



Douglas Partners

Geotechnics | Environment | Groundwater

Report on
Geotechnical Investigation

Proposed Commercial Development
Site 8C, Murray Rose Avenue
Sydney Olympic Park

Prepared for
FDC Construction & Fitout Pty Ltd

Project 73595
September 2013

Integrated Practical Solutions





Douglas Partners

Geotechnics | Environment | Groundwater

Document History

Document details

Project No.	73595	Document No.	1
Document title	Report on Geotechnical Investigation Proposed Commercial Development		
Site address	Site 8C, Murray Rose Avenue, Sydney Olympic Park		
Report prepared for	FDC Construction & Fitout Pty Ltd		
File name	2013-09-09 Site 8C Sydney Olympic Park Geotechnical Report		

Document status and review

Revision	Prepared by	Reviewed by	Date issued
1	R.Agnihotri	M.Thom	10 September 2013

Distribution of copies

Revision	Electronic	Paper	Issued to
1	1	3	FDC Construction & Fitout Pty Ltd

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	Date
Author		10.9.13
Reviewer		10.9.13



Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au
96 Hermitage Road
West Ryde NSW 2114
PO Box 472
West Ryde NSW 1685
Phone (02) 9809 0666
Fax (02) 9809 4095

Table of Contents

	Page
1. Introduction.....	1
2. Previous Investigations and Experience	1
3. Site Description and Geology	1
4. Field Work Methods	2
5. Field Work Results	2
6. Point Load Strength Tests.....	3
7. Proposed Development.....	3
8. Comments	4
8.1 Site Preparation and Earthworks	4
8.1.1 Excavation Conditions	4
8.1.2 Seepage	4
8.1.3 Dilapidation Surveys.....	4
8.1.4 Noise and Vibrations	4
8.1.5 Disposal of Excavated Material	5
8.2 Excavation Support.....	5
8.3 Foundations	7
8.4 Impacts on Rail Infrastructure	7
9. Limitations	8
 Appendix A: About this Report	
 Appendix B: Drawings	
 Appendix C: Results of Field Work	

Report on Geotechnical Investigation

Proposed Commercial Development

Site 8C, Murray Rose Avenue, Sydney Olympic Park

1. Introduction

This report presents the results of a geotechnical investigation undertaken for a proposed commercial development at Site 8C, located on Murray Rose Avenue within the commercial precinct of Sydney Olympic Park. The work was commissioned by FDC Construction & Fitout Pty Ltd.

The proposed development includes construction of a six-storey commercial building with two levels of basement carparking. The lowest basement level will require bulk excavation to depths of approximately 6 m to 6.5 m. The geotechnical investigation was carried out to provide information on the subsurface profile for design and planning purposes.

The investigation included four rock cored boreholes, and laboratory testing of selected rock core samples, followed by engineering analysis and reporting. Details of the field and laboratory work are provided in the report, together with comments on design and construction practice.

Douglas Partners Pty Ltd (DP) carried out an in situ waste classification of the site in conjunction with the geotechnical investigation. The results have been reported separately (Project No. 73595, dated 10 September 2013).

2. Previous Investigations and Experience

DP previously carried out a geotechnical investigation for Site 8B on the adjacent property to the east of the subject site, and prepared a report dated 28 February 2011 (Project 72233). The investigation included four rock cored boreholes, with one of the boreholes (BH104) located close to the north-eastern corner of Site 8C. The borehole log for BH104 is included in Appendix C of this report and the location is shown on Drawing 1 in Appendix B.

3. Site Description and Geology

The site is a rectangular shaped lot with a plan area of 1170 m² and a southern frontage along Murray Rose Avenue. At the time of the investigation the site was an asphaltic concrete paved carpark. The site is relatively level with the ground surface level typically varying from between RL 16.2 m to RL 16.5 m AHD.

On the adjacent property to the east of the site (Site 8B) is a six-storey commercial building with two levels of basement carparking. At the western end of the Site 8B basement, along the common

boundary with Site 8B, is a batter slope covered with a protective layer of shotcrete. The batter slope will be removed to link the basements below Site 8B and 8C.

The Olympic Park Train Station is located to the south of the site on the southern side of Murray Rose Avenue. Drawings provided by Bates Smart Pty Ltd indicate that the railway tunnel is located approximately 35 m from the site with the base of the tunnel approximately 10 m below the ground surface. Specific details of the tunnel and its construction have not been confirmed.

Reference to the Sydney 1:100 000 Geological Series Map indicates that the site is underlain by Ashfield Shale which typically comprises black to dark grey shale, siltstone and laminite (finely interbedded siltstone and fine grained sandstone). These rocks typically weather to a residual clay profile of medium to high plasticity. The geological mapping was confirmed by the field work which identified residual soils overlying bedrock including shale, laminite and siltstone.

4. Field Work Methods

The field work for the investigation included four boreholes (BH201 to 204, inclusive) drilled to depths of between 8.9 m to 9.1 m using a truck-mounted rig. The boreholes were initially drilled to depths of up to 2.8 m using spiral flight augers and rotary drilling within the soils and extremely low to very low strength rock. The boreholes were then cased and continued into the underlying rock using diamond core drilling techniques to obtain continuous core samples of the bedrock. Standard Penetration Tests (SPTs) were carried out within the boreholes at 1.5 m depth intervals to sample the soil and weathered rock and to assess the in situ strength of the materials.

The rock cores recovered from the boreholes were returned to the DP office where they were logged by a geologist, the cores photographed and Point Load Strength Index (IS_{50}) tests carried out on selected samples of the rock core.

The ground surface levels at the test locations were interpolated from surface levels shown on the survey plan by Land Partners Pty Ltd (Plan No 72664 Rev B, dated 06/04/11).

The borehole locations are shown on Drawing 1 in Appendix B.

5. Field Work Results

Details of the subsurface conditions encountered are given in the borehole logs in Appendix C, together with photographs of rock core samples and notes defining classification methods and descriptive terms.

