Douglas Partners
Geotechnics / Environment / Groundwater

Report on
Geotechnical Investigation

Carpark P6D (Site 2)
Cnr Australia Ave and Parkview Dr
Sydney Olympic Park

Prepared for
Ecove Group Pty Ltd

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Integrated Practical Solutions

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Report on Geotechnical Investigation
Carpark P6D (Site 2)
Cnr Australia Ave and Parkview Dr, Sydney Olympic Park

1. Introduction

This report presents the results of a geotechnical investigation undertaken by Douglas Partners Pty Ltd (DP) for a proposed commercial development at Car Park P6D, Sydney Olympic Park Parkview Precinct. The study was commissioned by Greg Hynd of Ecove Group Pty Ltd.

It is understood that the proposed development of the site will include the construction of two 30-storey mixed use towers with a four or five level common basement carpark. It is further understood that the proposed basement will extend to the existing property boundaries, except for the north-west and south-west corners of the site where a large tree is to remain and the T7 Olympic Park Line railway tunnels run beneath.

The aim of the investigation was to assess the subsurface conditions across the site in order to provide advice on:

- subsurface conditions and groundwater;
- excavation characteristics;
- suitable shoring options and retaining structures;
- suitable foundation systems and design parameters; and
- geotechnical considerations relating to the rail corridors.

The details of the field work undertaken are presented in this report, together with comments and recommendations on design and construction practice.

2. Site Description

The approximately 7,000 m^2 rectangular site is currently occupied by a single level asphaltic concrete carpark. The site encapsulates Sites 2 at the Sydney Olympic Park Parkview Precinct and is bound to the north, south and east by Murray Rose Avenue, Parkview Drive and Australia Avenue, respectively, and to the east by a commercial lot. The site slopes down gently to the north-east (refer to Drawings in Appendix E).

The south-west corner of the site is constrained by a rail easement from the T7 Olympic Park Line railway tunnel which passes beneath the south-west corner of the site, see Figure 1 and Drawing 1. The tunnels are understood to have been constructed using cut and cover methods, with the width of the tunnel increasing as it approaches the Olympic Park Station to the west. The tunnel crown and invert is understood to be at approximately RL 8 m and RL 15.5 m relative to Australian Height Datum (AHD).

3. Regional Geology

Reference to the Sydney 1:100,000 Geological Series Sheet indicates that the site is underlain by Ashfield Shale of the Triassic aged Wianamatta Group, which typically comprises black to dark grey shale and laminite. The Ashfield Shale is typically closely bedded and contains an orthogonal pair of steeply dipping (70° to 90°) joint sets typically striking NNE and ESE and spaced at 0.5 m to 5 m. Randomly oriented, 30° to 45° dipping slickensided joints are also ubiquitous.

The Homebush Bay Fault Zone, with a north-northeast trend, is mapped to the northeast of the site (see Figure 1). The fault zone contains sheared zones with closely spaced, steeply dipping joints and associated to thrust faults (Och et al, 2009).

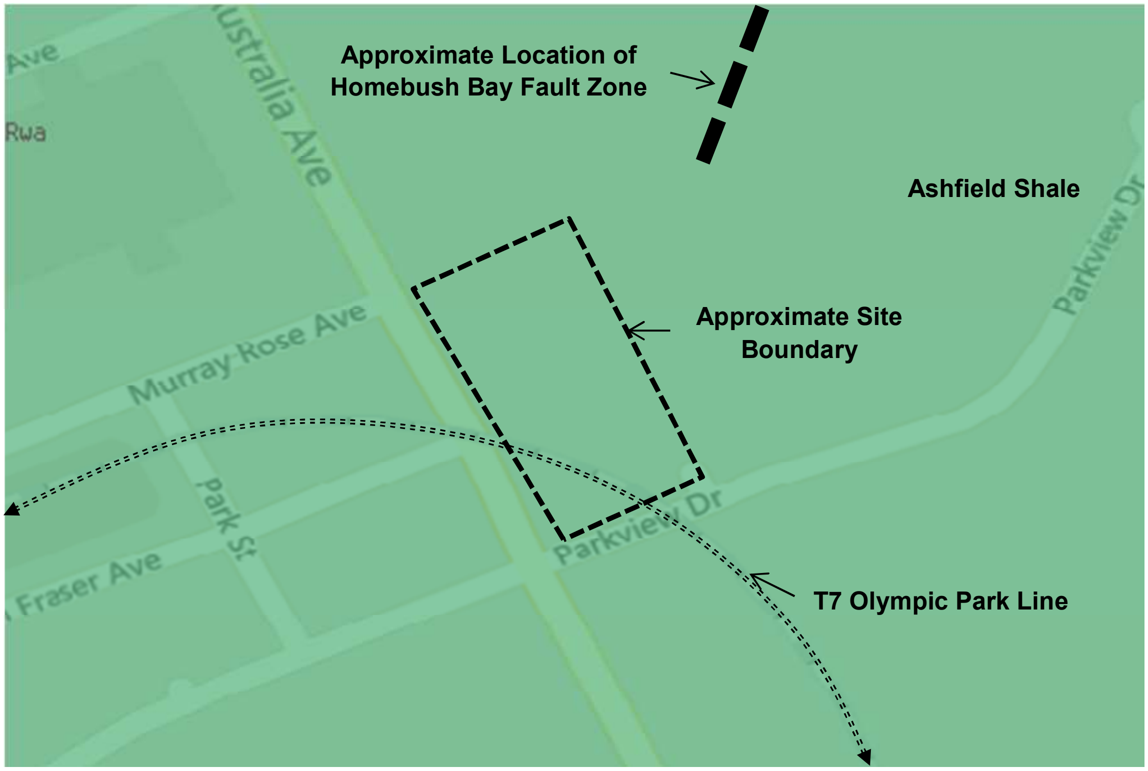


Figure 1: Extract from Geological Series with Homebush Bay Fault annotated

4. Field Work Methods

The field work for the investigation included six cored boreholes (BH1 to BH6, inclusive). The boreholes were all drilled using a truck-mounted drilling rig.

The boreholes were initially drilled to depths of 2.5 m to 5.5 m using spiral flight augers and rotary drilling techniques within the soil and extremely low to very low strength rock. Standard Penetration Tests (SPTs) were carried out at regular intervals to sample the soil and weathered rock and to assess the in situ strength of the materials. The boreholes were then cased and continued into the underlying rock using diamond core drilling techniques to obtain continuous core samples of the bedrock down to a maximum depth of 20.85 m.

The rock cores recovered from the boreholes were returned to the DP Workshop where they were logged by a geologist/geotechnical engineer, the cores photographed and Point Load Strength Index ($Is_{(50)}$) tests carried out on selected samples of the rock core at regular intervals.

Three standpipe piezometers were installed in BH2, BH3 and BH6 to allow measurements of groundwater levels.

