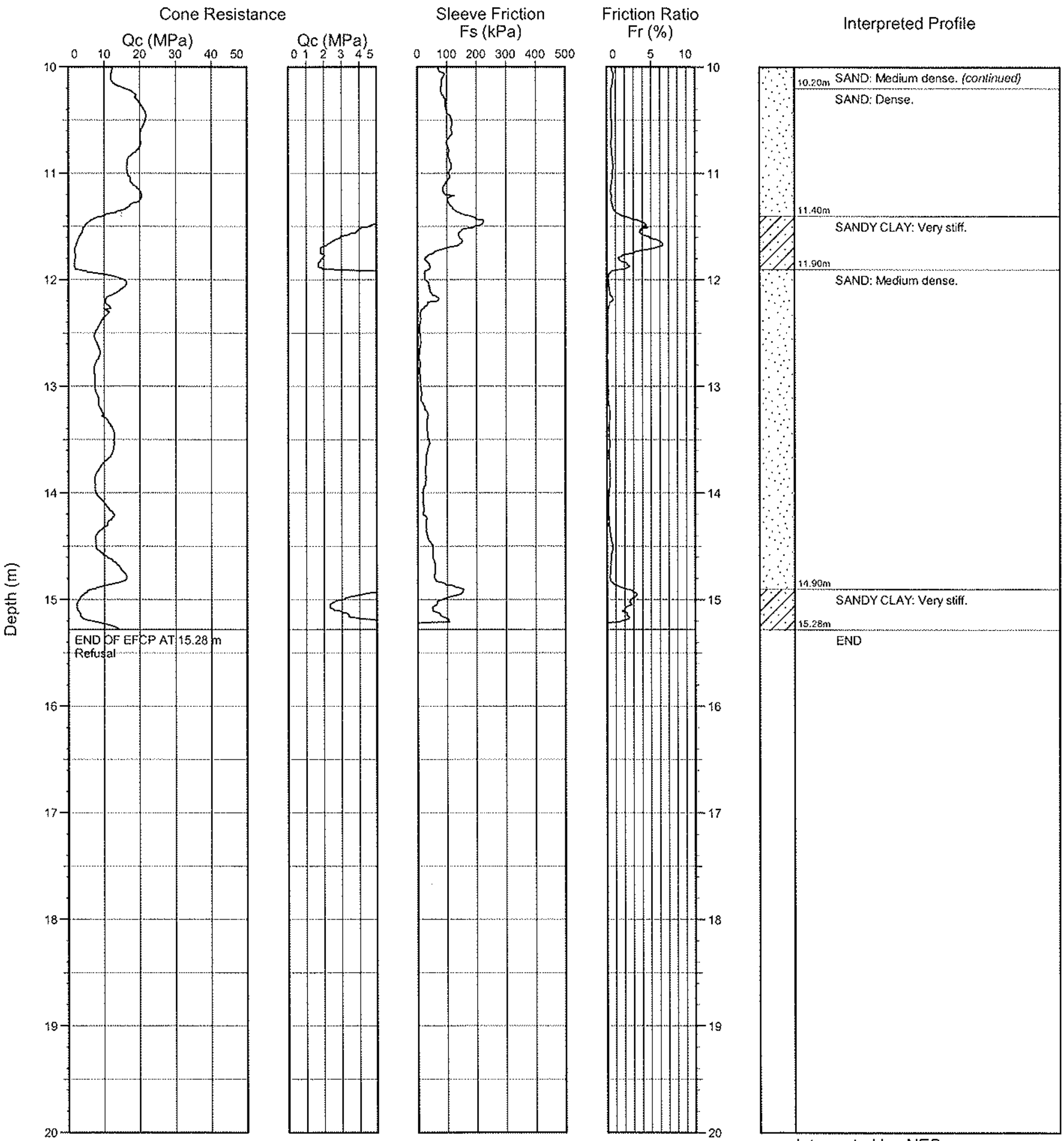
EFCP No.
2
2 / 2

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client: IGNACIO ABERASTURI
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 26 TUPIA STREET, BOTANY, NSW

Job No.: 21914W	**R.L. Surface:** 2.8 m	**Data File:** 21914W_2.GEF
Date: 27/02/08	**Datum:** AHD	**Operator:** JMK



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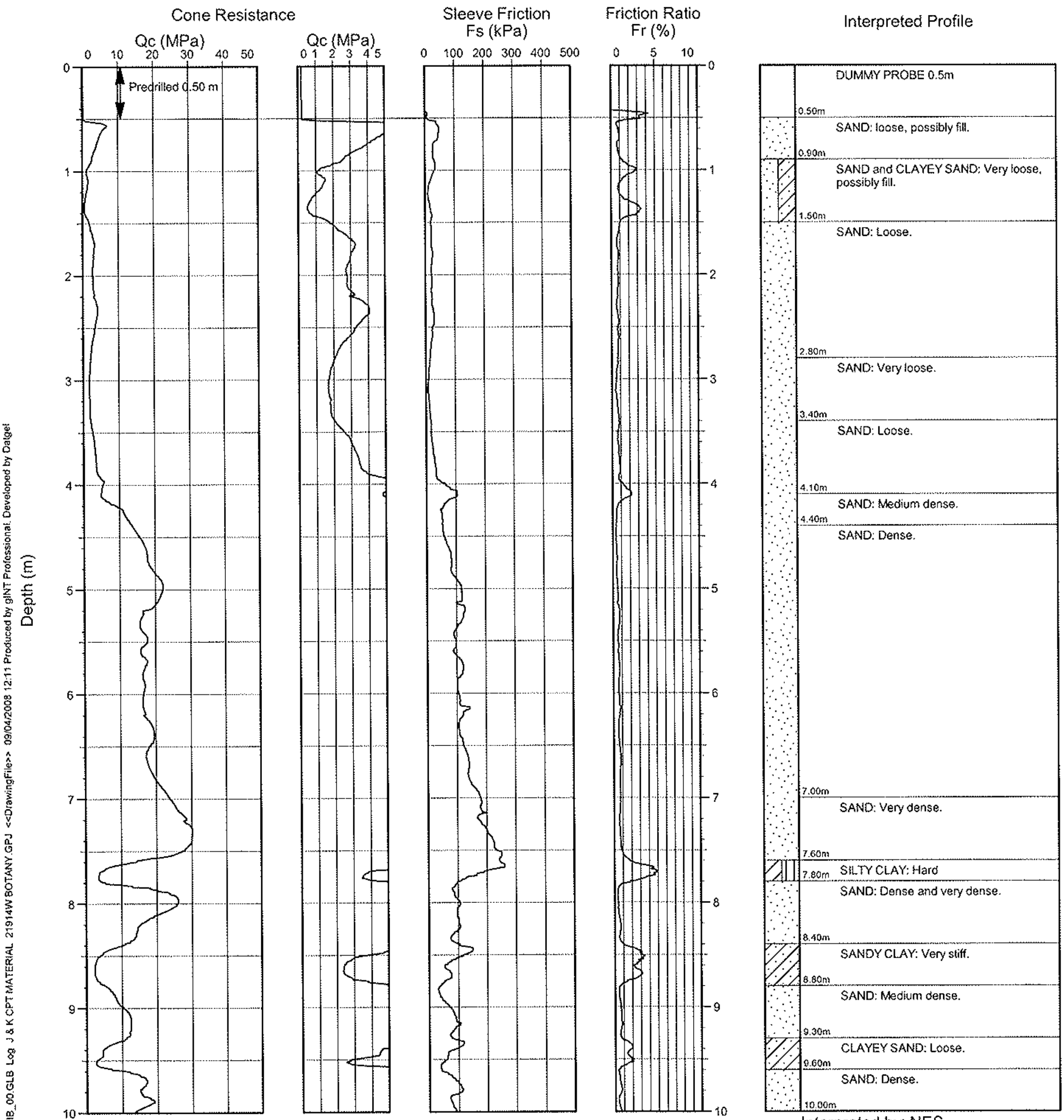
EFCP No.
3
1 / 2