The sequence of subsurface materials encountered within the boreholes, in increasing depth order, is described below:

PAVEMENT: Bitumen and blue metal gravel 50 mm thick, overlying road base between 50 mm and 150 mm in thickness;

- FILLING:** Clay and silty clay filling between 300 mm and 700 mm in thickness, including ironstone gravel (BH203) and crushed shale gravel (BH202);
- CLAY:** Hard clay was encountered in BH201 and BH204 only, to depths of 2 m (RL 114.3 m). Some extremely low strength laminite bands were present in BH201, and shaley clay bands were present in BH204;
- ROCK:** Shale and laminite were encountered below the clay at depths of between 0.5 m and 2.0 m (RL 115.8 m to RL 114.3 m). The two boreholes near the site's northern boundary (BH202 and BH204) indicated the presence of extremely low strength shale below the clay, to depths of between 2.0 m and 2.8 m (RL 114.3 m to RL 113.5 m). The other two boreholes, near the site's southern boundary (BH201 and BH203), indicated the presence of extremely low strength laminite below the clay, to depths of between 2.4 m and 2.9 m (RL 113.9 m to RL 113.4 m). The extremely low strength rock was underlain by fragmented to fractured, extremely low to medium strength laminite to depths of between 4.5 m and 6.9 m (RL 111.8 m to RL 109.4 m), grading to slightly fractured to unbroken laminite, of medium to high strength, below these depths.

No groundwater was observed during the auger drilling of the boreholes to depths of up to 2.8 m. The use of water for rotary drilling and coring prevented observation of groundwater below this depth.

An inferred geological cross-section through the site, based on the borehole logs, is presented in Appendix B. The geological strata boundaries indicated on the cross-section are relative to the Project Datum, which is 100 m lower than Australian Height Datum (AHD).

6. Point Load Strength Tests

Selected samples of the rock core were tested in the laboratory to determine the Point Load Strength Index (Is_{50}) values to assist with the rock strength classification. The results of the testing are shown on the borehole logs at the appropriate depth. The Is_{50} values were used to provide an estimate of the unconfined compressive strength (UCS) based on a UCS: Is_{50} ratio of 20:1. The Is_{50} values for the rock typically ranged from 0.1 MPa to 1.5 MPa, corresponding to a low to high strength classification (estimated UCS ranging from 2 MPa to 30 MPa).

7. Proposed Development

Architectural plans by Bates Smart Pty Ltd (dated 12/08/13) indicate that the proposed development includes the construction of a six-storey commercial building with two levels of basement carparking. Generally, the lowest basement level (approximately RL 110 m, Project Datum) will require bulk excavation to depths of approximately 6 m to 6.5 m. In localised areas, pits will be excavated for stormwater tanks, with the lowest excavation level proposed to be approximately RL 107 m. The basement excavation will extend up to the site boundaries.

8. Comments

8.1 Site Preparation and Earthworks

8.1.1 Excavation Conditions

The investigation indicates that the excavation for the basement will require the removal of soil and extremely low to medium-to-high strength rock. Slightly fractured to unbroken, medium and high strength rock is expected within the deeper parts of the excavation, particularly for the detailed excavation of footings, services and lift pits.

Excavation of soil and extremely low to low strength rock should be achievable using conventional earthmoving equipment, however, the assistance of rock hammering or ripping will probably be required for effective removal of medium strength bands within the weathered rock sequence. Excavation of low to medium strength rock may require moderate ripping with an excavator whilst excavation of medium and high strength rock will require heavy ripping with a large excavator or bulldozer. Productivity within slightly fractured to unbroken, medium to high strength rock may be low (even with large dozers) and therefore some pre-splitting or rock hammering may be necessary to improve efficiency.

8.1.2 Seepage

Some seepage may be expected along the top of the clay and rock surface and may also occur within fractured zones and joints within the underlying rock profile. Seepage flows are likely to increase following periods of extended wet weather. During construction and in the long term, it is anticipated that seepage flows should be readily controlled by perimeter drains connected to a "sump-and-pump" system. A drained basement could require permanent or temporary subfloor drainage below the basement floor slab, connected to a sump-and-pump system with periodic pumping.

8.1.3 Dilapidation Surveys

Consideration should be given to undertaking dilapidation surveys on surrounding buildings and pavements that may be affected by the basement construction. The dilapidation surveys should be undertaken before the commencement of any excavation work in order to document any existing defects so that any claims for damage due to construction related activities can be accurately assessed.

8.1.4 Noise and Vibrations

During excavation, it will be necessary to use appropriate methods and equipment to keep ground vibrations at adjacent buildings and structures within acceptable limits.

Ground vibration can be strongly perceptible to humans at levels above 2.5 mm/s component peak particle velocity (PPVi). This is generally much lower than the vibration levels required to cause structural damage to buildings. The Australian Standard AS2670.2-1990 "*Evaluation of human exposure to whole-body vibrations – continuous and shock induced vibrations in buildings (1-80 Hz)*" indicates an acceptable day time limit of 8 mm/s component PPVi for human comfort.

Based on the experience of DP and reference to AS2670, it is suggested that a maximum component PPVi of 8 mm/sec (applicable at the foundation level of existing buildings) be employed at this site to reduce the risk of structural damage to surrounding buildings.

As the magnitude of vibration transmission is site specific, it is recommended that a vibration trial be undertaken at the commencement of excavation. The trial may indicate that smaller or different types of excavation equipment should be used when undertaking rock excavation, and should be carried out before any heavy ripping of the rock is commenced. The initial stages of the excavation, during the vibration trial, should be undertaken in the centre of the site to minimise the risk of damage to surrounding structures.

8.1.5 Disposal of Excavated Material

All excavated materials will need to be disposed of in accordance with the current *Waste Classification Guidelines* (DECC, April 2008). Reference should be made to the DP in situ waste classification report (Project No. 73595, dated 10 September 2013) for details on the contamination status and waste classification of the site soils.

8.2 Excavation Support

Vertical excavations within the soils and rock will require both temporary and permanent lateral support during and after excavation. It is anticipated that a bored soldier pile wall with shotcrete infill panels would be suitable. Typically, soldier piles are spaced at approximately 2 m to 3 m centres however closer spaced piles may be required to limit wall movements, or collapse of infill materials, where structures or services are located in close proximity to the excavation. Generally, shotcrete panels should be constructed in 2 m maximum depth intervals within soil and extremely low to very low strength rock, and then in 3 m depth intervals within low to medium strength rock or better.