The borehole locations are shown on Drawing 1 in Appendix B and geotechnical logs and core photographs are provided in Appendix C. Borehole coordinates and ground surface levels were surveyed and provided by LTS Lockley.

5. Field Work Results

5.1 Boreholes

Details of the subsurface conditions encountered are given in the borehole logs included in Appendix C, with notes defining classification methods and descriptive terms. Photographs of the rock cores were taken and are presented with the relevant borehole logs.

The general sequence of materials encountered in the boreholes is summarised below:

Pavements:	Typically asphalt.
Filling:	Variably compacted silty sand, clayey sand and sandy gravel filling.
Residual Soil:	Clay, grey-brown, with occasional ironstone bands.
Weathered Bedrock:	Extremely low to low strength laminite and shale, with some ironstone bands.
Bedrock:	Medium and high strength, laminite and shale

No free groundwater or seepage was encountered in the boreholes during augering. The use of water as a drilling fluid precluded any observations of groundwater while coring the underlying rock.

Deeper filling overlying weathered bedrock was encountered in BH5 and BH6 in the southern portion of the site in near the cut and cover rail tunnel.

Discontinuities observed in the core included bedding panes, joints, and the occasional sheared zone. Bedding planes dipped between 0 and 10° and joints between 45° and 90°, which is typical for Ashfield Shale in Sydney. Sheared zones were also identified, in particular in BH6 between RL 2.5 m and RL -2.5 m.

Fracture spacing is typically described as slightly fractured to unbroken. Rock Quality Designation (RQD) of the medium strength or stronger rock ranged from 88 to 100 %.

5.1.1 Groundwater Readings

Three standpipe piezometers were installed in boreholes BH2, BH3 and BH6 to allow measurement of groundwater levels. The water level readings taken from each piezometer are shown in Table 1.

Table 1: Groundwater Level Readings

Date	Borehole BH2		Borehole BH3		Borehole BH6	
	Depth (m)	RL (m, AHD)	Depth (m)	RL (m, AHD)	Depth (m)	RL (m, AHD)
*11 Sept 2018	10.58	2.92	4.83	9.77	10.45	4.45
19 Sept 2018	10.64	2.86	10.48	4.12	10.50	4.40

* measured prior to purging (removing drilling water)

6. Laboratory Testing

Representative soil and rock samples were selected for laboratory testing. The soil testing report sheets are given in Appendix D and results from rock testing shown on the respective borehole logs in Appendix C.

6.1 Soil Testing

Four samples were selected for Atterberg Limit and Linear Shrinkage testing and four samples were selected to determine the aggressivity of the soil to buried structural elements.

6.1.1 Atterberg Limit Tests and Linear Shrinkage

Testing was carried out to determine the plasticity index and linear shrinkage of the residual clay. The results are summarised in Table 2.

Table 2: Summary of Atterberg Limit Test and Linear Shrinkage Results

Test Bore	Depth (m)	W_L (%)	W_P (%)	PI (%)	Linear Shrinkage (%)	Description
BH1	1.0 – 1.45	57	29	28	18	Silty Clay
BH2	1.0 – 1.45	57	30	27	17.5	Silty Clay
BH3	1.0 – 1.45	86	37	49	19.5	Silty Clay
BH4	1.0 – 1.45	84	32	52	20.5	Silty Clay

Where: W_L – liquid limit W_P – plastic limit PI – plasticity index

6.1.2 Aggressivity

Four samples were tested for pH, sulphate (SO_4), chloride (Cl) and electrical conductivity (EC). The results are summarised in Table 3.

Table 3: Summary of Aggressivity Test Results

Test Bore	Depth (m)	Material	pH	Cl (ppm)	SO_4 (ppm)	EC (µS/cm)	Resistivity (ohm.m)
BH1	1.0 – 1.45	Clay	4.6	310	23	200	50
BH2	0.4 – 0.5	Silty clay filling	5.3	49	140	150	66
BH3	2.5 – 2.75	Laminite – extremely low strength	5.1	21	44	56	180
BH6	4.0 – 4.45	Silty lay filling	5.6	30	51	71	140

Where: Cl= Chloride SO_4 = Sulphate EC= Electrical Conductivity

6.2 Rock Testing

The results of point load index testing ($Is_{(50)}$) carried out at regular intervals on rock cores, are shown on the respective borehole logs in Appendix C. The $Is_{(50)}$ values for the bedrock samples ranged from 0.5 MPa to 4.6 MPa, indicating medium strength to very high strength rock. The results of Point Load testing plotted against depth (RL) are shown in
Figure 2.

