



Douglas Partners

Geotechnics | Environment | Groundwater

Report on
Geotechnical Investigation

Proposed Stage 3B Car Park
67 to 69 Uralba Street and 24 to 28 Dalziell Street
Lismore

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John Holland Pty Ltd

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

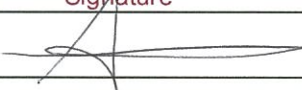
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Report on Geotechnical Investigation

Proposed Stage 3B Multi Storey Car Park

67 to 69 Uralba Street and 24 to 28 Dalziell Street, Lismore

1. Introduction

This report presents the results of a geotechnical investigation carried out by Douglas Partners Pty Ltd (DP) for the proposed Stage 3B multi storey car park to be located at 67 to 69 Uralba Street and 24 to 28 Dalziell Street, Lismore. The investigation was carried out at the request of John Holland Pty Ltd (JH), the builders for the project.

It is understood that the proposed development will comprise the construction of a six storey car park to be constructed in two stages. Stage 1 will comprise construction of a 250 space car park in the southern half of the development site and Stage 2 will comprise construction of an extension to the Stage 1 car park in the northern half of the site to provide an extra 250 car parking spaces.

Based on plans provided to DP, up to 5.3 m of bulk excavation is anticipated in order to achieve the basement car park design levels in the Stage 1 development area. Up to 2.0 m of cut and 3.0 m of filling will be required in the Stage 2 development area to achieve design levels.

The aim of this investigation as outlined in Douglas Partners Pty Ltd (DP) proposal GLD140225 (Rev01) dated 22 August 2014, was to provide information and comments on the following:

- regional geology and site description;
- field work procedures;
- results of all geotechnical laboratory testing;
- site preparation, earthworks, excavation conditions, unsuitable soils and reuse of site soils;
- suitable temporary and permanent batter slopes;
- basement retention options and geotechnical basement wall design parameters, design groundwater levels and rock anchor design parameters;
- allowable bearing pressures and estimated settlements for upper level footings;
- suitable pile foundations and pile design parameters for axially loaded piles in compression;
- comments on erosion potential and soil aggressivity in accordance with AS 2159-2009 (based on results of hospital investigation);
- indicative subgrade pavement California bearing ratio (CBR) and modulus of subgrade reaction parameters; and
- assessment of earthquake site sub-soil class to AS1170.4-2007 Part 4.

The investigation comprised the drilling and sampling of five boreholes followed by laboratory testing of selected samples. The details of the field work and laboratory testing are presented in this report, together with comments and recommendations on the items listed above.

This report should be read in conjunction with the notes entitled 'About this Report' in Appendix A along with any other explanatory notes and should be kept in its entirety without separation of individual pages or sections.

2. Site Description

The proposed development site comprises an 'L' – shaped area situated between Uralba Street to the north and Dalziell Street to the south. The site has an approximate frontage of 45 m along Uralba Street, some 55 m along Dalziell Street and extends approximately 100 m between both streets.

At the time of the investigation the northern half of the development site comprised two storey building housing medical practices with car parking areas located to the south. The southern half of the development site along its frontage with Dalziell Street comprised two storey brick built residential dwellings denoted as numbers 24 and 28. In between these residential dwellings at number 26 was a T-shaped area of vacant and which had been cleared of dwellings and vegetation.

Based on the site contour survey plan, ground surface elevations vary from approximately RL 34 mAHD along Uralba Street RL 20 mAHD along Dalziell Street.

An approximately 2.5 m high, 45° slope was located on the southern boundary between the Uralba Street medical practices and the vacant lot at 26 Dalziell Street. The site boundaries to the east and west were typically demarcated with wire or panel fencing.

3. Geology

Reference to the Geological Survey of New South Wales 1:250,000 series mapping indicates that the site lies on a 'peninsular' of early Miocene age Lamington Volcanics comprising "*basalt, sub-alkali basalt with members of rhyolite, trachyte, tuff, agglomerate, conglomerate and andesite*".

To the south of the site the lower ground is depicted as being underlain by Quarternary age alluvium comprising "*channel and flood plain alluvium; gravel, sand silt and clay*".

The natural ground conditions encountered during the field work (refer to Section 5 below) comprised residual soils underlain by basalt rock which are in general agreement with the published early Miocene geology.

4. Previous Investigation

A previous investigation was undertaken by DP at the main LBH site located on the northern side of the Uralba Street, directly opposite the proposed Stage 3B car park development site in July 2013. The results of the investigation are presented in "*Geotechnical Investigation: Proposed Eleven Storey Building, Lismore Base Hospital, Uralba Street, Lismore*", prepared for NSW Health Infrastructure, Reference 80243.00.

In summary, the investigation comprised three bores (designated Bores 4, 5 and 6) all of which encountered basalt rock at shallow depth. The basalt, which was found to be in excess of 20 m thick (limit of the investigation) comprised two flows separated by a tephra (ie: volcanic ash) layer up to 6 m thick.

The upper basalt flow was found to vary considerably across the site from high (or stronger) strength and moderately weathered to very low strength and highly weathered. Below the upper basalt flow a tephra layer was encountered and comprised extremely low strength to medium strength rock.

The lower basalt flow encountered beneath the tephra layer (in Bores 1 and 2 only) exhibited similar conditions to those observed within the upper basalt flow and comprised either low strength, highly weathered rock, or high strength 'fresher' rock.

The results of the laboratory testing undertaken as part of this previous DP investigation are summarised in Section 7 below.

5. Field Work Methods

Five boreholes (designated Bores 7 to 11) were drilled in accessible locations across the site between 15 and 21 October 2014 to depths of between 8 m and 20 m using a track mounted P120TB drilling rig at locations selected by JH.

The bores were commenced using continuous solid flight auger techniques to 1.0 m depth, and then advanced using washboring techniques then NMLC rock coring techniques to termination depths of the bores. The approximate test locations are shown on Drawing 1 in Appendix B.

Where high strength rock was not encountered within 1.0 m depth of the surface, standard penetration tests (SPTs) were undertaken, then at nominal 1.5 m intervals to provide samples for identification and an indication of relative density/strength and consistency.

On completion of drilling, and after making groundwater observations the bores were backfilled with any drill spoil, a plastic bore spider was installed and the surface reinstated to its pre-drilling condition.

The field work was undertaken by an experienced engineering geologist, who logged, sampled, photographed and performed point load strength testing of rock core samples.

6. Field Work Results

The subsurface conditions encountered in the bores are described in detail on the borehole logs presented in Appendix B. Photographs of the recovered core also follow the respective borehole log sheets in Appendix B. These should be read in conjunction with the notes 'About this Report' and other explanatory notes provided in Appendix A, which define the sampling methods, symbols and abbreviations and soil and rock descriptions used in their preparation.

In summary, the natural subsurface conditions encountered during this investigation typically comprised **residual clay soils** which were underlain by the variably weathered **upper basalt flow**. Underlying the upper basalt flow in Bore 7 only, a thin **basaltic tephra** layer was encountered and underlain by the highly weathered **lower basalt flow** which was generally of lower strength than the upper flow.

The above stratigraphic sequence varies across the site, not in composition but in the degree of weathering and fracturing, thus for the following sections, which describe the strata encountered in detail, for ease of the reader, the site has been divided into two zones (designated Stage 1 and Stage 2); refer Drawing 1 in Appendix B.

6.1 Stage 1 Development Area - Bores 7, 8 and 11

- **Filling:** uncompacted, clayey sand filling was encountered in all bores to between 0.05 m and 0.45 m depth.
- **Silty or Sandy Clay:** stiff, silty or sandy residual clay was encountered below the filling in all bores to between 1.0 m and 2.8 m depth.
- **Upper Basalt Flow:** the upper basalt flow was encountered below the residual clay soils in all bores and was variably weathered between the bores. Bores 7 and 8 initially comprised extremely low strength basalt which graded to very high strength rock at relatively shallow depth. The very high strength basalt was interbedded with bands of extremely low strength to medium strength basalt. The joints encountered within the upper basalt flow were typically at 0°, 10° and 45° and infilled with up to 5 mm of clay. Below depths of 9.0 m and 7.0 m in Boreholes 7 and 8 respectively, the basalt became less fractured and contained less 'weaker' bands. The less fractured basalt layer was encountered to 13.87 m depth in Bore 7 and to the base of Bore 8 at its termination depth of 8.0 m.

In Bore 11, the basalt encountered was initially very low strength with interbedded bands of medium strength rock, and was highly fractured to fragmented. Below 4.0 m the basalt became high strength, but was highly fractured and fragmented with numerous clay infilled joints and some interbedded bands of low and very low strength basalt. Below 8.3 m depth the basalt graded to very high strength and fractured to 8.85 m, and was underlain by medium strength basalt to bore termination at 9.0 m depth.

- **Tephra:** underlying the upper basalt flow in Bore 7 (at 13.87 m depth) a 0.13 m thick band of very low strength to low strength, highly weathered, purple grey tephra (volcanic ash) was encountered.
- **Lower Basalt Flow:** underlying the tephra layer in Bore 7, the lower basalt flow generally comprised interbedded bands of extremely low strength and medium strength, extremely weathered to moderately weathered, slightly fractured basalt. Bore 7 was terminated within this stratum at 20.0 m depth (RL 3 mAHD).

6.2 Stage 2 Development Area - Bores 9 and 10

- **Asphalt:** asphalt was encountered to 0.2 m depth on Bore 10 only.

- **Topsoil Filling:** uncompacted, silty sand topsoil filling was encountered in Bore 9 to 0.1 m depth.
- **Filling:** apparently uncompacted gravelly sandy clay or sandy clay filling was encountered to between 0.34 m and 0.8 m depth.
- **Residual Silty Clay:** estimated stiff, silty residual clay was encountered below the filling in Bore 9 to 1.0 m depth.
- **Upper Basalt Flow:** the upper basalt flow in the Stage 2 area of site was observed to have weathered considerably less than that beneath the 'lower' Stage 1 area of the site. The upper basalt flow comprised high strength, slightly weathered basalt which was initially slightly fractured with clay infilled joints typically at 0°, 10° and 45°. The upper basalt flow was encountered in both bores to their termination at 12 m and 8.14 m depth.

6.3 Groundwater

Due to washboring drilling techniques being used it was not possible to determine groundwater ingress during the drilling of the bores. Following completion of the drilling of the DP bores, they were left open for one day to allow monitoring of groundwater levels. The results of monitoring are presented in Table 1.

Table 1: Summary of Groundwater Levels

Borehole	Borehole Level (mAHD)	Depth of Groundwater (m)	Groundwater Elevation (mAHD)
7	23.00	4.20	18.80
8	26.75	2.10	24.65
9	33.50	0.94	32.56
10	30.50	0.90	29.60
11	25.75	1.20	24.55

Based on the limited groundwater monitoring undertaken, it is anticipated that perched groundwater could be expected at between approximately 1.0 m and 4.5 m below the current surface levels. It should be noted, however, that groundwater depths and ground moisture conditions are affected by climatic conditions and soil permeability, and will therefore vary with time.

7. Laboratory Testing

During the previous DP investigation for the LBH site geotechnical laboratory testing was performed on selected samples from the test bores, and comprised the following:

- Emerson class dispersion tests; and
- Soil pH, chloride and sulfate content tests.

The results of this laboratory testing are provided summarised in Tables 2 and 3 below.

Table 2: Emerson Class Dispersion Test Results

Borehole	Depth (m)	Material	Emerson Class Number	pH	Potential for Erosion
4	0.5	Filling- sandy clay with some gravel	4	7.6	Moderate
5	2.1-2.2	Weathered basalt	3	6.6	Moderate

Table 3: pH, Chloride and Sulfate Test Results

Borehole	Depth (m)	Material	pH	Chloride (mg/kg)	Sulfate, as SO ₄ (mg/kg)
5	2 – 2.15	Medium strength basalt	8.8	<10	<10
6	14.1 – 14.2	Low strength tephra	7.4	<10	<10

Selected lengths of rock core recovered from the bores undertaken during this current investigation were tested in the laboratory for point load index (I_s), both in axial and diametral orientations, to assess intact rock strength. The results [corrected to $I_{s(50)}$] are given on the borehole log report sheets presented in Appendix B and are in the range 0.25 MPa to 7.10 MPa, indicating variable conditions of low strength to very high strength rock.

