Douglas Partners
Geotechnics | Environment | Groundwater

Report on
Preliminary Geotechnical Investigation

Proposed Mixed Use Development
86 – 94 Castlereagh Street & 77 – 79 Bathurst Street
Liverpool, NSW

Prepared for
Il Capitano Investments Pty Ltd

Project 92327.01
December 2018

Integrated Practical Solutions

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Document History

Document details

Project No.	92327.01	Document No.	R.001.Rev0
Document title	Report on Preliminary Geotechnical Investigation Proposed Mixed Use Development		
Site address	86 – 94 Castlereagh Street & 77 – 79 Bathurst Street Liverpool, NSW		
Report prepared for	Il Capitano Investments Pty Ltd		
File name	92327.01.R.001.Rev0		

Document status and review

Status	Prepared by	Reviewed by	Date issued
Revision 0	Rasoul Machiani	Michael J Thom	18 December 2018

Distribution of copies

Status	Electronic	Paper	Issued to
Revision 0	1	0	Il Capitano Investments Pty Ltd – Mr Matthew Sofi

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.

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Report on Preliminary Geotechnical Investigation
Proposed Mixed Use Development
86 – 94 Castlereagh Street & 77 – 79 Bathurst Street Liverpool, NSW

1. Introduction

This report presents the results of a preliminary geotechnical investigation undertaken for a proposed mixed use development at 86 – 94 Castlereagh Street & 77 – 79 Bathurst Street, Liverpool. The investigation was commissioned in an email dated 16 October 2018 by Mr Brian Mariotti of Allen Jack + Cottier (project's Architects) on behalf of Il Capitano Investments Pty Ltd and was undertaken in accordance with Douglas Partners Pty Ltd's (DP) proposal MAC180343 dated 17 October 2018.

DP understands that the proposed development comprises two high rise towers including basement levels with cut depths of up to 10 m. The architectural concept drawings were provided for the geotechnical investigation that shows the location and design levels of the proposed development.

The investigation included the drilling of cored boreholes and laboratory testing of selected samples. Details of the work undertaken and the results obtained are given within this report, together with comments relating to foundation design and earthworks.

2. Site Description

The site is located at the corner of Memorial Avenue and Castlereagh Street, Liverpool and covers an area of some 0.4 ha with maximum north-south and east-west dimensions of approximately 60 m and 100 m respectively. The site has an approximate 30 m frontage to Bathurst Street to the east.

At the time of the investigation the site was relatively level. The property at 86 Castlereagh Street is located at the north-western part of the site, was at the time of the investigation used as a service station and for residential purposes. A two-storey brick building was noted and the entire external are was covered by a concrete slab. A two-storey commercial brick building was located at the southern part of the site and was used for a restaurant and offices and on grade carparks.

3. Regional Geology

Reference to the 1:100 000 Penrith Geological Series Sheet (Ref 1) indicates that the site is underlain by Bringelly Shale of the Wianamatta Group of Triassic age, which in the vicinity of the site includes an unnamed, fine to medium grained quartz-lithic sandstone member. The Bringelly Shale typically comprises shale, siltstone, claystone and laminite with coal bands, all of which weather to form clays of high plasticity.

4. Field Work Methods

The field work comprised the drilling of four cored boreholes (Bores 1 – 4) to a maximum depth of 10.2 m. The boreholes were drilled using a Hydrapower Scout drilling rig using 125 mm diameter solid flight augers to bedrock and 'NMLC' rotary coring technique and water flush with steel casing thereafter to obtain continuous rock core samples. Standard penetration tests (AS 1289.6.3.1) were also carried out at a depth of 1.0 m and then at 1.5 m depth intervals within all boreholes whilst augering. The standard penetration test procedure is given in the attached notes and the penetration 'N' value obtained during testing is shown on the borehole logs.

The field work was supervised by a geotechnical engineer who logged the boreholes and collected disturbed soil samples to assist in strata identification and for laboratory testing. Following logging, testing and sampling, all test locations were backfilled and the ground surface reinstated to its previous level.

The test locations were nominated by DP and located on site prior to the investigation using a differential GPS unit for which an accuracy of ± 20 mm is typical. The locations of boreholes are shown on Drawing 1 (Appendix A). The surface levels were obtained using the differential GPS unit.

All field measurements and mapping for this project have been carried out using the Geodetic Datum of Australia 1994 (GDA94) and the Map Grid of Australia 1994 (MGA94 Zone 56). All reduced levels are given in relation to Australian Height Datum (AHD).

5. Field Work Results

- CONCRETE – 150 – 200 mm thick concrete slab in Bores 1, 3 & 4. The ground surface at the location of Bore 2 was covered with 50 mm asphalt;
- FILLING – sandy silty clay filling with some gravel to depths of 0.7 – 0.8 m in all boreholes;
- SILTY CLAY – stiff to very stiff silty clay to depths of 3.0 – 3.1 m in all boreholes; and
- BEDROCK – variable in strength comprising extremely low to very low strength shale and siltstone becoming low to medium strength with high strength bands of sandstone to the termination depth of boreholes.

No groundwater seepage was observed in any of boreholes whilst auguring. The introduction of water into the boreholes during the rotary coring precluded any observations of groundwater that might have been present. Thus, standpipe piezometers were installed at the location of Bores 2 and 4 which allow longer term monitoring of groundwater and were inspected on 3 December 2018. The groundwater level was recorded at RL's 13.9 m AHD and 12.4 m AHD within Bores 2 and 4 respectively. It is noted that groundwater levels are affected by factors such as soil permeability and weather conditions and can fluctuate with time.

6. Laboratory Testing

6.1 Point Test Testing

Selected rock core samples were tested in the laboratory for measurement of point load strength index ($Is_{(50)}$) to estimate rock strength at variable depths. The detailed laboratory test report sheets are given in Appendix C and the values of $Is_{(50)}$ are shown on the borehole logs.

6.2 Soil Aggressivity

Selected samples from the boreholes were tested in the laboratory for aggressivity assessment by measuring pH, sulphates, chlorides and electrical conductivity. The detailed test report sheets are given in Appendix D, with the results summarised in Table 1.

Table 1: Results of Laboratory Testing – Aggressivity

Bore	Depth (m)	pH	Chloride (mg/kg)	Sulphate (mg/kg)	EC (µS/cm)	Material
1	0.5	8.5	<10	130	160	Filling
1	1.0	5.6	–	–	64	Silty clay
2	1.0	6.3	–	–	72	Filling
3	0.5	7.4	–	–	200	Silty Clay
3	1.0	5.2	160	190	200	Filling
4	0.5	8.8	–	–	170	Silty clay
4	1.0	5.4	–	–	160	Silty clay
4	3.9	6.8	–	–	64	XW Shale

The exposure classification of the surface of concrete and steel piles was determined in accordance with AS 2159 – 1996 (Ref 2) as detailed in Table 6.4.2 (c) and Table 6.5.2 (c) which indicates the soils tested would be classified as *“non aggressive”* to concrete and steel.

7. Proposed Development

It is understood that the proposed development comprises the construction of two high rise towers, including three basement levels for commercial and residential purposes.

Concept plans of the development have been provided for the investigation, which indicate the finished design levels and excavation depths. Although, the column design working loads are yet to be determined, the preliminary architectural design drawings show the finished excavation level will be at RL 11.0 m AHD. Bulk excavation plans are yet to be completed, however, in accordance with the brief and the concept plans, bulk earthworks on this site may result in cutting of up to approximately 10 m deep as the current ground level of the site is at approximate RL 21 m AHD.