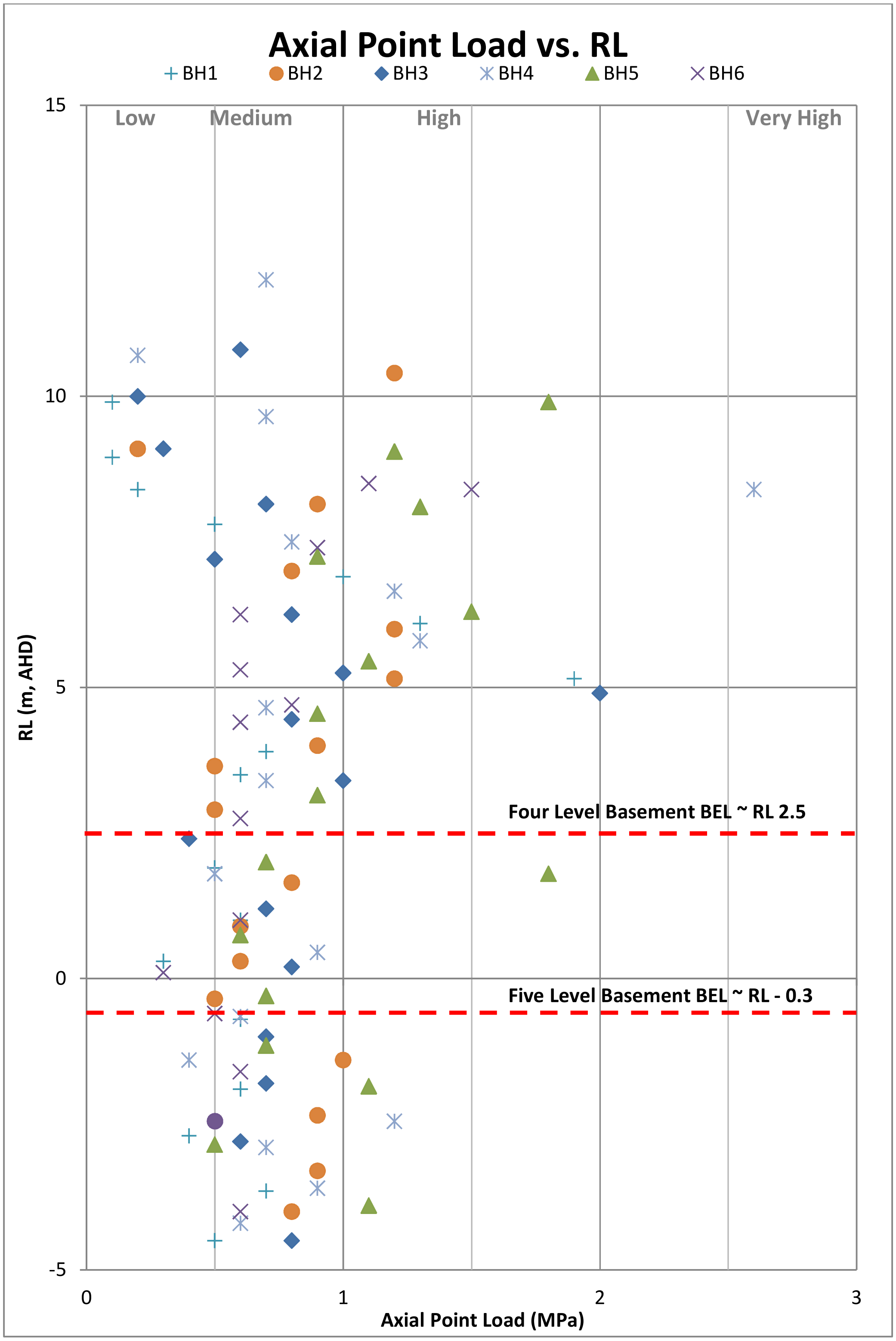


Figure 2: Point Load Values

The $Is_{(50)}$ values can be used to provide an estimate of the Unconfined Compressive Strength (UCS) of the rock, based on a UCS:$Is_{(50)}$ ratio of 20:1 for excavatability. The estimated UCS values for the laminite and shale in the core typically ranged from 4 MPa to 78 MPa, indicating that the rock tested range from low strength to very high strength. The very high strength values are probably related to siderite bands in the shale. For assessment of foundations, batters and rock faces it is recommended that a conversion ratio of no more than 16:1 is used.

7. Geotechnical Model

7.1 Subsurface Profile

The subsurface profile encountered in the boreholes has been summarised into four units as outlined in Table 4.

Table 4: Geotechnical Model

Unit	Material	Description	Depth Range to Top of Unit m bgl / (RL)
1	Filling	Generally silty sand, clayey sand and sandy gravel filling	0 m (RL 16.6 m – RL 13.5 m)
2	Residual soil	Generally stiff to very stiff grey brown clay	0.6 m – 1.1 m (RL 14.25 – RL 12.9 m)
3	Extremely low to low strength laminite and shale	Generally extremely to moderately weathered, fragmented to fractured, grey and brown, laminite and shale, with some ironstone bands	1.5 m – 5.5 m (RL 12.8 m – RL 9.4 m)
4	Medium strength or stronger laminite and shale	Generally slightly weathered to fresh, slightly fractured and unbroken, grey, laminite and shale	4.7 m – 6.5 m (RL 10.1 - RL 8.3 m)

7.2 Groundwater

The initial readings taken from the three standpipe piezometers installed in boreholes BH2, BH3 and BH6 indicate that the groundwater level at the site is between RL 2.9 m and RL 4.4 m. The difference between levels across the site suggest that the hydraulic gradient slopes north/north-east towards the Parramatta River and the Brickpit Park.

The recorded groundwater levels are all above the bulk excavation in rock of at least medium strength. The flow of groundwater through this rock will be controlled by the discontinuities (bedding planes and joints) in the rock mass. Seepage into the excavation is expected to be along bedding planes and to a lesser extent through joints exposed in the rock faces. The RQD of the core indicates the rock is of good to excellent quality with minimal discontinues and hence the seepage is expected to be low. Seepage into the excavation may also occur through filling and along the soil:rock interface in response to periods of rainfall.

It should be noted that groundwater levels can vary with time due to changes in climatic conditions. Reference should be made to the NSW Office of Water for prediction of future rainfall changes.

A geotechnical model of the site is presented in the form of interpreted cross-sections (Drawings 2 - 4 in Appendix B) based on the investigation carried out.

8. Comments

8.1 Proposed Development

It is understood that the proposed development of the site will include the construction of two 30-storey mixed use towers with either a four or five level common basement carpark. It is further understood the basement will extend across mostof the site, except for the north-west and south-west corners of the site where a large tree is to remain and the T7 Olympic Park Line railway tunnel runs beneath. Minor landscaping works at street level are also proposed, though it is understood there are no structures or buildings which will apply a significant load onto the underlying rail tunnel (refer to Drawings in Appendix E)..

The drawings indicate that the basement will extend up to the first reserve (refer to Section 8.6). The bulk excavation level will be approximately RL 2.5 for a four level basement and RL -0.3 for a five level basement, requiring a maximum excavation depth of approximately 14 m or 17 m, below the existing ground surface. The bulk excavation level will be approximately 5.5 m below the invert of the adjacent rail tunnel for a four level basement and 8 m for a five level basement.

8.2 Excavation

8.2.1 Ground Conditions

It is expected that the filling and clayey residual soils, together with extremely low to low strength rock should be readily excavated using conventional earthmoving equipment, such as excavators. Excavation of the low to medium strength and fractured rock should be achieved by moderate ripping aided by the use of excavator mounted rock hammers. Excavation of slightly fractured, medium strength or stronger rock will require moderate to heavy ripping with a large bulldozer (space permitting) and/or excavators in conjunction with hydraulic rock hammers.

8.2.2 Groundwater

During construction, seepage into the excavation should be expected at the soil/rock interface particularly following heavy rainfall and along the bedding planes and joints within the rock especially below the groundwater level (about RL 4.5 to RL 2.5). It is expected that seepage into the excavation during construction will be relatively minor and should be controllable by perimeter drains feeding into sumps, from where it can be discharged into the stormwater system. The amount of water seeping into the basement structure should be monitored during construction for reference in relation to the design of the permanent drainage system (if the basement is designed as drained).

The need for ongoing dewatering, after construction, will depend on whether the basement is designed as a drained or water-tight (tanked) basement as described below:

- A drained basement will require permanent sub-slab drainage below the basement floor slab to prevent the development of hydrostatic pressure, and to direct any seepage towards the drainage system. The sub-slab drainage should be connected to a sump which regularly pumps out the water. The disposal requirements of water collected on-site will be dependent on the chemical composition of the water. Normally, water is disposed of to a stormwater or sewer system in accordance with council and EPA regulations and will be subject to approval from the relevant government authority. Approval from the relevant government authority may also be required to construct a basement below the groundwater level. A drained basement will, however, act as a low point to which groundwater will flow. If present, contamination within the surrounding groundwater system could be drawn into the basement and adversely affect the quality of the water collected on site.
- A tanked basement would avoid the need for on-going dewatering but is likely to be more expensive than a drained basement. A tanked basement would need to be designed to resist uplift forces beneath the basement floor slab associated with groundwater pressure, for which preliminary design could be based on a groundwater level at approximately 10.5 m below the surface level.