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client:	IGNACIO ABERASTURI
Project:	PROPOSED RESIDENTIAL DEVELOPMENT
Location:	26 TUPIA STREET, BOTANY, NSW

Job No.: 21914W	**R.L. Surface:** 1.7 m	**Data File:** 21914W_3.GEF
Date: 27/02/08	**Datum:** AHD	**Operator:** JMK



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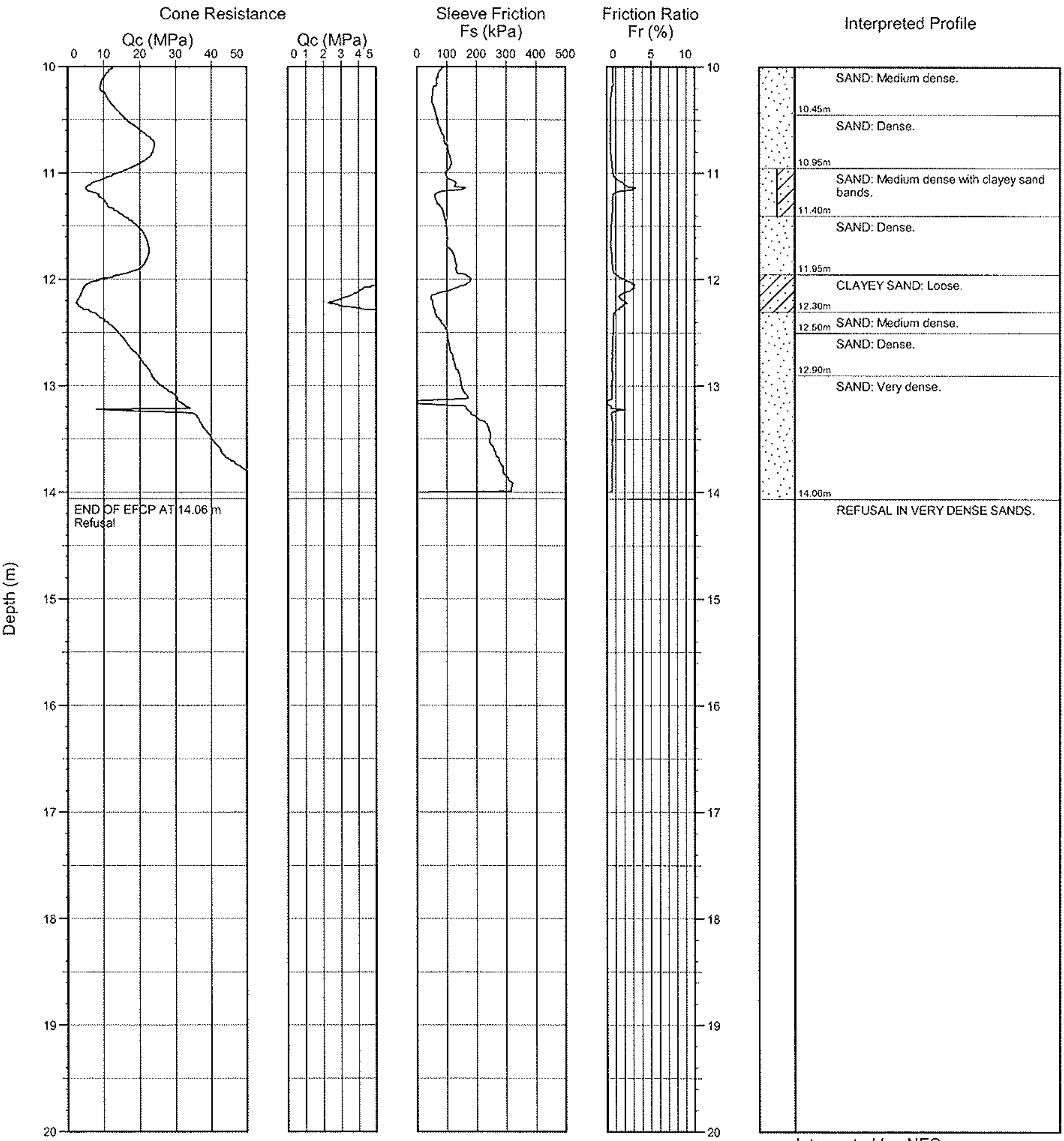
EFCP No. 3

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client: IGNACIO ABERASTURI
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 26 TUPIA STREET, BOTANY, NSW

Job No.: 21914W	**R.L. Surface:** 1.7 m	**Data File:** 21914W_3.GEF
Date: 27/02/08	**Datum:** AHD	**Operator:** JMK



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Checked by: [signature]