8. Comments

8.1 Proposed Development

It is understood that the proposed development will comprise the construction of a six storey car park to be constructed in two stages. Stage 1 will comprise construction of a 250 space car park in the southern half of the development site and Stage 2 will comprise construction of an extension to the Stage 1 car park in the northern half of the site to provide an extra 250 car parking spaces.

Stage 1 will require between 3.8 m and 5.3 m of bulk excavation in order to achieve basement car park levels of RL 23.1 mAHD and RL 21.7 mAHD at approximately the centre of the site and the southern end of the site respectively.

The construction of the Stage 2 will require up to 2.0 m of bulk excavation along the northern elevation of the car park in order to achieve a basement car park level of RL 31.5 mAHD. From plans provided to DP it is expected that up to 3.0 m of filling will be required along the northern elevation of the Stage 1 building in order to level the site prior to the commencement of Stage 2 construction.

Locally deeper excavations may be required for lift overrun pits, services installation and footings. It is expected that the excavation for the lower car park levels will extend close to the eastern and western boundaries.

8.2 Appreciation of Ground Conditions

In summary the subsurface conditions across the site are relatively uniform in depth, thickness and composition. The weathering, strength and consequent fracturing profile changes significantly across the site. A summary of the conditions underlying the shallow covering 0.05 m to 0.8 m of generally gravelly sandy clay, sandy clay gravelly or clayey sand filling, then silty or sandy residual clays to between 1.0 m and 2.8 m is given below:

- **Stage 1:** A variable deeper weathering profile comprising very low strength, highly fractured to fragmented basalt with clay seams and some decomposed zones were encountered to 4.0 m depth in Bore 11. Below this depth high strength basalt was encountered to 8.85 m depth and underlain by medium strength basalt to bore termination at 9.0 m depth. Bores 7 and 8 encountered high strength basalt from a relatively shallow depth, however, it was highly fractured to fragmented and contained interbedded bands of extremely low to medium strength basalt. Bore 8 was terminated in this stratum at 8.0 m depth.

Below 10.4 m depth in Bore 7 the basalt became slightly weathered and slightly fractured, and did not contain the interbedded weaker zones. A 0.13 m thick band of tephra was encountered at approximately 13.87 m depth and comprised very low to low strength ash. Beneath the tephra layer, the lower basalt flow was encountered and typically comprised very low and low strength, highly weathered basalt to 20.0 m depth.

- **Stage 2:** Limited weathering and a high strength (or stronger) more 'massive' basalt profile.

Based on the limited groundwater monitoring undertaken, it is anticipated that groundwater could be expected at between approximately 1 m and 4.5 m below the current surface levels. It should be noted, however, that groundwater depths and ground moisture conditions are affected by climatic conditions and soil permeability, and will therefore vary with time.

It should also be noted that following a number of batter and pad footing inspections undertaken at the main LBH site, no groundwater seepages were noted from the excavation faces or at the base of any footings. Therefore, groundwater seepages within the high strength basalt rock are unlikely however, groundwater seepages in the lower strength and fractured basalt (particularly where clay seams are present) are expected.

Due to the subsurface conditions encountered and the requirement to excavate close to the boundaries of the site, the anticipated major implications for the design and construction of the proposed building and basement are:

- Excavatability;
- Stability of excavated faces during construction; and
- Foundation options, particularly high level footings.

8.3 Basement Level Construction

It is understood that in order to achieve the proposed design levels for the basement car park levels in Stage 1, bulk excavation depths to between 3.8 m (RL 23.1 mAHD) and 5.3 m (RL 21.7 mAHD) will be undertaken at approximately the southern end of the site and centre of the site respectively.

Stage 2 construction will require up to 2.0 m (RL 31.5 mAHD) of bulk excavation in order to achieve the proposed design level for the basement car park level at the northern elevation of the building.

Based on the plans provided, it is anticipated that the proposed basement level excavations will extend close to the western and eastern site boundaries and therefore close to neighbouring properties. The need therefore, to maintain stability of the excavation faces and of the adjacent roads, buildings, footpaths and in-ground services will impact on the method of construction adopted.

Where space permits and surcharging from neighbouring structures does not occur, then battering for both long term and short term stability of the excavation faces could be undertaken.

Where battering of the excavation faces cannot be undertaken, temporary support of near vertically cut excavation faces will be required to prevent instability of the near surface clay soils, the weathered basalt and the highly fractured basalt anticipated in the top sections of the faces (particularly in the Stage 1 development area) to prevent potentially unstable block and wedge failure of rock deeper down.

Progressive excavation with installation of support of the near-surface soils, weathered and highly fractured basalt to the required bulk excavation levels is considered feasible. It is anticipated that such support could comprise soil nails and rock bolts with mesh reinforced shotcrete panels.

Alternative methods could comprise anchored contiguous or soldier piles and shotcrete panels, although installation within the high strength (or stronger) basalt would be slow. This method would, however, reduce any delays with regard to potential instability in excavation faces, allow vertical excavation and the piles to be incorporated in the design of the permanent basement walls to support proposed building loads.

It is anticipated that in the Stage 2 development area, beyond the retention of the surface soils, only mapping and some anchoring or bolting with mesh and shotcrete would be required within the high strength, fractured basalt.

8.4 Excavatability

Excavation of the filling, residual clay soils and extremely low to low strength basalt should be achievable using conventional medium sized earthmoving plant (ie. 20-25 tonne hydraulic excavators with toothed buckets and ripping tynes etc). Where medium to high strength, highly fractured rock is encountered, heavy rock breakers will be required to facilitate excavation.

It is anticipated that excavation of the high strength (or stronger) slightly fractured rock could be undertaken with using large sized earthmoving plant ((ie. 80 tonne hydraulic excavators fitted with large rock breakers).

Inspection of core samples by tendering contractors is considered essential prior to the selection of the most suitable excavation technique and finalising of tender price.

Rock breaking or blasting may require the need for vibration monitoring at adjoining buildings and the undertaking of building condition reports of adjoining structures prior to bulk excavation. Predrilling or

rock sawing of the high (or stronger) strength rock at the site boundaries could be undertaken to reduce the transmission of ground vibrations and to avoid over-excavation.

It should be recognised that the above excavatability estimates are based on materials encountered at test locations only and that excavation conditions may prove more difficult (or easier) between and beyond these test locations.

8.5 Construction Vibration, Noise and Movements

Vibration will result from excavation, earthworks and construction work on this site. There is significant debate as to the maximum amount of vibration that buildings can accommodate; however, vibration restrictions must be set with a realistic appreciation for the normal operational environment of the site. Tolerance to vibration will also depend upon the nature of the materials used in construction (ductile or brittle), the age of the buildings, and whether or not the buildings are already cracked or in disrepair.

Development of a peak particle velocity (PPV) criterion in conjunction with a frequency component is considered essential to enable adequate monitoring and control of demolition and construction vibrations. The development of a site-specific vibration monitoring program is outside the scope of this report, but it is anticipated that the following could be adopted as an initial guide:

Type of Building or Structure	Historical/Heritage	Sound Construction
Maximum PPV (mm/sec)	2 – 3 ⁽¹⁾	10
Frequency range (Hz)	<10	10 – 50

Note: ⁽¹⁾ or below background levels.

A properly designed vibration monitoring program will need to be implemented during demolition, excavation and new construction phases of the project. Demolition and construction noise and its impact upon nearby tenants and residents will also need to be considered.

A movement monitoring program is suggested to ensure excavation, earthworks and construction works do not affect adjacent structures. A dilapidation/building condition survey of the adjacent buildings prior to commencing site work, coupled with vibration, noise and movement monitoring, is also suggested. DP can undertake the vibration monitoring works if required.

8.6 Temporary Slope Batters

Full height battering of excavation faces may not be suitable due to the close proximity of the adjoining buildings, access roads, footpaths and in ground services.

If space permits, however, non-surcharged dry batter slopes cut into the various soil profiles encountered during the field work, may be designed for short and long term conditions, using the ratios of horizontal (H) to vertical (V) measurements as presented in Table 4 below.

Table 4: Temporary Batter Slopes

Material	Safe Batter Slope (H:V)		Proposed Temporary Batter Heights and Depth to Base of Strata	
	Short Term	Long Term	Stage 1 5.3 m ⁽¹⁾	Stage 2 2.0 m ⁽¹⁾
Uncontrolled filling	2:1	2.5:1	0.2 – 2.8	0.8 – 1.0
Controlled filling, natural stiff or stronger clays and extremely low strength basalt	1:1	2:1		
Very low to low strength weathered basalt	1:1	1:1	2.8 – 4.0 ⁽³⁾	-
High strength (or stronger) fractured basalt	0.75:1 to 1:1 ⁽²⁾		1.7 ⁽⁴⁾ – 5.3	2.0

Notes: ⁽¹⁾ Excavation depth based on proposed bulk excavation depth and bore elevation.

⁽²⁾ Design using 1:1 but may be steepened to 0.75:1 following geotechnical inspection and analysis.

⁽³⁾ Based on the ground conditions encountered in Bore 11 only

⁽⁴⁾ Based on the ground conditions encountered Bores 7 and 8 only

Specific slope stability assessment, including stability analysis, should be undertaken prior to any final design, particularly those adopting a battered excavation in excess of the batter heights given in the table above.

It is strongly recommended that all batters be inspected and geologically mapped during excavation to identify the condition and orientation of rock mass defects along the rock faces. Measured orientations and representative averages should then be plotted and analysed to assess the potential for wedge or block failure. All batters where rock is exposed, particularly the fractured and highly fractured basalt, which contained numerous clay filled fractures, joints and seams some near vertical, will need to be scaled back in a controlled manner to remove any loose blocks or wedges of rock.

Furthermore, it should also be expected that some particularly fragmented and fractured zones will be encountered within the basement excavation faces (most likely during excavation in the Stage 1 development area), thus provision should be made for some spot bolts, mesh and shotcrete. All completed batters will require geotechnical inspection during which recommendations with regard to spot bolts etc (if required) could be made.

All batters should be completed with lined open drains across the top of cut batter slopes to prevent overland water flow down the batters and subsequent erosion of the near surface soils.

For dry, temporary, confined excavations (ie. services trenches, footing excavations, lift overrun pit etc) excavated with vertical sides to a maximum height of 2.0 m should remain stable, provided adversely dipping joints or clay seams are not present, which should be confirmed on-site by a geotechnical engineer. Based on the limited groundwater monitoring undertaken it is possible that groundwater may seep from the faces of confined excavations and will require temporary dewatering and possibly the installation of positive support to ensure personnel safety.

8.7 Positive Support

Positive excavation support may be required to minimise ground movements behind the excavation faces to ensure adjoining structures, footpaths, access roads and in-ground services are not affected as a result of basement construction.

The near surface soils, very low to low strength basalt encountered to 4.0 m depth (Bore 11 only) in the Stage 1 development area could be retained near vertically by regularly spaced, anchored soldier piles with shotcrete panels between, or by shotcrete panels supported by a regular grid of soil nails/rock bolts and rock anchors.

It is expected that a full depth, bored soldier pile retaining wall founded into the very low to low strength, fractured rock would be feasible. However, due to the variation in rock strengths across the site, installation into a uniform stratum such as high strength rock would be necessary. Installation into high strength rock, would be extremely slow, and costly to construct and is therefore not recommended. However, if adopted anchored or propped piled walls could be designed using the parameters given in the following sections.

8.7.1 Shotcrete Panels Supported by Soil Nails and Rock Bolts

Shotcrete panels supported by a regular grid of soil nails and rock anchors could be used to temporarily support the near surface soils and very low strength rock anticipated in the top sections of the faces. Shotcrete panels would be required over the entire extent of these weaker soils and rock to ensure they do not deteriorate and become unstable with time.

8.7.2 Regular Grid of Rock Anchors

Where excavation into high strength, fractured to slightly fractured basalt is required (primarily in the Stage 2 development area) it is anticipated that rock anchors possibly with the use of mesh and shotcrete in any particularly loose zones would be sufficient for short term stability of cut faces.

Determination of rock anchor spacing and lengths is a matter for detailed design, DP could undertake this design if required.

8.7.3 Bond Stresses for Rock Anchors and Dowels

Temporary rock anchors and rock dowels may be required to support the faces of the excavation and it is envisaged that such anchors will be multi-strand cable anchors bonded into rock.

Ultimate bond stresses for the design of rock bolts and anchors are given in Table 5 below. These stresses should be divided by a factor of safety of 2 to assess suitable working bond stresses in the design of fixed anchor lengths. All ground anchors should be bonded into rock below a 45° line rising up from the toe of the excavation, and should be load tested to 1.3 times their working load after installation.