Shoring piles should be founded on rock at least 1.0 m below the base of the bulk excavation and nearby localised excavation level in order to provide lateral restraint at the base of the excavation. Suitably sized drilling rigs fitted with rock augers will be required to penetrate medium and high strength rock. Shoring piles may be used to carry vertical structural loads and may be designed on the basis of the allowable foundation pressures given in Section 8.3.

The design of the shoring will depend somewhat upon whether it is cantilevered or restrained by rows of temporary rock anchors. It is anticipated that at least one or possibly two rows of rock anchors will be required to provide lateral restraint to shoring piles.

It is suggested that design of shoring systems with a single row of anchors be based on a triangular earth pressure distribution based on earth pressure coefficients provided in Table 1. 'Active' earth pressures (K_a) may be used where some wall movement is acceptable, and 'at rest' earth pressures (K_0) should be used where wall movement needs to be limited. Cantilevered shoring piles are unlikely to be suitable for use with this excavation.

Table 1: Recommended Earth Pressure Coefficients and Bulk Unit Weights

Retained Material	Bulk Density kN/m ³	Earth Pressure Coefficient	
		Active (K_a)	At Rest (K_0)
Filling	20	0.4	0.6
Stiff clay	20	0.35	0.5
Extremely to very low strength rock	22	0.2	0.3
Low to medium strength rock	22	0.1	0.2
Medium to high strength rock	24	0	0

All surcharge loads should be allowed for in the shoring design, including building footings, inclined slopes behind the wall, traffic and construction related activities.

Preliminary design for lateral earth pressures for walls with more than one row of anchors may be based on a uniform rectangular earth pressure distribution. The additional lateral pressures due to surcharge loading behind the wall and hydrostatic pressures (if appropriate) must also be considered. Where lateral movement is less critical a pressure distribution of 4H may be considered (where H is the depth to the top of the slightly fractured to unbroken, medium to high strength rock). For situations where movements are critical, a higher uniform pressure of 6H is suggested. For detailed design a computer analysis package such as WALLAP, FLAC, PLAXIS or similar should be used to model the excavation and anchoring sequence, to refine the design and provide estimates of possible lateral movements.

The design of temporary and permanent support will also need to consider the possibility that 45° joints in the rock will daylight near the base of the shoring wall leading to wedges of rock which need to be supported by the temporary and permanent retaining structures. The support system would typically comprise rock bolts or anchors spaced at 2 m to 3 m centres over the rock face. These anchors should have their bond lengths formed in rock behind a line projected up at 45° from the base of the shoring. As a guide, the support system should be designed to withstand an anchor load per unit width of $4.2H^2$ (kN) where H is the height of the excavation in metres. This approximation of the horizontal force required to support a 45° wedge is based on an anchor inclination of 10° below horizontal, an average bulk weight of 21 kN/m³, and friction angle of 25° and cohesion of 0 kPa along the failure plane. Given that there is a low probability that a joint would run the full length and height of the excavation it suggested that this aspect of design may be carried out for a factor of safety of 1.1.

Shoring walls should be designed for full hydrostatic pressures unless drainage of the ground behind impermeable walls can be provided. Drainage could comprise 150 mm wide strip drains pinned to the face at 2 m centres behind shotcrete in-fill panels. The base of the strip drains should extend out from the shoring wall to allow any seepage to flow into a perimeter toe drain which is connected to a sump dewatering system.

Passive resistance for piles founded in medium to high strength rock below the base of the excavation (including allowance for detailed excavation for services or footings) may be based on an ultimate passive restraint equal 4000 kPa in medium to high strength or greater rock. A factor of safety must

be applied to these ultimate values. The top 0.5 m of the rock socket should be ignored due to possible disturbance and tolerance effects.

Temporary ground anchoring or alternative means of support are required until permanent support can be provided by the completed structure. Ground anchors could be constructed in the conventional manner by drilling and grouting the anchors which may be designed on the basis of the maximum allowable bond stresses given in Table 2.

Table 2: Allowable Bond Stresses for Rock Anchor Design

Material Description	Maximum Allowable Bond Stress (kPa)
Extremely low to very low strength	50
Low to medium strength rock	200
Medium to high strength rock	500

The parameters given in Table 2 assume that drilled holes are clean and adequately flushed. Lift-off tests should be carried out to confirm the bond capacities. Higher bond stress values may be adopted if trial anchors are used to prove higher capacities.

8.3 Foundations

It is expected that the proposed bulk excavation level at approximately RL 110 m (Project Datum) will expose fractured to unbroken, medium to high strength laminite at, or close to the base of the excavation. BH201 at the eastern end of the site encountered medium strength laminite, with some very low strength bands, to a depth of approximately 0.6 m below the proposed bulk excavation level.

All structural loads should be uniformly supported on the underlying medium to high strength rock, for which pad footings should be appropriate. Footings founded on medium to high strength rock may be designed for an allowable end bearing pressure of 6000 kPa. Footings proportioned on this basis would be expected to experience total settlements of less than 1% of the footing width under applied working loads, with differential settlements between adjacent columns expected to be less than half of this value.

All footings should be inspected by a geotechnical engineer to confirm that foundation conditions are suitable for the design parameters. Spoon testing should be undertaken in at least one-third of the footings which are proportioned on the basis of allowable bearing pressures of greater than 6000 kPa. The purpose of spoon testing is to check that no significant weak seams exist below the base of the footing within a depth equal to 1.5 times the least footing dimension. If the testing identifies the presence of weak seams then the footing will either have to be deepened or widened to reduce the actual bearing pressure.

8.4 Impacts on Rail Infrastructure

The Olympic Park Train Station is located to the south of the site on the southern side of Murray Rose Avenue. Based on information provided by Bates Smart Pty Ltd and FDC Construction and Fitout Pty Ltd, it is understood that the edge of the railway tunnel is located approximately 30 m from the site,

with the base of the tunnel approximately 10 m below the ground surface. The railway station walls are set back approximately 24 m from the site.

