

Report on Geotechnical Assessment

Moriah College Queens Park Road, Queens Park

> Prepared for Moriah College

Project 86890.00 August 2019



Douglas Partners Geotechnics | Environment | Groundwater

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The undersigned, on behalf of Douglas Partners Pty Ltd, confirm that this document and all attached drawings, logs and test results have been checked and reviewed for errors, omissions and inaccuracies.

Signature Signature	Date
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Report on Geotechnical Assessment Moriah College Queens Park Road, Queens Park

1. Introduction

This report presents the results of a desktop geotechnical assessment undertaken for Moriah College at Queens Park Road, Queens Park. The assessment was commissioned in an email dated 1 August 2019 by Michael Carbone of Aver Pty Ltd, project managers, on behalf of Moriah College and was undertaken in accordance with Douglas Partners' proposal SYD190694 dated 11/07/2019.

It is understood that the development of the site will include staged demolition of some existing buildings and demountable buildings and staged construction of new school buildings. Stage 1 is construction of a 3/4 storey STEAM building while Stage 2 is construction of a three storey Early Learning Centre (ELC) building and administration offices.

The aim of the assessment was to determine the ground conditions across the site in order to provide:

- Comment on the geotechnical suitability of the site for the proposed development;
- Make recommendations on site preparation and earthworks;
- Make recommendations on excavations and retaining structures; and
- Provide an appropriate foundation system for the proposed development, including an assessment of allowable bearing pressures.

It is understood that this report will form part of an SSDA submission (SSD-10352) for the staged upgrade of the existing College site.

A previous geotechnical investigation report (Report 28900, June 2000) was prepared for Moriah College which included the area of the currently proposed works. The previously proposed works in the area of the currently proposed works were never undertaken. The report has been written using the previous field work results for the currently proposed development.

2. Site Description

Moriah College is located on Queens Park Road, Queens Park and covers an area of some 4.5 ha. The site is bounded by Queens Park Road to the north and north-east, York Road to the west and south, and Baronga Avenue to the east. The proposed development is located in the south-eastern quadrant of the College site.

The site of the proposed works is currently occupied by an open car park, driveway, tennis courts and some school buildings. Much of the site has been developed with site levels generally rising to the north with a difference in levels of some 4 m over 80 m, which is less than 5 degrees.



3. Geology

The 1:100,000 Series Geological Map for Sydney indicates that the site is underlain by transgressive dunes which comprise medium to fine grained marine sands.

The Acid Sulphate Soil Risk Map for Botany Bay published by the Department of Conservation and Soil Management indicates that the site is in an area of no known occurrence of acid sulphate soils.

4. Field Work Methods

The field work for the assessment undertaken in June 2000 included four cone penetration tests (T10-13) to depths between 15 m and 22 m and one bore drilled to 10 m with a standpipe installed in the bore.

The test locations are shown on Drawing No. 1 in Appendix B.

During a cone penetration test (CPT) a ballasted truck-mounted test rig is used to push a 35 mm diameter instrumented cone into the soil using a hydraulic ram system. Continuous measurements are made of the end-bearing pressure on the cone and the friction on a 135 mm long sleeve located immediately behind the cone. The cone resistance and friction readings are displayed on a digital monitor and stored on computer for subsequent plotting of results and interpretation.

The borehole was drilled using a truck mounted rig, wash boring a 100 mm diameter bore.

The depth to groundwater was recorded upon extraction of the CPT rods, and a standpipe installed in the bore.

The ground surface levels at the test locations were determined at the time of the field work by levelling in relation to the floor slab of the administration building, indicated to be RL 58.536 m AHD on the survey plan provided by the school in 2000.

5. Field Work Results

Details of the subsurface conditions encountered are given on the CPT report sheets and borehole log presented in Appendix C, together with information on the CPT method and interpretation of the results and Notes defining descriptive terms and classification methods. The CPT report sheets also show the interpreted soil stratification.

Relatively uniform conditions were encountered in the CPTs and the bore, with sands extending for the full depth of the tests to 22 m.

The CPT results for locations T10 and T11 indicated that the sands encountered in these areas below the pavement to depths of 2.8 m and 4.0 m respectively, had either been disturbed, possibly as a result of previous construction work or, had been placed as filling.



Groundwater was not observed in the holes left after CPT rods were withdrawn. Groundwater was, however, measured in Bore BH3 at 8 m depth or RL 41.3 m (relative to AHD) in June 2000.

Groundwater levels will fluctuate with climatic conditions and are likely to increase following periods of extended wet weather.

A falling head test was carried out in Bore BH3 for the purpose of determining the insitu soil permeability and produced a result of approximately 2x10⁻⁴ m/sec.

6. **Proposed Development**

It is understood that the development of the site will include staged demolition of some existing buildings and demountable buildings and staged construction of new school buildings. Stage 1 involves the construction of a 3/4 storey STEAM building while Stage 2 is construction of a three storey Early Learning Centre (ELC) building and administration offices. The concept design drawings by FJMT Architects indicate that the Stage 1 development will include a lower level or basement car park, but that no basement is shown in the Stage 2 area.

The proposed basement level will be at RL 49.72 m relative to Australian Height Datum (AHD).

The geotechnical issues considered relevant to the proposed development include groundwater, excavation, excavation support and foundations. With a proposed basement level at RL 49.72 mAHD, excavation depths are expected to vary from about 1 - 2m at the north- eastern corner of the development area increasing to 3 - 4m in the western and north-western sections of the site.

7. Comments

7.1 Groundwater

The groundwater level has been measured on the site at approximately RL 41 m in June 2000. Further investigation will be required to confirm the current water level. Based on historical data, the current groundwater level is expected to be within about 2 m of the measured value in June 2000 as groundwater levels do change with changes in climatic conditions and over time.

For a proposed (Stage 1) development at approximately RL 49.7 m, and groundwater levels assumed at RL 41 m (plus or minus 2 m), groundwater is not expected to be a significant issue for the proposed development.

7.2 Excavation

Excavation is expected to be required within filling and sandy soils up to about 4 m depth. Excavation of these materials should be readily achieved using conventional earthmoving equipment such as excavators and bulldozers. Dry sands may make the site un-trafficable especially to tyred vehicles and



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some form of bridging layer or working platform may be required for constructability purposes (e.g. as access tracks for the trucks removing excavated material).

It should be noted that any off-site disposal of spoil with generally require assessment for use or classification in accordance with the current Waste Classification Guidelines (EPA2014/0796).

7.3 Excavation Support

7.3.1 General

Where there is adequate room, batters could be used for the sides of excavation. For depths to 3 m, temporary batters of 1.5H:1V could be adopted increasing to 2H:1V for deeper excavations.