8. Comments

8.1 General

Comments are provided in the following sections on development constraints related to geotechnical and geological factors to assist in the foundation design of the proposed mix-used towers. As detailed design of the proposed redevelopment works has not been undertaken, the comments given must also be considered as being preliminary in nature. Once details are available, they should be forwarded to DP for review to determine if comments given within this report require revision.

8.2 Subsurface Conditions and Rock Strength

The following comments are based on the surface and subsurface profiles encountered during the investigation and the results of laboratory testing of selected samples collected at the borehole locations. The boreholes have indicated that subsurface conditions underlying the site typically comprise filling to depths of 1.7 – 3.7 m. Filling was underlain by silty clays with traces of extremely weathered rock to depths of 4.7 – 5.5 m, where bedrock of variable strength and weathering condition were encountered and continued to the final depth of boreholes.

The bedrock from the cored boreholes has been classified in accordance with Reference 3 and depths/RLs of each rock class are summarised in Table 2.

Table 2: Depth/Level of Rock Classes

Bore	RL Depth (m)	Thickness (m)	Rock Class (Shale)
1 Surface Level: 21.0m AHD	17.9 – 17.4	0.5	V
	17.4 – 16.9	0.5	IV
	16.9 – 16.2	0.7	II
	16.2 – 15.2	1.0	II
	15.2 – 13.8	1.4	III
	13.8 – 11.8	2.0	II
2 Surface Level: 21.2m AHD	18.1 – 16.9	1.2	IV
	16.9 – 15.5	1.4	III
	15.5 – 13.5	2.0	II
3 Surface Level: 21.0m AHD	18.0 – 17.4	0.6	V
	17.4 – 16.0	1.4	IV
	16.0 – 13.5	2.5	III
	13.5 – 10.7	2.8	II
4 Surface Level: 20.5m AHD	16.7 – 13.3	3.4	IV
	13.3 – 11.8	1.5	III
	11.8 – 10.5	1.3	II

Note: Bands of higher or lower strength rock are expected within each rock class category.

The cored borehole logs indicate that the rock structure is mainly governed by horizontal to sub-horizontal (0º – 10º) bedding and occasional steeply-inclined (up to 45°) jointing. The fracture spacings on the recovered core samples show generally 'fractured' shale with seams of extremely weathered rock and 'fragmented' zones were encountered to approximate RL's 16.9 – 13.3 m AHD, and 'slightly fractured' shale with 'fractured' seams thereafter to the final termination depths. Occasional bands of high strength sandstone were encountered in Bore 1 at RL's 16 m and 9.3 m AHD

8.3 Foundations

The results of the investigation and point load test results indicate that the depth of medium to high strength rock vary within the boreholes. In general, considering the design excavation level (RL 11.0 m AHD) and anticipated depth of Class III or greater quality rock as encountered in the boreholes, between RL 16.9 m AHD (Bores 1 & 2), RL 16.0 m AHD (Bore 3) and RL 13.3 m AHD (Bore 4), which are well above excavation level, pad footings of suitable size and depths are considered suitable options to support the columns. Depending on the final design loads, consideration may be given to the design of bore piles for the proposed development.

Based on the results of the field investigation and laboratory testing, retaining wall and building footings could be proportioned using the maximum design parameters presented in Table 3. The footing recommendations and design parameters for any given strata will need to be confirmed following the completion of the design stage when the final excavation depth, footing size and design loads are specified.

Table 3: Estimated Design Parameters

Material		Ultimate Base Bearing Pressures (kPa) (1)	Ultimate Shaft Adhesion Pressures (kPa) (2)	Allowable Base Bearing Pressures (kPa) (3)	Allowable Shaft Adhesion Pressures (kPa)	Allowable Lateral Resistance (kPa)	Young Modulus, E' (MPa)
Controlled fill		–	–	100	–	–	4 – 10
Very stiff to hard clay		–	–	200	–	–	20 – 50
Shale	Class V	3000	100	700	70	200	30 – 60
	Class IV	6000	150	1000	100	300	50 – 100
	Class III	20000	750	3500	350	1200	500 – 3000
	Class II	70000	1500	10000	1000	4500	5000 – 7000

Notes (1) The values are in accordance with Pells et al- 1998 (Ref3);

(2) Ultimate values occur at large settlements (generally >5% of the minimum footing width);

(3) Values can only be adopted for clean sockets of roughness category R2 or better. Values may need to be reduced to account for smear;

(4) Value for rock based on settlements of <1% of minimum footing width.

Base bearing and shaft adhesion values have also been provided for Limit State design. The geotechnical strength reduction factor Φg of 0.45 shoul be applied in accordance with AS 2159 – 2009, Table 4.3.2 based on the available information. Pile testing will be required by AS 2159 for piles designed with Φg>0.4.

Reference should be made to the borehole logs (Appendix B) and Table 2 with respect to the depth/levels of the various bearing strata.

8.4 Earthworks

It is considered that significant bulk earthworks including cutting to a depth of 10.0 m for the basement level would be required, however the final earthworks plans have not been finalized at the time of preparing this report.

8.4.1 Site Preparation

To prepare the site for the construction of pavement and ground carparks, the following procedures are suggested:

- Stripping of existing pavement and filling (to a minimum depth of 0.5 m below subgrade level) and inspection of the stripped surface by a geotechnical engineer;
- Compaction of the exposed surface with at least of 8 passes of a 12 tonne (minimum dead weight) roller, followed by test rolling in the presence of a geotechnical engineer. Where soft spots are identified, they should be excavated and then backfilled using a suitable granular material. All filling should be placed in 250 mm (loose thickness) layers and compacted with placement moisture contents within the range of -2% to +2% of OMC in order to limit surface deflection during proof rolling.
- Surface drainage should be maintained at all times by adopting appropriate surface cross-falls. Surface drainage should be installed as soon as is practicable in order to capture and remove surface flows to prevent erosion and softening of the exposed surface.

Site won materials are expected to be used for filling. Any imported filling must be approved by the geotechnical consultant prior to delivery to site.

Conventional sediment and erosion control measures should be implemented during the construction phase, with exposed surfaces to be topsoiled and vegetated as soon as practicable following the completion of earthworks. Alternatively, a layer of compacted high strength gravel could be suitable.

8.4.2 Excavation

Subsurface material to the design excavation level of RL 11.0 m AHD is expected to be comprised of filling, silty clay and bedrock of variable weathering and strength conditions. All topsoil, filling, natural soils and bedrock up to very low to low strength should be readily removed using a conventional medium sized excavator (or equivalent) fitted with a toothed bucket, possibly with some light ripping.

Low to medium strength rock encountered at approximate RL’s 16.9 – 13.3 m AHD is expected and will likely required medium to heavy ripping and/ or rock breaking equipment to assist in bulk earthworks with the potential for very low production rates within high strength bands (e.g. sandstone in Bore 1).

Equipment required for excavations are given as a guide only. Rock strength and quality are expected to vary within the footprint and depth of the proposed excavation. Assessment of excavation difficulties are best determined by intending contractors based on inspection of the core samples, the equipment they have at their disposal and the experience of the operators. For information on soil and rock types and indicative strength, reference must be made to the individual logs which are included in Appendix B.