Based on previous experience in the Sydney area, it is anticipated that the weathered rock profile within the proposed depth of excavation will have relatively low in-situ horizontal stresses. Therefore we do not expect any stress relief effects to impact the adjacent rail infrastructure which is set back at least 24 m from the proposed excavation.

Rock hammers will probably be required to remove medium to high strength rock in the base of the excavation. Based on DP's experience, it is expected that ground vibrations resulting from rock hammers will be less than 1 to 2 mm/sec (peak particle velocities) at the rail tunnel and station (i.e. at distances of more than 24 m from the rock hammers).

In summary, we expect that the proposed development will have no adverse impact on the adjacent rail infrastructure which is set back 24 m or more from the site boundary.

9. Limitations

Douglas Partners (DP) has prepared this report for the proposed commercial development at Site 8C, Murray Rose Avenue, Sydney Olympic Park, in accordance with DP's proposal dated 22 July 2013. The work was carried out under DP Conditions of Engagement. This report is provided for the exclusive use of FDC Construction and Fitout Pty Ltd and their agents for the specific project and purpose 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.

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

DP's advice is based upon the conditions encountered during this investigation. The accuracy of the advice provided by DP in this report may be limited by undetected variations in ground conditions between sampling 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 given in this report.

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

About this Report

Douglas Partners



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

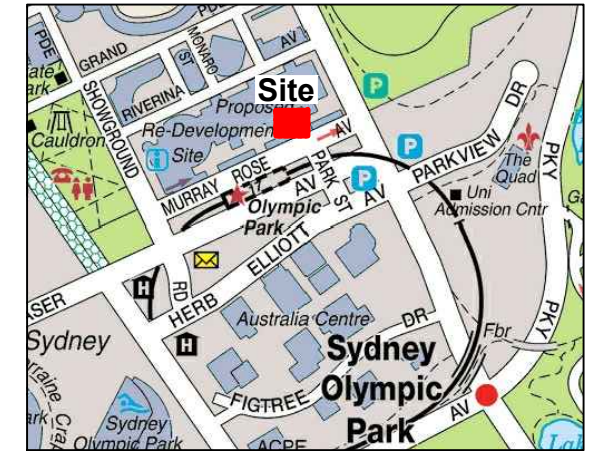
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate 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

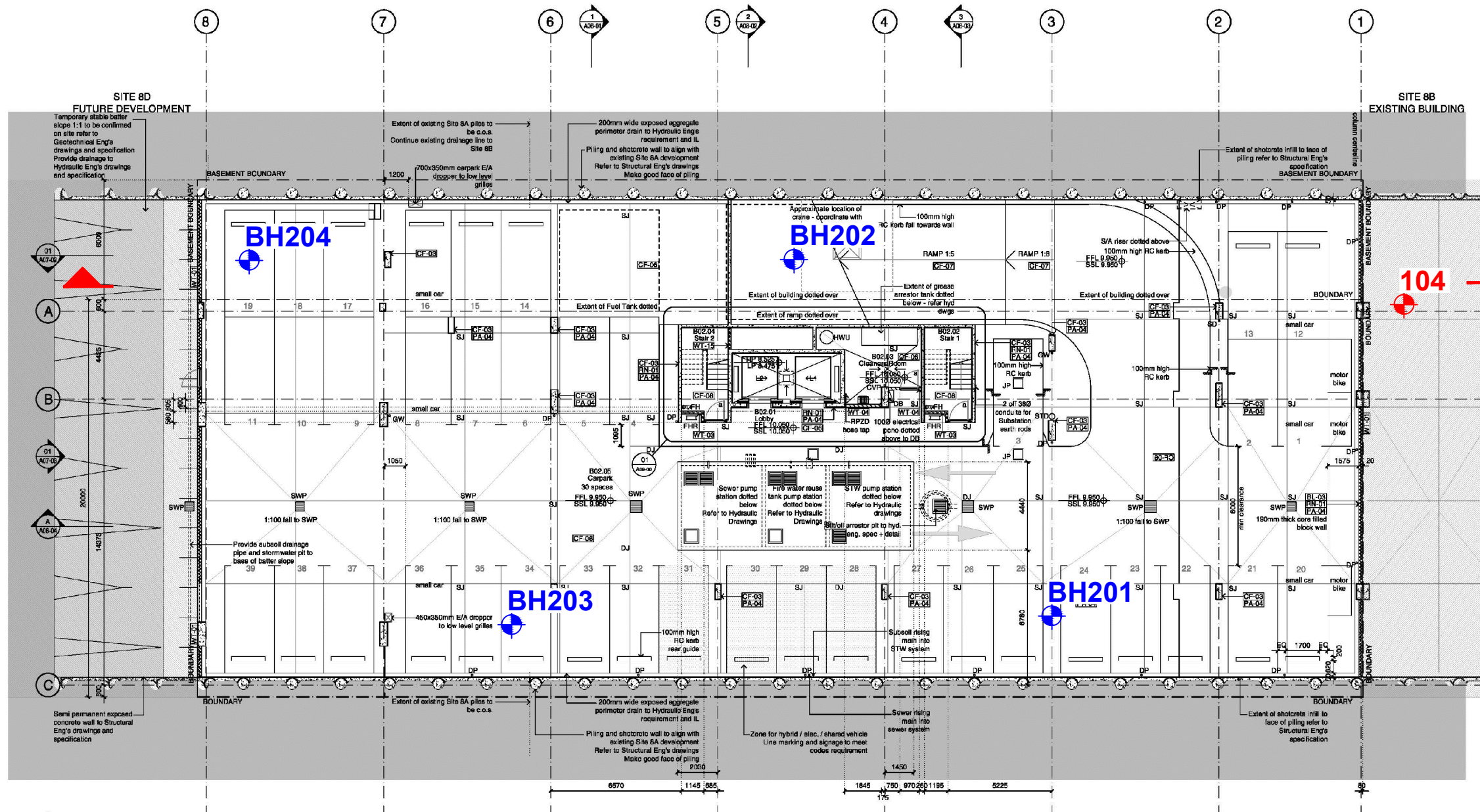
The company will always be pleased to provide engineering inspection services for geotechnical and environmental aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