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EFCP No.
4
1 / 2

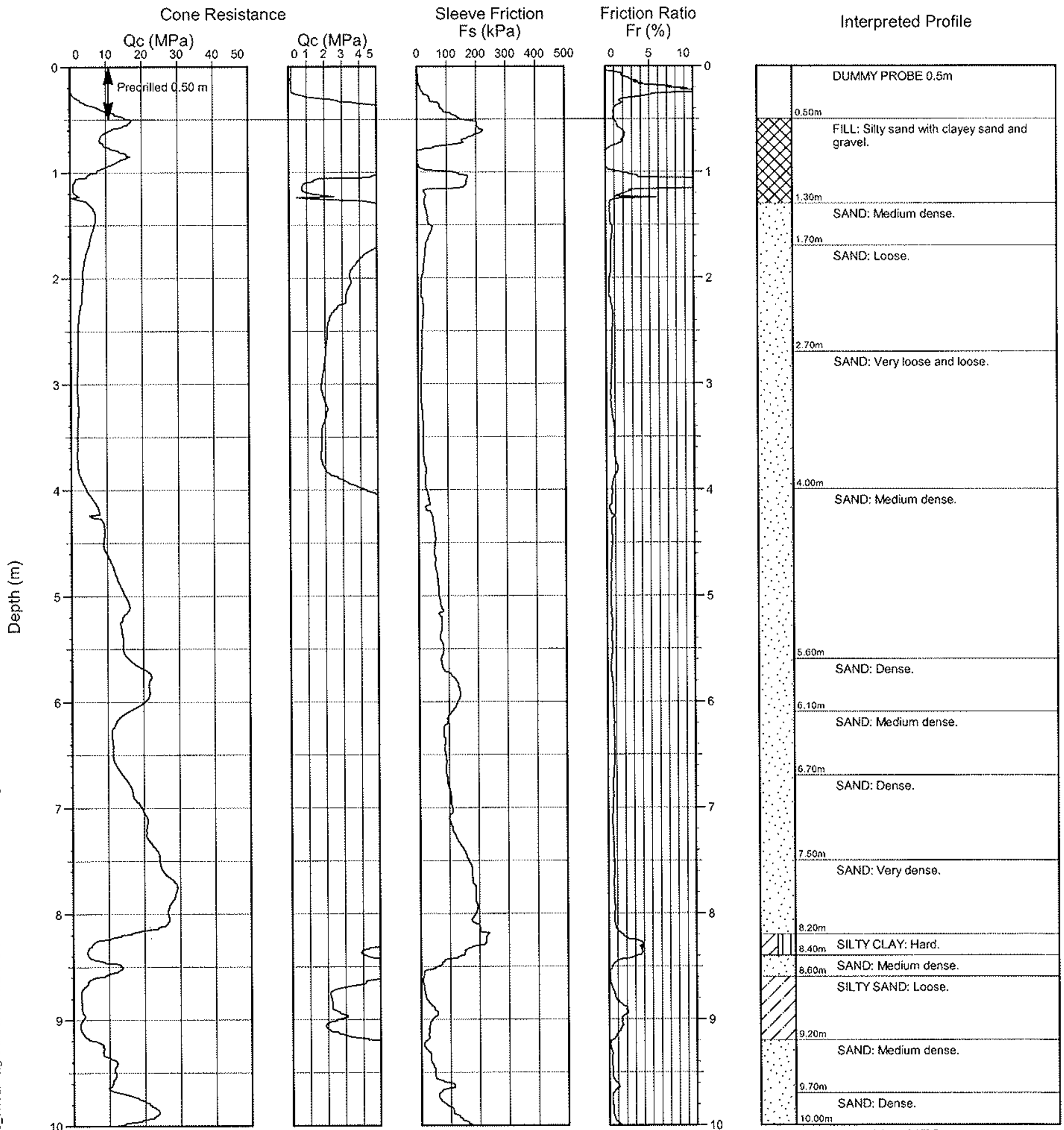
ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client: IGNACIO ABERASTURI

Project: PROPOSED RESIDENTIAL DEVELOPMENT

Location: 26 TUPIA STREET, BOTANY, NSW

Job No.: 21914W	**R.L. Surface:** 1.9 m	**Data File:** 21914W_4.GEF
Date: 27/02/08	**Datum:** AHD	**Operator:** JMK



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Checked by: *[signature]*

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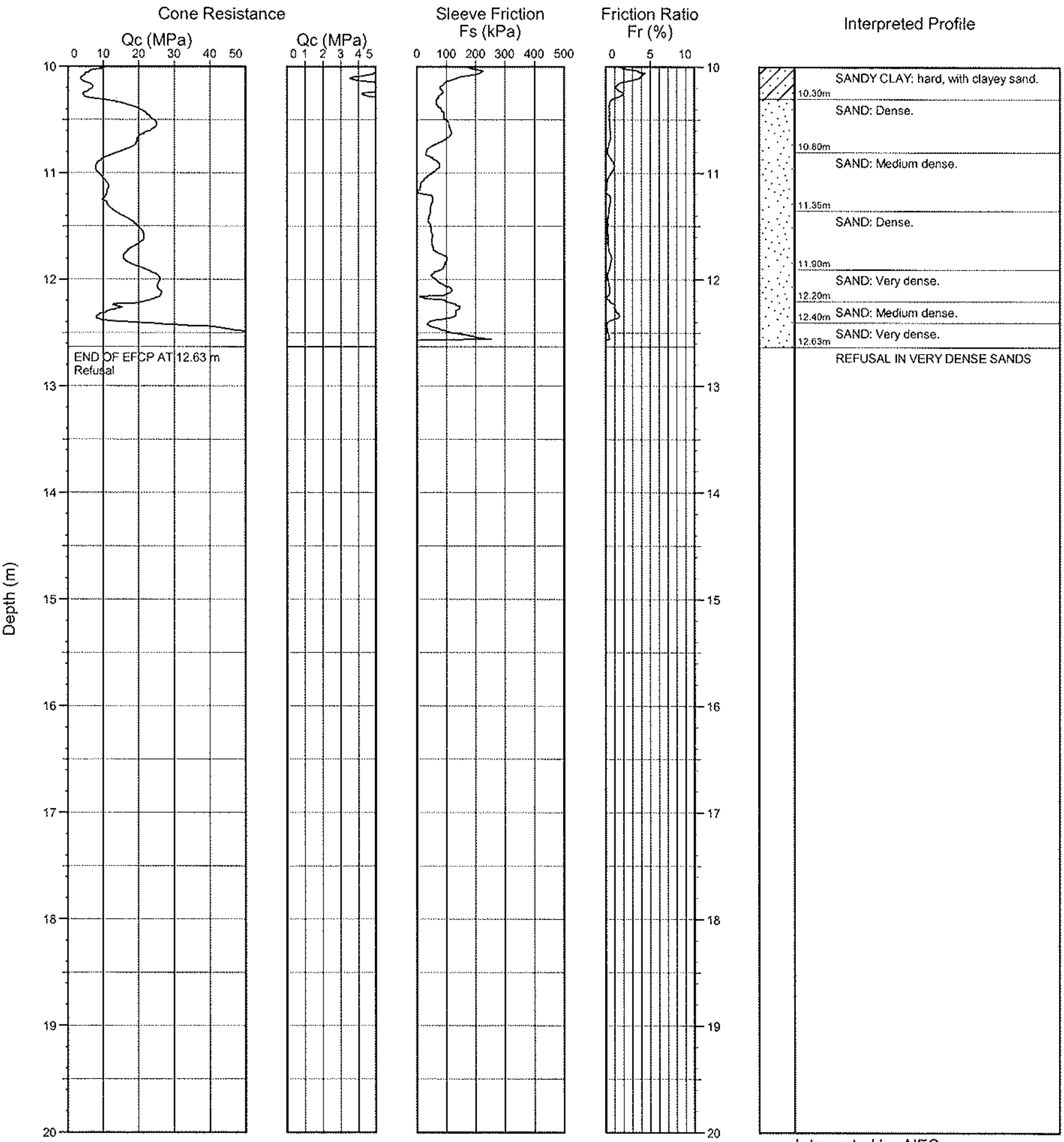
EFCP No.
4
2 / 2