8.4.3 Batter Slopes

Considering the proximity of adjacent properties and infrastructures (e.g. roads, underground services) and proposed depth of excavations (10 m), it is expected that the design of shoring will be required to support the batters. Where space permits the use of permanent or temporary batters within the bulk excavation area, suggested batter slopes are provided in Table 4.

Table 4: Suggested Safe Batter Slopes

Material	Temporary	Permanent
Stiff to very stiff clay or greater	1H:1V	2H:1V
Extremely low strength rock	1H:1V	2H:1V
Very low to low strength rock	0.75H:1V	1H:1V*

* These batter slope angles are subject to inspection by a qualified geotechnical engineer or engineering geologist.

The values in Table 4 are largely dependent on groundwater level, joint orientation and would be subject to verification after an inspection by a qualified engineering geologist or geotechnical engineer during the excavation process. In order to maintain long term stability of the slopes and reduce the effects of scour and erosion, any batter greater than 4 m in height should include a 3 m wide intermediate bench every 4 m in height.

The maximum batter slope for temporary batters in uncontrolled filling should be reduced to 3(H):1(V).

8.5 Excavation Support

Bulk excavations should be constructed to the suggested safe batters and considering the nominated design level, this may be not be achievable around the perimeter of the building. Where temporary or permanent batters at recommended batter angles are not feasible due to insufficient space for batters adjacent of the excavation, the design of shoring will be required as batters steeper that those suggested in Section 8.4 are not expected to remain stable for a long period of time.

Based on the investigation findings, the feasible options would include either anchored soldier piles (drilled at maximum 2.4 m spacings) with close shuttering/shotcrete infill panels or contiguous piling. Contiguous piling is a cost effective form of concrete pile wall, however, is not a water retaining structure and may not be suitable for all material due to gaps between piles.

Excavation of panels for shotcreting at anchored soldier piles option should be staged to allow a hit and miss approach, with the first panel extending no more than 1.5 m below the ground, and subsequent panels not exceeding 1.5 – 2.0 m in height.

Drainage is normally provided behind shotcrete walls. The sprayed concrete wall should provide adequate structural support, however it may be appropriate to install a false wall (single brickwork or block work) for aesthetic purposes and to manage dampness. Care should be exercised in construction to ensure that anchors are installed progressively with excavation (and stressed up) and that the shotcreting is carried out at regular intervals to limit the exposed sections. The first row of anchors should be installed as high as possible and stressed up to 80% of its working load prior to excavation of the next row of panels.

Any groundwater inflows during the excavation should be removed by pumping from sumps within the excavations.

As a guide, in addition to the soldier piles, preliminary design of infilled panel sections should allow for the application of a steel mesh-reinforced shotcrete layer with a minimum nominal thickness of 150 mm where permanent support is required or 75 mm for temporary support.

Earth pressures acting on multi-anchored shoring structures and retaining walls can be estimated on the basis of a trapezoidal pressure distribution (ie: triangular to 0.25 H, uniform from 0.25 H to 0.75 H and triangular decreasing to zero from 0.75 H to H) with depth using appropriate values of bulk density and active (Ka) or 'at rest' (Ko) lateral earth pressure coefficients as set out in Table 5.

Table 5: Suggested Lateral Earth Pressure Design Parameters – Retaining Structures

Retained Material	Bulk Density (kN/m^3)	K_0	Ka		Drained Friction Angle, Φ' (degrees)	Drained Cohesion, c' (kPa)
			Short Term	Long Term		
Controlled filling/ stiff clay	18	0.6	0.25	0.3	25	2 – 5
Stiff to hard clay and extremely weathered rock	20	0.6	0.25	0.3	25	5 – 10
Very low strength shale/siltstone	22	0.45	0.3	0.35	28	50 – 100
Medium strength or greater rock	22	-	10 kPa*	10 kPa*	32	150 – 200

* A uniform pressure of 10 kPa should be adopted for the support of the medium strength sandstone to account for possible defects, but subject to inspection during the early stages of excavation to confirm bedding/jointing and revision of lateral restraint, if appropriate.

'At rest' pressure coefficients are appropriate where support must be provided to boundaries and where movement intolerant services or adjacent structures are present. Surcharge lateral pressure due to any adjacent structure will also need to be taken into account where the footings found on low strength or weaker rock or unfavourably orientated jointing is encountered.

The groundwater table was recorded at approximately RL's 13.9 m AHD and 12.4 m AHD within Bores 2 and 4 which is above the design excavation level of RL 11.0 m AHD. Consequently, it is anticipated that, as a minimum a drained basement would be required for this project. Approval should be sought from relevant authority (Liverpool Council) regarding the suitability of a drained basement for the subject site and the need for any ongoing licence from DPI Water. A second option for groundwater management would be the construction of a tanked basement which is a more expensive option. Full hydrostatic pressure should be allowed for in design of tanked basements and densities of the retained soils will need to be appropriately reduced to the buoyant values. Where applicable, superimposed surcharge loads due to adjacent driveways and future developments should also be accommodated in the design of such structures.

Where appropriate, lateral restraint may also be developed by embedding piles below the base of the excavation and developing passive pressure. Suggested ultimate passive resistance values are given in Table 6 may be adopted below one pile diameter beneath the bulk excavation level and should incorporate a factor of safety to limit wall movement.

Table 6: Suggested Ultimate Passive Pressure Values

Material	Ultimate Passive Pressure (kPa)
Extremely low and very low strength siltstone	300
Low strength siltstone and sandstone	1200
Medium or greater strength siltstone and sandstone	4000

Where engineer-designed retaining walls are proposed, the following measures should be incorporated into the design:

- Backfilling of the void between the wall and the slope using imported, free draining granular material connected into a drainage pipe at the base of the wall;
- Capping of the backfill (where exposed) with compacted clay or concrete to prevent surface runoff entering the backfill;
- Provision of an open drain to collect and divert surface runoff from ponding above the wall;
- For horizontal backfill or retained soils, design based on an average bulk unit weight for retained material of 20 kN/m^3 and on a triangular earth pressure distribution based on an active earth pressure coefficient of (K_a) 0.3 for compacted filling and natural clay where no movement sensitive structures are located within a horizontal distance of 2H (where H is the vertical height of the retained zone) of the rear of the wall;
- Where there are movement sensitive structures located within the abovementioned critical zone, an at rest pressure coefficient (K_0) of 0.6 should be adopted; and
- If an adequate drainage medium is not provided behind the retaining wall, then hydrostatic pressures must be incorporated within the design with soil parameters reduced to their buoyant values.

8.6 Earthquake Actions – Sub-soil Class

The site stratigraphy comprises filling underlain by stiff to hard silty clays, overlying bedrock at depths less than 10 m. The proposed development will be founded on bedrock and therefore, the site's sub-soil class when assessed in accordance with AS 1170.4 – 2007 (Ref 4) is considered a rock site and a classification of Class B_e is suggested.

9. Summary

The investigation included the drilling of four cored boreholes to a maximum depth of 10.2 m across the site. The boreholes have indicated that subsurface conditions underlying the site generally comprise variable depths of filling overlying silty clay and clay of very stiff to hard consistency. Bedrock comprising shale of variable strength and weathering condition were encountered in all boreholes.

Bearing capacity recommendations are provided in Section 8.3. The site preparation, earthworks and excavation support recommendations are to be undertaken in accordance with Sections 8.4 and 8.5.