Table 5: Ultimate Bond Stresses for Ground Anchors

Material	Ultimate Unfactored Bond Stress (kPa)
Filling, residual soils and extremely low strength basalt	N/A
Very low and low strength basalt	200
Medium strength (or stronger) highly fractured to fragmented basalt	500
High strength (or stronger) slightly fractured to fractured basalt ⁽¹⁾	1500
Very high strength slightly fractured (or less) basalt ⁽¹⁾	3000

Notes: ⁽¹⁾ Anticipated to be encountered only in excavations in the Stage 2 development area.

8.7.4 Wall Design Pressures

It is envisaged that the faces of the excavation (if required) would be either battered or supported by anchored shotcrete panels around the top sections of the faces followed by a regular grid of ground anchors below. The permanent basement walls will then be constructed inside the temporarily supported faces. The basement walls will be propped internally by floor slabs, and it is assumed that the structure (ie. walls and floor slabs) will support the long term excavation earth pressures.

For preliminary design, walls supported by multiple rows of anchors or props could be designed using a rectangular earth pressure distribution of $0.3\gamma H$ kPa over the full height of the wall (where H is the total vertical height of the wall in metres and γ is the bulk unit weight of the soil retained). Suggested value for soil is 18kN/m^3 , very low strength rock 20kN/m^3 and high strength rock is 22kN/m^3 . It is probable that the earth pressure will be below this value at the top and toward the base of the wall. DP should be contacted if more refined and sophisticated wall analysis is required.

Surcharge loading should be multiplied by a lateral coefficient of 0.3 to calculate the resulting additional lateral pressure on the walls.

Adequate drainage will need to be provided behind temporary shoring and/or permanent walls (for 'drained' basement) via strip drains and weepholes to ensure water pressure does not build up behind these walls. If the permanent basement walls are to be 'tanked', then the walls and base slabs will need to be designed for full hydrostatic pressures.

8.8 Re-Use of Cut Materials

The field work results indicate that the materials won from excavations on the site would vary between near surface sandy or silty residual clays, extremely low strength to very high strength basalt. It is considered that the majority of these materials would be suitable for reuse as structural or general filling, subject to meeting minimum placement and compaction requirements

The existing gravelly sandy clay filling, residual soils and extremely low strength rock could be reused as controlled filling provided that any oversized inert material (e.g. gravel, rock, brick, concrete etc greater than 75 mm) is crushed or removed or any deleterious material (e.g. organic matter, steel, plastic etc), if encountered, is removed.

The weathered highly fractured and fragmented very low to low strength basalt is expected to be recovered as a gravelly clay mixture and would be suitable for reuse with little working provided that

any material greater than 75 mm is crushed or removed. Where 'fresher' high and very high strength, slightly fractured basalt is encountered (particularly in the Stage 2 development area) it is likely to require crushing prior to reuse.

8.9 Site Preparation at the Base of the Excavation

The exposed subgrades at bulk excavation levels of RL 23.1 mAHD and RL 21.7 mAHD in the Stage 1 development area are expected to comprise residual clay soils, or high strength basalt with interbedded extremely low to medium strength bands. The exposed subgrade at the bulk excavation level of RL 31.5 mAHD in the Stage 2 development area is expected to comprise very high strength, slightly fractured basalt.

It would be prudent to over excavate any areas below proposed ground bearing floor slabs in the high strength (or stronger) basalt typically encountered in Stage 2 area. It is suggested that over excavation by at least 0.1 m should occur prior to the placement of select filling. This will ensure that any ground bearing floor slabs will be constructed on a uniform platform to avoid any problems associated with 'hard' spots directly beneath proposed floor slabs.

The exposed subgrade at the base of the excavations should be air blasted to remove all loose material and then inspected. Any soft or loose material should be removed and replaced with approved and suitably compacted fill prior to any platform fill placement.

It is recommended that all residual clay soils (encountered in the area of Bore 11 in the Stage 1 development area) be removed and the underlying weathered very low strength basalt exposed. If any soft or loose zones are present, they should be removed and replaced with approved and suitably compacted filling. Test rolling should be carried out with a smooth drum roller with a minimum static weight of 12-tonne.

Any new filling, if required to achieve design levels, should be undertaken under 'Level 2' supervision and testing as detailed in AS 3798–2007 (Ref 3). Filling should be placed in layers not exceeding 300 mm 'loose' thickness, with each layer compacted to a minimum dry density ratio of 100% Standard compaction within $\pm 2\%$ of optimum moisture content.

Trafficability problems on the weathered very low strength basalt should also be expected during and after periods of rainfall or other increases in subgrade moisture content. It will be essential to keep this lower part of the site well drained during construction. A granular working platform is recommended to reduce potential lost time during or following wet weather, and to reduce wetting or drying of the subgrade soils.

8.10 Foundations

With anticipated column working loads in the order of 4000 kN, it is considered that building foundations will generally need to be supported in very low strength or stronger rock. Based on the subsurface conditions encountered in the bores, very low and high strength basalt is likely to be present at bulk excavation depth in the Stage 1 development area and high strength basalt is likely to be present at bulk excavation depths in the Stage 2 development area.

It is anticipated that due to the varied rock strengths and weathering encountered in the Stage 1 development area that a combination of shallow and deepened pad footings and/or piles in conjunction with a suspended raft may be required to minimise differential settlement of the structure across the site. All building footings should be founded in similar strength strata to reduce the risk of differential settlement between adjacent footings which could damage the new structures, however as some differential settlement is expected to occur, the structure should be designed to incorporate movement joints to allow for some articulation.

8.10.1 Pad and Strip Footings

The use, performance and associated bearing pressure of pad and strip footings on this site will be controlled by both the strength, degree of weathering and fracturing of the basalt rock. Rock strengths at the anticipated founding depths are typically varied in strength and weathering beneath the Stage 1 development area and generally uniform beneath the Stage 2 development area.

Therefore, the governing factors with regard to the use of pad footings in the Stage 1 development area will be presence of lower strength bands, clay seams, persistence of fracture spacing and the resulting settlement caused. It would therefore be essential to base the design of pad footings on the lowest strength material encountered within 1.5 x the depth of the breadth of footing base order to limit punching failure into the weaker strata bands and potential excessive settlements.

Provided site preparation is carried out in accordance with the recommendations in this report, it is considered that pad or strip footings keyed in to the respective rock strengths could be designed using the allowable bearing pressures given in Table 6.

It is expected that all 'uncontrolled' filling under the footprint of the proposed structures will be removed during bulk excavation to achieve the proposed design levels.

Table 6: Maximum Allowable Bearing Pressures for Pad and Strip Footings

Material	Maximum Allowable Bearing Pressure (kPa)	Base of Strata Depths in Proposed Excavations	
		Stage 1 5.3 m ⁽¹⁾	Stage 2 2.0 m ⁽¹⁾
Filling, residual soils and extremely low strength rock	N/A	1.7 m - 2.8 m	0.8 m – 1.0 m
Very low to low strength weathered basalt	1000	To 4.0 m depth in Bore 11 only	-
Medium (or stronger) highly fractured to fragmented basalt	1500	To the base of the proposed excavation at 5.3 m depth	-
High strength (or stronger) slightly fractured to fractured basalt	3000	-	To the base of the proposed excavation in Bore 9 To 1.2 m depth in Bore 10
Very high strength slightly fractured (or less) basalt	6000	-	To the base of the proposed excavation in Bore 10

Notes: ⁽¹⁾ Estimated excavation depth below existing site levels required to achieve proposed bulk excavation levels.

At the above allowable bearing pressures, total settlements would not be expected to exceed 1% to 2% of footing width. However, as noted previously some allowance should be made for differential settlement (possibly through the use of movement joints and careful articulation of the structure) due to both the lateral and vertical variation in basalt weathering profile.

All footing excavations will need to be inspected by a qualified geotechnical engineer/engineering geologist prior to casting of concrete to confirm rock quality. As part of the inspection process, probe drilling and spoon testing must be carried out to ensure the percentage of clay seams and highly fractured zones present beneath footings do not exceed that assumed in the design.

8.10.2 Bored Piles Founded in Rock

Bored piles may be used should the allowable bearing pressures presented in Section 8.10.1 prove inadequate for the structural loadings.

The most suitable pile types are expected to be bored piles founded in the slightly fractured, high strength basalt. Care will be required, however, when designing piles, particularly if they are to be founded in the upper basalt flow above any weaker zones, to ensure that punching failure with settlement does not occur. It is therefore recommended that piles in this situation are founded a minimum of four pile diameters above any decrease in rock strength within the upper basalt flow or from the upper basalt flow/tephra interface.

Bored piles founding into the nominated founding strata a minimum of one pile diameter for rock, can be designed using the ultimate geotechnical strengths (R_{ug}) given in Table 7. During design the upper 1.5 D length of shaft (where D = pile diameter) should be ignored in shaft capacity calculations to allow for load development.

Pile capacities and suitable pile types should be confirmed by prospective piling contractors. Contractors should also be made aware of the potential for groundwater inflow at depth.

Table 7 – Bored Pile Design Pressures (Ultimate Unfactored)

Material	Ultimate Unfactored Shaft Adhesion (MPa)	Ultimate Unfactored End Bearing (MPa)	Base of Strata Depths in Proposed Excavations	
			Stage 1 5.3 m ⁽¹⁾	Stage 2 2.0 m ⁽¹⁾
Filling, residual soils and extremely low strength rock	N/A	N/A	1.7 m - 2.8 m	0.8 m – 1.0 m
Very low to low strength weathered basalt	0.15	1	To 4.0 m depth in Bore 11 only	-
Medium (or stronger) highly fractured to fragmented basalt	0.2	6	To the base of the proposed excavation at 5.3 m depth	-
High strength (or stronger) slightly fractured to fractured basalt	0.8	25	-	To the base of the proposed excavation in Bore 9 To 1.2 m depth in Bore 10
Very high strength slightly fractured (or less) basalt	2	60	-	To the base of the proposed excavation in Bore 10

Notes: ⁽¹⁾ Estimated excavation depth below existing site levels required to achieve proposed bulk excavation levels

Where limit state methods are used to design the piles, the ultimate geotechnical strength ($R_{d,ug}$) must be multiplied by a suitable geotechnical strength reduction factor (ϕ_g) to obtain the design geotechnical strength ($R_{d,g}$). The geotechnical strength reduction factor is dependent upon several factors which were unknown at the time of preparation of this report, including incorporation of pile testing (if any) into the construction sequence and method of pile testing. As a guide, where the average risk rating is assessed to be high, there is no pile testing and the system has low redundancy, a ϕ_g value of 0.45 would apply. Guidance on the choice of ϕ_g factor is provided in Section 4 of AS2159–2009.

Where working stress methods are used to design piles and no pile testing is carried out, the above ultimate values should be divided by a factor of safety of 2.5.

Experience indicates that settlements of properly designed and constructed piles designed using suitably factored shaft adhesion and end bearing values, are unlikely to exceed 1% of the pile diameter.

It should be noted that the ability to drill bored piles in rock is not only dependent on the characteristics of the rock (strength, fracture spacing etc) but also the type (power and size) of the drilling rig and the size (diameter) of piles. Bored pile installation in medium or stronger strength rock will require the use of heavy drilling plant such as Casagrande, Soilmecc or Bauer rigs. It is recommended that the drilling

contractors allow for slow drilling rates, the use of coring buckets, high bit wear and also confirm the size of equipment required prior to commencing works on the site.

8.10.3 Verification of Design Pressures

It is essential that both pile and foundation excavations (where applicable) be inspected by experienced geotechnical personnel to ensure the design parameters adopted are suitable for the ground conditions and to ensure that there is no soft or loose material remaining at the base of the excavations or smear on the side walls. As previously stated ground conditions can vary, and it is essential that adequate provision be made throughout the project to vary foundations to suit differing ground conditions.

8.11 Aggressivity of Foundation Soils

The results of the aggressivity testing undertaken during the previous DP geotechnical investigation for the main LBH site were compared to values presented in AS 2159 to arrive at exposure classifications for buried concrete and steel. These suggest that soil conditions are generally 'non-aggressive' for buried concrete and 'non-aggressive' for buried steel. It is anticipated that the same values can be adopted for the ground conditions encountered below Stage 1 and Stage 2 of the proposed Stage 3B car park.