ELECTRICAL FRICTION CONE PENETROMETER TEST RESULTS

Client: IGNACIO ABERASTURI
Project: PROPOSED RESIDENTIAL DEVELOPMENT
Location: 26 TUPIA STREET, BOTANY, NSW

Job No.: 21914W	**R.L. Surface:** 1.9 m	**Data File:** 21914W_4.GEF
Date: 27/02/08	**Datum:** AHD	**Operator:** JMK



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Checked by: [signature]

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AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM, 25 APR 2019.

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

Title:	SITE LOCATION PLAN
Location:	26 TUPIA STREET BOTANY, NSW
Report No: 32491PN	Figure No: 1

JKGeotechnics

PLOT DATE: 4/07/2019 2:31:38 PM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\32000'S\32491PN BOTANY\CAD\32491PN.DWG

LEGEND

- ● BOREHOLE AND ELECTRICAL FRICTION CONE PENETRATION TEST
- ⊙ ELECTRICAL FRICTION CONE PENETROMETER TEST

AERIAL IMAGE SOURCE: MAPS.AU.NEARMAP.COM

0 5 10 15 20 25

SCALE 1:500 @A3 METRES

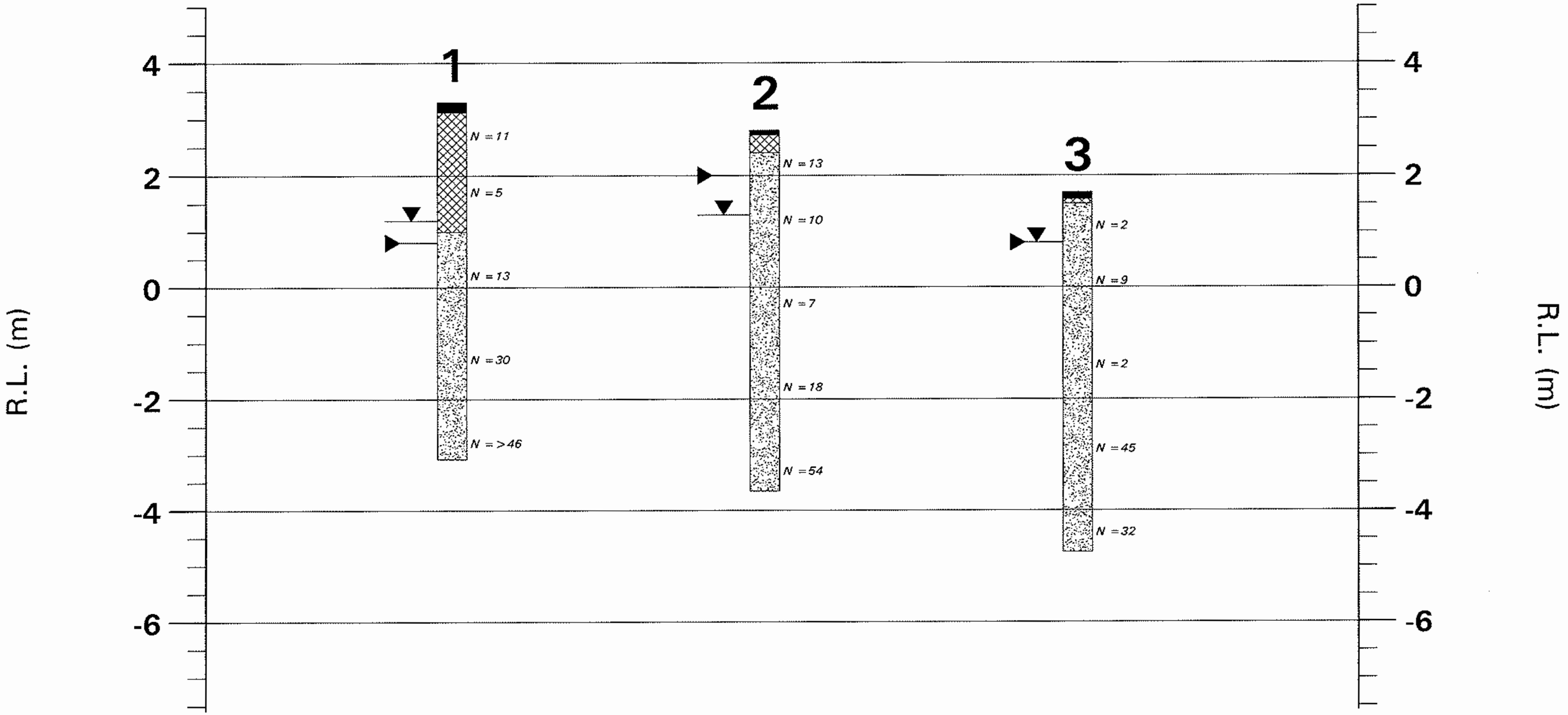
This plan should be read in conjunction with the JK Geotechnics report.