Appendix B

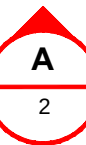
Drawings



Locality Plan



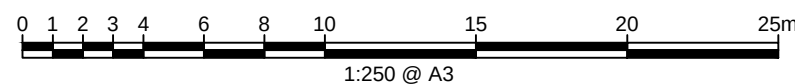
104



NOTE:
Base drawing from Bates Smart Pty Ltd
(Drawing A03.002[A], dated 12/08/2013)

LEGEND

- Previous DP Borehole (Site 8B, Project 72233)
- Current DP Borehole (Site 8C)



CLIENT: FDC Construction & Fitout Pty Ltd

OFFICE: Sydney

DRAWN BY: JCP

SCALE: As shown

DATE: 5.9.2013

TITLE: Location of Boreholes

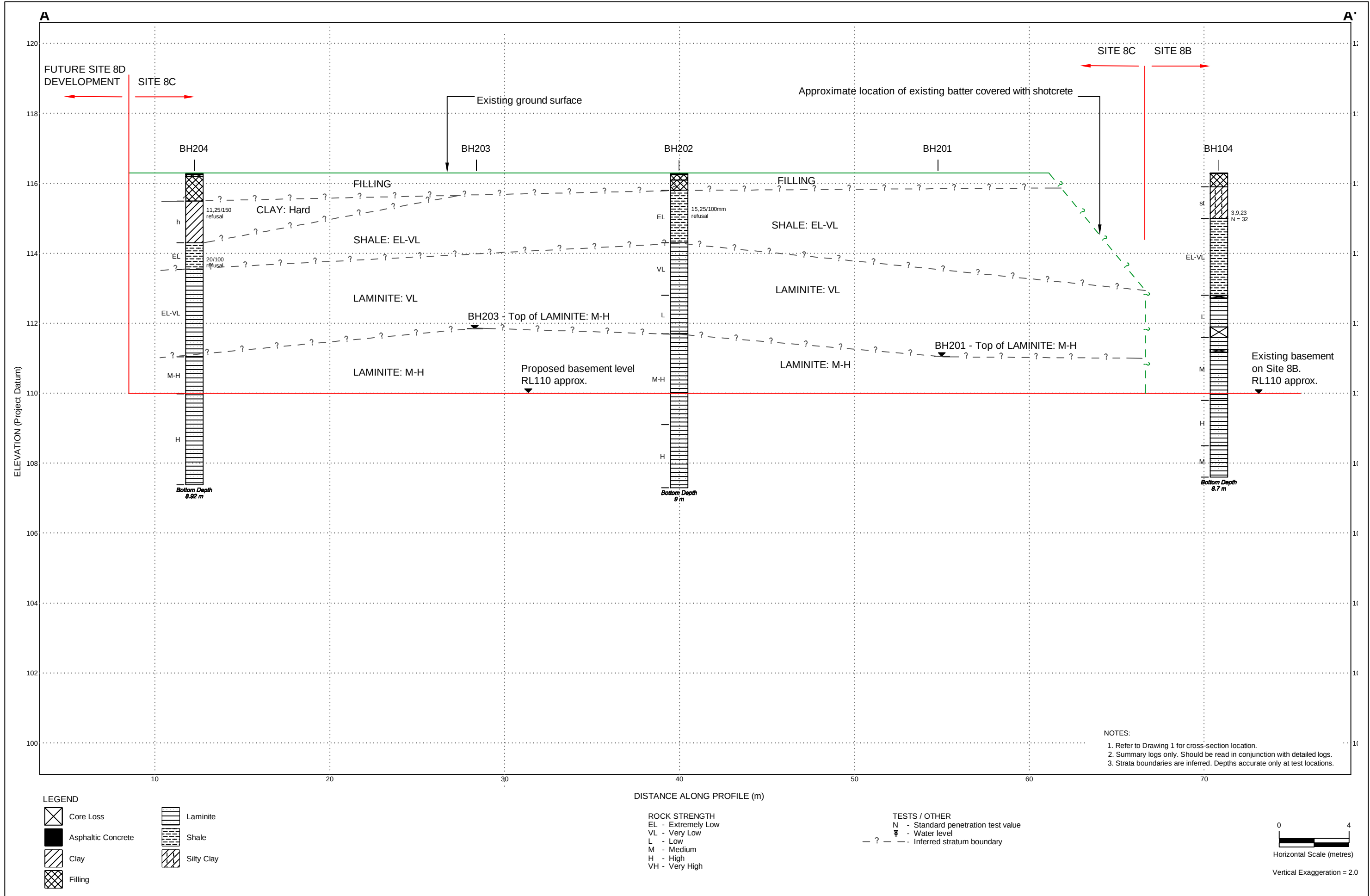
Proposed Commercial Development

Site 8C, Murray Rose Avenue, Sydney Olympic Park

PROJECT No: 73595

DRAWING No: 1

REVISION: 0



Appendix C

Results of Field Work



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 thin-walled 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 in-situ 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.



Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard AS 1726, Geotechnical Site Investigations Code. 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:

Type	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:

Type	Particle size (mm)
Coarse gravel	20 - 63
Medium gravel	6 - 20
Fine gravel	2.36 - 6
Coarse sand	0.6 - 2.36
Medium sand	0.2 - 0.6
Fine sand	0.075 - 0.2

The proportions of secondary constituents of soils are described as:

Term	Proportion	Example
And	Specify	Clay (60%) and Sand (40%)
Adjective	20 - 35%	Sandy Clay
Slightly	12 - 20%	Slightly Sandy Clay
With some	5 - 12%	Clay with some sand
With a trace of	0 - 5%	Clay with a trace of sand

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

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	h	>200

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	SPT N value	CPT qc value (MPa)
Very loose	vl	<4	<2
Loose	l	4 - 10	2 - 5
Medium dense	md	10 - 30	5 - 15
Dense	d	30 - 50	15 - 25
Very dense	vd	>50	>25

Soil Descriptions

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;
- Transported soils - formed somewhere else and transported by nature to the site; or
- Filling - moved by man.