Recommendations are presented in AS 2159 for appropriate minimum concrete strengths and minimum cover to reinforcing steel for various aggressivity categories.

8.12 Earthquake Site Factor

In accordance with AS 1170.4 (Ref 2), it is recommended that a site sub-soil classification of "Class B_e– Rock Site" be adopted, in accordance with the definitions presented in *Section 4.2 – Class Definitions*.

A site hazard factor (z) of 0.05 has been assessed for the site based on the above standard.

8.13 Erodability and Erosion Control

Fine grained soils are prevalent near surface at the site and would generally be susceptible to erosion and dispersivity. Given the proposed depth of excavation at the site and the fine grained soils present, it is expected that there is a moderate erosion risk associated with surface water flow, as the soils exposed to water runoff will be mainly contained within any basement excavation.

Erosion control measures at the surface will require detailed design. It is expected, however, that as a minimum, measures will need to include silt fences, hay bales and measures to limit water runoff velocity (such as swales or benches).

It is recommended that adequately lined collector drainage be installed at the top/crest of all batters and that all clean drainage is discharged off-site via pipes or lined channels.

8.14 On-Ground Floor Slabs

Following basement excavation, the exposed subgrade is expected to comprise high to very high strength Basalt in Stage 2, and very low strength weathered basalt in Stage 1.

Provided subgrade preparation is carried out in accordance with the recommendations given in this report, a modulus of subgrade reaction (k) of 50kPa/mm could be adopted for the design of floor slabs subjected to standard wheel loads (i.e. car traffic). This is based on a soaked California bearing ratio (CBR) of 10% where the subgrade comprises medium strength (or stronger) basalt subjected to standard wheel loads (ie. car traffic). A modulus of subgrade reaction of 40kPa/mm or soaked CBR of 7% could be used for slabs founded on the very low strength basalt based on its potential high plasticity clay content. For loaded areas of different proportion or different load intensity to standard wheel loads, DP should be contacted for further advice.

It is envisaged that the basement will be 'drained' and therefore on-ground basement floor slabs will need to be designed with appropriate hydrostatic pressure relief such as a gravel drainage layer with a grid of 'ag' pipes linked to sumps for removal by pumps.

8.15 Further Investigation and Geotechnical Inspection

Based on the subsurface conditions encountered and the findings of this report it is recommended that all batters, excavations and footing excavations (particularly those personnel are to enter) are inspected by an experienced geotechnical engineer. Inspection is particularly required along excavation faces and batters to ensure that there are no adversely orientated joints or shear planes, which could lead to failure especially within the highly fractured to fractured basalt which contained numerous clay filled fractures, joints and some near vertical seams. Footing inspections will also particularly important to confirm base material strength of all footings in the Stage 1 area, where very low strength basalt was encountered.

9. References

1. Australian Standard AS 3798–2007 "Guidelines on earthworks for commercial and residential developments", Standards Australia.
2. Australian Standard AS 2159 – 2009 "Piling – Design and Installation", Standards.
3. Australian Standard AS 1170.4–2007, "*Structural Design Actions, Part 4: Earthquake actions in Australia*", Standards Australia.

10. Limitations

Douglas Partners (DP) has prepared this report for the proposed Stage 3B multi-storey car park located at 67 to 69 Uralba Street and 24 to 28 Dalziell Street, Lismore in accordance with DP's revised proposal GLD140225 (Rev. 01) dated 22 August 2014 and acceptance received from John Holland Pty Ltd. The report is provided for the exclusive use of John Holland Pty Ltd for this project only and for the purpose(s) described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its

exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

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

DP's advice is based upon the conditions encountered during 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 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.

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 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
Notes on Sampling Methods
Symbols and Abbreviations
Soil and Rock Descriptions

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.



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.

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


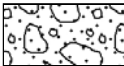
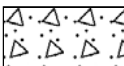

Other

fg	fragmented
bnd	band
qtz	quartz


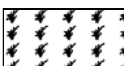
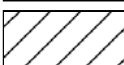
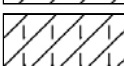
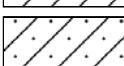
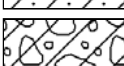
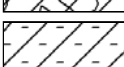

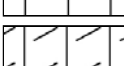
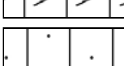

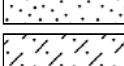
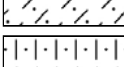
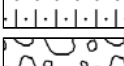
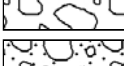
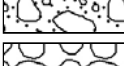

Symbols & Abbreviations

Graphic Symbols for Soil and Rock




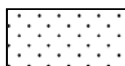
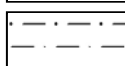
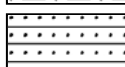
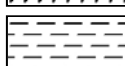
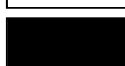
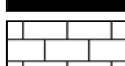
General

	Asphalt
	Road base
	Concrete
	Filling

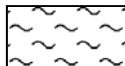
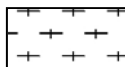

Soils

	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

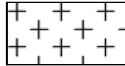
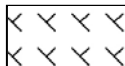
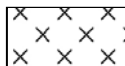
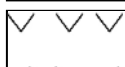
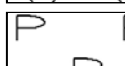
Sedimentary Rocks

	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



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 loner 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.

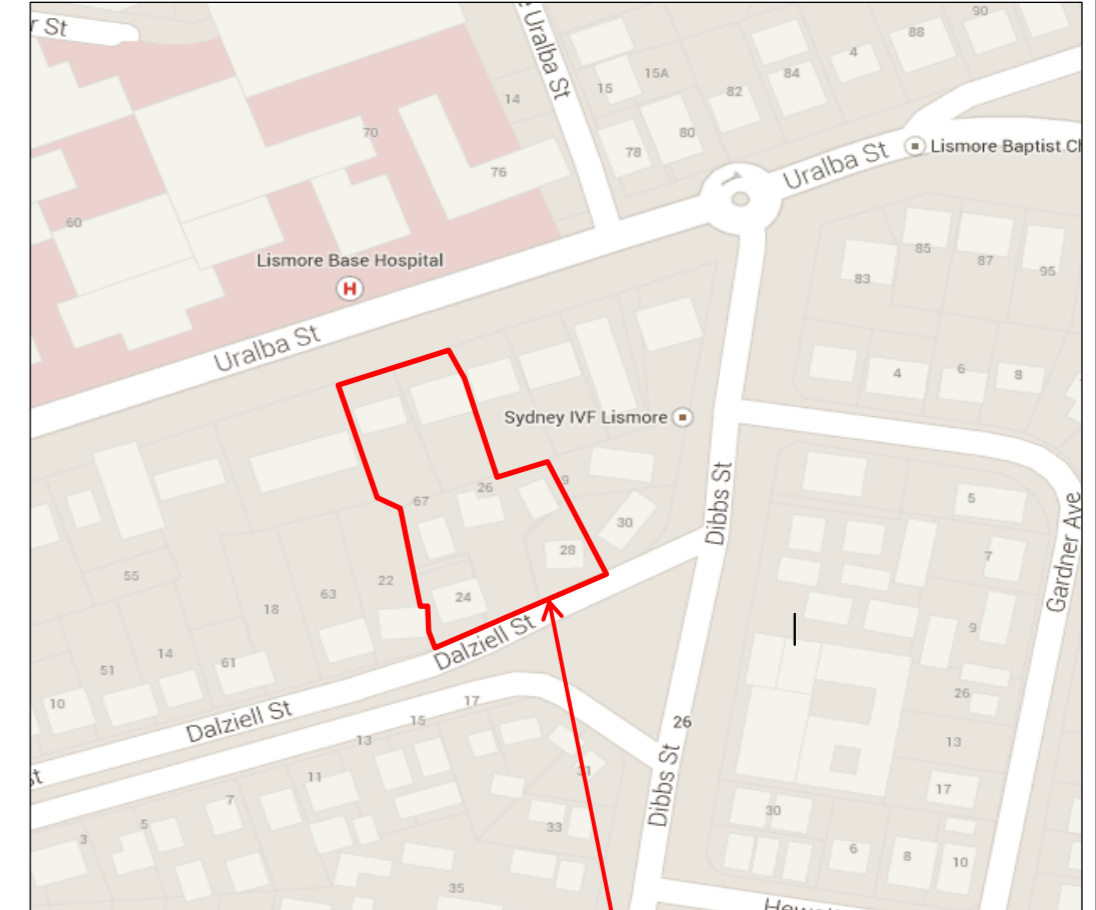
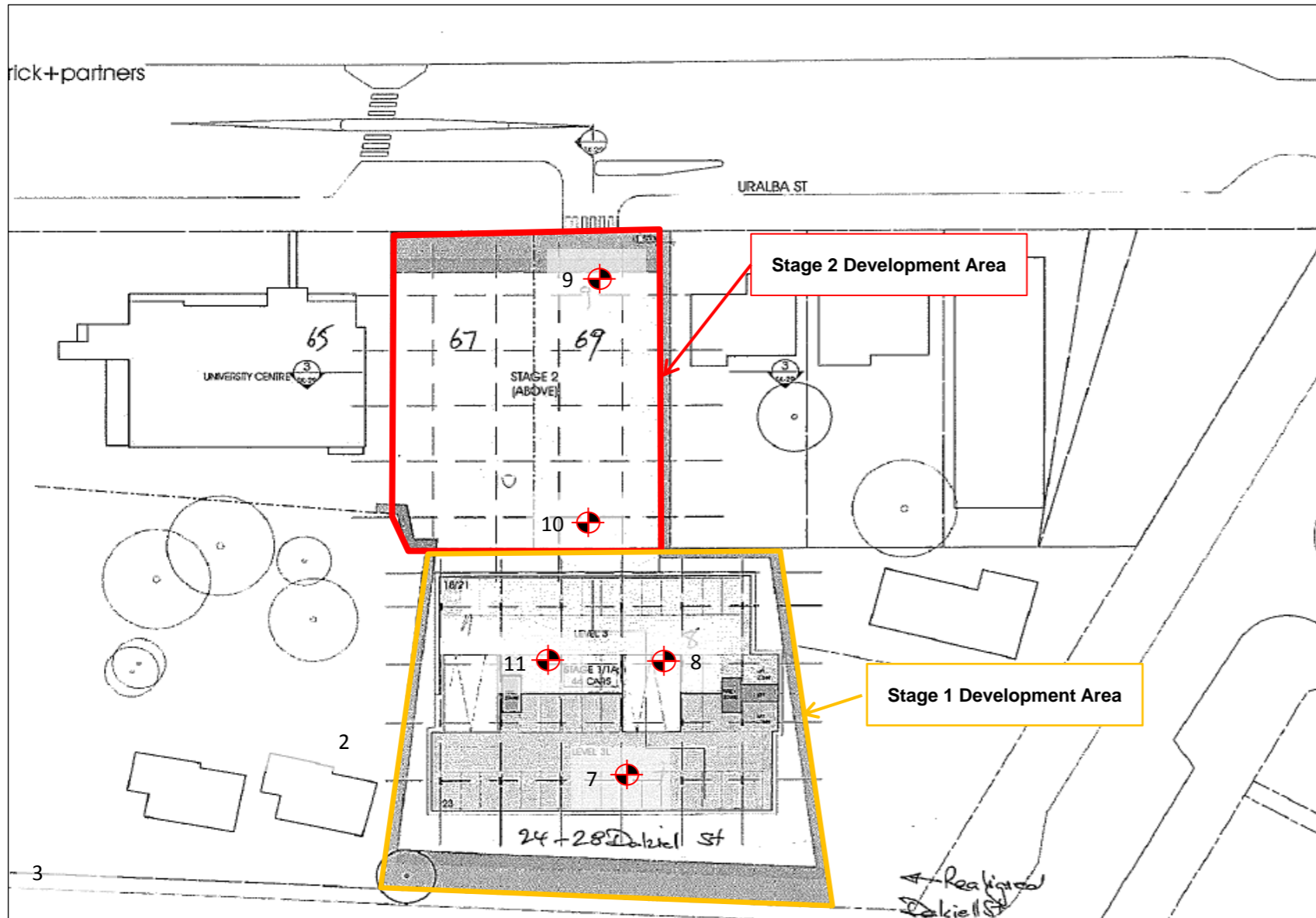
Stratification Spacing

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


Appendix B

Drawing 1 – Site and Test location Plan
Borehole Log Sheets



Approximate Site Location

- Notes:**
1. Test locations are approximate only and are shown with reference to existing and proposed site features.
 2. Adapted from drawing number SK-23 provided by John Holland Pty Ltd