Title:	**TEST LOCATION PLAN**
Location:	26 TUPIA STREET BOTANY, NSW
Report No: 32491PN	Figure No: 2

JKGeotechnics

PLOT DATE: 2/11/2022 10:10:26 AM DWG FILE: S:\6 GEOTECHNICAL\6F GEOTECHNICAL JOBS\32000'S\32491PN BOTANY\CAD\32491PN.DWG

GRAPHICAL BOREHOLE SUMMARY



Asphaltic/ Bituminous Paving or Coal

Fill

Sand

Observed water level

Groundwater seepage level

N SPT "N" VALUE

Nc SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 100 (vert) ; NTS (horiz)

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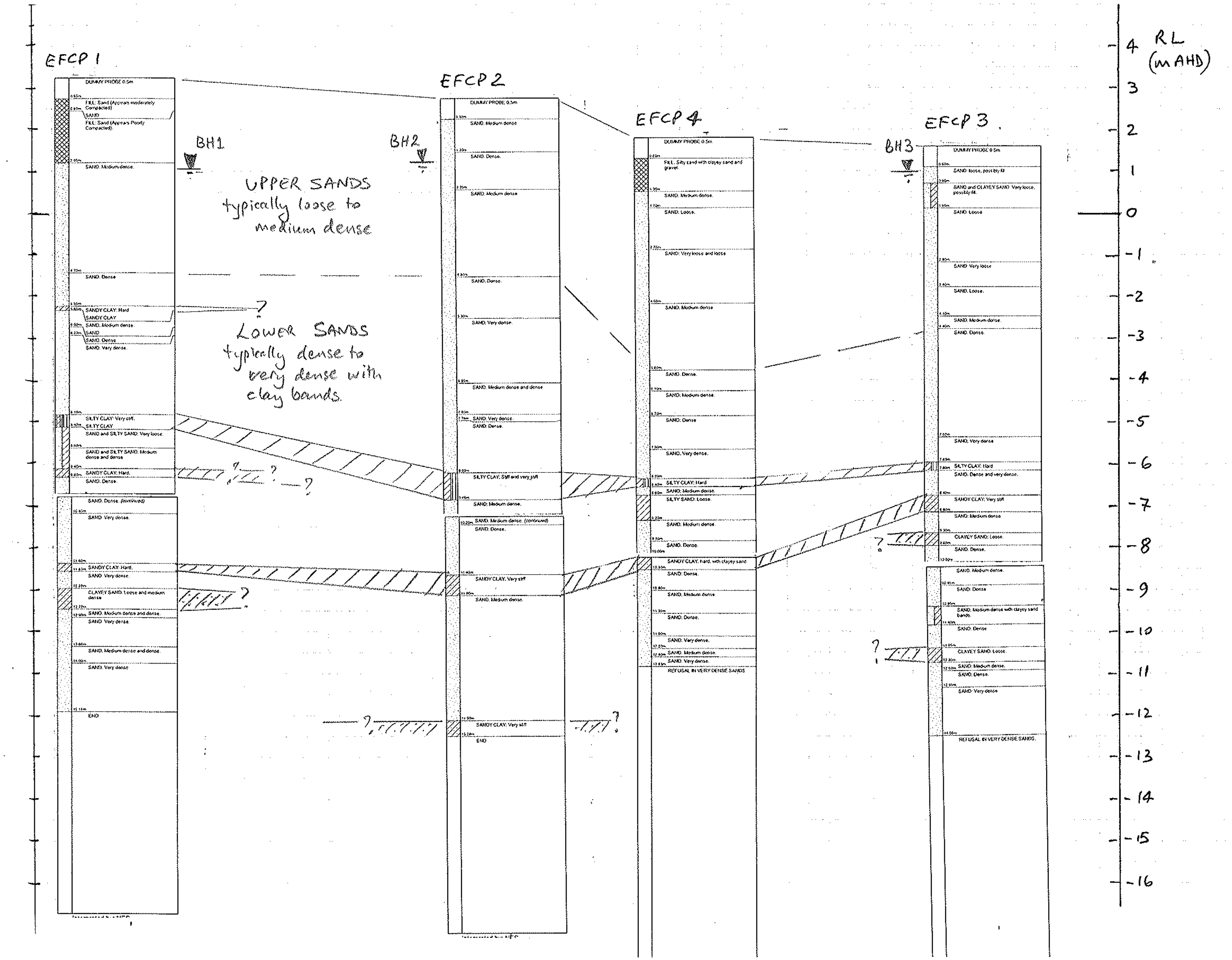
Job No.: 21914W

Figure No.: 2



GRAPHICAL EFCP TEST SUMMARY

NOTES

1. Refer to attached report for discussion. Clay layers may not be continuous laterally.
2. Refer to detailed EFCP test result sheets.
3. Interpreted material types and strength/density is based on usual correlations, but should only be regarded as 'approximate'.
4. Not to horizontal scale.

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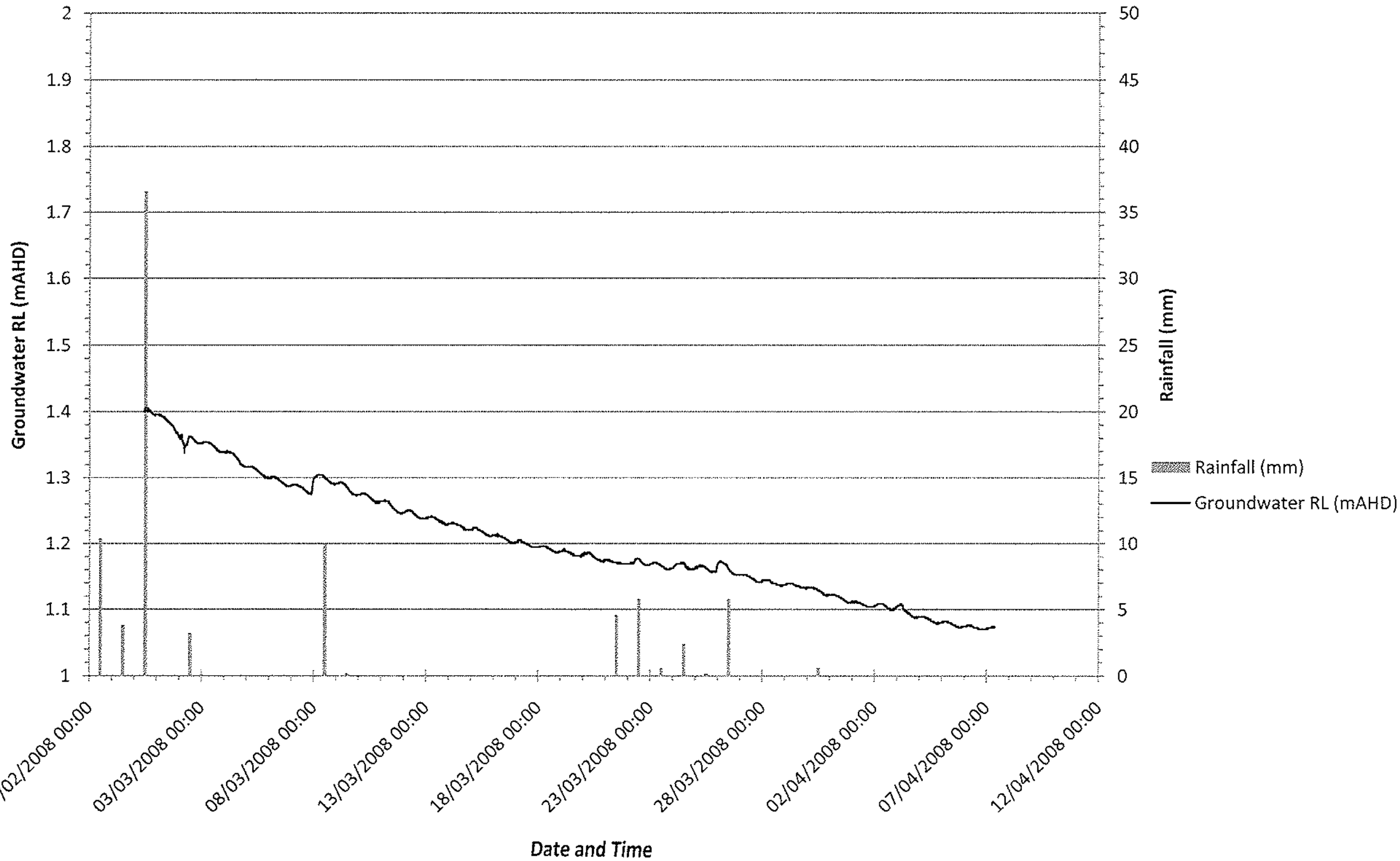
Report No. 21914W **Figure No. 3**