Consideration must be given to the preliminary nature of the investigation and potential for variability in the subsurface condition across the site. Once design is suitably advanced and design loads, earthworks details and footing locations are known, further investigation will be required to confirm the suitability of the recommendations given in this report.

10. References

1. Geology of 1:100 000 Penrith Geological Series Sheet No 9029 – 9129, Dept of Mines, (1985).
2. Australian Standard AS 2159 – 2009 "*Piling – Design and Installation*".
3. Foundations on Shales and Sandstones in the Sydney Region, Pells *et al*, Australian Geomechanics Journal (1998).
4. AS 1170.4 – 2007, "Structural Design Actions – Part 4: Earthquake Actions in Australia".
5. AS 1170.4 – 1993, "Structural Design Actions – Part 4: Earthquake Actions in Australia".
6. AS 3798 – 2007, "Guidelines on Earthworks for Commercial and Residential Developments".

11. Limitations

Douglas Partners Pty Ltd (DP) has prepared this report (or services) for this project at 86 – 94 Castlereagh Street & 77 – 79 Bathurst Street Liverpool, NSW in accordance with DP's proposal MAC180343 dated 17 October 2018 and acceptance received from Mr Brian Mariotti of Allen Jack + Cottier Pty Ltd on behalf of the client. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of Il Capitano Investments Pty Ltd 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 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 subsurface 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. Subsurface 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 this 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.

This report must be read in conjunction with all of the attached 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 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/groundwater 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
Drawing 1

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.

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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.

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.

Rock Descriptions

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 - 2007. The terms used to describe rock strength are as follows:

Term	Abbreviation	Point Load Index $Is_{(50)}$ MPa	Approximate 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 $Is_{(50)}$. It should be noted that the UCS to $Is_{(50)}$ ratio varies significantly for different rock types and specific ratios should be determined for each site.

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

Sampling Methods

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

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 based on Australian Standard AS 1726-1993, 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.

Symbols & Abbreviations

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

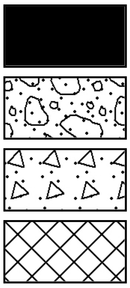
Other

fg	fragmented
bnd	band
qtz	quartz

Symbols & Abbreviations

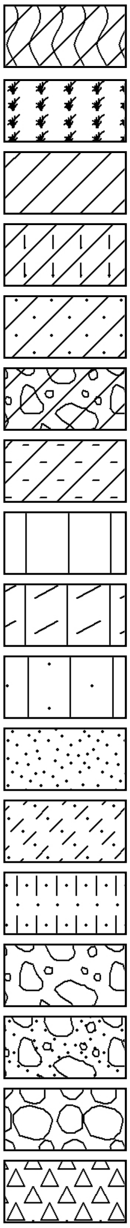
Graphic Symbols for Soil and Rock

General



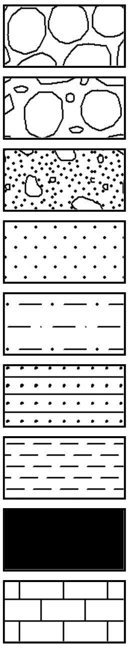
- Asphalt
- Road base
- Concrete
- Filling

Soils



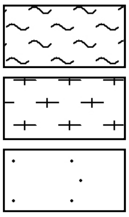
- Topsoil
- Peat
- Clay
- Silty clay
- Sandy clay
- Gravelly clay
- Shaly clay
- Silt
- Clayey silt
- Sandy silt
- Sand
- Clayey sand
- Silty sand
- Gravel
- Sandy gravel
- Cobbles, boulders
- Talus

Sedimentary Rocks



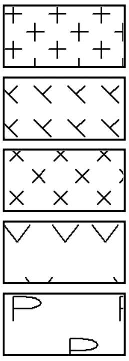
- Boulder conglomerate
- Conglomerate
- Conglomeratic sandstone
- Sandstone
- Siltstone
- Laminite
- Mudstone, claystone, shale
- Coal
- Limestone

Metamorphic Rocks

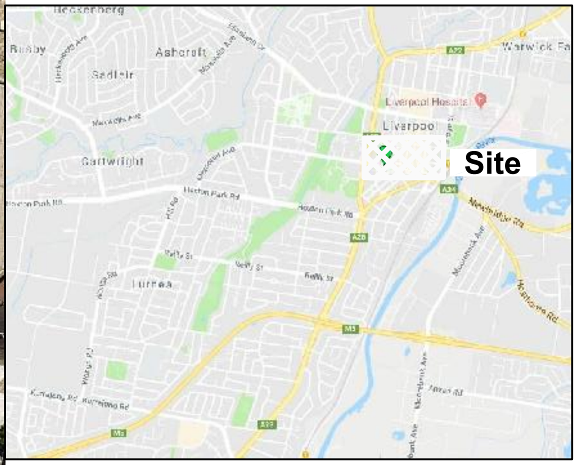


- Slate, phyllite, schist
- Gneiss
- Quartzite

Igneous Rocks



- Granite
- Dolerite, basalt, andesite
- Dacite, epidote
- Tuff, breccia
- Porphyry



Locality Plan

LEGEND

⌖ Borehole Location

Douglas Partners
Geotechnics | Environment | Groundwater

CLIENT: Il Capitano Investment Pty Ltd		TITLE: **Approximate Borehole Locations**
OFFICE: Macarthur	DRAWN BY: RMM	**Proposed Mixed Use Development**
SCALE: As shown	DATE: 17.12.2018	**77 - 79 Bathurst Street ,Liverpool**

N
MAG
ISG
MGA

PROJECT No:	92327.00
DRAWING No:	1
REVISION:	A

Appendix B

Borehole Logs (Bores 1 – 4)
Rock Core Photographs

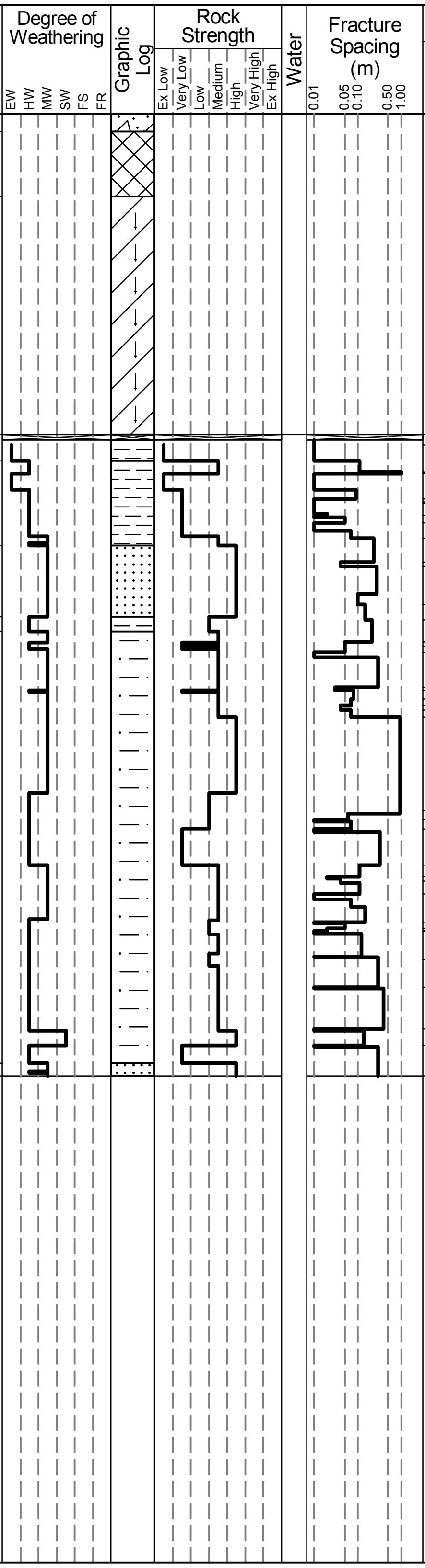
BOREHOLE LOG

CLIENT: Il Capitano Investments Pty Ltd
PROJECT: Proposed Multi-storey Residential Development
LOCATION: 86-94 Castlereagh Street &
77-79 Bathurst Street, Liverpool, NSW