Where the excavation is to extend to the site boundaries, or where there are adjacent structures, it will be necessary to provide temporary support (e.g. shoring) during construction and long-term support in the form of retaining walls. If the footings for the adjacent structures or utilities are founded above an imaginary 'influence' line rising at 45 degree from the base of the excavation or pit, provision should be made for designing the retaining system to adequately resist the lateral earth pressures from the loadings applied by the adjacent building footings. Consideration could be given to underpinning the footings, however, it is often difficult and expensive to carry out underpinning in sand.

For dry sandy conditions above the groundwater table, a contiguous pile wall is considered to be an appropriate retaining wall method.

7.3.2 Earth Pressures

The shoring/basement wall will be subject to earth pressures from the ground surface down to the base of the excavation. It is expected that basement retaining walls supporting 3 m or more of soil will require a row of anchors until floor slab is used to prop the wall.

The lateral earth pressure distribution for a wall with a single row of lateral support is often modelled as a triangular pressure distribution. For preliminary design purposes, a coefficient of active soil pressure (Ka) of 0.3 could be adopted using a bulk density of 20 kN/m³. Where retaining walls are supporting adjacent footings, they should be design using 'at rest' conditions and K₀ of 0.5. Surcharge pressures from adjacent structures to the north, construction machinery and traffic should also be incorporated into the design of the shoring wall as necessary.

If batters are used and the void between the batter and retaining wall backfilled, the design of the retaining wall should consider the effects of the machinery used for backfilling.

7.3.3 Temporary Ground Anchors

Inclined tie-back (ground) anchors could be used for the temporary lateral restraint of the shoring/permanent basement walls. The ground anchors should be inclined below the horizontal to allow anchorage into the denser materials. The preliminary design of temporary ground anchors may be carried out using an ultimate average bond stress at the grout-soil interface of 30 kPa for the sand.



Secondary-grouted anchors could be used to increase the anchor capacity in sands. This technique involves installing a conventionally-grouted anchor and then, once cured, injecting grout into the anchor at a higher pressure to crack the primary grout and densify the surrounding materials. This technique is specialised and only experienced contractors should be engaged for the design and installation of secondary-grouted anchors.

The parameter given above assumes that the anchor holes are clean, with grouting and other installation procedures carried out in accordance with good anchoring practice. Careful installation and close supervision by a geotechnical specialist may allow increased bond stresses to be adopted during construction, subject to testing.

It will be necessary to obtain permission from neighbouring landowners prior to installing anchors that will extend beyond the perimeter of the site. In addition, care should be taken to avoid damaging buried services and pipes during anchor installation.

Only experienced contractors should be engaged to install anchors because anchors in sand often "slip" a little resulting in some wall movement. Proof stressing /testing should be undertaken by the anchoring contractor to demonstrate a 'reserve' capacity above their nominated design bond stress values.

7.4 Foundations

The footing loads are expected to vary across the site as the levels of the building above the footings vary. Where loads are relatively light, shallow spread footings could be adopted. Otherwise piles are the preferred footing types. It is generally preferable to adopt the same footing types across the site to control differential settlement. On the school site, it may be possible to adopt different footing types provided properly designed slip joints and articulation are incorporated into the construction to allow for differential settlement.

7.4.1 Shallow Footings

Spread footings founded in natural medium dense sand could be considered. For spread footings in sand, the allowable bearing pressure is dependent on the size of the footing, the depth of embedment of the footing as well as the density of the sand and the position of the groundwater table. For the Moriah College site, the ground at approximately RL 49 m is expected to be medium dense sand with the groundwater table many metres below. Typical allowable bearing pressures for varying sized strip and pad footings on the College site are given in Table 1.

Embedment	Pad F	ooting	Strip F	ooting
Depth	1 m square	1.5 m square	0.5 m wide	1 m wide
0.5 m	300 kPa	350 kPa	200 kPa	275 kPa
1 m	450 kPa	500 kPa	300 kPa	350 kPa

Table 1:	Typical Allowable bearin	a Pressures for Spread F	ootinas.
	· · · · · · · · · · · · · · · · · · ·	g i roccurco ror oproud r	ootingo.



For different sized footings and embedment depths to those given in Table 1, the allowable bearing pressures will vary. Settlements for spread footings is typically about 1% to 2% of the footing width.

A raft slab may be appropriate but consideration needs to be given to the stress increases under a raft and the settlement. Further analysis would generally will be required to develop this option.

7.4.2 Deep Footings

Continuous flight auger (CFA) piles are often used nowadays in similar geological conditions and would be a suitable pile type for supporting the higher column loads. CFA piles are constructed by inserting a hollow stem auger into the ground to the nominated depth. Concrete or grout is then injected through the stem of the augers as the auger is withdrawn. A column of concrete or grout is then formed upon completion of the auger withdrawal when a steel reinforcement cage can be lowered into the grout column to complete the pile. As rock is greater than 22 m, piles are expected to be designed as 'friction piles' with the friction values varying with depth.

Using an in-house (DP) computer program (ConePile®), typical Design Geotechnical Strengths, R_{g}^{*} , using a geotechnical strength reduction factor of 0.5 for 450 mm and 600 mm diameter CFA piles at varying depths from RL 49 m are given in Table 2.

Founding Depth	450 mm diameter Pile	600 mm diameter Pile
5 m	750 kN	1200 kN
7.5 m	1000 kN	1500 kN
10 m	1250 kN	2000 kN

Table 2 : Typical Design Geotechnical Strengths for CFA piles

Other pile sizes and depths can be assessed upon request.

Selection of the geotechnical strength reduction factor (Φ_g) is based on a series of individual risk ratings which are weighted and lead to an average risk rating. For preliminary purposes, a geotechnical strength reduction factor of 0.5 was assumed for the values given in Table 2. For detailed design purposes, an appropriate geotechnical strength reduction factor should be applied when using the limit-state approach as outlined in AS 2159 – 2009 *Piling – Design and installation.*

Soil decompression can occur during CFA piling when a strong stratum is encountered. This occurs when the augers continue to rotate but the rate of auger progression decreases, displacing soil from around the auger upwards towards the surface. Decompression can cause weakening and settlement of the soils adjacent to the pile and should be avoided by monitoring auger speed and progression closely.

Settlement of a pile is dependent on the loads applied to the pile and the foundation conditions. Settlement analysis should be undertaken during the detailed design phase to provide settlement estimates to refine pile spacing and founding levels.