 Approximate Bore Location



CLIENT: John Holland Pty Ltd	
OFFICE: Gold Coast	DRAWN BY: APS
SCALE: N.T.S	DATE: 10 Nov 2014

TITLE: Site and Test Location Plan Proposed Stage 3B Car Park 67 to 69 Uralba Street and 24 to 28 Dalziel Street, Lismore
--

PROJECT No:	80760.01
Drawing No:	1
REVISION:	0

BOREHOLE LOG

CLIENT: John Holland Pty Ltd
PROJECT: Proposed Stage 3B Car Park
LOCATION: 67-69 Uralba Street & 24-28 Dalziell Street, Lismore

SURFACE LEVEL: 23.00 AHD
EASTING: 528548
NORTHING: 6813007
DIP/AZIMUTH: 90°/-

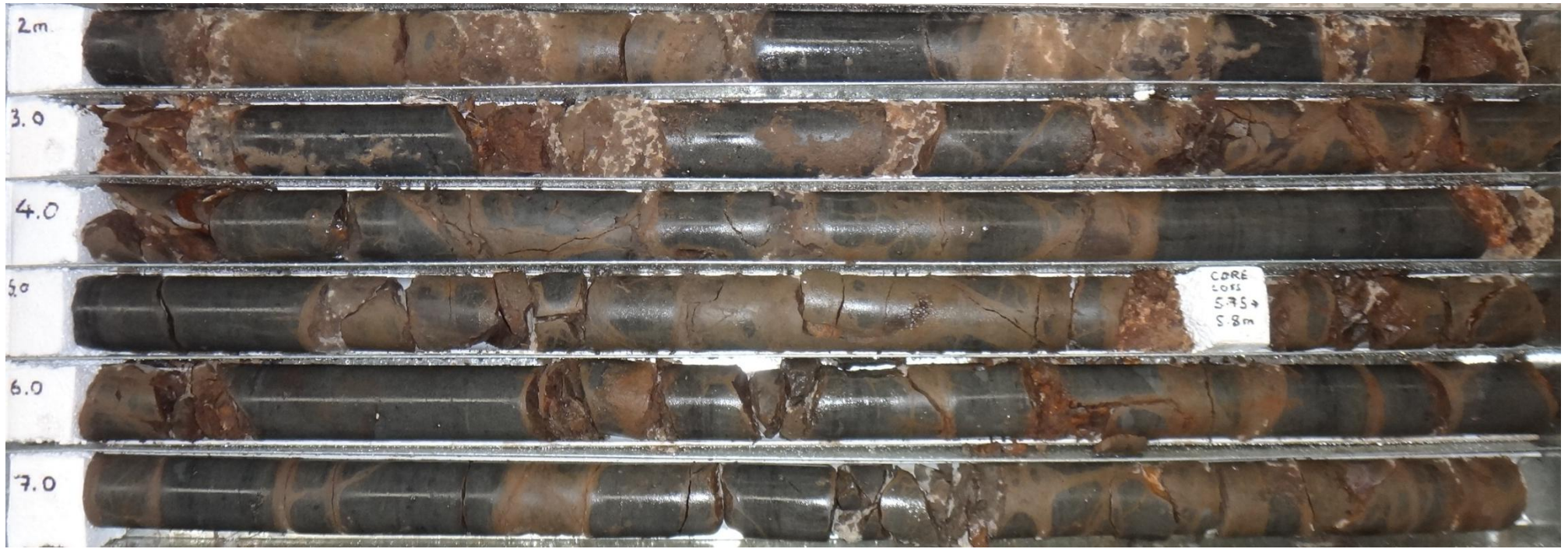
BORE No: 7
PROJECT No: 80760.01
DATE: 15 - 16/10/2014
SHEET 1 OF 5

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	B - Bedding	J - Joint
23	0.2	TOPSOIL FILLING - uncompacted brown and red brown sandy silt topsoil filling with some basalt cobbles, glass fragments and wood fragments SANDY CLAY - hard, red brown and orange brown and yellow brown sandy clay															A			
21	1.0	BASALT - extremely low strength, extremely weathered dark red brown and brown basalt (Upper Basalt Flow) - becoming very high strength, slightly weathered, grey basalt															S			30 for 140mm refusal
20	2	- extremely low strength, extremely weathered band between 2.25m to 2.30m - very low strength, highly weathered band between 2.42m to 2.45m																		
20	3	- very low strength band, highly weathered band between 2.84m to 2.87m - very low strength band, highly weathered band between 3.25m to 3.35m - extremely low strength, extremely weathered band 3.53m to 3.56m																		
														2.07m: J @ 0° clay coring - 5mm 2.21m: J @ 0° clean 2.34m: J @ 0° clean 2.56m: J @ 5° clay veneer 2.66m: highly fractured zone between 2.66m and 2.75m 2.9m: J @ 5° stained 3.18m: J @ 10° clay veneer 3.53m: J @ 20° extremely low strength zone 3.67m: highly fractured zone between 3.67m and 4.1m typically with extremely low strength zones up to 10mm thick	C	100	29	PL(D) = 4.4		
																	C	100	15	

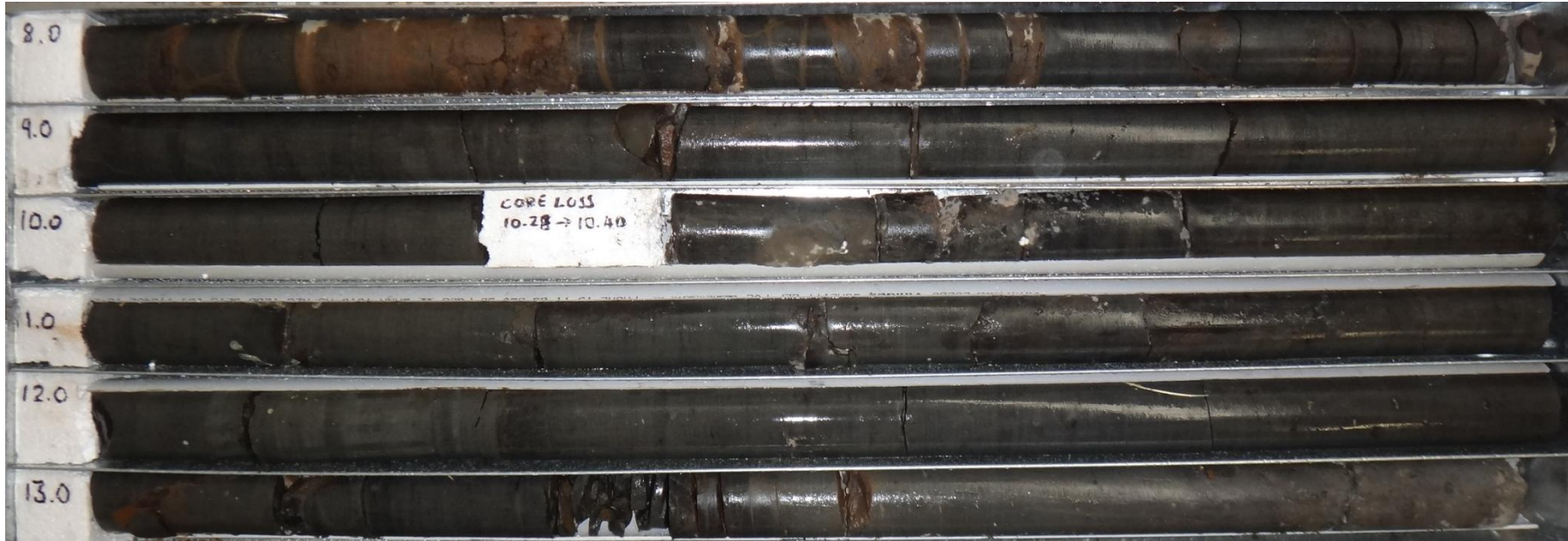
RIG: Track Mounted P120TB **DRILLER:** NCD **LOGGED:** AS **CASING:** 0.8m
TYPE OF BORING: Auger 0 - 1m, washboring 1m - 2m, NMLC coring 2m - 20m
WATER OBSERVATIONS:
REMARKS:

A Auger sample	G Gas sample	PLD Photo ionisation detector (ppm)
B Bulk sample	P Piston sample	PL(A) Point load axial test (50) (MPa)
BLK Block sample	U Tube sample (x mm dia.)	PL(D) Point load diametral test (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)





Bore 7 – 2.0 m to 8.0 m depth



Bore 7 – 8.0 m to 13.0 m depth



CLIENT: John Holland Pty Ltd

OFFICE: Gold Coast

DATE: November 2014

Core Photographs

**Proposed Lismore Base Hospital Stage 3B
Car Park**

Uralba Street and Dalziell Street, Lismore

PROJECT No: 80760.01

PLATE No: 2

REVISION: 0



Bore 7 – 14.0 m to 19.0 m depth



CLIENT: John Holland Pty Ltd

OFFICE: Gold Coast

DATE: November 2014

Core Photographs

Proposed Lismore Base Hospital Stage 3B Car Park

Uralba Street and Dalziell Street, Lismore

PROJECT No: 80760.01

PLATE No: 3

REVISION: 0

BOREHOLE LOG

CLIENT: John Holland Pty Ltd
PROJECT: Proposed Stage 3B Car Park
LOCATION: 67-69 Uralba Street & 24-28 Dalziell Street, Lismore

SURFACE LEVEL: 26.75 AHD
EASTING: 528543
NORTHING: 6813020
DIP/AZIMUTH: 90°/-

BORE No: 8
PROJECT No: 80760.01
DATE: 16/10/2014
SHEET 1 OF 2

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	FW	MW	SW		FS	FR	Ex. Low	Very Low	Low			Medium	High	Very High	Ex. High	B - Bedding		J - Joint	S - Shear	D - Drill Break	Type
	0.45	FILLING - uncompacted brown and red brown, clayey sand filling with some basalt cobbles, glass fragments and wood fragments																						
	0.45	SILTY CLAY - firm, orange brown and brown silty clay																						
1	1.0	BASALT - extremely low strength, extremely weathered dark red brown and brown basalt (Upper Basalt Flow)																						11,30 for 140mm refusal
2	2.0	- becoming very high strength, slightly weathered blue grey basalt. Fractured with fractures typically at 0°, 10° and 45°, and infilled with stiff clay. Micro fractures also present throughout and typically intersecting at 0°, 10°, 45°																						
	2.22	- very low strength, highly weathered band between 2.1m and 2.22m																						
	2.44	- very low strength band, highly weathered band between 2.44m and 2.6m																						
	2.68	- very low strength band, highly weathered band between 2.76m and 2.85m																						
3	3.0	- very low strength band, highly weathered band between 2.95m and 3.24m																						
	3.35	- extremely low strength, extremely weathered band between 3.3m and 3.35m																						
	3.35	- medium strength, highly weathered band between 3.35m and 3.8m. Typically recovered as gravel size fragments																						
	3.8																							
	4.0																							

RIG: Track Mounted P120TB DRILLER: NCD LOGGED: AS CASING: 0.8m

TYPE OF BORING: Auger 0 - 1m, washbore 1m - 2m, NMLC coring 2m - 8m

WATER OBSERVATIONS:

REMARKS:

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)	



Bore 8 – 2.0 m to 8.0 m depth



CLIENT: John Holland Pty Ltd

OFFICE: Gold Coast

DATE: November 2014

Core Photographs

Proposed Lismore Base Hospital Stage 3B Car Park

Uralba Street and Dalziell Street, Lismore

PROJECT No: 80760.01

PLATE No: 4

REVISION: 0

BOREHOLE LOG

CLIENT: John Holland Pty Ltd
PROJECT: Proposed Stage 3B Car Park
LOCATION: 67-69 Uralba Street & 24-28 Dalziell Street, Lismore

SURFACE LEVEL: 33.50 AHD
EASTING: 528511
NORTHING: 6813076
DIP/AZIMUTH: 90°/-

BORE No: 9
PROJECT No: 80760.01
DATE: 17/10/2014
SHEET 3 OF 3

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	PS		FR	Ex Low	Very Low	Low	Medium			High	Very High	Ex High	B - Bedding	J - Joint	S - Shear	D - Drill Break
	8.14	Bore discontinued at 8.14m, target depth reached																	C	100	100	
	25																					
	9																					
	24																					
	10																					
	23																					
	11																					
	22																					