Transported soils may be further subdivided into:

- Alluvium - river deposits
- Lacustrine - lake deposits
- Aeolian - wind deposits
- Littoral - beach deposits
- Estuarine - tidal river deposits
- Talus - scree or coarse colluvium
- Slopewash or Colluvium - transported downslope by gravity assisted by water. Often includes angular rock fragments and boulders.



Rock Strength

Rock strength is defined by the Point Load Strength Index ($IS_{(50)}$) and refers to the strength of the rock substance and not the strength of the overall rock mass, which may be considerably weaker due to defects. The test procedure is described by Australian Standard 4133.4.1 - 1993. The terms used to describe rock strength are as follows:

Term	Abbreviation	Point Load Index $IS_{(50)}$ MPa	Approx Unconfined Compressive Strength MPa*
Extremely low	EL	<0.03	<0.6
Very low	VL	0.03 - 0.1	0.6 - 2
Low	L	0.1 - 0.3	2 - 6
Medium	M	0.3 - 1.0	6 - 20
High	H	1 - 3	20 - 60
Very high	VH	3 - 10	60 - 200
Extremely high	EH	>10	>200

* Assumes a ratio of 20:1 for UCS to $IS_{(50)}$

Degree of Weathering

The degree of weathering of rock is classified as follows:

Term	Abbreviation	Description
Extremely weathered	EW	Rock substance has soil properties, i.e. it can be remoulded and classified as a soil but the texture of the original rock is still evident.
Highly weathered	HW	Limonite staining or bleaching affects whole of rock substance and other signs of decomposition are evident. Porosity and strength may be altered as a result of iron leaching or deposition. Colour and strength of original fresh rock is not recognisable
Moderately weathered	MW	Staining and discolouration of rock substance has taken place
Slightly weathered	SW	Rock substance is slightly discoloured but shows little or no change of strength from fresh rock
Fresh stained	Fs	Rock substance unaffected by weathering but staining visible along defects
Fresh	Fr	No signs of decomposition or staining

Degree of Fracturing

The following classification applies to the spacing of natural fractures in diamond drill cores. It includes bedding plane partings, joints and other defects, but excludes drilling breaks.

Term	Description
Fragmented	Fragments of <20 mm
Highly Fractured	Core lengths of 20-40 mm with some fragments
Fractured	Core lengths of 40-200 mm with some shorter and longer sections
Slightly Fractured	Core lengths of 200-1000 mm with some shorter and longer sections
Unbroken	Core lengths mostly > 1000 mm

Rock Descriptions

Rock Quality Designation

The quality of the cored rock can be measured using the Rock Quality Designation (RQD) index, defined as:

$$\text{RQD \%} = \frac{\text{cumulative length of 'sound' core sections} \geq 100 \text{ mm long}}{\text{total drilled length of section being assessed}}$$

where 'sound' rock is assessed to be rock of low strength or better. The RQD applies only to natural fractures. If the core is broken by drilling or handling (i.e. drilling breaks) then the broken pieces are fitted back together and are not included in the calculation of RQD.

Stratification Spacing

For sedimentary rocks the following terms may be used to describe the spacing of bedding partings:

Term	Separation of Stratification Planes
Thinly laminated	< 6 mm
Laminated	6 mm to 20 mm
Very thinly bedded	20 mm to 60 mm
Thinly bedded	60 mm to 0.2 m
Medium bedded	0.2 m to 0.6 m
Thickly bedded	0.6 m to 2 m
Very thickly bedded	> 2 m

Symbols & Abbreviations

Douglas Partners



Introduction

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

Drilling or Excavation Methods

C	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

▷	Water seep
▽	Water level

Sampling and Testing

A	Auger sample
B	Bulk sample
D	Disturbed sample
E	Environmental sample
U ₅₀	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

B	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
sv	sub-vertical

Coating or Infilling Term

cln	clean
co	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

po	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



Asphalt



Road base



Concrete



Filling

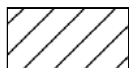
Soils



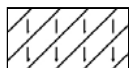
Topsoil



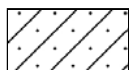
Peat



Clay



Silty clay



Sandy clay



Gravelly clay



Shaly clay



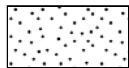
Silt



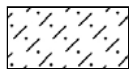
Clayey silt



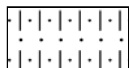
Sandy silt



Sand



Clayey sand



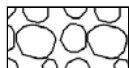
Silty sand



Gravel



Sandy gravel

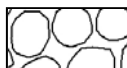


Cobbles, boulders



Talus

Sedimentary Rocks



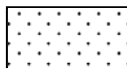
Boulder conglomerate



Conglomerate



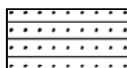
Conglomeratic sandstone



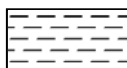
Sandstone



Siltstone



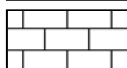
Laminite



Mudstone, claystone, shale

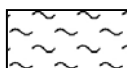


Coal

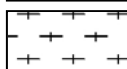


Limestone

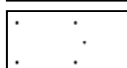
Metamorphic Rocks



Slate, phyllite, schist

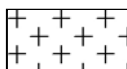


Gneiss

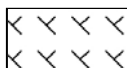


Quartzite

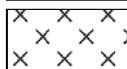
Igneous Rocks



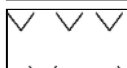
Granite



Dolerite, basalt, andesite



Dacite, epidote



Tuff, breccia



Porphyry

DOUGLAS PARTNERS PTY LTD

SITE 8C, MURRAY ROSE AVENUE, SYDNEY OLYMPIC PARK

BORE 201

PROJECT 73595

AUGUST 2013



2.80m – 8.97m

BOREHOLE LOG

CLIENT: FDC Construction and Fitout Pty Ltd
PROJECT: Sydney Olympic Park Site 8C
LOCATION: Murrumbidgee Avenue, Sydney Olympic Park