SURFACE LEVEL: 21.0 mAHD
EASTING: 307719
NORTHING: 6244232
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 92327.00
DATE: 26/11/2018
SHEET 1 OF 1

Depth (m)	Description of Strata
0.17	CONCRETE
0.8	FILLING - dark grey sandy silty clay with some gravel, MC>PL
3.1	SILTY CLAY - stiff, light brown mottled red silty clay with some ironstone gravel, MC>PL - becoming grey mottled red below 1.0m - becoming MC<PL below 1.5m - with very low strength, highly weathered shale bands below 2.5m
3.15	CORE LOSS
3.35	SHALE - extremely low strength, extremely weathered, grey shale
4.17	SHALE - very low strength, highly weathered, grey shale - becoming extremely low to very low strength, extremely to highly weathered below 3.48m - becoming medium strength, highly to moderately weathered below 4.08m
4.86	SANDSTONE - high strength, moderately weathered, grey fine grained sandstone
5.0	SHALE - low to medium strength, highly weathered, grey shale
9.17	SILTSTONE - medium strength, moderately weathered, grey siltstone with some very low strength, highly weathered shale bands - becoming low to medium strength, highly weathered between 6.56 - 7.26m - becoming high strength, slightly weathered below 8.86m
9.3	SANDSTONE - high strength, moderately weathered, grey fine grained sandstone
	Bore discontinued at 9.3m - limit of investigation



Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault):

3.1m: CORE LOSS: 50mm
3.2m: Cs 150mm thick
3.46m: fg zone 20mm thick
3.48m: Cs 150mm thick
3.72m: Cs 30mm thick
3.75m: fg zone 10mm thick
3.76m: Cs 100mm thick
3.88m: fg zone 20mm thick
3.95m: Cs 80mm thick
4.1m: B, h, pl, sm
4.33m: B, h, pl, sm
4.37m: B, h, pl, sm
4.74m: B, h, pl, sm
4.89m: B, h, pl, sm
5.1m: B, h, pl, sm
5.15m: B, h, pl, sm
5.2m: fg zone 50mm thick
5.54m: B, pl, h, sm
5.57m: B, pl, h, sm
5.65m: B, pl, h, sm
5.72m: B, pl, h, sm
5.76m: B, pl, h, sm
5.83m: B, pl, h, sm
6.73m: J, 45°, sv, pl, sm 60mm long
6.82m: fg zone, Cs 20mm thick
6.91m: Cs 30mm thick
7.26m: B, h, pl, sm
7.37m: J, 40°, sv, pl, sm 40mm long, B, h, ir, sm
7.41m: J, 40°, sh, pl, sm 30mm long
7.54m: fg zone 50mm thick
7.81m: fg zone 10mm thick
7.87m: B, h, pl, sm
7.89m: B, h, pl, sm
7.9m: fg zone, Cs 20mm thick
8.17m: Cs 10mm thick
8.44m: Cs 10mm thick
8.84m: fg zone 20mm thick
9m: fg zone 10mm thick

Sampling & In Situ Testing:

Type	Core Rec. %	RQD %	Test Results & Comments
D			
D			
D			pp = 100-240
S			2,4,6 N = 10
S			7,9,12 N = 21
C	93.5	39.6	
C	100	62.9	
C	100	74.4	

RIG: Scout 4 **DRILLER:** Groundtest **LOGGED:** LAH **CASING:**
TYPE OF BORING: 115mm diameter SFA to 2.5m, wash boring to 3.1m, NMLC coring to 9.3m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: Location coordinates are in MGA94 Zone 56.

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_x	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	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

Douglas Partners
Geotechnics / Environment / Groundwater

Douglas Partners Pty Ltd

Proposed Mixed Use Development
86 – 94 Castlereagh Street & 77 – 79 Bathurst Street Liverpool, NSW

BORE: 1 DEPTH: 3.1 m – 9.3 m PROJECT: 92327.00 Dec 2018

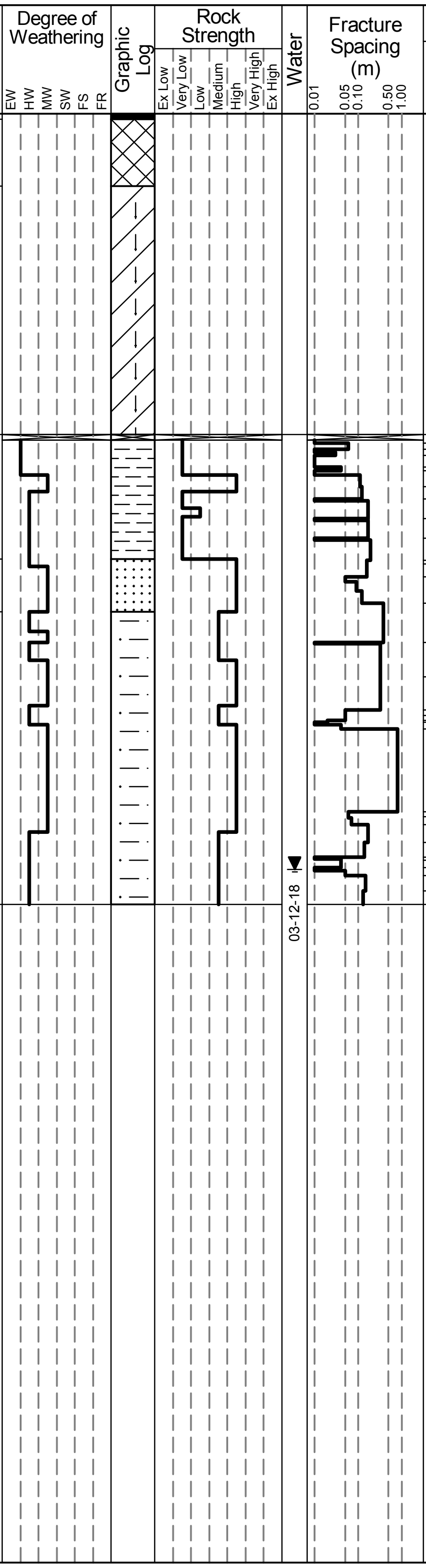
BOREHOLE LOG

CLIENT: Il Capitano Investments Pty Ltd
PROJECT: Proposed Multi-storey Residential Development
LOCATION: 86-94 Castlereagh Street &
77-79 Bathurst Street, Liverpool, NSW