Note that the groundwater may have significant concentrations of iron which will tend to precipitate on exposure to air, giving rise to a gelatinous mass of iron oxide/hydroxide sludge. This precipitate will need to be taken into account when designing the permanent drainage lines and pump-out systems with allowance for access to periodically clean out of the sludge or the installation of water treatment plant.

It should be noted that groundwater levels can vary with time due to changes in climatic conditions. Reference should be made to the NSW Office of Water for prediction of future rainfall changes.

8.2.3 Ground-borne Vibration

During excavation, it will be necessary to use appropriate methods and equipment to keep ground vibration at adjacent buildings and structures, sensitive services in the footpaths and, in particular, the railway tunnel within acceptable limits. The level of acceptable vibration is dependent on various factors including the type of structure (e.g. reinforced concrete, brick, etc.), its structural condition, the frequency range of vibration produced by the construction equipment, the natural frequency of the structure and the vibration transmitting medium.

Ground-borne vibration can be strongly perceptible to humans at levels above 3 mm/s component peak particle velocity (PPVi). This is generally much lower than the vibration levels required to cause structural damage to buildings. The Australian Standard AS2670.2-1990 *“Evaluation of human exposure to whole-body vibrations – continuous and shock induced vibrations in buildings (1-80 Hz)”* indicates an acceptable day-time limit of 8 mm/s component PPVi for human comfort. This should be adopted for the occupied building to the east of the site (about 4 m from the site boundary).

Ground-borne vibration will need to be controlled when excavating in the vicinity of the Olympic Park rail tunnel. It should be noted that, based on previous experience, TfNSW usually require real-time vibration monitoring within the tunnel (see Section 9).

As the magnitude of vibration transmission is site specific, it is recommended that a vibration trial be undertaken at the commencement of rock excavation.

8.2.4 Excavation Plant

Trial data is dependent on site conditions and equipment, hence actual vibration levels may differ from predictions and a trial is therefore recommended at the commencement of rock excavation on site. DP maintains a database of vibration trial results which can provide guidance for the selection of plant. The database suggests buffer distance ranges, such as those shown for selected plant in Table 5, which should be maintained between excavation plant and adjacent structures. The buffer distance should be assessed during the vibration trial.

Table 5: Approximate Buffer Distances for Selected Plant (Provisional Allowed Limit 8 mm/s)

Excavation Plant	Distance from plant by which vibration normally attenuates to the Provisional Allowed Limit of 8 mm/s	
	From DP trial maxima[1]	From DP trial averages
Rock Saw on Excavator [2]	1.1 m	0.6 m
Ripper on 20t Excavator	3.4 m	1.2 m
Rock Hammer < 500 kg operating weight	7.4 m	3.0 m
Rock Hammer 501 - 1000 kg operating	7.5 m	3.3 m
Rock Hammer 1001 - 2000 kg operating	12.4 m	5.4 m
Rock Hammer > 2000 kg operating	7.4 m	4.9 m

Note:

1. Smaller distances can generally be determined from individual trials, as indicated by the trial averages;
2. Buffer distances for rock hammers may be reduced by prior saw cutting along, or parallel to, excavation boundaries.

8.2.5 Disposal of Excavated Material

All surplus excavated materials will need to be disposed of in accordance with the Protection of the Environment Operations Act 1997 (POEO Act). All materials removed from the site are defined as waste under the POEO Act and must be disposed of in accordance with one of the following:

- virgin excavated natural materials (VENM) as defined under the POEO Act, permitting reuse on site; or,
- a waste category meeting the criteria set out in the NSW EPA Waste Classification Guidelines 2014, with the materials disposed to a landfill licenced to receive the waste under the assigned classification; or,

- material complying with a Resource Recovery Order (RRO) as defined under the Protection of the Environment Operations (Waste) Regulation 2014, with complying materials able to be reused under certain conditions.

Accordingly, environmental testing will need to be carried out to determine the most appropriate off-site destination(s) for the surplus excavated material.

It is noted that a contamination assessment has been carried out by DP concurrently with the geotechnical investigation, which will be reported separately.

8.2.6 Stress Relief

The proposed excavation works cannot be accomplished without some lateral movement at the excavation boundaries. The release of locked-in stresses in the rock is accompanied by lateral movement of the rock faces along the boundaries, particularly on the northern and southern boundaries (direction of the maximum principal stress).

Previous experience in shale, expected to have similar movement to the Laminite present at this this site, has shown that boundary movement in the order of 0.5 to 1.5 mm per metre depth of rock excavation can occur. The amount of horizontal movement will diminish along the crest away from the midpoint, down the excavated face and back from the crest.

Movement resulting from stress relief generally occurs over a horizontal distance of up to three times the excavation depth, back from the excavation. Stress relief movements are an unavoidable consequence of large excavations. Stress relief will induce some movement of adjacent structures. The extent of movement can be predicted by carrying out numerical modelling. The actual movement can be assessed during excavation by the use of survey points or inclinometers. Monitoring of such movements should be included in the Inspection and Test Plan for the site (see Section 10).

The bulk of the movement is expected to occur progressively during the excavation with only minor creep expected after completion.

8.2.7 Excavation Support

Careful consideration must be given to the planning and design of excavation and excavation retention system(s) to reduce the risks of destabilising and causing damage to the adjacent buildings, surrounding public footpaths/roads and the adjacent rail tunnel.

As excavation will be required to the boundaries of the site, battering the sides of the excavation will not be feasible. Vertical excavation in the overburden materials and rock of less than medium strength (Units 1 to 3 inclusive) will require both temporary and permanent lateral support during excavation and as part of the final construction. Shoring is therefore likely to be required down to the medium strength or stronger rock (Unit 4). The depth to the top of Unit 4 will depend on the levels that medium strength rock is encountered around the perimeter of the site. At the borehole locations, the top of Unit 4 varied from about RL 8.3 to 10.1 m.

Excavated faces in medium strength or stronger laminite/shale (Unit 4) are generally expected to be self-supporting, apart from where adverse jointing is present. Where adverse jointing is identified, it will require spot rockbolting or anchors with/without shotcrete support. Joint dips measured in the core varied from 30 to 90° indicating that shallow dipping joints should be expected in excavation faces. It should be noted that the vertical boreholes drilled as part of the investigation provide information about the joints at that point only and further jointing with different dips may be present away from the borehole.

Note that, there is always a slight risk of a 45° fault daylighting at the base of the excavation which would only become evident once the excavation has reached bulk level. To minimize this risk pattern rockbolting or anchoring would be required to support the 45 degree wedge initially designed for a lower factor of safety commensurate with the low risk of such eventuality. However, the excavation would need to be carefully inspected during excavation to check that there are no signs of such a fault because if there are, additional anchors may be required to increase the factor of safety.