Other pile types such as concrete-injected screw piles or cast-insitu driven piles could be considered for the site. These pile types are proprietary products and their suppliers should be consulted concerning



their load capacity. It is noted that all driven piles do cause some vibration and therefore may be unsuitable for the site.

7.5 Pavements and Slabs

Where slabs or pavements on ground are proposed, it is suggested to undertake a program of proving of the existing ground following excavation works or the removal of existing pavements. The areas should be subjected to proof rolling using a roller of at least 10 tonne dead weight capacity. The rolling should be accompanied by visual inspection by a geotechnical engineer to allow detection and suitable treatment, where necessary, of any weak or soft layers identified by rolling.

Following preparation of the subgrade as outlined above, the preliminary design of slabs and pavements may be based on the subgrade having a CBR of 8% and a modulus of subgrade reaction of 40 kPa per mm relevant to point of wheel loads, for the medium dense sand subgrade anticipated.

7.6 Seismicity

A Hazard Factor (*Z*) of 0.08 would be appropriate for the development site in accordance with Australian Standard AS 1170.4 – 2007 *Structural design actions – Part 4: Earthquake actions in Australia.* The site sub-soil class is considered to be Class C_e based on testing carried out on the site and the assumed depth to rock being less than 45 m.

7.7 Further Investigation

The above comments are based on tests carried out approximately twenty years ago. The surface conditions do not appear to have change significantly since the investigation, however, further investigation should be carried out to determine the groundwater level as groundwater levels do change over time.

If deep foundations are adopted for the site, cone penetration testing (CPT) can be undertaken (if required) prior to demolition works, to better identify the depths of dense and very dense sand for founding purposes because the depths do vary across the site.

8. Limitations

Douglas Partners (DP) has prepared this report for this project at Moriah College at Queens Park in accordance with DP's proposal dated 11 July 2019 and acceptance received from Aver on behalf of Moriah College dated 1 August 2019. The work was carried out under an amended DP's Conditions of Engagement. This report is provided for the exclusive use of Moriah College for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so



entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

The results provided in the report are indicative of the sub-surface conditions on the site only at the specific sampling and/or testing locations, and then only to the depths investigated and at the time the work was carried out. Sub-surface conditions can change abruptly due to variable geological processes and also as a result of human influences. Such changes may occur after DP's field testing has been completed.

DP's advice is based upon the conditions encountered during a previous investigation. The accuracy of the advice provided by DP in this report may be affected by undetected variations in ground conditions across the site between and beyond the sampling and/or testing locations. The advice may also be limited by budget constraints imposed by others or by site accessibility.

This report must be read in conjunction with all of the attached notes and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by DP. This is because this report has been written as advice and opinion rather than instructions for construction.

The scope for work for this investigation/report did not include the assessment of surface or sub-surface materials or groundwater for contaminants, within or adjacent to the site. Should evidence of filling of unknown origin be noted in the report, and in particular the presence of building demolition materials, it should be recognised that there may be some risk that such filling may contain contaminants and hazardous building materials.

The contents of this report do not constitute formal design components such as are required, by the Health and Safety Legislation and Regulations, to be included in a Safety Report specifying the hazards likely to be encountered during construction and the controls required to mitigate risk. This design process requires risk assessment to be undertaken, with such assessment being dependent upon factors relating to likelihood of occurrence and consequences of damage to property and to life. This, in turn, requires project data and analysis presently beyond the knowledge and project role respectively of DP. DP may be able, however, to assist the client in carrying out a risk assessment of potential hazards contained in the Comments section of this report, as an extension to the current scope of works, if so requested, and provided that suitable additional information is made available to DP. Any such risk assessment would, however, be necessarily restricted to the geotechnical components set out in this report and to their application by the project designers to project design, construction, maintenance and demolition.

Douglas Partners Pty Ltd

Appendix A

About This Report



Introduction

These notes have been provided to amplify DP's report in regard to classification methods, field procedures and the comments section. Not all are necessarily relevant to all reports.

DP's reports are based on information gained from limited subsurface excavations and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Copyright

This report is the property of Douglas Partners Pty Ltd. The report may only be used for the purpose for which it was commissioned and in accordance with the Conditions of Engagement for the commission supplied at the time of proposal. Unauthorised use of this report in any form whatsoever is prohibited.

Borehole and Test Pit Logs

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

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes or pits, the frequency of sampling, and the possibility of other than 'straight line' variations between the test locations.

Groundwater

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

 In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole 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. They may not be the same at the time of construction as are indicated in the report; and
- 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 first be washed out of the hole if water measurements are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or 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 a perched water table.

Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP 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 and environmental aspects, and recommendations or suggestions for design and construction. However, DP cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

If these occur, DP will be pleased to assist with investigations or advice to resolve the matter.

About this Report

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, DP requests that it be immediately notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