RIG: Track Mounted P120TB **DRILLER:** NCD **LOGGED:** AS **CASING:** 0.8m

TYPE OF BORING: Auger 0 - 0.8m, washbore 0.8m - 1m, NMLC coring 1m - 8.14m

WATER OBSERVATIONS:

REMARKS:

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 _t 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)





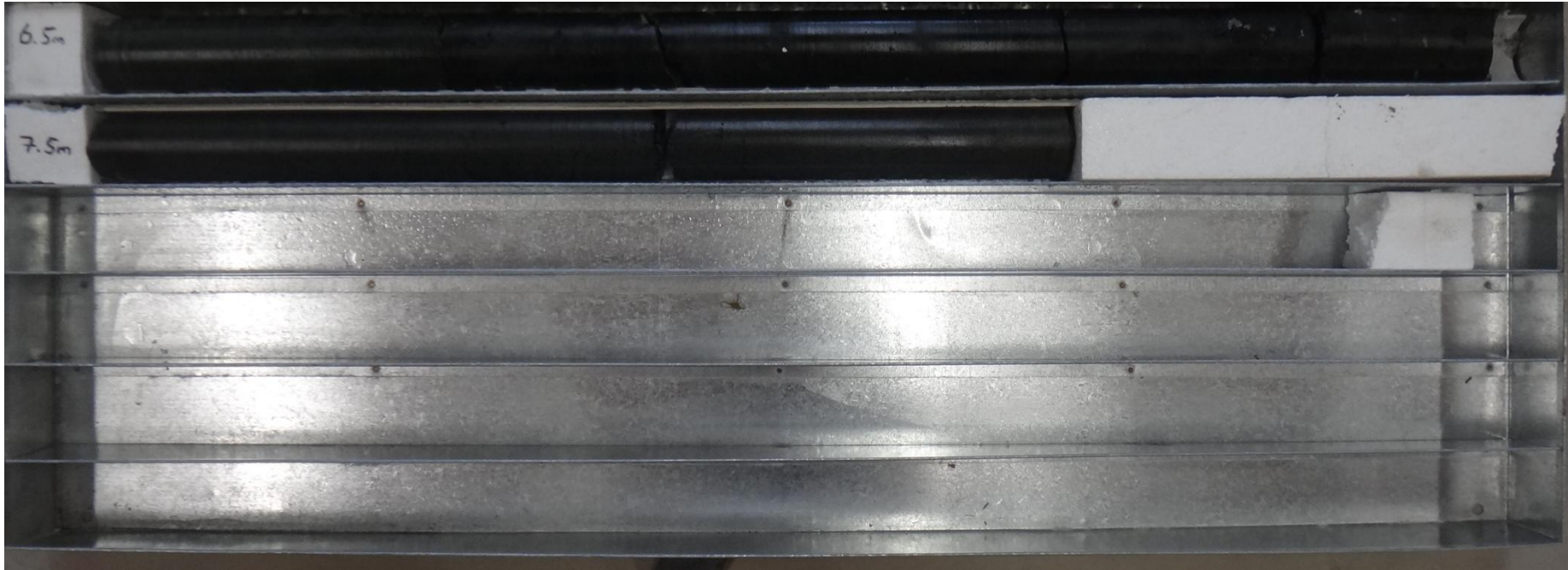
Bore 9 – 1.0 m to 6.5 m depth



CLIENT: John Holland Pty Ltd
 OFFICE: Gold Coast
 DATE: November 2014

Core Photographs
Proposed Lismore Base Hospital Stage 3B Car Park
Uralba Street and Dalziell Street, Lismore

PROJECT No: 80760.01
 PLATE No: 5
 REVISION: 0



Bore 9 – 6.5 m to 8.14 m depth

BOREHOLE LOG

CLIENT: John Holland Pty Ltd
PROJECT: Proposed Stage 3B Car Park
LOCATION: 67-69 Uralba Street & 24-28 Dalziell Street,
 Lismore

SURFACE LEVEL: 30.50 AHD
EASTING: 528526
NORTHING: 6813046
DIP/AZIMUTH: 90°/--

BORE No: 10
PROJECT No: 80760.01
DATE: 20 - 21/10/2014
SHEET 2 OF 3

RL	Depth (m)	Description of Strata	Degree of Weathering				Graphic Log	Rock Strength					Water	Fracture Spacing (m)	Discontinuities		Sampling & In Situ Testing					
			EW	HW	SW	FS		EX	Low	Medium	High	Very High			Ext High	B - Bedding	J - Joint	S - Shear	D - Drill Break	Type	Core Rec. %	RQD %
		BASALT - very high strength, slightly weathered dark grey basalt (continued)													3.98m: J @ 10° clay veneer 4m: J @ 30° clay veneer 4.1m: highly fractured zone between 4.1m and 4.46m							
25															4.59m: J @ 10° clay ven							
	5														4.76m: highly fractured zone between 4.76m and 4.82m 4.92m: dif	C	100	40.6				
															5.03m: dif							
															5.15m: J @ 5° stained							
25															5.52m: J @ 0° clean							
															5.81m: J @ 0° clean							
	6														6m: dif							
															6.12m: J @ 12° clean	C	100	100				
															6.38m: J @ 5° clean							
24															6.61m: J @ 0° stained							
															6.97m: J @ 20° dif							
	7														7.43m: J @ 0° clean	C	100	100				
															7.57m: dif							
23															7.84m: J @ 0° clean							

RIG: Track Mounted P120TB **DRILLER:** NCD **LOGGED:** AS **CASING:** 1.0m

TYPE OF BORING: Auger 0 - 0.8m, washbore 0.8m - 1m, NMLC coring 1m - 12.0m

WATER OBSERVATIONS:

REMARKS:

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)

BOREHOLE LOG

CLIENT: John Holland Pty Ltd
PROJECT: Proposed Stage 3B Car Park
LOCATION: 67-69 Uralba Street & 24-28 Dalziell Street,
 Lismore

SURFACE LEVEL: 30.50 AHD
EASTING: 528526
NORTHING: 6813046
DIP/AZIMUTH: 90°/--

BORE No: 10
PROJECT No: 80760.01
DATE: 20 - 21/10/2014
SHEET 3 OF 3

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	J - Joint	S - Shear	D - Drill Break	Type
		BASALT - very high strength, slightly weathered dark grey basalt (continued)																									PL(D) = 5.56
	22																										
	9																										
	21																										
	10																										
	20																										
	11																										PL(A) = 3.03
	19																										
	12.0																										

Bore discontinued at 12.0m, target

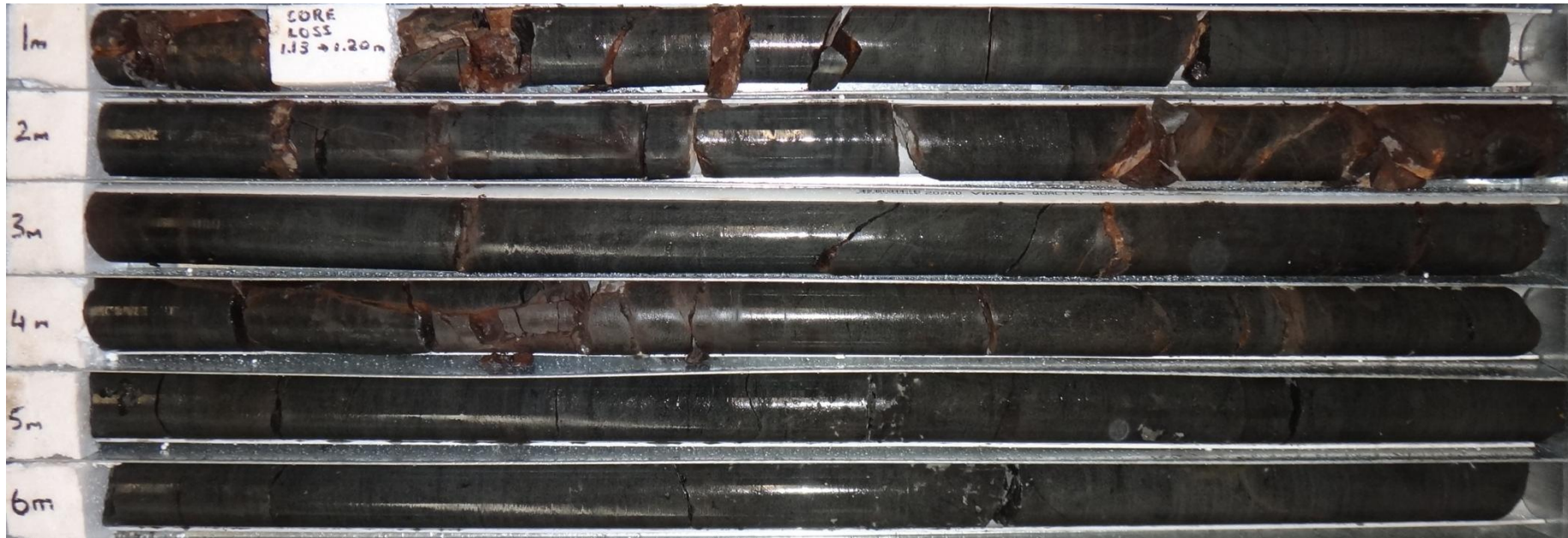
RIG: Track Mounted 20TB **DRILLER:** NCD **LOGGED:** AS **CASING:** 1.0m

TYPE OF BORING: Auger 0 - 0.8m, washbore 0.8m - 1m, NMLC coring 1m - 12.0m

WATER OBSERVATIONS:

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	ƒ	Water level
		PID	Photo ionisation detector (ppm)
		PL(A)	Point load axial test (50) (MPa)
		PL(D)	Point load diametral test (50) (MPa)
		pp	Pocket penetrometer (kPa)
		S	Standard penetration test
		V	Shear vane (kPa)



Bore 10 – 1.0 m to 7.0 m depth



CLIENT: John Holland Pty Ltd
 OFFICE: Gold Coast
 DATE: November 2014

Core Photographs
Proposed Lismore Base Hospital Stage 3B Car Park
Uralba Street and Dalziell Street, Lismore

PROJECT No: 80760.01
 PLATE No: 7
 REVISION: 0



Bore 10 – 7.0 m to 11.0 m depth

BOREHOLE LOG

CLIENT: John Holland Pty Ltd
PROJECT: Proposed Stage 3B Car Park
LOCATION: 67-69 Uralba Street & 24-28 Dalziell Street,
 Lismore

SURFACE LEVEL: 25.75 AHD
EASTING: 528523
NORTHING: 6813012
DIP/AZIMUTH: 90°/--

BORE No: 11
PROJECT No: 80760.01
DATE: 21/10/2014
SHEET 1 OF 3

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	B - Bedding	J - Joint	S - Shear	D - Drill Break	Type	Core Rec. %
	0.05	FILLING - uncompacted, brown and red brown, clayey sand filling with some basalt cobbles, glass fragments and wood fragments SANDY CLAY - stiff, orange brown, sandy clay with trace angular-sub-angular, fine and medium basalt gravel																		A				
	1.3	- becoming red brown and light grey with some basalt lithorelics (Residual Soil)																		S				2,6,9 N = 15
	2.8	BASALT - very low strength, highly weathered, dark brown, fragmented basalt.																		S				5,7,20/100mm
	3.0	- medium strength, moderately weathered band between 3.64m and 3.69m																		C	100	0		PL(A) = 0.48
	3.64																			C	100	0		

RIG: Track Mounted P120TB DRILLER: NCD LOGGED: AS CASING: 1.0m

TYPE OF BORING: Auger 0 - 1m, washbore 1.0m - 3.0m, NMLC coring 3.0m - 9.0m

WATER OBSERVATIONS:

REMARKS:

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)

BOREHOLE LOG

CLIENT: John Holland Pty Ltd
PROJECT: Proposed Stage 3B Car Park
LOCATION: 67-69 Uralba Street & 24-28 Dalziell Street,
 Lismore