It is recommended that all rock faces be inspected by an engineering geologist at 1.5 m drops during excavation to confirm that the site conditions are consistent with the design assumptions and to verify the stability of the faces and advise on any bolting or anchoring requirements. Note, the shale and siltstone, if left unprotected will slowly dry out, fret and ravel. Hence it is recommended that the full face be protected with a minimum thickness of 80 mm of dowel supported mesh reinforced shotcrete.

8.2.7.1 Shoring

Temporary support of the excavation faces in Units 1 to 3 inclusive will be required along all four site boundaries. Based on the expected ground conditions, suitable support could be provided by anchored soldier piles with shotcrete infill panels. This system should be capable of supporting typically sized wedges that may be present in the weathered rock. Note that an alternative approach is expected to be required in the south-west corner of the site where it may not be possible to install ground anchors in the rail first reserve (a rail protection zone closest to the rail tunnel).

The drawings indicate the site boundary is adjacent to the rail first reserve with the tunnel about 7 m away from the proposed basement excavation. It is assumed that the tunnel has been constructed using cut and cover techniques mostly in Units 1 to 3, with the invert in Unit 4 material. Boreholes 5 and 6 indicate that there is a deeper profile of Unit 1 (Filling) adjacent to the tunnel, than in other boreholes, which may be associated with the tunnel construction.

A suitable shoring system along this section of the boundary could be a contiguous pile wall temporarily supported by a soil berm, internal props and/or diagonal bracing. Permanent support would be provided by the building structure. At this initial stage, it is expected that the piles would extend below the bulk level to benefit from the passive resistance provided by the shale below bulk level. Depending on the extent of the first reserve below the tunnel invert, however, it may be possible to found the piles higher by installing anchors below the first reserve. Confirmation of the extent of the first reserve by Sydney Trains will be required to assess this possibility.

8.2.7.2 Shoring Wall Design

Where more than 1 row of anchors is required, a trapezoidal earth pressure distribution should be used. In this case, the pressure distribution should increase from zero at the surface to the maximum value at a depth of 0.25 H and decrease from the maximum value at a depth of 0.75 H back to zero at the base of the excavation.

Where there are no movement-sensitive structures in close proximity to the excavation the maximum pressure should be calculated using 4H kPa (where H equals the depth to the top of self-supporting medium strength or stronger rock – note, fractured shale is not self-supporting). Where the wall movement is to be minimised (i.e. close to adjacent buildings) the maximum pressure should be calculated using 6H kPa. For the tunnel or other movement-sensitive structures, where it is critical that deformation is controlled, it is recommended that the maximum pressure is calculated using 8H kPa, applied as a rectangular pressure distribution.

Additional surcharge loads, such as new and existing footings, and loads from construction related activities, must also be allowed for in the design as a rectangular earth pressure distribution, applied over the depth of influence, using the earth pressure coefficient in Table 6.

Table 6: Recommended Permanent Design Parameters for Shoring Systems

Material	**Unit Weight (kN/m^3)**	**Earth Pressure Coefficient (Permanent)**		**Effective Cohesion c' (kPa)**	**Effective Friction Angle (°)**
		Active (K_a)	**At Rest (K_o)**		
Filling/Clay	20	0.4	0.5	2	25
Extremely Low to Low Strength Laminite/Shale	22	0.25	0.4	10	25
Low Strength Laminite/Shale	23	0.25	0.15	-	-
Medium Strength or Stronger Laminite/Shale	24	0	0	-	-

Notes: All values assume a level surface behind the wall and that the rock is not affected by adverse dipping joints.

The triangular earth pressure distribution on the wall can be calculated as follows:

$$H_z = K (\gamma z + p)$$

Where: H_z = horizontal pressure at depth z

γ = unit weight of soil or rock

K = earth pressure coefficient

z = depth (m)

p = vertical surcharge pressure

The earth pressure loading described above does not include either earthquake loads or hydrostatic pressure due to the build-up of groundwater behind impermeable walls, both of which must also be considered in the design. Unless positive drainage measures are incorporated to prevent water pressure build-up behind the walls, the full hydrostatic head should be allowed for in design while, at the same time, allowing for the soil unit weight to reduce to the buoyant condition.

Passive resistance for piles founded in rock below the base of the bulk excavation may be based on a working passive restraint of 3000 kPa in medium strength or stronger shale, not adversely affected by discontinuities. The minimum socket depth should be equal to the greater of one pile diameter or 1.0 m below the lowest level of any nearby excavation (including any detailed excavations) unless there is sufficient evidence available to confirm otherwise. The first 0.5 m of rock socket below the bulk excavation level should not be taken into account for the purpose of passive restraint. See also the comments in Section 8.2.7 for rock support below the base of the weathered rock.

Staged excavation and inspection by a suitably qualified geotechnical engineer will be required to confirm that the rock is not adversely affected by discontinuities, especially where passive resistance is relied upon.

8.2.7.3 Anchoring

Pre-stressed ground anchors, rockbolts and dowels (support elements) can be used to laterally support existing walls, new shoring, underpinning or unstable rock masses. These support elements should be bonded in the stronger rock, inclined as required, but preferably not steeper than 30° below the horizontal. Table 7 provides allowable bond stresses for anchor design.

Table 7: Ultimate Bond Stresses for Anchor Design

Material Description	Ultimate Bond Stress (kPa)
Very stiff to hard clay	50
Extremely low to very low strength rock	100
Low strength rock	300
Medium strength rock	1000

These values should be confirmed by pull-out tests prior to installation of support elements. Ultimately, it is the contractor's responsibility to ensure that the correct design values (specific to the support system and method of installation) are used and that the support element holes are carefully cleaned prior to grouting.

After support elements have been installed, it is recommended that they are tested to 125% of their nominal working load. Where stress relief or further unavoidable movement of the shoring and rock face is expected, it is recommended that the support elements are locked-off between 60% and 80% of their working loads, as required to accommodate the additional movement and subsequent increase in stress in the support elements.

Shorter support elements (rockbolts, dowels and pins) may be required to support localised unstable rock wedges, slivers or blocks. Short dowels and pins may be required to support feather edges where sub-parallel joints intersect the face.

Care should be exercised to ensure that anchors are installed progressively during excavation and stressed prior to excavation of the next drop to ensure that stability is maintained at all times.

8.3 Foundations

Column loads have not been provided, however, for the structure proposed it is recommended that all loads be taken down to rock. If there are any high level loads founding in close proximity to the rail tunnel, they will need to be transferred down to below the area of influence on the rail tunnel (i.e. to below a 45° line drawn upwards from the invert of the tunnel). Typical maximum allowable bearing pressures for shale, based on the foundation classification methods of Pells et al. (1998), are shown Table 8. Note that shaft adhesion values for uplift (tension) may be taken as being equal to 70% of the values for compression.