Information for Contractual Purposes

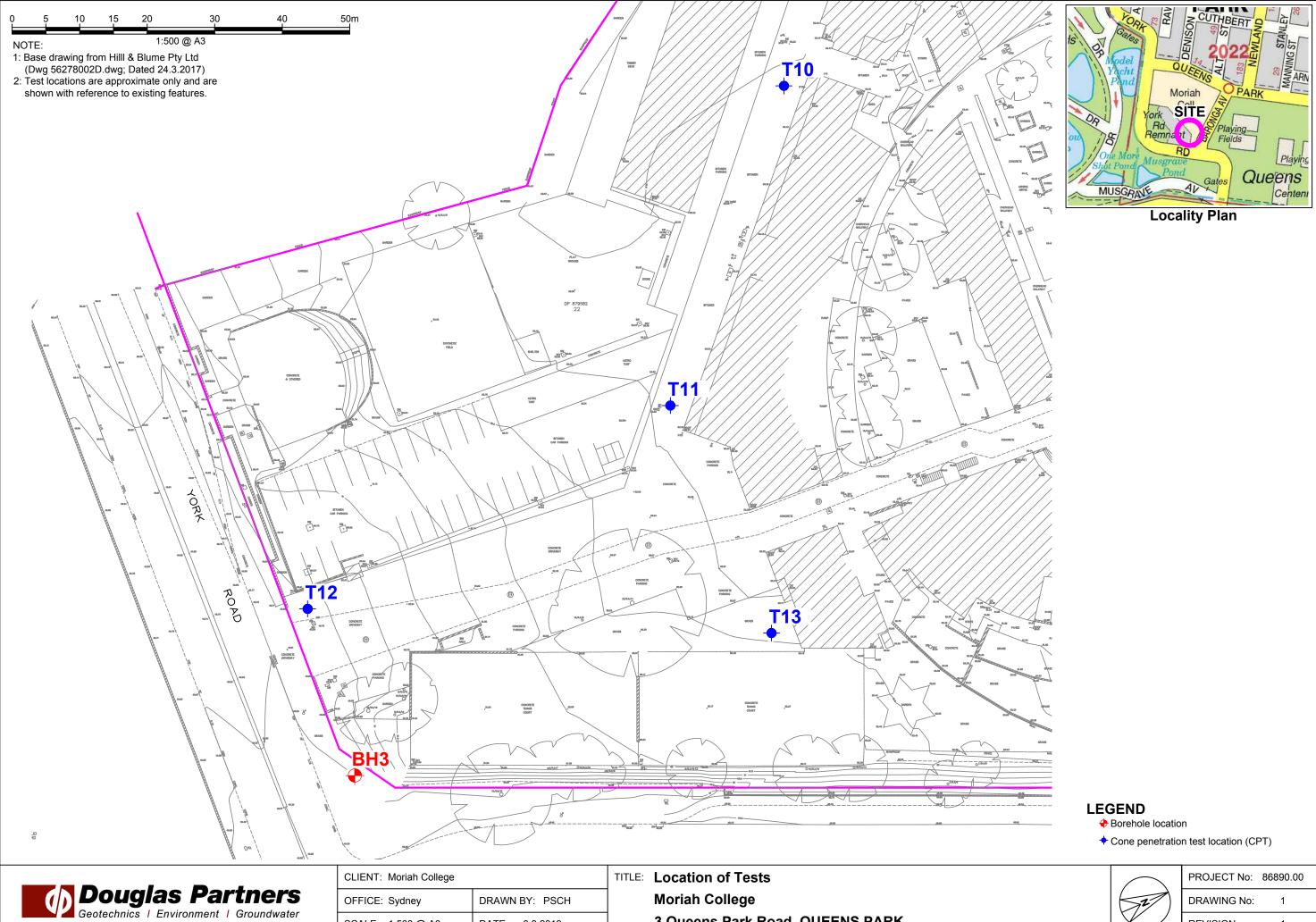
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate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

Site Inspection

The company will always be pleased to provide engineering inspection services for geotechnical and environmental aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

Appendix B

Drawing



3 Queens Park Road, QUEENS PARK

OFFICE: Sydney

SCALE: 1:500 @ A3

DRAWN BY: PSCH

DATE: 8.8.2019

		PROJECT No:	86890.00
DIVAWING NO. 1	2	DRAWING No:	1
REVISION: 1		REVISION:	1

Appendix C

Field Work Results

Sampling

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

Disturbed samples taken during drilling provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thinwalled sample tube into the soil and withdrawing it to obtain a sample of the soil 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.

Test Pits

Test pits are usually excavated with a backhoe or an excavator, allowing close examination of the insitu soil if it is safe to enter into the pit. The depth of excavation is limited to about 3 m for a backhoe and up to 6 m for a large excavator. A potential disadvantage of this investigation method is the larger area of disturbance to the site.

Large Diameter Augers

Boreholes can be drilled using a rotating plate or short spiral auger, generally 300 mm or larger in diameter commonly mounted on a standard piling rig. The cuttings are returned to the surface at intervals (generally not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube samples.

Continuous Spiral Flight Augers

The borehole is advanced using 90-115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are disturbed and may be mixed with soils from the sides of the hole. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively low reliability, due to the remoulding, possible mixing or softening of samples by groundwater.

Non-core Rotary Drilling

The borehole is advanced using a rotary bit, with water or drilling mud 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 the rate of penetration. Where drilling mud is used this can mask the cuttings and reliable identification is only possible from separate sampling such as SPTs.

Continuous Core Drilling

A continuous core sample can be obtained using a diamond tipped core barrel, usually with a 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in weak rocks and granular soils), this technique provides a very reliable method of investigation.

Standard Penetration Tests

Standard penetration tests (SPT) are used as a means of estimating the density or strength of soils 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 6.3.1.

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm 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 150 mm of, say, 4, 6 and 7 as:

4,6,7 N=13

In the case where the test is discontinued before the full penetration depth, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm as:

15, 30/40 mm

Sampling Methods

The results of the SPT tests can be related empirically to the engineering properties of the soils.

Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests

Dynamic penetrometer tests (DCP or PSP) are carried out by driving a steel rod into the ground using a standard weight of hammer falling a specified distance. As the rod penetrates the soil the number of blows required to penetrate each successive 150 mm depth are recorded. Normally there is a depth limitation of 1.2 m, but this may be extended in certain conditions by the use of extension rods. Two types of penetrometer are commonly used.

- Perth sand penetrometer a 16 mm diameter flat ended rod is driven using a 9 kg hammer dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands and is mainly used in granular soils and filling.
- Cone penetrometer a 16 mm diameter rod with a 20 mm diameter cone end is driven using a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). This 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.

Soil Descriptions

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are generally based on Australian Standard AS1726:2017, Geotechnical Site Investigations. In general, the descriptions include strength or density, colour, structure, soil or rock type and inclusions.

Soil Types

Soil types are described according to the predominant particle size, qualified by the grading of other particles present:

Туре	Particle size (mm)
Boulder	>200
Cobble	63 - 200
Gravel	2.36 - 63
Sand	0.075 - 2.36
Silt	0.002 - 0.075
Clay	<0.002

The sand and gravel sizes can be further subdivided as follows:

Туре	Particle size (mm)
Coarse gravel	19 - 63
Medium gravel	6.7 - 19
Fine gravel	2.36 - 6.7
Coarse sand	0.6 - 2.36
Medium sand	0.21 - 0.6
Fine sand	0.075 - 0.21

Definitions of grading terms used are:

- Well graded a good representation of all particle sizes
- Poorly graded an excess or deficiency of particular sizes within the specified range
- Uniformly graded an excess of a particular particle size
- Gap graded a deficiency of a particular particle size with the range

The proportions of secondary constituents of soils are described as follows:

In fine grained soils	(>35% fines)
-----------------------	--------------

Term	Proportion	Example
	of sand or	
	gravel	
And	Specify	Clay (60%) and
		Sand (40%)
Adjective	>30%	Sandy Clay
With	15 – 30%	Clay with sand
Trace	0 - 15%	Clay with trace
		sand

In coarse grained soils (>65% coarse)

with	clays	or	silts	

man olaye er ena		
Term	Proportion of fines	Example
And	Specify	Sand (70%) and Clay (30%)
Adjective	>12%	Clayey Sand
With	5 - 12%	Sand with clay
Trace	0 - 5%	Sand with trace
		clay

In coarse grained soils (>65% coarse)
 with coarser fraction

Term	Proportion	Example
	of coarser	
	fraction	
And	Specify	Sand (60%) and
		Gravel (40%)
Adjective	>30%	Gravelly Sand
With	15 - 30%	Sand with gravel
Trace	0 - 15%	Sand with trace
		gravel

The presence of cobbles and boulders shall be specifically noted by beginning the description with 'Mix of Soil and Cobbles/Boulders' with the word order indicating the dominant first and the proportion of cobbles and boulders described together.