SURFACE LEVEL: 21.2 mAHD
EASTING: 307707
NORTHING: 6244213
DIP/AZIMUTH: 90°/--

BORE No: 2
PROJECT No: 92327.00
DATE: 26/11/2018
SHEET 1 OF 1

Depth (m)	Description of Strata
0.05	ASPHALTIC CONCRETE
0.7	FILLING - dark grey sandy silty clay with some gravel, MC>PL
3.1	SILTY CLAY - stiff, light brown mottled red silty clay with some ironstone gravel, MC~PL - becoming grey mottled red, MC<PL below 1.0m - becoming very stiff, with very low strength, highly weathered shale bands below 2.1m
3.15	
4.3	SHALE - extremely low to very low strength, extremely to highly weathered, grey shale - becoming high strength, moderately weathered below 3.49m - becoming very low to low strength, highly to moderately weathered below 3.65m
4.81	SANDSTONE - high strength, moderately weathered, grey and brown fine grained sandstone
7.64	SILTSTONE - medium strength, highly to moderately weathered, grey siltstone - becoming high strength, moderately weathered below 5.28m - becoming medium strength, highly weathered below 5.72m - becoming high strength, moderately weathered below 5.9m - becoming medium strength, highly weathered below 6.94m
	Bore discontinued at 7.64m - limit of investigation



Discontinuities

B - Bedding J - Joint
S - Shear F - Fault

3.1m: CORE LOSS: 50mm
3.18m: fg zone 30mm thick
3.24m: Cs 30mm thick
3.3m: Cs 100mm thick
3.41m: B, h, pl, sm
3.45m: Cs 40mm thick
3.6m: B, h, pl, sm
3.72m: fg zone 20mm thick
3.91m: Cs 20mm thick
4.1m: Cs 20mm thick
4.31m: fg zone 40mm thick
4.35m: J, v, ir, ro, he 340mm long
4.47m: B, h, ir, sm
4.73m: B, h, pl, sm
5.11m: fg zone 10mm thick
5.44m: B, h, pl, sm
5.76m: B, h, pl, sm
5.81m: B, h, pl, sm
5.86m: B, h, pl, sm
5.88m: Cs 20mm thick
5.94m: B, h, pl, sm
6.74m: B, h, pl, sm
6.79m: J, 5°, sv, pl, sm 20mm long
6.87m: B, h, ir, sm
7.04m: B, h, pl, sm
7.18m: Cs 20mm long
7.21m: J, 45°, sv, ir, un 50mm long
7.28m: fg zone 30mm long
7.36m: B, h, pl, sm
7.51m: B, h, pl, sm

Sampling & In Situ Testing

Type	Core Rec. %	RQD %	Test Results & Comments
D			
D			
D			pp = 170-200
S			3,4,7 N = 11
S			7,9,15 N = 24
C	96.7	17.9	
C	100	83.5	

RIG: Scout 4 **DRILLER:** Groundtest **LOGGED:** LAH **CASING:**
TYPE OF BORING: 115mm diameter SFA to 2.5m, wash boring to 3.1m, NMLC coring to 7.64m
WATER OBSERVATIONS: Free groundwater at 7.3m, measured in well on 3/12/18
REMARKS: Location coordinates are in MGA94 Zone 56. Standpipe piezometer installed: 0 - 2.0m bentonite; 2.0 - 7.64m gravel; 0 - 2.6m casing; 2.6m - 7.64m slotted

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_x	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	▷	Water seep	S	Standard penetration test		
E	Environmental sample	▼	Water level	V	Shear vane (kPa)		

Douglas Partners Pty Ltd

Proposed Mixed Use Development
86 – 94 Castlereagh Street & 77 – 79 Bathurst Street Liverpool, NSW

BORE: 2 DEPTH: 3.1 m – 7.6 m PROJECT: 92327.00 Dec 2018

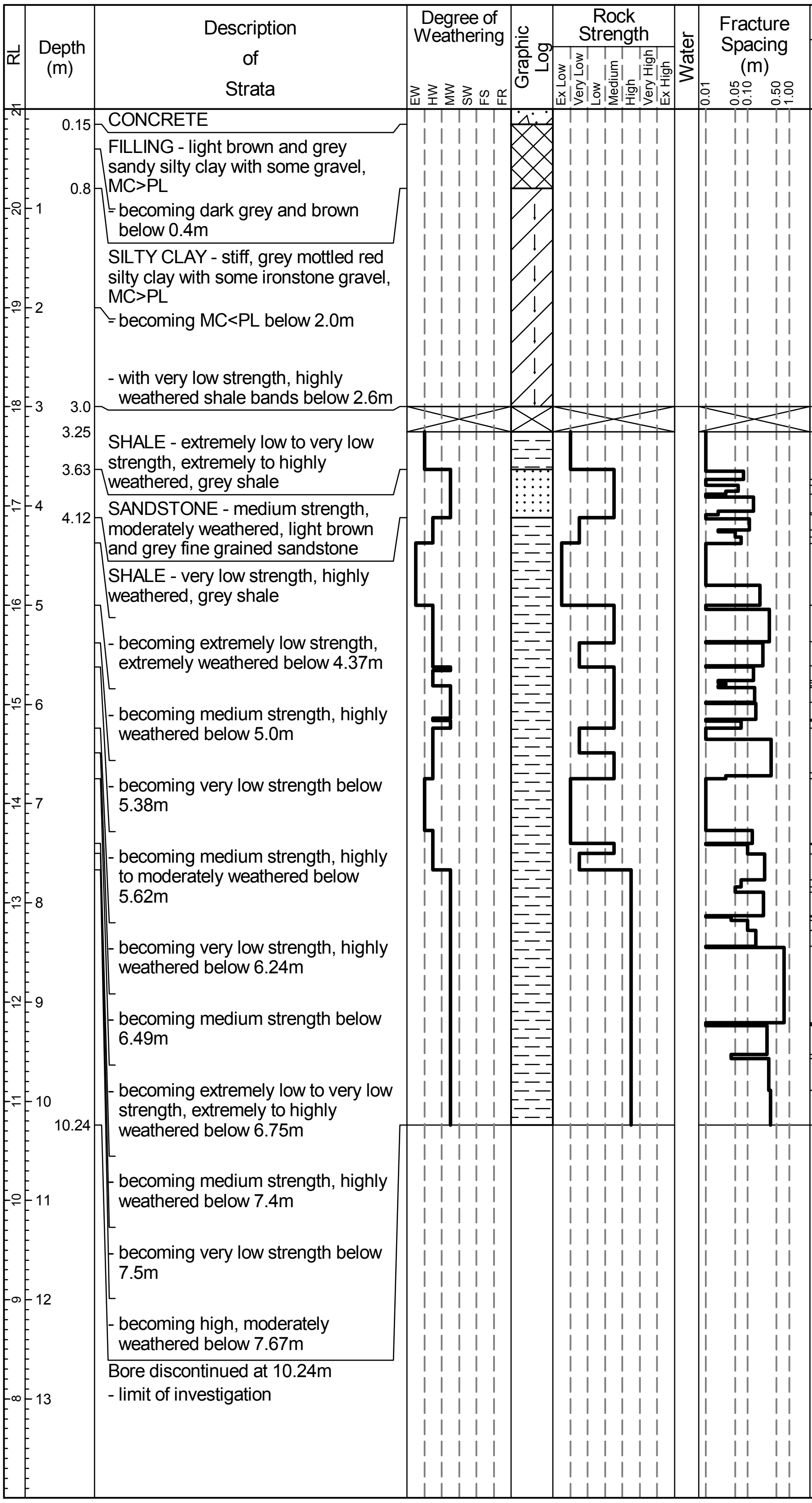
BOREHOLE LOG

CLIENT: Il Capitano Investments Pty Ltd
PROJECT: Proposed Multi-storey Residential Development
LOCATION: 86-94 Castlereagh Street &
77-79 Bathurst Street, Liverpool, NSW