Summary of BH1 Groundwater Levels



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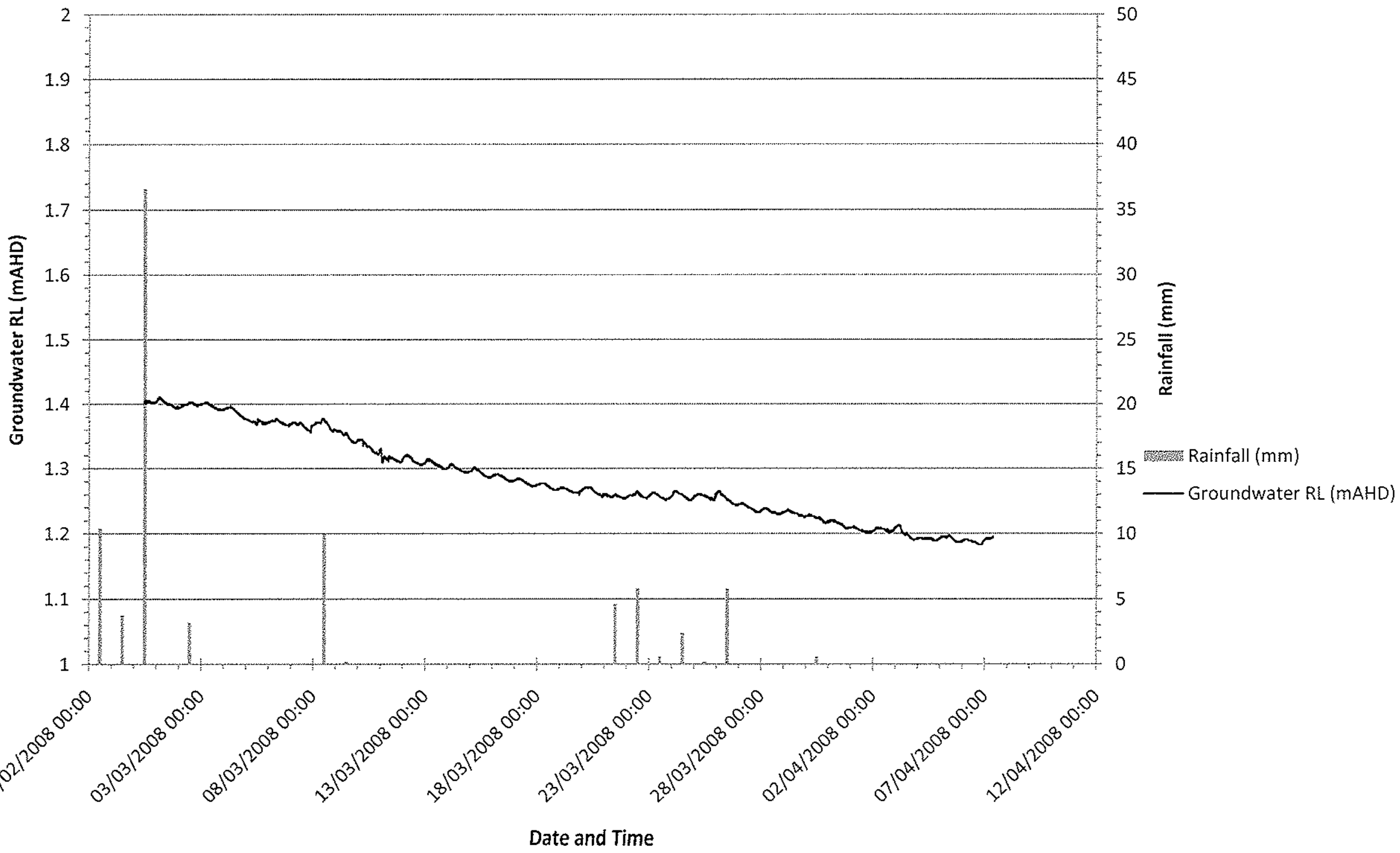
Report No. 21914W Figure No. 4