SURFACE LEVEL: 16.3 AHD
EASTING: 321425
NORTHING: 6253252
DIP/AZIMUTH: 90°/-

BORE No: 201
PROJECT No: 73595
DATE: 27/8/2013
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Degree of Weathering					Graphic Log	Rock Strength					Water	Fracture Spacing (m)				Discontinuities		Sampling & In Situ Testing																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
			EW	HW	MW	SW	FS		FR	Ex Low	Very Low	Low	Medium		High	Very High	Ex High	0.01	0.05	0.10	0.50	1.00	B - Bedding S - Shear	J - Joint F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
16 15	0.05	FILLING - Bitumen and blue metal gravel																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
	0.1																										FILLING - Road base	E																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
	0.5																																FILLING - Brown-grey clay filling	S																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
	1	CLAY - Hard, red-brown and grey clay with extremely low strength laminite bands																																																			25/150 refusal																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
	2	2.0																									LAMINITE - Extremely low strength, extremely weathered, red-brown and grey, laminite with ironstone bands																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									

RIG: Scout 4

DRILLER: G. Cooper

LOGGED: L. James-Hall

CASING: HQ to 2.80m

TYPE OF BORING: SFA to 2.80m; NMLC to 9.00m

WATER OBSERVATIONS: No groundwater observed whilst augering

REMARKS: 30mm core left down hole on completion of drilling.

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	W	Water seep	S	Standard penetration test
E	Environmental sample	W	Water level	V	Shear vane (kPa)



Douglas Partners
 Geotechnics | Environment | Groundwater

DOUGLAS PARTNERS PTY LTD

SITE 8C, MURRAY ROSE AVENUE, SYDNEY OLYMPIC PARK

BORE 202

PROJECT 73595

AUGUST 2013



2.60m – 9.00m

BOREHOLE LOG

CLIENT: FDC Construction and Fitout Pty Ltd
PROJECT: Sydney Olympic Park Site 8C
LOCATION: Murrumbidgee Avenue, Sydney Olympic Park

SURFACE LEVEL: 16.3 AHD
EASTING: 321409
NORTHING: 6253261
DIP/AZIMUTH: 90°/--

BORE No: 202
PROJECT No: 73595
DATE: 26/8/2013
SHEET 1 OF 1

[illegible]

RIG: Scout 4

DRILLER: G. Cooper

LOGGED: L. James-Hall

CASING: HQ to 2.60m

TYPE OF BORING: SFA to 2.60m; NMLC to 9.00m

WATER OBSERVATIONS: No groundwater observed whilst augering

REMARKS:

SAMPLING & IN SITU TESTING LEGEND			
A	Auger sample	G	Gas sample
B	Bulk sample	P	Piston sample
BLK	Block sample	U _t	Tube sample (x mm dia.)
C	Core drilling	W	Water sample
D	Disturbed sample	W _{seep}	Water seep
E	Environmental sample	W _{level}	Water level
		PID	Photo ionisation detector (ppm)
		PL(A)	Point load axial test (s(50) (MPa)
		PL(D)	Point load diametral test (s(50) (MPa)
		pp	Pocket penetrometer (kPa)
		S	Standard penetration test
		V	Shear vane (kPa)



Douglas Partners
Geotechnics | Environment | Groundwater

DOUGLAS PARTNERS PTY LTD

SITE 8C, MURRAY ROSE AVENUE, SYDNEY OLYMPIC PARK

BORE 203

PROJECT 73595

AUGUST 2013



2.65m – 9.08m

BOREHOLE LOG

CLIENT: FDC Construction and Fitout Pty Ltd
PROJECT: Sydney Olympic Park Site 8C
LOCATION: Murrury Rose Avenue, Sydney Olympic Park

SURFACE LEVEL: 16.3 AHD
EASTING: 321402
NORTHING: 6253242
DIP/AZIMUTH: 90°/-

BORE No: 203
PROJECT No: 73595
DATE: 27/8/2013
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Degree of Weathering					Graphic Log	Rock Strength					Water	Fracture Spacing (m)				Discontinuities		Sampling & In Situ Testing					
			EW	HW	MW	SW	FS		FR	Ex Low	Very Low	Low	Medium		High	Very High	Ex High	0.01	0.05	0.10	0.50	1.00	B - Bedding S - Shear	J - Joint F - Fault	Type	Core Rec. %
16	0.05	FILLING - Bitumen and blue metal gravel																								10,25/100mm refusal
	0.1	FILLING - Road base																				E				
15	0.5	FILLING - Brown-grey clay filling with some ironstone gravel																								
	1	LAMINITE - Extremely low strength, extremely weathered, brown-grey, laminite with ironstone bands																				S				
14	2.4	LAMINITE - Very low strength, moderately weathered, slightly fractured, brown and grey, laminite (20% fine grained sandstone, 80% shale)																				S				25/150 refusal
13	3																					C	100			PL(A) = 0.06
12	4.3	Low strength below 4.00m																								
11	4.34	LAMINITE - As above																								PL(A) = 1
	4.47	LAMINITE - Medium to high strength, slightly weathered and fresh, grey, laminite (20% fine grained sandstone, 80% shale)																								PL(A) = 0.8
10	5																					C	98	94		PL(A) = 0.6
9	6																									
8	7																									PL(A) = 1.3
8	8																					C	100	96		PL(A) = 1
7	9																									PL(A) = 1
7	9.08	Bore discontinued at 9.08m - target depth reached																								

RIG: Scout 4 **DRILLER:** G. Cooper **LOGGED:** L. James-Hall **CASING:** HQ to 2.65m
TYPE OF BORING: SFA to 2.65m; NMLC to 9.08m
WATER OBSERVATIONS: No groundwater observed whilst augering
REMARKS:

SAMPLING & IN SITU TESTING LEGEND			
A	Auger sample	G	Gas sample
B	Bulk sample	P	Piston sample
BLK	Block sample	U	Tube sample (x mm dia.)
C	Core drilling	W	Water sample
D	Disturbed sample	W	Water seep
E	Environmental sample	W	Water level
		PID	Photo ionisation detector (ppm)
		PL(A)	Point load axial test Is(50) (MPa)
		PL(D)	Point load diametral test Is(50) (MPa)
		pp	Pocket penetrometer (kPa)
		S	Standard penetration test
		V	Shear vane (kPa)