Table 8: Preliminary Design Parameters for Foundation Materials

Material	Ultimate End Bearing Pressure (MPa)	Allowable End Bearing Pressure (MPa)	Allowable Shaft Adhesion (kPa)	Elastic Modulus (MPa)	Minimum Investigation
Very Low strength shale (Class IV)	3	1	150	350	As below
Low strength shale (Class III)	15	2.25	250	700	Site inspection with at least 2 cored boreholes
Medium strength shale (Class II)	65	4.5	575	1500	Minimum 4 cored bores with spoon testing in at least 50% of footings
Medium to high strength shale (Class I)	90	8	800	2000	Cored bores at maximum 10 m grid spacing or cored bores for 50% of footings and spoon testing of remainder.

Notes:

- Ultimate parameters mobilized at large settlements (i.e. >5% of footing width)
- Allowable pressures for “Working Stress Design Values” are based on a ‘limiting settlement’ of <1% of the footing diameter or width.
- All shaft adhesion parameters are based on adequately clean and rough sockets of category “R2”, or better.
- Shaft Adhesion Values should only be adopted from 2 pile diameters below the bulk excavation level

Foundations proportioned on the basis of the above allowable bearing pressures would be expected to experience total settlements of less than 1% of the minimum footing width under the applied working load, with differential settlement between adjacent columns expected to be less than half this value.

The design of footings is usually governed by settlement criteria and performance rather than the ultimate bearing capacity. The Serviceability limit should be assessed, for normal ‘static’ load cases, using the elastic modulus values given in Table 8. This modulus value is appropriate for the anticipated working stress values or strain expected under serviceability loading.

All footing excavations should be inspected by a geotechnical engineer to confirm that foundation conditions are in accordance with the design parameters used. Minimum investigation requirements for the respective bearing pressures are shown in Table 8.

8.4 Soil Aggressivity

The results of aggressivity testing, and reference to Table 6.4.2(C) in AS2159-2009 "Piling: Design and Installation" indicates that an Exposure Classification for concrete piles of 'mild' for soil conditions Type B (low permeability soil) is appropriate.

8.5 Basement Floor Slabs and External Pavements

Basement slab-on-ground construction will require a compacted granular sub-base layer. The sub-base layer will act as the sub-floor drainage layer provided the material consists of free-draining material with a low percentage of fines.

Details of any proposed on-grade pavements have not been provided. The subgrade will vary across the site and will depend on the design level and also the subgrade preparation and type of engineered filling used to form the subgrade. From the borehole information, however, it is likely that pavement subgrades may comprise clayey filling and generally stiff clays overlying extremely low to very low strength rock at relatively shallow depth.

During construction of pavements, it is recommended that all topsoil, organic and deleterious material be stripped and stockpiled separately for disposal or use in landscaping areas. Proof rolling of the exposed subgrade should then be carried out under the supervision of a geotechnical engineer to detect any soft or heaving areas. Any soft spots detected during proof rolling will need to be stripped to a stiff base or maximum depth of 0.5 m and replaced with engineered filling.

Subject to the subgrade preparation outlined above, the design of pavements on clay and extremely low strength rock subgrade may be based on a CBR value of say 3%. The design of pavements on very low strength rock, if exposed, may be based on a CBR value of 10%. These CBR values assume all pavements are protected by adequate surface and subsoil drainage to minimise the risk of water infiltration and softening of pavement materials. Further inspection of the earthworks should be carried out during the earthworks to confirm the appropriate CBR values.

Using CBR values of 3%, car park or pavements to be used by cars and light commercial vehicles (i.e. delivery vans up to 3 tonne gross weight), will require flexible pavement thickness in the order of 150 to 200 mm.

Should any engineered filling be required, it should be placed in maximum 300 mm thick loose layers and compacted to a minimum dry density ratio of 98% Standard compaction with moisture contents within 2% of optimum moisture content (OMC). The compaction should be increased to a dry density ratio of 100% Standard compaction within 0.3 m of the subgrade surface. The existing filling and clay on site should generally be suitable for re-use as engineered filling, provided it has a maximum particle size of 70 mm and moisture content between OMC –4% and OMC.

8.6 Earthquake Design

A hazard factor of 0.08 is appropriate for the site in accordance with the Earthquake Loading Standard, AS1170.4 – 2007. Based on the available information, the site has been assessed as having a Site Sub-Soil Class of C_e where foundations are located on residual soils. For foundations located on rock at the base of the excavation a Site Sub-Soil Class of B_e may be adopted provided that the building is not designed to rely on the lateral support provided by the soil.

9. Considerations Relating to the Rail Corridor

As specified in TfNSW's T HR CI 12051 ST Standard the first reserve zone comprises the immediate surrounds of the tunnel. This zone represents the area that shall not be encroached upon by any future construction or development. The first reserve at any given location adjacent to the tunnel is half the maximum tunnel width, which is up to 8.25 m along the alignment adjacent to the site. As the tunnel is cut and cover the first reserve above the tunnel extends to the existing ground surface. The standard states the first reserve below the tunnel is the greater of 1 m from the lowest invert including cable and drainage trenches or the existing predefined easement. From the available information it appears that there is no existing predefined easement so the first reserve is assumed to be 1 m below the lowest invert level. This should be confirmed by Sydney Trains.

It is noted, that the current basement design does not encroach into the first reserve, however, the landscaping works at street level may. It is understood only minor works are proposed above the tunnel and within the zone of influence (imaginary line drawn upwards at 45° from the base of the tunnel). Permission from Sydney Trains, however, may be required and an assessment made as to whether the works will have an adverse effect on the tunnels.

The second reserve zone covers the areas where development works have the potential to affect the performance and operation of the tunnel. At this site, the second reserve is the first reserve plus 25 m. Development works or any future construction within the second reserve requires an engineering assessment to determine their effects on the underground rail infrastructure. Thus it is likely that TfNSW will require a numerical analysis to determine the effect that the proposed development will have on the railway tunnel.

It is noted that Sydney Trains typically require the following before excavation works can commence:

- An engineering assessment using numerical modelling that demonstrates the excavation will not cause any adverse effect on the rail tunnel and associated infrastructure;
- A detailed work method statement with hold points at various stages of excavation that are subject to review of satisfactory monitoring results;
- A detailed monitoring plan for ground deformation, tunnel convergence, stress, crack width monitoring, vibration monitoring and reporting protocol for each party;
- Risk assessment and contingency plans; and,
- Dilapidation surveys of the tunnel lining prior, during and on completion of construction with joint Sydney Trains sign-off typically of the baseline and final surveys.

As part of this process, cross section(s) showing development details and the tunnels prepared by a NSW registered surveyor and in ground instrumentation such as an inclinometer(s) may be required.

Instrumentation in the tunnel may include tunnel convergence, crack meter, crack tell-tale gauges, vibration sensor, rail track distortion monitoring, strain gauges in the tunnel lining, pressure cells in the lining and real time monitoring such as tilt sensors or optical prism laser scanning. Baseline data will be required for all instrumentation prior to excavation commencing.