Summary of BH2 Groundwater Levels



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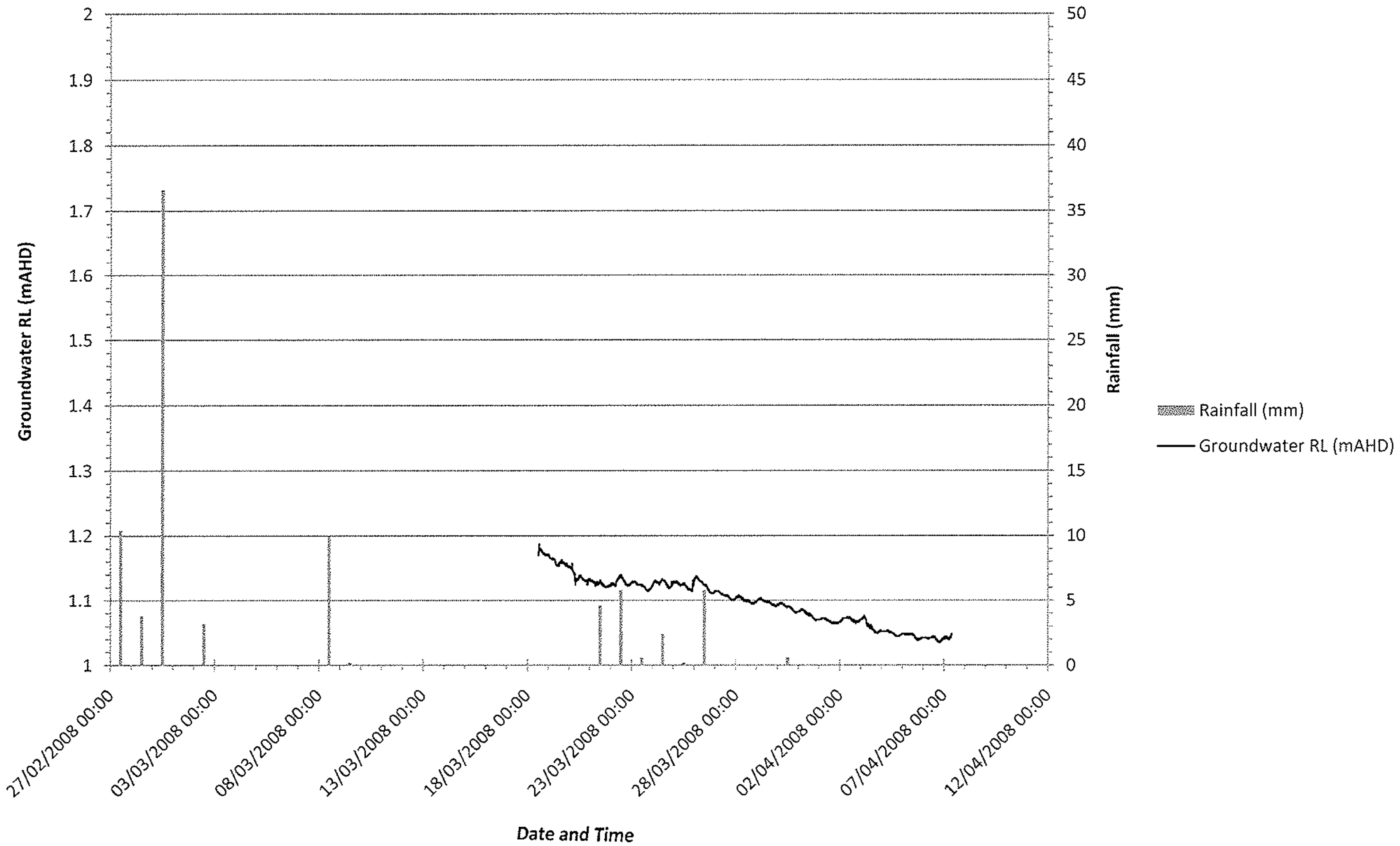
Report No. 21914W Figure No. 5

Summary of BH3 Groundwater Levels



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Report No. 21914W Figure No. 6

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ABN 17 003 550 801

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, the SAA Site Investigation Code. 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 Unified Soil Classification Table qualified by the grading of other particles present (eg sandy clay) as set out below:

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.06mm
Sand	0.06 to 2mm
Gravel	2 to 60mm

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	less than 4
Loose	4 – 10
Medium dense	10 – 30
Dense	30 – 50
Very Dense	greater than 50

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

Classification	Unconfined Compressive Strength kPa
Very Soft	less than 25
Soft	25 – 50
Firm	50 – 100
Stiff	100 – 200
Very Stiff	200 – 400
Hard	Greater than 400
Friable	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 thinly bedded to laminated siltstone.

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 shear 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 except test pits, hand auger drilling and portable dynamic cone penetrometers require the use of a mechanical drilling rig which is commonly mounted on a truck chassis.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils 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 an 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. Premature refusal of the hand augers can occur on a variety of materials such as hard clay, gravel or ironstone, 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 relatively lower 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 fragments. 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 determined 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 such as Revert or Biogel. 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, an NMLC triple tube core barrel, which gives a core of about 50mm diameter, 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 CORE LOSS. The location of losses are determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the top end 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, "Methods of Testing Soils for Engineering Purposes" – Test F3.1.

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 63kg 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.

Occasionally, the drop hammer is used to drive 50mm diameter thin walled sample tubes (U50) in clays. In such circumstances, the test results are shown on the borehole logs in brackets.

A modification to the SPT test 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.

Static Cone Penetrometer Testing and Interpretation: Cone penetrometer testing (sometimes referred to as a Dutch Cone) described in this report has been carried out using an Electronic Friction Cone Penetrometer (EFCP). The test is described in Australian Standard 1289, Test F5.1.

In the tests, a 35mm 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 an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm 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.

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.
- 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 EFCP and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of EFCP 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.

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a rod into the ground with a sliding hammer and counting the blows for successive 100mm increments of penetration.

Two relatively similar tests are used:

- Cone penetrometer (commonly known as the Scala Penetrometer) – a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS1289, Test F3.2). The test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various Road Authorities.
- Perth sand penetrometer – a 16mm diameter flat ended rod is driven with a 9kg hammer, dropping 600mm (AS1289, Test F3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.

LOGS

The borehole or test pit logs presented herein are an engineering and/or geological interpretation of the sub-surface 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 attached explanatory notes define the terms and symbols used in preparation of the logs.

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 water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after stabilising 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 determine 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 Soil for Engineering Purposes'*. 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.

Every 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.

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 that at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Attention is drawn to the document *'Guidelines for the Provision of Geotechnical Information in Tender Documents'*, published by the Institution of Engineers, Australia. 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. License to use the documents may be revoked without notice if the Client is in breach of any objection to make a payment to us.

REVIEW OF DESIGN

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

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 such as appropriate footing or pier founding depths, or
iii) full time engineering presence on site.

GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS

SOIL

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

ROCK

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

DEFECTS AND INCLUSIONS

- CLAY SEAM
- SHEARED OR CRUSHED SEAM
- BRECCIATED OR SHATTERED SEAM/ZONE
- IRONSTONE GRAVEL
- ORGANIC MATERIAL

OTHER MATERIALS

- CONCRETE
- BITUMINOUS CONCRETE, COAL
- COLLUVIUM

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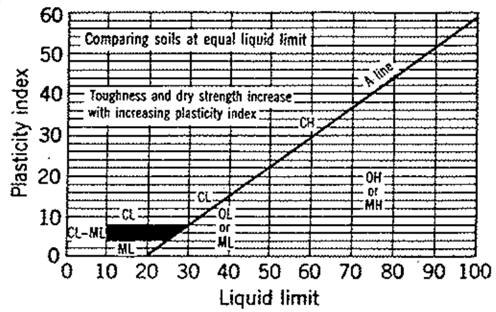
CONSULTING GEOTECHNICAL & ENVIRONMENTAL ENGINEERS

UNIFIED SOIL CLASSIFICATION TABLE

Major divisions			Field Identification Procedures (Excluding particles larger than 75 μm and basing fractions on estimated weights)	Group Symbols a	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria
Coarse-grained soils More than half of material is *larger* than 75 μm sieve size[b] (The 75 μm sieve size is about the smallest particle visible to naked eye)	Gravels More than half of coarse fraction is larger than 4 mm sieve size	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	*GW*	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics Example: *Silty sand*, gravelly; about 20% hard, angular gravel particles 12 mm maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (*SM*)	Use grain size curve in identifying the fractions as given under field identification. Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than 75 μm sieve size) coarse grained soils are classified as follows: Less than 5% *GW, GP, SW, SP*; More than 12% *GM, GC, SM, SC*; 5% to 12% *Borderline* cases requiring use of dual symbols $C_U = \frac{D_{60}}{D_{10}}$ Greater than 4 $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing	*GP*	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for *GW*
		Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures see *ML* below)	*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures		Atterberg limits below "A" line, or *PI* less than 4 — Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)	*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures		Atterberg limits above "A" line, with *PI* greater than 7
	Sands More than half of coarse fraction is smaller than 4 mm sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	*SW*	Well graded sands, gravelly sands, little or no fines		$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6 $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing	*SP*	Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for *SW*
		Sands with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures, see *ML* below)	*SM*	Silty sands, poorly graded sand-silt mixtures		Atterberg limits below "A" line or *PI* less than 5 — Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)	*SC*	Clayey sands, poorly graded sand-clay mixtures		Atterberg limits below "A" line with *PI* greater than 7

Major divisions		Identification Procedures on Fraction Smaller than 380 μm Sieve Size: Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	Group Symbols a	Typical Names	Information Required for Describing Soils
Fine-grained soils More than half of material is *smaller* than 75 μm sieve size	Silts and clays liquid limit less than 50	None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)
		Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
		Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity	
	Silts and clays liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	
		High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays	
		Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity	
Highly Organic Soils		Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils	



Plasticity chart
for laboratory classification of fine grained soils

NOTE: 1) Soils possessing characteristics of two groups are designated by combinations of group symbols (e.g. GW-GC, well graded gravel-sand mixture with clay fines).

2) Soils with liquid limits of the order of 35 to 50 may be visually classified as being of medium plasticity.

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LOG SYMBOLS

LOG COLUMN	SYMBOL	DEFINITION
Groundwater Record		Standing water level. Time delay following completion of drilling may be shown.
		Extent of borehole collapse shortly after drilling.
		Groundwater seepage into borehole or excavation noted during drilling or excavation.
Samples	ES	Soil 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 screening.
	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. 'R' as noted below.
	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 degree 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 (Cohesive Soils)	MC>PL	Moisture content estimated to be greater than plastic limit.
	MC≈PL	Moisture content estimated to be approximately equal to plastic limit.
	MC<PL	Moisture content estimated to be less than plastic limit.
(Cohesionless 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 less than 25kPa
	S	SOFT - Unconfined compressive strength 25-50kPa
	F	FIRM - Unconfined compressive strength 50-100kPa
	St	STIFF - Unconfined compressive strength 100-200kPa
	VSt	VERY STIFF - Unconfined compressive strength 200-400kPa
	H	HARD - Unconfined compressive strength greater than 400kPa
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other tests.
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 — 15-35 — 4-10
	MD	Medium Dense — 35-65 — 10-30
	D	Dense — 65-85 — 30-50
	VD	Very Dense — >85 — >50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other tests.
Hand Penetrometer Readings	300 250	Numbers indicate individual test results in kPa on representative undisturbed material unless noted otherwise.
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Tungsten carbide wing bit.
	T 60	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.

Ref: Standard Sheets/Log Symbols
November 2007

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
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LOG SYMBOLS

ROCK MATERIAL WEATHERING CLASSIFICATION

TERM	SYMBOL	DEFINITION
Residual Soil	RS	Soil developed on extremely weathered rock; the mass structure and substance fabric are no longer evident; there is a large change in volume but the soil has not been significantly transported.
Extremely weathered rock	XW	Rock is weathered to such an extent that it has "soil" properties, ie it either disintegrates or can be remoulded, in water.
Distinctly weathered rock	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by ironstaining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Slightly weathered rock	SW	Rock is slightly discoloured but shows little or no change of strength from fresh rock.
Fresh rock	FR	Rock shows no sign of decomposition or staining.

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index (Is 50) and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described by the International Journal of Rock Mechanics, Mining, Science and Geomechanics. Abstract Volume 22, No 2, 1985.

TERM	SYMBOL	Is (50) MPa	FIELD GUIDE
Extremely Low:	EL		Easily remoulded by hand to a material with soil properties.
		0.03	
Very Low:	VL		May be crumbled in the hand. Sandstone is "sugary" and friable.
		0.1	
Low:	L		A piece of core 150mm long x 50mm dia. may be broken by hand and easily scored with a knife. Sharp edges of core may be friable and break during handling.
		0.3	
Medium Strength:	M		A piece of core 150mm long x 50mm dia. can be broken by hand with difficulty. Readily scored with knife.
		1	
High:	H		A piece of core 150mm long x 50mm dia. core cannot be broken by hand, can be slightly scratched or scored with knife; rock rings under hammer.
		3	
Very High:	VH		A piece of core 150mm long x 50mm dia. may be broken with hand-held pick after more than one blow. Cannot be scratched with pen knife; rock rings under hammer.
		10	
Extremely High:	EH		A piece of core 150mm long x 50mm dia. is very difficult to break with hand-held hammer. Rings when struck with a hammer.

ABBREVIATIONS USED IN DEFECT DESCRIPTION

ABBREVIATION	DESCRIPTION	NOTES
Be	Bedding Plane Parting	Defect orientations measured relative to the normal to the long core axis (ie relative to horizontal for vertical holes)
CS	Clay Seam	
J	Joint	
P	Planar	
Un	Undulating	
S	Smooth	
R	Rough	
IS	Ironstained	
XWS	Extremely Weathered Seam	
Cr	Crushed Seam	
60t	Thickness of defect in millimetres	