SURFACE LEVEL: 21.0 mAHD
EASTING: 307754
NORTHING: 6244206
DIP/AZIMUTH: 90°/--

BORE No: 3
PROJECT No: 92327.00
DATE: 27/11/2018
SHEET 1 OF 1

Depth (m)	Description of Strata
0.15	CONCRETE
0.8	FILLING - light brown and grey sandy silty clay with some gravel, MC>PL - becoming dark grey and brown below 0.4m
3.0	SILTY CLAY - stiff, grey mottled red silty clay with some ironstone gravel, MC>PL - becoming MC<PL below 2.0m - with very low strength, highly weathered shale bands below 2.6m
3.25	
3.63	SHALE - extremely low to very low strength, extremely to highly weathered, grey shale
4.12	SANDSTONE - medium strength, moderately weathered, light brown and grey fine grained sandstone
10.24	SHALE - very low strength, highly weathered, grey shale - becoming extremely low strength, extremely weathered below 4.37m - becoming medium strength, highly weathered below 5.0m - becoming very low strength below 5.38m - becoming medium strength, highly to moderately weathered below 5.62m - becoming very low strength, highly weathered below 6.24m - becoming medium strength below 6.49m - becoming extremely low to very low strength, extremely to highly weathered below 6.75m - becoming medium strength, highly weathered below 7.4m - becoming very low strength below 7.5m - becoming high, moderately weathered below 7.67m
	Bore discontinued at 10.24m - limit of investigation



Discontinuities
B - Bedding J - Joint
S - Shear F - Fault

3m: CORE LOSS: 250mm
3.25m: Cs 380mm thick
3.63m: Cs, fg zone 20mm thick
3.73m: Cs, fg zone 60mm thick
3.79m: J, h, cu, ro 90mm long
3.88m: Cs, fg zone 30mm long
4.07m: B, h, pl, sm
4.09m: fg zone 40mm thick
4.24m: Cs 20mm thick
4.31m: B, h, pl, sm
4.38m: Cs 420mm thick
5m: fg zone 40mm thick
5.37m: fg zone 10mm thick
5.61m: Cs 10mm thick
5.76m: B, h, pl, he, clay inf
5.78m: B, h, pl, he, clay inf
5.81m: B, h, pl, he, clay inf
5.83m: B, h, pl, he, clay inf
5.98m: Cs 10mm thick
6.15m: B, h, pl, clay inf
6.16m: B, h, pl, clay inf
6.17m: B, h, pl, clay inf
6.24m: heavily bedded with clay seams 110mm thick
6.69m: J, 40°, sv, pl, sm, clay inf 60mm long
6.75m: Cs 520mm thick
7.4m: B, h, ir, sm
7.41m: B, h, pl, sm
7.51m: B, h, pl, sm
7.77m: B, h, pl, sm
7.84m: B, h, pl, sm
7.89m: B, h, pl, clay inf
8.13m: B, h, pl, clay inf
8.14m: B, h, pl, clay inf
8.18m: B, h, pl, clay inf
8.28m: B, h, pl, sm
8.44m: fg zone 10mm thick
9.21m: B, h, pl, sm
9.22m: B, h, pl, sm
9.23m: B, h, pl, sm
9.24m: B, h, pl, sm
9.53m: B, h, pl, sm
9.57m: B, h, pl, sm
9.89m: B, h, pl, sm

Sampling & In Situ Testing

Type	Core Rec. %	RQD %	Test Results & Comments
D			
D			
D			
S			pp = 100-150 3,5,6 N = 11
S			25/90mm,-,- refusal
C	76.2	13.3	
C	100	32.7	
C	100	74.8	

RIG: Scout 4 **DRILLER:** Groundtest **LOGGED:** LAH **CASING:**

TYPE OF BORING: 115mm diameter SFA to 2.5m, wash boring to 3.0m, NMLC coring to 10.24m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Location coordinates are in MGA94 Zone 56.

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_x	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	▷	Water seep	S	Standard penetration test
E	Environmental sample	⊻	Water level	V	Shear vane (kPa)

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Douglas Partners Pty Ltd

Proposed Mixed Use Development
86 – 94 Castlereagh Street & 77 – 79 Bathurst Street Liverpool, NSW

BORE: 3 DEPTH: 3.0 m – 10.2 m PROJECT: 92327.00 Dec 2018

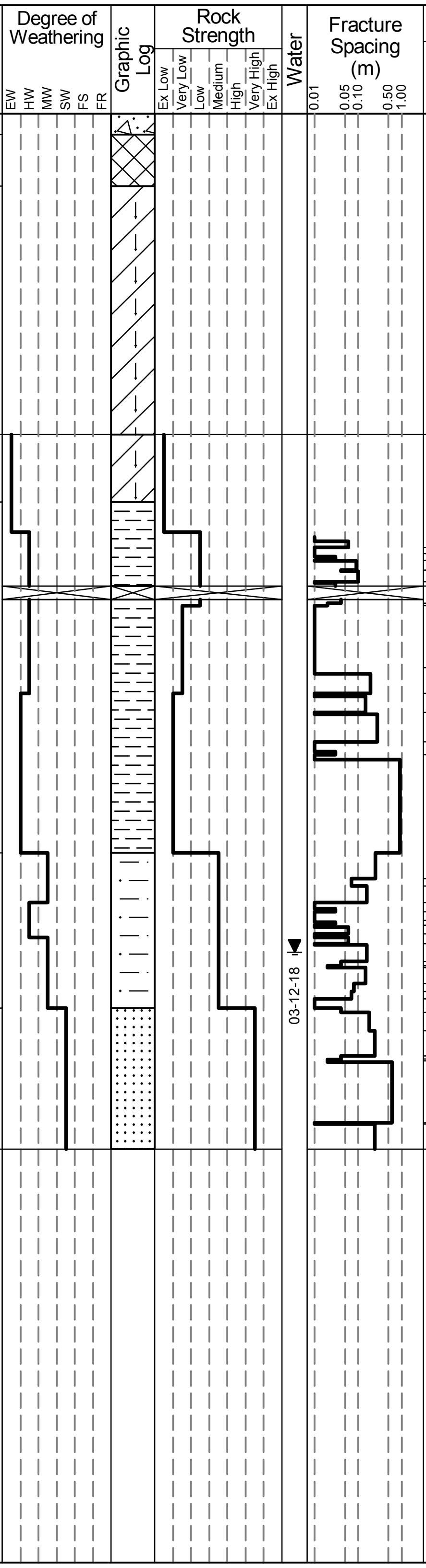
BOREHOLE LOG

CLIENT: Il Capitano Investments Pty Ltd
PROJECT: Proposed Multi-storey Residential Development
LOCATION: 86-94 Castlereagh Street &
77-79 Bathurst Street, Liverpool, NSW