10. Additional Geotechnical Works

10.1 Additional Site Investigation

Additional geotechnical investigation may be required depending on the final structural design.

10.2 Geotechnical Inspection and Monitoring during Construction

It is suggested that the following be carried out either prior to or during the construction phase of the project, as appropriate. This is usually part of the inspection and test plan that should be prepared for the geotechnical work.

10.2.1 Monitoring

It is recommended that survey points be installed on the top of the excavation support wall to monitor wall deflection during the works. Readings will generally need to be taken in advance of any excavation, at intervals during excavation works, and after completion of all excavation works. Movement of the shoring walls should be recorded by a surveyor at every 1.5 m drop in excavation level and forwarded on to the geotechnical engineer for assessment.

10.2.2 Excavation

It is recommended that regular inspections during drilling, installation and stressing of anchors for the shoring wall is carried out by a geotechnical engineer/ engineering geologist.

All rock faces should be inspected by a geotechnical engineer/engineering geologist every 1.5 m drop in excavation to confirm that the site conditions are consistent with the design assumptions and to verify the stability of the faces and advise on any bolting or anchoring requirements. The purpose of these inspections is to identify and assess any adverse dipping joints (wedges) or defects in the rock face and determine if any additional support is required.

10.2.3 Foundations

Footing inspections will be required to assess the bearing capacity of the founding materials. Spoon testing in pre-drilled holes to assess the rock below the footing for any seams or defects may also be required depending on the footing bearing pressure. The geotechnical engineer/engineering geologist should immediately be notified of any discrepancy in material consistency so that conditions can be re-assessed and amendments made, should it be required.

11. Limitations

Douglas Partners (DP) has prepared this report for this project in accordance with DP's proposal SYD180435 dated 7 August 2018 and acceptance received from Greg Hynd. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of this project only and for the purposes as described in the report. It should not be used by or be relied upon for other projects or purposes on the same or another 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 likely conditions provided in the report are indicative of the likely sub-surface conditions based on information from test locations only. Sub-surface conditions can change laterally due to variable geological processes and the accuracy of the advice provided by DP in this report may be affected by undetected variations in ground.

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

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

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

12. References

Braybrooke, J.C. (1990 a) – Some geotechnical phenomena related to high stresses in the Hawkesbury Sandstone. *24th Newcastle Symposium, 1990*.

Braybrooke, J.C. (1990 b) – Excavations in a high horizontal stress field. *Proc. 6th Cong. Int. Assoc. Eng. Geol. Amsterdam, Aug 1990*.

Och et al. (2009) – "Timing of brittle faulting and thermal events, Sydney Region: Association with early stages of extension of East Gondwana". Australian Jornal of Earth Sciences 56(7):873-877, Oct 2009

Pells et al (1998) "Foundations on Sandstone and Shale in the Sydney Region" Aust. Geomech Jrnl., Dec, 1998.

Douglas Partners Pty Ltd

Appendix A

About This Report

About this Report

Introduction

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

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

Copyright

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

Borehole and Test Pit Logs

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

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

Groundwater

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

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole is left open;
- A localised, perched water table may lead to an erroneous indication of the true water table;
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report; and
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water measurements are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.

Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical and environmental aspects, and recommendations or suggestions for design and construction. However, DP cannot always anticipate or assume responsibility for:

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

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

Site Anomalies

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, DP requests that it be immediately notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

Information for Contractual Purposes

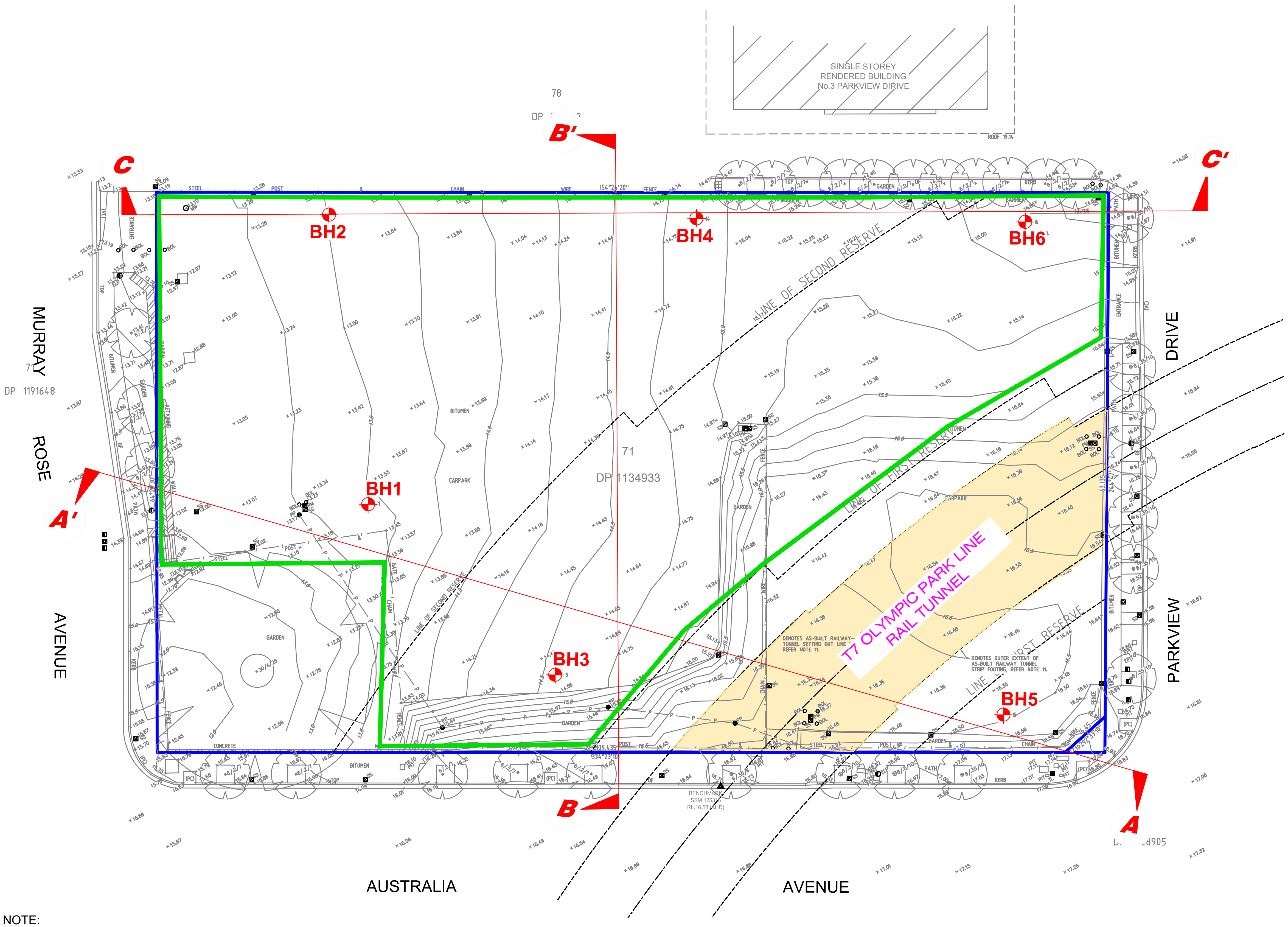
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate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