DOUGLAS PARTNERS PTY LTD

SITE 8C, MURRAY ROSE AVENUE, SYDNEY OLYMPIC PARK

BORE 204

PROJECT 73595

AUGUST 2013



2.60m – 8.92m

BOREHOLE LOG

CLIENT: FDC Construction and Fitout Pty Ltd
PROJECT: Sydney Olympic Park Site 8C
LOCATION: Murrury Rose Avenue, Sydney Olympic Park

SURFACE LEVEL: 16.3 AHD
EASTING: 321384
NORTHING: 6253249
DIP/AZIMUTH: 90°/-

BORE No: 204
PROJECT No: 73595
DATE: 26/8/2013
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Degree of Weathering					Graphic Log	Rock Strength					Water	Fracture Spacing (m)				Discontinuities		Sampling & In Situ Testing						
			EW	HW	MW	SW	FS		FR	Ex Low	Very Low	Low	Medium		High	Very High	Ex High	0.01	0.05	0.10	0.50	1.00	B - Bedding S - Shear	J - Joint F - Fault	Type	Core Rec. %	RQD %
16	0.05	FILLING - Bitumen and blue metal gravel FILLING - Road base FILLING - Brown-grey clay filling																								11,25/150 refusal	
16	0.1																										
15	0.8																										
15	1	CLAY - Hard, red-brown and grey, clay with some shaley clay bands																									
14	2.0	SHALE - Extremely low strength, extremely weathered, brown-grey, shale																								20/100 refusal	
13	2.75	LAMINITE - Extremely low to low strength, moderately weathered, highly fractured, grey and brown, laminite (30% fine grained sandstone, 70% shale)																								PL(A) = 0.7 PL(A) = 1 PL(A) = 1.1 PL(A) = 1.2 PL(A) = 1 PL(A) = 0.9	
3																											
13																											
4																											
12																											
5																											
11	5.26																										
6																											
10																											
7																											
9																											
8																											
8																											
8																											
9	8.92	Bore discontinued at 8.92m - target depth reached																									

RIG: Scout 4

DRILLER: G. Cooper

LOGGED: L. James-Hall

CASING: HQ to 2.60m

TYPE OF BORING: SFA to 2.60m; NMLC to 9.00m

WATER OBSERVATIONS: No groundwater observed whilst augering

REMARKS: 80mm core left down hole on completion of drilling.

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	Δ	Water seep	S	Standard penetration test
E	Environmental sample	≡	Water level	V	Shear vane (kPa)



Douglas Partners
 Geotechnics | Environment | Groundwater

DOUGLAS PARTNERS PTY
SITE 8B, SYDNEY OLYMPIC PARK - HOMEBUSH
BORE 104 PROJECT 72233 JAN 2011



3.5 – 8.0m

DOUGLAS PARTNERS PTY LTD
SITE 8B, SYDNEY OLYMPIC PARK - HOMEBUSH
BORE 104 PROJECT 72233 JAN 2011



8.0 – 8.7m

BOREHOLE LOG

CLIENT: Fitzpatrick Investments Pty Ltd
PROJECT: Proposed Commercial Development
LOCATION: Site 8B, Sydney Olympic Park

SURFACE LEVEL: 116.3 ^
EASTING:
NORTHING:
DIP/AZIMUTH: 90°/-

BORE No: 104
PROJECT No: 72233
DATE: 20/1/2011
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Degree of Weathering					Graphic Log	Rock Strength					Water	Fracture Spacing (m)			Discontinuities		Sampling & In Situ Testing			Test Results & Comments		
			EW	HW	MW	SW	FS		FR	Ex Low	Very Low	Low	Medium		High	Very High	Ex High	0.01	0.05	0.10	0.50	1.00		B - Bedding S - Shear	J - Joint F - Fault
116	0.02	ASPHALTIC CONCRETE																				A			3,9,23 N = 32
	0.4	FILLING - brown sand filling with some igneous and sandstone gravel, humid																				A/E			
1		SILTY CLAY - stiff, light grey and orange brown, silty clay with a trace of ironstone gravel, Mc<PL																				A/E			
115	1.3	SHALE - extremely low to very low strength, grey shale with some medium strength, red brown ironstone bands																				S			
2																						A			
114																									
3		- becoming very low strength from approximately 3.0m																							
113																									
3.5	3.59	LAMINITE - low strength, highly to moderately weathered, fragmented to fractured, grey brown laminite with approximately 20% fine grained sandstone laminations and some very low strength bands																							
4																						C	75	0	PL(A) = 0.2
112	4.7	LAMINITE - medium strength, moderately weathered then fresh stained, fragmented to fractured then slightly fractured, light grey to grey brown laminite with approximately 20% fine grained sandstone laminations																							PL(A) = 0.4
5	5.13																								PL(A) = 0.5
6																									
110	6.5	LAMINITE - high strength, fresh, unbroken, grey laminite with approximately 30% fine grained sandstone laminations																							PL(A) = 0.9
7																									PL(A) = 1.2
109																									
8	7.8	LAMINITE - medium strength, fresh, slightly fractured, grey laminite with approximately 20% fine grained sandstone laminations																							PL(A) = 0.6
108																									
8.7		Bore discontinued at 8.7m																							
9																									
107																									

RIG: DT 100

DRILLER: Steve Salib

LOGGED: SI

CASING: HW to 1.6m

TYPE OF BORING: Solid flight auger (TC-bit) to 1.6m; Rotary to 3.5m; NMLC-Coring to 8.7m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Soft bone 4.4m to 4.7m. Mc = Moisture content; PL = Plastic limit. ^Project Datum

SURVEY DATUM:

SAMPLING & IN SITU TESTING LEGEND			
A Auger sample	G Gas sample	PID Photo ionisation detector (ppm)	
BB Bulk sample	P Piston sample	PL(A) Point load axial test Is(50) (MPa)	
BLK Block sample	U Tube sample (x mm dia.)	PL(D) Point load diametral test Is(50) (MPa)	
CLC Core drilling	W Water sample	pp Pocket penetrometer (kPa)	
D Disturbed sample	W Water seep	S Standard penetration test	
E Environmental sample	W Water level	V Shear vane (kPa)	