SURFACE LEVEL: 25.75 AHD
EASTING: 528523
NORTHING: 6813012
DIP/AZIMUTH: 90°/-

BORE No: 11
PROJECT No: 80760.01
DATE: 21/10/2014
SHEET 3 OF 3

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	B - Bedding	J - Joint	S - Shear	D - Drill Break
		weathered band between 7.9m and 8.3m = becoming very high strength and slightly weathered below 8.3m - medium strength band, highly weathered band between 8.85m to 9.0m Bore discontinued at 9.0m, target depth reached													0.01 0.05 0.10 0.50 1.00	8.15m: J @ 0° clay infill 1mm 8.19m: J @ 0° clay infill 1mm 8.2m: J @ 0° clay infill 1mm 8.3m: J @ 0° clay ven 8.33m: J @ 0° clay ven 8.4m: J @ 0° clay ven 8.45m: J @ 0° clay ven 8.55m: J @ 0° clay ven 8.6m: J @ 0° clay ven 8.75m: J @ 0° clay ven 8.78m: highly fractured with clay infill up to 2mm 8.9m: highly fractured zone between 8.9m and 8.94m	C	100	0	PL(A) = 3.47		
																			C	100	0	PL(A) = 0.57

RIG: Track Mounted P120TB **DRILLER:** NCD **LOGGED:** AS **CASING:** 1.0m

TYPE OF BORING: Auger 0 - 1m, washbore 1.0m - 3.0m, NMLC coring 3.0m - 9.0m

WATER OBSERVATIONS:

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 (50) (MPa)
		PL(D)	Point load diametral test (50) (MPa)
		pp	Pocket penetrometer (kPa)
		S	Standard penetration test
		V	Shear vane (kPa)



Bore 11 – 3.0 m to 9.0 m depth



CLIENT: John Holland Pty Ltd

OFFICE: Gold Coast

DATE: November 2014

Core Photographs

Proposed Lismore Base Hospital Stage 3B Car Park

Uralba Street and Dalziell Street, Lismore

PROJECT No: 80760.01

PLATE No: 9

REVISION: 0

Appendix C

Results of Laboratory Testing

Determination of Emerson Class Number and pH Value of Soil

Client:	NSW Health Infrastructure	Project No:	80243
Project:	Lismore Base Hospital	Report No:	BO13-0723
Location:	60 Uralba Street, Lismore	Report Date:	17.06.2013
		Date Sampled:	28-30.05.2013
		Date of Test:	14.06.2013
		Page:	1 of 1

SAMPLE NO	DEPTH (m)	DESCRIPTION	WATER TYPE	WATER TEMP	CLASS NO.	PH VALUE
Bore 4	0.50	Filling- sandy clay with some gravel	De-ionised	22°C	4	7.61
Bore 5	2.10-2.20	Extremely low strength basalt	De-ionised	22°C	3	6.60

Test Method(s): AS 1289 3.8.1, AS 1289 4.3.1

Sampling Method(s): Sampled by DP Engineering

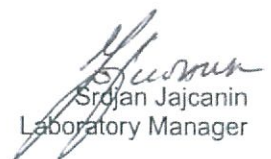
Remarks:



NATA Accredited Laboratory
Number: 828

The results of the tests, calibrations and/or measurements included in this document are traceable to Australian/national standards. Accredited for compliance with ISO/IEC 17025

Tested:	AC
Checked:	SJ


Srojan Jajcanin
Laboratory Manager

CERTIFICATE OF ANALYSIS

<p>Work Order : EB1314399</p> <p>Client : DOUGLAS PARTNERS PTY LTD</p> <p>Contact : MR TIM BARRITT</p> <p>Address : 439 MONTAGUE ROAD WEST END QLD, AUSTRALIA 4101</p> <p>E-mail : tim.barritt@douglaspartners.com.au</p> <p>Telephone : +61 07 32378900</p> <p>Facsimile : +61 07 32378999</p> <p>Project : 80243</p> <p>Order number : 103097</p> <p>C-O-C number : ----</p> <p>Sampler : Mark Hayes</p> <p>Site : 60 Uralba Street, Lismore</p> <p>Quote number : EN/020/13</p>	<p>Page : 1 of 3</p> <p>Laboratory : Environmental Division Brisbane</p> <p>Contact : Customer Services</p> <p>Address : 2 Byth Street Stafford QLD Australia 4053</p> <p>E-mail : Brisbane.Enviro.Services@alsglobal.com</p> <p>Telephone : +61 7 3243 7222</p> <p>Facsimile : +61 7 3243 7218</p> <p>QC Level : NEPM 1999 Schedule B(3) and ALS QCS3 requirement</p> <p>Date Samples Received : 13-JUN-2013</p> <p>Issue Date : 24-JUN-2013</p> <p>No. of samples received : 2</p> <p>No. of samples analysed : 2</p>
---	---

This report supersedes any previous report(s) with this reference. Results apply to the sample(s) as submitted. All pages of this report have been checked and approved for release.

This Certificate of Analysis contains the following information:

- General Comments
- Analytical Results



NATA Accredited Laboratory 825

Accredited for compliance with
ISO/IEC 17025.

Signatories

This document has been electronically signed by the authorized signatories indicated below. Electronic signing has been carried out in compliance with procedures specified in 21 CFR Part 11.

Signatories	Position	Accreditation Category
Kim McCabe	Senior Inorganic Chemist	Brisbane Inorganics
Kim McCabe	Senior Inorganic Chemist	Brisbane Inorganics
Kim McCabe	Senior Inorganic Chemist	Brisbane Inorganics
Kim McCabe	Senior Inorganic Chemist	Brisbane Inorganics



Page : 2 of 3
Work Order : EB1314399
Client : DOUGLAS PARTNERS PTY LTD
Project : 80243

General Comments

The analytical procedures used by the Environmental Division have been developed from established internationally recognized procedures such as those published by the USEPA, APHA, AS and NEPM. In house developed procedures are employed in the absence of documented standards or by client request.

Where moisture determination has been performed, results are reported on a dry weight basis.

Where a reported less than (<) result is higher than the LOR, this may be due to primary sample extract/digestate dilution and/or insufficient sample for analysis.

Where the LOR of a reported result differs from standard LOR, this may be due to high moisture content, insufficient sample (reduced weight employed) or matrix interference.

When sampling time information is not provided by the client, sampling dates are shown without a time component. In these instances, the time component has been assumed by the laboratory for processing purposes.

Key : CAS Number = CAS registry number from database maintained by Chemical Abstracts Services. The Chemical Abstracts Service is a division of the American Chemical Society.
LOR = Limit of reporting

^ = This result is computed from individual analyte detections at or above the level of reporting



Page : 3 of 3
 Work Order : EB1314399
 Client : DOUGLAS PARTNERS PTY LTD
 Project : 80243

Analytical Results

Sub-Matrix: SOIL (Matrix: SOIL)

Compound	CAS Number	LOR	Unit	Client sample ID	
				Bore 5	Bore 6
		Client sampling date / time			
EA002 : pH (Soils)		0.1	pH Unit	28-MAY-2013 15:00 EB1314399-001	14.1m - 14.2m 30-MAY-2013 15:00 EB1314399-002
EA055: Moisture Content		1.0	%	8.8	7.4
ED040S : Soluble Sulfate by ICPAES	14808-79-8	10	mg/kg	12.3	19.2
ED045G: Chloride Discrete analyser	16887-00-6	10	mg/kg	<10	<10

QUALITY CONTROL REPORT

Work Order	: EB1314399	Page	: 1 of 4
Client	: DOUGLAS PARTNERS PTY LTD	Laboratory	: Environmental Division Brisbane
Contact	: MR TIM BARRITT	Contact	: Customer Services
Address	: 439 MONTAGUE ROAD WEST END QLD, AUSTRALIA 4101	Address	: 2 Byth Street Stafford QLD Australia 4053
E-mail	: tim.barritt@douglaspartners.com.au	E-mail	: Brisbane.Enviro.Services@alsglobal.com
Telephone	: +61 07 32378900	Telephone	: +61 7 3243 7222
Facsimile	: +61 07 32378999	Facsimile	: +61 7 3243 7218
Project	: 80243	QC Level	: NEPM 1999 Schedule B(3) and ALS QCS3 requirement
Site	: 60 Uralba Street, Lismore	Date Samples Received	: 13-JUN-2013
C-O-C number	: ---	Issue Date	: 24-JUN-2013
Sampler	: Mark Hayes	No. of samples received	: 2
Order number	: 103097	No. of samples analysed	: 2
Quote number	: EN/020/13		

This report supersedes any previous report(s) with this reference. Results apply to the sample(s) as submitted. All pages of this report have been checked and approved for release.

This Quality Control Report contains the following information:

- Laboratory Duplicate (DUP) Report; Relative Percentage Difference (RPD) and Acceptance Limits
- Method Blank (MB) and Laboratory Control Spike (LCS) Report; Recovery and Acceptance Limits
- Matrix Spike (MS) Report; Recovery and Acceptance Limits



WORLD RECOGNISED
ACCREDITATION

NATA Accredited Laboratory 825

Accredited for compliance with
ISO/IEC 17025.

Signatories

This document has been electronically signed by the authorized signatories indicated below. Electronic signing has been carried out in compliance with procedures specified in 21 CFR Part 11.

Signatories	Position	Accreditation Category
Kim McCabe	Senior Inorganic Chemist	Brisbane Inorganics
Kim McCabe	Senior Inorganic Chemist	Brisbane Inorganics
Kim McCabe	Senior Inorganic Chemist	Brisbane Inorganics
Kim McCabe	Senior Inorganic Chemist	Brisbane Inorganics



Page : 2 of 4
Work Order : EB1314399
Client : DOUGLAS PARTNERS PTY LTD
Project : 80243

General Comments

The analytical procedures used by the Environmental Division have been developed from established internationally recognized procedures such as those published by the USEPA, APHA, AS and NEPM. In house developed procedures are employed in the absence of documented standards or by client request.

Where moisture determination has been performed, results are reported on a dry weight basis.

Where a reported less than (<) result is higher than the LOR, this may be due to primary sample extract/digestate dilution and/or insufficient sample for analysis.

Where the LOR of a reported result differs from standard LOR, this may be due to high moisture content, insufficient sample (reduced weight employed) or matrix interference.

Key :
Anonymous = Refers to samples which are not specifically part of this work order but formed part of the QC process lot
CAS Number = CAS registry number from database maintained by Chemical Abstracts Services. The Chemical Abstracts Service is a division of the American Chemical Society.
LOR = Limit of reporting
RPD = Relative Percentage Difference
= Indicates failed QC



Page : 3 of 4
 Work Order : EB1314399
 Client : DOUGLAS PARTNERS PTY LTD
 Project : 80243

Laboratory Duplicate (DUP) Report

The quality control term Laboratory Duplicate refers to a randomly selected intralaboratory split. Laboratory duplicates provide information regarding method precision and sample heterogeneity. The permitted ranges for the Relative Percent Deviation (RPD) of Laboratory Duplicates are specified in ALS Method QWI-EN/38 and are dependent on the magnitude of results in comparison to the level of reporting. Result < 10 times LOR, No Limit; Result between 10 and 20 times LOR: 0% - 50%; Result > 20 times LOR: 0% - 20%.

Sub-Matrix: SOIL

Laboratory sample ID	Client sample ID	Method: Compound	CAS Number	LOR	Unit	Laboratory Duplicate (DUP) Report			Recovery Limits (%)
						Original Result	Duplicate Result	RPD (%)	
EA002 : pH (Solis) (QC Lot: 2920233)									
EB1314399-001	Bore 5 2m - 2.15m	EA002: pH Value	---	0.1	pH Unit	8.8	8.8	0.0	0% - 20%
EA055: Moisture Content (QC Lot: 2920257)									
EB1314355-020	Anonymous	EA055-103: Moisture Content (dried @ 103°C)	---	1.0	%	4.4	4.4	0.0	No Limit
EB1314355-046	Anonymous	EA055-103: Moisture Content (dried @ 103°C)	---	1.0	%	3.6	3.5	0.0	No Limit
ED040S: Soluble Major Anions (QC Lot: 2920234)									
EB1314399-001	Bore 5 2m - 2.15m	ED040S: Sulfate as SO4 2-	14808-79-8	10	mg/kg	<10	<10	0.0	No Limit
ED045G: Chloride by Discrete Analyser (QC Lot: 2920235)									
EB1314399-001	Bore 5 2m - 2.15m	ED045G: Chloride	16887-00-6	10	mg/kg	<10	<10	0.0	0% - 20%



Page : 4 of 4
 Work Order : EB1314399
 Client : DOUGLAS PARTNERS PTY LTD
 Project : 80243

Method Blank (MB) and Laboratory Control Spike (LCS) Report

The quality control term Method / Laboratory Blank refers to an analyte free matrix to which all reagents are added in the same volumes or proportions as used in standard sample preparation. The purpose of this QC parameter is to monitor potential laboratory contamination. The quality control term Laboratory Control Sample (LCS) refers to a certified reference material, or a known interference free matrix spiked with target analytes. The purpose of this QC parameter is to monitor method precision and accuracy independent of sample matrix. Dynamic Recovery Limits are based on statistical evaluation of processed LCS.