Soil Descriptions

Cohesive Soils

Cohesive soils, such as clays, are classified on the basis of undrained shear strength. The strength may be measured by laboratory testing, or estimated by field tests or engineering examination. The strength terms are defined as follows:

Description	Abbreviation	Undrained shear strength (kPa)
Very soft	VS	<12
Soft	S	12 - 25
Firm	F	25 - 50
Stiff	St	50 - 100
Very stiff	VSt	100 - 200
Hard	Н	>200
Friable	Fr	-

Cohesionless Soils

Cohesionless soils, such as clean sands, are classified on the basis of relative density, generally from the results of standard penetration tests (SPT), cone penetration tests (CPT) or dynamic penetrometers (PSP). The relative density terms are given below:

Relative Density	Abbreviation	Density Index (%)
Very loose	VL	<15
Loose	L	15-35
Medium dense	MD	35-65
Dense	D	65-85
Very dense	VD	>85

Soil Origin

It is often difficult to accurately determine the origin of a soil. Soils can generally be classified as:

- Residual soil derived from in-situ weathering of the underlying rock;
- Extremely weathered material formed from in-situ weathering of geological formations. Has soil strength but retains the structure or fabric of the parent rock;
- Alluvial soil deposited by streams and rivers;

- Estuarine soil deposited in coastal estuaries;
- Marine soil deposited in a marine environment;
- Lacustrine soil deposited in freshwater lakes;
- Aeolian soil carried and deposited by wind;
- Colluvial soil soil and rock debris transported down slopes by gravity;
- Topsoil mantle of surface soil, often with high levels of organic material.
- Fill any material which has been moved by man.

Moisture Condition – Coarse Grained Soils For coarse grained soils the moisture condition

should be described by appearance and feel using the following terms:

- Dry (D) Non-cohesive and free-running.
- Moist (M) Soil feels cool, darkened in colour.

Soil tends to stick together. Sand forms weak ball but breaks easily.

Wet (W) Soil feels cool, darkened in colour.

Soil tends to stick together, free water forms when handling.

Moisture Condition – Fine Grained Soils

For fine grained soils the assessment of moisture content is relative to their plastic limit or liquid limit, as follows:

- 'Moist, dry of plastic limit' or 'w <PL' (i.e. hard and friable or powdery).
- 'Moist, near plastic limit' or 'w ≈ PL (i.e. soil can be moulded at moisture content approximately equal to the plastic limit).
- 'Moist, wet of plastic limit' or 'w >PL' (i.e. soils usually weakened and free water forms on the hands when handling).
- 'Wet' or 'w ≈LL' (i.e. near the liquid limit).
- 'Wet' or 'w >LL' (i.e. wet of the liquid limit).

Symbols & Abbreviations

Introduction

These notes summarise abbreviations commonly used on borehole logs and test pit reports.

Drilling or Excavation Methods

С	Core drilling
R	Rotary drilling
SFA	Spiral flight augers
NMLC	Diamond core - 52 mm dia
NQ	Diamond core - 47 mm dia
HQ	Diamond core - 63 mm dia
PQ	Diamond core - 81 mm dia

Water

\triangleright	Water seep
\bigtriangledown	Water level

Sampling and Testing

- A Auger sample
- B Bulk sample
- D Disturbed sample
- E Environmental sample
- U₅₀ Undisturbed tube sample (50mm)
- W Water sample
- pp Pocket penetrometer (kPa)
- PID Photo ionisation detector
- PL Point load strength Is(50) MPa
- S Standard Penetration Test
- V Shear vane (kPa)

Description of Defects in Rock

The abbreviated descriptions of the defects should be in the following order: Depth, Type, Orientation, Coating, Shape, Roughness and Other. Drilling and handling breaks are not usually included on the logs.

Defect Type

В	Bedding plane
Cs	Clay seam
Cv	Cleavage
Cz	Crushed zone
Ds	Decomposed seam
F	Fault
J	Joint
Lam	Lamination
Pt	Parting
Sz	Sheared Zone
V	Vein

Orientation

The inclination of defects is always measured from the perpendicular to the core axis.

- h horizontal
- v vertical
- sh sub-horizontal

art

sv sub-vertical

Coating or Infilling Term

cln	clean
со	coating
he	healed
inf	infilled
stn	stained
ti	tight
vn	veneer

Coating Descriptor

ca	calcite
cbs	carbonaceous
cly	clay
fe	iron oxide
mn	manganese
slt	silty

Shape

cu	curved
ir	irregular
pl	planar
st	stepped
un	undulating

Roughness

ро	polished
ro	rough
sl	slickensided
sm	smooth
vr	very rough

Other

fg	fragmented
bnd	band
qtz	quartz

Symbols & Abbreviations

Graphic Symbols for Soil and Rock

General

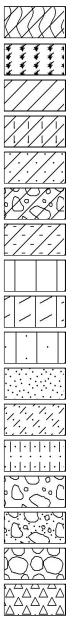
A. A. A. Z	

Asphalt Road base

Concrete

Filling

Soils



Topsoil
Peat
Clay
Silty clay
Sandy clay
Gravelly clay
Shaly clay

Silt

Clayey silt

Sandy silt

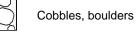
Sand

Clayey sand

Silty sand

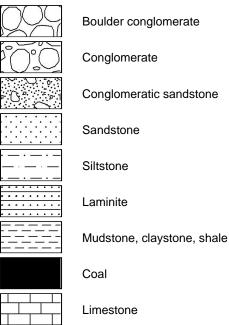
Gravel

Sandy gravel



Talus

Sedimentary Rocks



Metamorphic Rocks

Slate, phyllite, schist

Quartzite

Gneiss

Igneous Rocks

Granite

Dolerite, basalt, andesite

Dacite, epidote

Tuff, breccia

Porphyry

Cone Penetration Tests

Introduction

The Cone Penetration Test (CPT) is a sophisticated soil profiling test carried out in-situ. A special cone shaped probe is used which is connected to a digital data acquisition system. The cone and adjoining sleeve section contain a series of strain gauges and other transducers which continuously monitor and record various soil parameters as the cone penetrates the soils.