SURFACE LEVEL: 20.5 mAHD
EASTING: 307790
NORTHING: 6244221
DIP/AZIMUTH: 90°/--

BORE No: 4
PROJECT No: 92327.00
DATE: 27/11/2018
SHEET 1 OF 1

Depth (m)	Description of Strata
	CONCRETE
0.2	FILLING - dark grey and brown sandy silty clay with some gravel, MC>PL
0.7	SILTY CLAY - stiff, red mottled grey silty clay with some ironstone gravel, MC~PL - becoming grey mottled red, MC<PL below 1.5m - becoming very stiff, with iron indurated bands below 2.5m
3.1	SILTY CLAY - very stiff, grey silty clay with iron indurated shale bands, MC<PL
3.75	SHALE - extremely low strength, extremely weathered, grey shale - becoming low strength, highly weathered below 4.04m
4.56	
4.69	- becoming very low strength below 4.75m - becoming extremely low to very low strength, extremely to highly weathered below 5.6m
7.14	SILTSTONE - medium strength, moderately weathered, dark grey siltstone - becoming highly weathered below 7.62m - becoming moderately weathered below 7.96m
8.64	SANDSTONE - very high strength, slightly weathered, grey fine grained sandstone with a trace of coal
10.0	Bore discontinued at 10.0m - limit of investigation



Discontinuities

B - Bedding J - Joint
S - Shear F - Fault

4.09m: Cs 40mm thick
4.19m: Cs 60mm thick
4.25m: fg zone 30mm thick
4.31m: fg zone 10mm thick
4.41m: Cs 10mm thick
4.52m: Cs 10mm thick
4.56m: CORE LOSS: 130mm
4.73m: B, h, pl, sm, ir, st
4.75m: heavily bedded/clay seam zone 600mm thick
5.35m: Cs 60mm thick
5.6m: fg zone 30mm thick
5.78m: Cs 20mm thick
6.07m: Cs 90mm thick
6.19m: Cs 50mm thick
7.39m: B, h, pl, sm
7.46m: B, h, pl, sm
7.62m: heavily bedded/clay seam 60mm thick
7.71m: heavily bedded/clay seam 80mm thick
7.79m: Cs 20mm thick
7.84m: Cs 20mm thick
7.92m: Cs 40mm thick
8.02m: Cs 10mm thick
8.19m: B, h pl, sm
8.23m: B, h pl, sm
8.25m: B, h, pl, clay inf
8.4m: B, h, pl, clay inf
8.48m: B, h, pl, clay inf
8.55m: Cs, fg zone 90mm thick
8.68m: B, pl, h, ro
8.86m: B, pl, h, ro
9.1m: B, h, cu
9.13m: J, 20°, sh, pl, ro 20mm long
9.15m: J, 20°, sh, pl, ro 20mm long
9.75m: B, h, pl, sm
9.76m: B, h, pl, sm

Sampling & In Situ Testing

Type	Core Rec. %	RQD %	Test Results & Comments
D			
D			
S			pp = 220-260 4,5,6 N = 11
S			6,8,11 N = 19
C	100	68	
C	95	0	
C	100	68.9	

RIG: Scout 4 **DRILLER:** Groundtest **LOGGED:** LAH **CASING:**

TYPE OF BORING: Concrete coring to 0.2m; 115mm diameter SFA to 2.5m, wash boring to 3.1m, NMLC coring to 10.0m

WATER OBSERVATIONS: Free groundwater at 8.1m, measured in well on 3/12/18

REMARKS: Location coordinates are in MGA94 Zone 56. Standpipe piezometer installed: 0 - 7.15m bentonite; 7.15 - 10.0m sand; 0 - 6.5m casing; 6.5m - 10.0m slotted

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_x	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	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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Proposed Mixed Use Development
86 – 94 Castlereagh Street & 77 – 79 Bathurst Street Liverpool, NSW

BORE: 4 DEPTH: 3.7 m – 10.0 m PROJECT: 92327.00 Dec 2018

Appendix C

Aggressivity Test Results

Envirolab Services Pty Ltd
ABN 37 112 535 645
12 Ashley St Chatswood NSW 2067
ph 02 9910 6200   fax 02 9910 6201
customerservice@envirolab.com.au
www.envirolab.com.au

CERTIFICATE OF ANALYSIS 207282

Client Details

Client	Douglas Partners Pty Ltd Smeaton Grange
Attention	Ludvig Arentz-Hansen
Address	18 Waler Crescent, Smeaton Grange, NSW, 2567

Sample Details

Your Reference	92327.00, Liverpool
Number of Samples	8 Soil
Date samples received	03/12/2018
Date completed instructions received	03/12/2018

Analysis Details

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

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

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

Report Details

Date results requested by	11/12/2018
Date of Issue	10/12/2018

NATA Accreditation Number 2901. This document shall not be reproduced except in full.

Accredited for compliance with ISO/IEC 17025 - Testing. **Tests not covered by NATA are denoted with ***

Results Approved By
Nick Sarlamis, Inorganics Supervisor

Authorised By

Jacinta Hurst, Laboratory Manager

ACCREDITED FOR TECHNICAL COMPETENCE

Misc Inorg - Soil						
Our Reference		207282-1	207282-2	207282-3	207282-4	207282-5
Your Reference	UNITS	BH1	BH1	BH2	BH3	BH3
Depth		0.5	1.0	1.0	0.5	1.0
Date Sampled		26/11/2018	26/11/2018	26/11/2018	27/11/2018	27/11/2018
Type of sample		Soil	Soil	Soil	Soil	Soil
Date prepared	-	07/12/2018	07/12/2018	07/12/2018	07/12/2018	07/12/2018
Date analysed	-	07/12/2018	07/12/2018	07/12/2018	07/12/2018	07/12/2018
pH 1:5 soil:water	pH Units	8.5	5.6	6.3	7.4	5.2
Electrical Conductivity 1:5 soil:water	µS/cm	160	64	72	200	200
Chloride, Cl 1:5 soil:water	mg/kg	<10	[NA]	[NA]	[NA]	160
Sulphate, SO4 1:5 soil:water	mg/kg	130	[NA]	[NA]	[NA]	190

Misc Inorg - Soil				
Our Reference		207282-6	207282-7	207282-8
Your Reference	UNITS	BH4	BH4	BH4
Depth		0.5	1.0	3.9-4
Date Sampled		27/11/2018	27/11/2018	27/11/2018
Type of sample		Soil	Soil	Soil
Date prepared	-	07/12/2018	07/12/2018	07/12/2018
Date analysed	-	07/12/2018	07/12/2018	07/12/2018
pH 1:5 soil:water	pH Units	8.8	5.4	6.8
Electrical Conductivity 1:5 soil:water	µS/cm	170	160	64

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25°C in accordance with APHA latest edition 2510 and Rayment & Lyons.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Alternatively determined by colourimetry/turbidity using Discrete Analyer.

QUALITY CONTROL: Misc Inorg - Soil						Duplicate			Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			07/12/2018	5	07/12/2018	07/12/2018		07/12/2018	[NT]
Date analysed	-			07/12/2018	5	07/12/2018	07/12/2018		07/12/2018	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	5	5.2	5.2	0	103	[NT]
Electrical Conductivity 1:5 soil:water	µS/cm	1	Inorg-002	<1	5	200	210	5	99	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	5	160	170	6	105	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	5	190	200	5	109	[NT]

Result Definitions

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

Quality Control Definitions

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

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

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

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

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals; 60-140% for organics (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.