Sub-Matrix: SOIL

Method/Compound	CAS Number	LOR	Unit	Method Blank (MB) Report		Laboratory Control Spike (LCS) Report			
				Result	Concentration	Spike Recovery (%)	LCS	Low	High
EA002 : pH (Soils) (QCLot: 2920233)	---	0.1	pH Unit	---	4.0 pH Unit	100	100	98	102
ED040S: Soluble Major Anions (QCLot: 2920234)	14808-79-8	10	mg/kg	<10	476 mg/kg	103	103	86	118
ED045G: Chloride by Discrete Analyser (QCLot: 2920235)	16887-00-6	10	mg/kg	<10	5000 mg/kg	92.8	92.8	77	115

Matrix Spike (MS) Report

The quality control term Matrix Spike (MS) refers to an intralaboratory split sample spiked with a representative set of target analytes. The purpose of this QC parameter is to monitor potential matrix effects on analyte recoveries. Static Recovery Limits as per laboratory Data Quality Objectives (DQOs). Ideal recovery ranges stated may be waived in the event of sample matrix interference.

- No Matrix Spike (MS) Results are required to be reported.

Matrix Spike (MS) and Matrix Spike Duplicate (MSD) Report

The quality control term Matrix Spike (MS) and Matrix Spike Duplicate (MSD) refers to intralaboratory split samples spiked with a representative set of target analytes. The purpose of these QC parameters are to monitor potential matrix effects on analyte recoveries. Static Recovery Limits as per laboratory Data Quality Objectives (DQOs). Ideal recovery ranges stated may be waived in the event of sample matrix interference.

- No Matrix Spike (MS) or Matrix Spike Duplicate (MSD) Results are required to be reported.

INTERPRETIVE QUALITY CONTROL REPORT

Work Order	: EB1314399	Page	: 1 of 6
Client	: DOUGLAS PARTNERS PTY LTD	Laboratory	: Environmental Division Brisbane
Contact	: MR TIM BARRITT	Contact	: Customer Services
Address	: 439 MONTAGUE ROAD WEST END QLD, AUSTRALIA 4101	Address	: 2 Byth Street Stafford QLD Australia 4053
E-mail	: tim.barritt@douglaspartners.com.au	E-mail	: Brisbane.Enviro.Services@alsglobal.com
Telephone	: +61 07 32378900	Telephone	: +61 7 3243 7222
Facsimile	: +61 07 32378999	Facsimile	: +61 7 3243 7218
Project	: 80243	QC Level	: NEPM 1999 Schedule B(3) and ALS QCS3 requirement
Site	: 60 Uralba Street, Lismore	Date Samples Received	: 13-JUN-2013
C-O-C number	: ----	Issue Date	: 24-JUN-2013
Sampler	: Mark Hayes	No. of samples received	: 2
Order number	: 103097	No. of samples analysed	: 2
Quote number	: EN/020/13		

This report supersedes any previous report(s) with this reference. Results apply to the sample(s) as submitted. All pages of this report have been checked and approved for release.

This Interpretive Quality Control Report contains the following information:

- Analysis Holding Time Compliance
- Quality Control Parameter Frequency Compliance
- Brief Method Summaries
- Summary of Outliers



Page : 2 of 6
 Work Order : EB1314399
 Client : DOUGLAS PARTNERS PTY LTD
 Project : 80243

Analysis Holding Time Compliance

The following report summarises extraction / preparation and analysis times and compares with recommended holding times. Dates reported represent first date of extraction or analysis and precludes subsequent dilutions and reruns. Information is also provided re the sample container (preservative) from which the analysis aliquot was taken. Elapsed period to analysis represents number of days from sampling where no extraction / digestion is involved or period from extraction / digestion where this is present. For composite samples, sampling date is assumed to be that of the oldest sample contributing to the composite. Sample date for laboratory produced leachates is assumed as the completion date of the leaching process. Outliers for holding time are based on USEPA SW 846, APHA, AS and NEPM (1999). A listing of breaches is provided in the Summary of Outliers.

Holding times for leachate methods (excluding elutriates) vary according to the analytes being determined on the resulting solution. For non-volatile analytes, the holding time compliance assessment compares the leach date with the shortest analyte holding time for the equivalent soil method. These soil holding times are: Organics (14 days); Mercury (28 days) & other metals (180 days). A recorded breach therefore does not guarantee a breach for all non-volatile parameters.

Matrix: SOIL

Evaluation: * = Holding time breach ; ✓ = Within holding time.

Method Container / Client Sample ID(s)	Sample Date		Extraction / Preparation		Analysis	
	Date extracted	Due for extraction	Date analysed	Due for analysis	Evaluation	Evaluation
EA002 : pH (Soils)						
Snap Lock Bag (EA002) Bore 5 2m - 2.15m	19-JUN-2013	04-JUN-2013	20-JUN-2013	19-JUN-2013	*	*
Snap Lock Bag (EA002) Bore 6 14.1m - 14.2m	19-JUN-2013	06-JUN-2013	20-JUN-2013	19-JUN-2013	*	*
EA055: Moisture Content						
Snap Lock Bag (EA055-103) Bore 5 2m - 2.15m	---	---	17-JUN-2013	11-JUN-2013	---	*
Snap Lock Bag (EA055-103) Bore 6 14.1m - 14.2m	---	---	17-JUN-2013	13-JUN-2013	---	*
ED040S : Soluble Sulfate by ICPAES						
Snap Lock Bag (ED040S) Bore 5 2m - 2.15m	19-JUN-2013	04-JUN-2013	20-JUN-2013	17-JUL-2013	*	✓
Snap Lock Bag (ED040S) Bore 6 14.1m - 14.2m	19-JUN-2013	06-JUN-2013	20-JUN-2013	17-JUL-2013	*	✓
EDD45G: Chloride Discrete analyser						
Snap Lock Bag (EDD45G) Bore 5 2m - 2.15m	19-JUN-2013	04-JUN-2013	20-JUN-2013	17-JUL-2013	*	✓
Snap Lock Bag (EDD45G) Bore 6 14.1m - 14.2m	19-JUN-2013	06-JUN-2013	20-JUN-2013	17-JUL-2013	*	✓



Quality Control Parameter Frequency Compliance

The following report summarises the frequency of laboratory QC samples analysed within the analytical lot(s) in which the submitted sample(s) was(were) processed. Actual rate should be greater than or equal to the expected rate. A listing of breaches is provided in the Summary of Outliers.

Matrix: SOIL

Quality Control Sample Type Analytical Methods	Method	Count		Rate (%)		Evaluation	Quality Control Specification
		QC	Regular	Actual	Expected		
Laboratory Duplicates (DUP)							
Chloride Soluble By Discrete Analyser	ED045G	1	2	50.0	10.0	✓	NEPM 1999 Schedule B(3) and ALS QCS3 requirement
Major Anions - Soluble	ED040S	1	2	50.0	10.0	✓	NEPM 1999 Schedule B(3) and ALS QCS3 requirement
Moisture Content	EA055-103	2	20	10.0	10.0	✓	NEPM 1999 Schedule B(3) and ALS QCS3 requirement
pH (1:5)	EA002	1	2	50.0	10.0	✓	NEPM 1999 Schedule B(3) and ALS QCS3 requirement
Laboratory Control Samples (LCS)							
Chloride Soluble By Discrete Analyser	ED045G	2	2	100.0	10.0	✓	NEPM 1999 Schedule B(3) and ALS QCS3 requirement
Major Anions - Soluble	ED040S	1	2	50.0	5.0	✓	NEPM 1999 Schedule B(3) and ALS QCS3 requirement
pH (1:5)	EA002	1	2	50.0	5.0	✓	NEPM 1999 Schedule B(3) and ALS QCS3 requirement
Method Blanks (MB)							
Chloride Soluble By Discrete Analyser	ED045G	1	2	50.0	5.0	✓	NEPM 1999 Schedule B(3) and ALS QCS3 requirement
Major Anions - Soluble	ED040S	1	2	50.0	5.0	✓	NEPM 1999 Schedule B(3) and ALS QCS3 requirement

Evaluation: * = Quality Control frequency not within specification ; ✓ = Quality Control frequency within specification



Page : 4 of 6
Work Order : EB1314399
Client : DOUGLAS PARTNERS PTY LTD
Project : 80243

Brief Method Summaries

The analytical procedures used by the Environmental Division have been developed from established internationally recognized procedures such as those published by the US EPA, APHA, AS and NEPM. In house developed procedures are employed in the absence of documented standards or by client request. The following report provides brief descriptions of the analytical procedures employed for results reported in the Certificate of Analysis. Sources from which ALS methods have been developed are provided within the Method Descriptions.

Analytical Methods	Method	Matrix	Method Descriptions
pH (1:5)	EA002	SOIL	(APHA 21st ed., 4500H+) pH is determined on soil samples after a 1:5 soil/water leach. This method is compliant with NEPM (1999) Schedule B(3) (Method 103)
Moisture Content	EA055-103	SOIL	A gravimetric procedure based on weight loss over a 12 hour drying period at 103-105 degrees C. This method is compliant with NEPM (2010 Draft) Schedule B(3) Section 7.1 and Table 1 (14 day holding time).
Major Anions - Soluble	ED040S	SOIL	In-house. Soluble Anions are determined off a 1:5 soil / water extract by ICPAES.
Chloride Soluble By Discrete Analyser	ED045G	SOIL	APHA 21st edition 4500-CI- E. The thiocyanate ion is liberated from mercuric thiocyanate through sequestration of mercury by the chloride ion to form non-ionised mercuric chloride.in the presence of ferric ions the liberated thiocyanate forms highly-coloured ferric thiocyanate which is measured at 480 nm. Analysis is performed on a 1:5 soil / water leachate.



Summary of Outliers

Outliers : Quality Control Samples

The following report highlights outliers flagged in the Quality Control (QC) Report. Surrogate recovery limits are static and based on USEPA SW846 or ALS-QWIEN/38 (in the absence of specific USEPA limits). This report displays QC Outliers (breaches) only.

Duplicates, Method Blanks, Laboratory Control Samples and Matrix Spikes

- For all matrices, no Method Blank value outliers occur.
- For all matrices, no Duplicate outliers occur.
- For all matrices, no Laboratory Control outliers occur.
- For all matrices, no Matrix Spike outliers occur.

Regular Sample Surrogates

- For all regular sample matrices, no surrogate recovery outliers occur.

Outliers : Analysis Holding Time Compliance

This report displays Holding Time breaches only. Only the respective Extraction / Preparation and/or Analysis component is/are displayed.

Matrix: SOIL

Method Container / Client Sample ID(s)	Extraction / Preparation		Analysis	
	Date extracted	Due for extraction Days overdue	Date analysed	Due for analysis Days overdue
EA002 : pH (Soils)				
Snap Lock Bag				
Bore 5 2m - 2.15m	19-JUN-2013	04-JUN-2013 15	20-JUN-2013	19-JUN-2013 1
Snap Lock Bag				
Bore 6 14.1m - 14.2m	19-JUN-2013	06-JUN-2013 13	20-JUN-2013	19-JUN-2013 1
EA055: Moisture Content				
Snap Lock Bag				
Bore 5 2m - 2.15m	---	---	17-JUN-2013	11-JUN-2013 6
Snap Lock Bag				
Bore 6 14.1m - 14.2m	---	---	17-JUN-2013	13-JUN-2013 4
ED040S : Soluble Sulfate by ICPAES				
Snap Lock Bag				
Bore 5 2m - 2.15m	19-JUN-2013	04-JUN-2013 15	---	---
Snap Lock Bag				
Bore 6 14.1m - 14.2m	19-JUN-2013	06-JUN-2013 13	---	---
ED045G: Chloride Discrete analyser				
Snap Lock Bag				
Bore 5 2m - 2.15m	19-JUN-2013	04-JUN-2013 15	---	---
Snap Lock Bag				
Bore 6 14.1m - 14.2m	19-JUN-2013	06-JUN-2013 13	---	---



Outliers : Frequency of Quality Control Samples

The following report highlights breaches in the Frequency of Quality Control Samples.

- **No Quality Control Sample Frequency Outliers exist.**