The soil parameters measured depend on the type of cone being used, however they always include the following basic measurements

qc

fs

i

7

- Cone tip resistance
- Sleeve friction
- Inclination (from vertical)
- Depth below ground

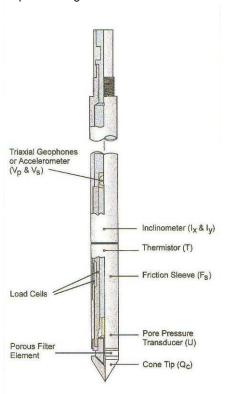


Figure 1: Cone Diagram

The inclinometer in the cone enables the verticality of the test to be confirmed and, if required, the vertical depth can be corrected.

The cone is thrust into the ground at a steady rate of about 20 mm/sec, usually using the hydraulic rams of a purpose built CPT rig, or a drilling rig. The testing is carried out in accordance with the Australian Standard AS1289 Test 6.5.1.



Figure 2: Purpose built CPT rig

The CPT can penetrate most soil types and is particularly suited to alluvial soils, being able to detect fine layering and strength variations. With sufficient thrust the cone can often penetrate a short distance into weathered rock. The cone will usually reach refusal in coarse filling, medium to coarse gravel and on very low strength or better rock. Tests have been successfully completed to more than 60 m.

Types of CPTs

Douglas Partners (and its subsidiary GroundTest) owns and operates the following types of CPT cones:

Туре	Measures
Standard	Basic parameters (q _c , f _s , i & z)
Piezocone	Dynamic pore pressure (u) plus basic parameters. Dissipation tests estimate consolidation parameters
Conductivity	Bulk soil electrical conductivity (σ) plus basic parameters
Seismic	Shear wave velocity (V_s) , compression wave velocity (V_p) , plus basic parameters

Strata Interpretation

The CPT parameters can be used to infer the Soil Behaviour Type (SBT), based on normalised values of cone resistance (Qt) and friction ratio (Fr). These are used in conjunction with soil classification charts, such as the one below (after Robertson 1990)

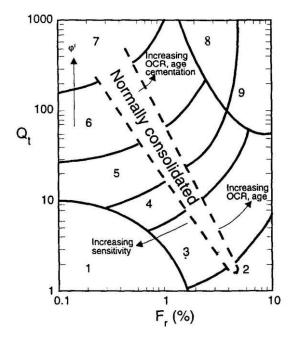


Figure 3: Soil Classification Chart

DP's in-house CPT software provides computer aided interpretation of soil strata, generating soil descriptions and strengths for each layer. The software can also produce plots of estimated soil parameters, including modulus, friction angle, relative density, shear strength and over consolidation ratio.

DP's CPT software helps our engineers quickly evaluate the critical soil layers and then focus on developing practical solutions for the client's project.

Engineering Applications

There are many uses for CPT data. The main applications are briefly introduced below:

Settlement

CPT provides a continuous profile of soil type and strength, providing an excellent basis for settlement analysis. Soil compressibility can be estimated from cone derived moduli, or known consolidation parameters for the critical layers (eg. from laboratory testing). Further, if pore pressure dissipation tests are undertaken using a piezocone, in-situ consolidation coefficients can be estimated to aid analysis.

Pile Capacity

The cone is, in effect, a small scale pile and, therefore, ideal for direct estimation of pile capacity. DP's in-house program ConePile can analyse most pile types and produces pile capacity versus depth plots. The analysis methods are based on proven static theory and empirical studies, taking account of scale effects, pile materials and method of installation. The results are expressed in limit state format, consistent with the Piling Code AS2159.

Dynamic or Earthquake Analysis

CPT and, in particular, Seismic CPT are suitable for dynamic foundation studies and earthquake response analyses, by profiling the low strain shear modulus G_0 . Techniques have also been developed relating CPT results to the risk of soil liquefaction.

Other Applications

Other applications of CPT include ground improvement monitoring (testing before and after works), salinity and contaminant plume mapping (conductivity cone), preloading studies and verification of strength gain.

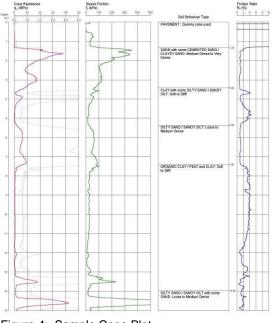


Figure 4: Sample Cone Plot

BOREHOLE LOG

SURFACE LEVEL: 49.3 AHD EASTING: NORTHING:

DIP/AZIMUTH: 90°/--

BORE No: BH3 PROJECT No: 28900 DATE: 28-4-2000 SHEET 1 OF 1

_	Description	Degree of Weathering			Fracture	Discontinuities	Sa	ampli	ng &	In Situ Testing	
Depth (m)	of	g	Log	Ex Low Very Low Medium Very High Xery High	Spacing State (m)	B - Bedding J - Joint	Type	ore :. %	RQD %	Test Results &	
N/		FR S W W W	G	Ex Low Medic Very FX High	0.01 0.10 1.00 1.00	S - Shear F - Fault	ļ	ပိမ္မိ	R %	& Comments	
	FILL: light grey sand, concrete and brick fill		\boxtimes								
			\bigotimes		1 11 11						
0.5	SAIND: Very loose and loose, numid										
	to damp, light yellow brown, fine to medium grained sand										
- 1								-			
							s			2,2,2 N = 4	
								-			
-2											
					1 11 11						
							s			3,3,3 N = 6	
- 3							<u> </u>	1		N - 0	
-4											
	4.0m to 5.5m: medium dense				i ii ii		s			4,6,7 N = 13	
								-		IN - 13	
- 5											
					1 11 11						
5.5	SAND: medium dense inep verv	$\begin{array}{c} 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 $					s			8.,9,10	
	dense, humid to damp, light grey, fine to medium grained sand with a						Ľ			N = 19	
- 6	trace of clay										
-7								1		14 13 13	
							S			14,13,13 N = 26	
- 8											
			····								
-9								-		17.05/400	
							s			17,25/130 refusal	
							<u> </u>	1			
10.0	Bore discontinued at 10.0m										
G: Scou		LER: Kiernar			GGED: Parma	CASING:					

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Surface level interpolated from survey plan

Colin Ging & Partners

LOCATION: Queens Park Road, Bondi Junction

Moriah War Memorial College

CLIENT: PROJECT:

 SAMPLING & IN SITU TESTING LEGEND

 A Auger sample
 G Gas sample
 Pliston sample</t

CLIENT: COLIN GING AND PARTNERS

PROJECT: MORIAH WAR MEMORIAL COLLEGE

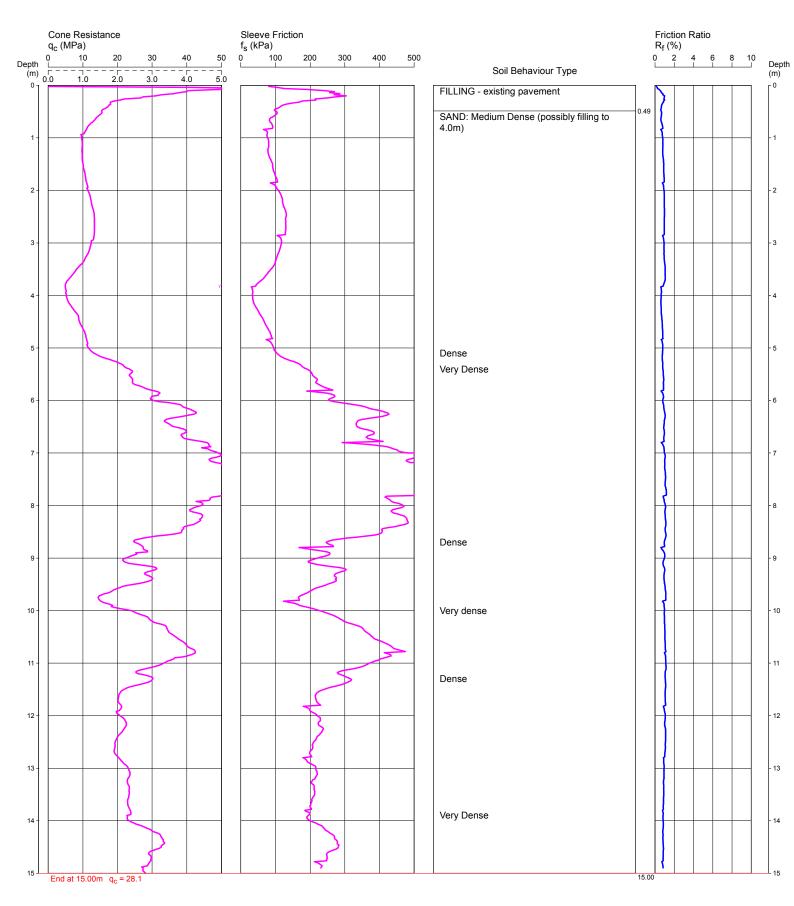
LOCATION: QUEENS PARK ROAD, BONDI JUNCTION

AHD

REDUCED LEVEL:53.5

COORDINATES:

CPT T10 Page 1 of 1 DATE 12 APR 2000 PROJECT No: 28900



REMARKS: NO FREE GROUND WATER OBSERVED TO 8 METRES DEPTH

 File:
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 Cone ID:
 CONE-157
 Type: 2 Standard

Douglas Partners Geotechnics | Environment | Groundwster

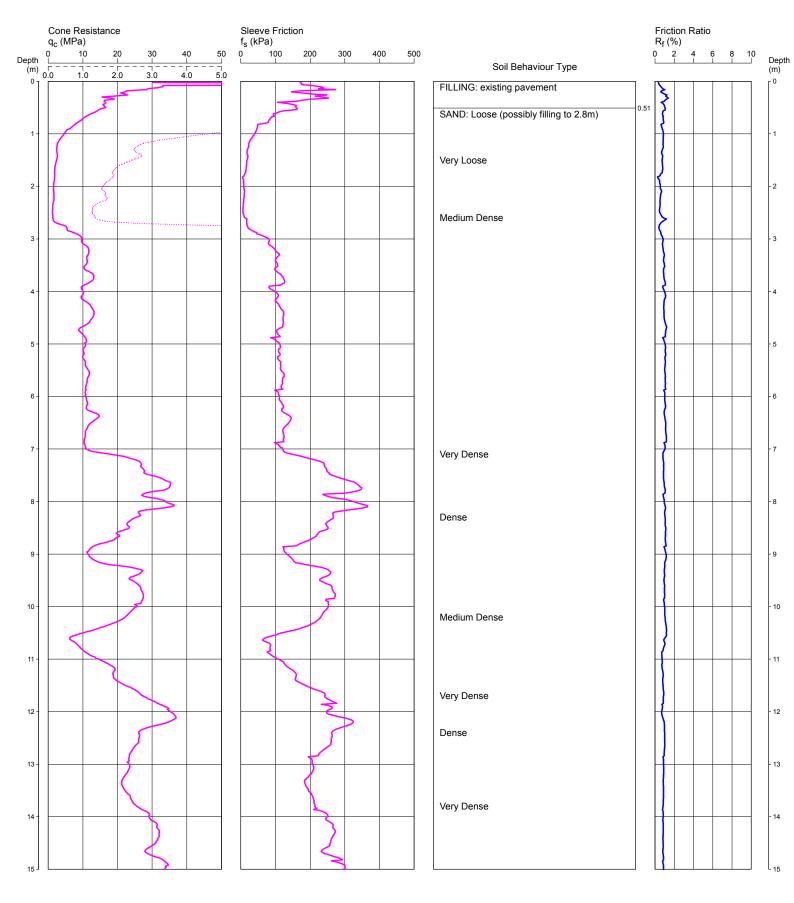
CLIENT: COLIN GING AND PARTNERS

PROJECT: MORIAH WAR MEMORIAL COLLEGE

LOCATION: QUEENS PARK ROAD, BONDI JUNCTION



REDUCED LEVEL:53.0 COORDINATES: AHD



REMARKS: HOLE COLLAPSE AT 3.8 METRES DEPTH

File: P:\86890.00 - QUEENS PARK, 3 Queens Park Road, Geo\4.0 Field Work\4.2 Testing\CPT11.CP5
Cone ID: CONE-157
Type: 2 Standard

ConePlot Version 5.9.2 © 2003 Douglas Partners Pty Ltd



CLIENT: COLIN GING AND PARTNERS

PROJECT: MORIAH WAR MEMORIAL COLLEGE

LOCATION: QUEENS PARK ROAD, BONDI JUNCTION



REDUCED LEVEL:53.0 COORDINATES: AHD

(Cone Resistance q _c (MPa)	Sleeve Friction f _s (kPa)		F	Frictio	n Rat	io		
0 pth נ	0 10 20 30 40 50	0 100 200 300 400 500	Soil Behaviour Type	0	2	4	6	8 10	Depth (m)
(m) [15] [0 1.0 2.0 3.0 4.0 5.0		SAND: Loose (possibly filling to 2.8m)	[(III) ¹⁵
16			Dense						- 16
		2	Dense		Ì				10
17 -			Very Dense	-	$\left\{ \right\}$				- 17
			SANDY CLAY: Stiff	17.60	ξ				
18			SAND- Dense	18.24	7	•			- 18
19									- 19
		2	CLAYEY SAND: medium dense	19.50	Y				
20 -	3		SAND: Very Dense	19.92					- 20
21 -									- 21
2	End at 22.00m q _c = 45.7			22.00	*				22
3									- 23
-									- 24
5									- 25
;									- 26
7-									- 27
3-							+	+	- 28
									~
9									- 29
									30

REMARKS: HOLE COLLAPSE AT 3.8 METRES DEPTH

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 Cone ID:
 CONE-157
 Type: 2 Standard



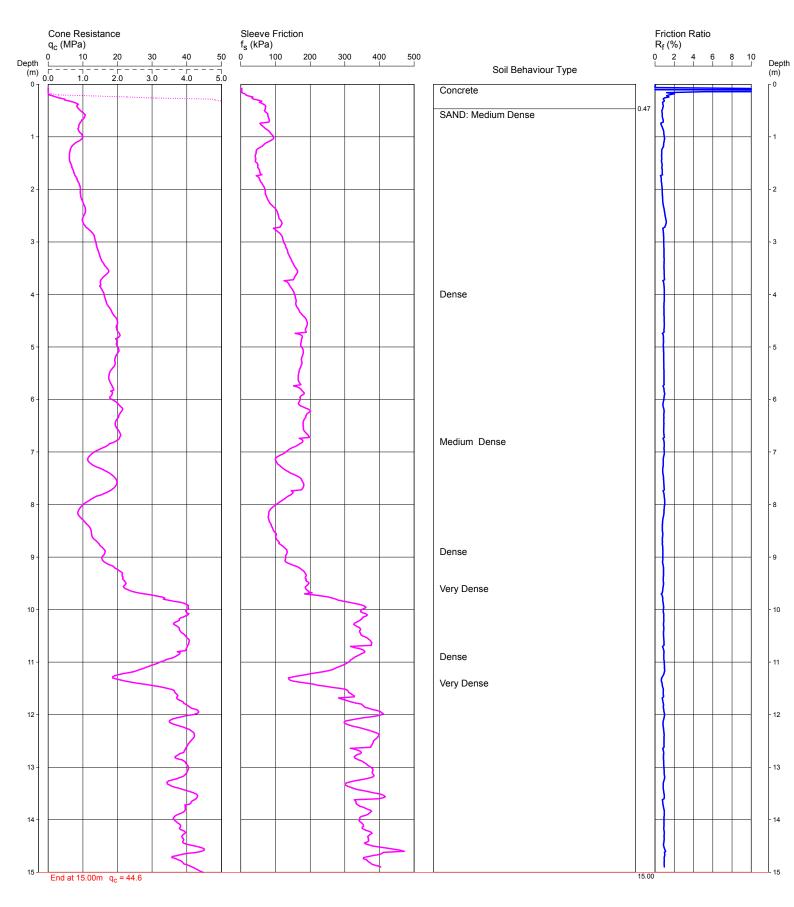
CLIENT: COLIN GING AND PARTNERS

PROJECT: MORIAH WAR MEMORIAL COLLEGE

LOCATION: QUEENS PARK ROAD, BONDI JUNCTION



REDUCED LEVEL:50.0 COORDINATES: AHD



REMARKS: HOLE COLLAPSE AT 7.2 METRES DEPTH

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 Cone ID: CONE-157
 Type: 2 Standard



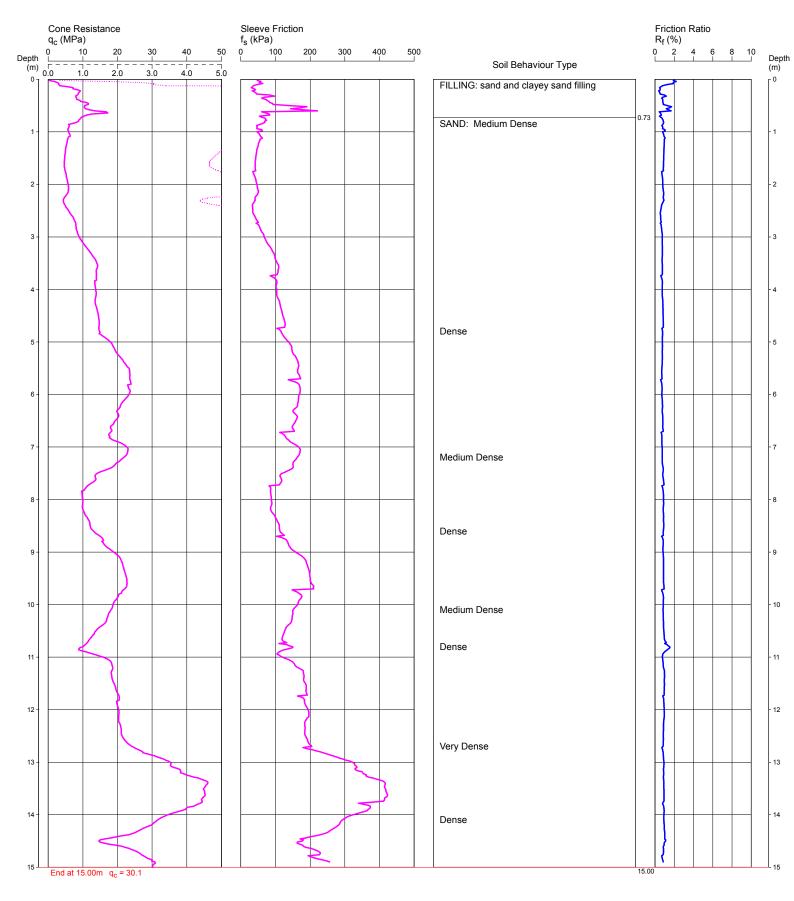
CLIENT: COLIN GING AND PARTNERS

PROJECT: MORIAH WAR MEMORIAL COLLEGE

LOCATION: QUEENS PARK ROAD, BONDI JUNCTION CPT T13 Page 1 of 1 DATE 13 APR 2000 PROJECT No: 28900

COORDINATES:

REDUCED LEVEL: 52.9



REMARKS: NO FREE GROUND WATER TO 8 METRES DEPTH

 File:
 P:\86890.00 - QUEENS PARK, 3 Queens Park Road, Geo\4.0 Field Work\4.2 Testing\CPT13.CP5

 Cone ID:
 CONE-157
 Type: 2 Standard