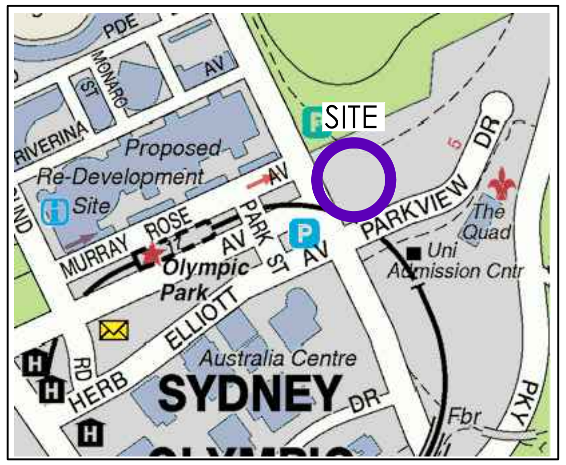
Site Inspection

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

Appendix B

Drawings





Locality Plan

NOTE:
1: Base drawing from LTS Lockley Pty Ltd
(Ref 50357 002DT, dated 14.6.2018)
2: Test locations are approximate only and
are shown with reference to existing features.

0 5 10 15 20 30 40 50m

1:500 @ A3

LEGEND

⊕ Borehole location

A —— A' Geotechnical Cross Section A-A'

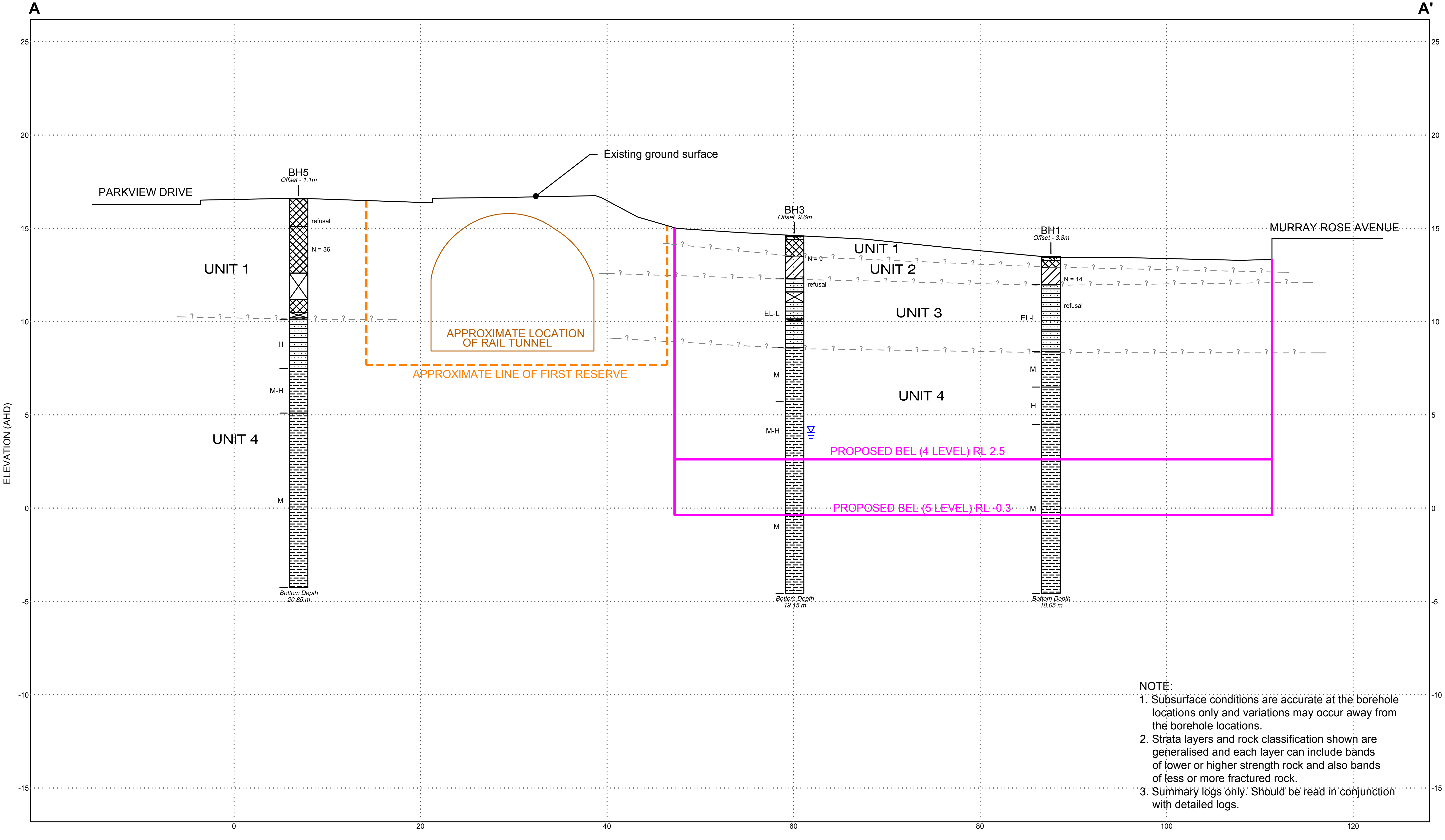
🟩 Approx. extent of proposed basement

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Ecove Pty Ltd
OFFICE: Sydney
DRAWN BY: PSCH
SCALE: 1:500 @ A3
DATE: 17.9.2018

TITLE: **Test Location Plan**
Carpark P6D (Site 2)
Olympic Park, SYDNEY

PROJECT No: 86411.00
DRAWING No: 1
REVISION: 0



NOTE:

1. Subsurface conditions are accurate at the borehole locations only and variations may occur away from the borehole locations.
2. Strata layers and rock classification shown are generalised and each layer can include bands of lower or higher strength rock and also bands of less or more fractured rock.
3. Summary logs only. Should be read in conjunction with detailed logs.

LEGEND

- ☒ Core Loss
- ■ Asphaltic Concrete
- ▨ Clay
- ▦ Filling
- ▤ Laminite
- ⁘ Roadbase
- ☰ Shale

ROCK STRENGTH

EL	Extremely low strength
VL	Very low strength
L	Low strength
M	Medium strength
H	High strength

UNIT 1 - Filling
UNIT 2 - Residual soil
UNIT 3 - Extremely low strength to low strength Laminite/ Shale
UNIT 4 - Medium strength or stronger Laminite/ Shale

TESTS / OTHER
N - Standard penetration test value
▽ - Water level
PP - Pocket penetrometer test value

0 8
Horizontal Scale (metres)

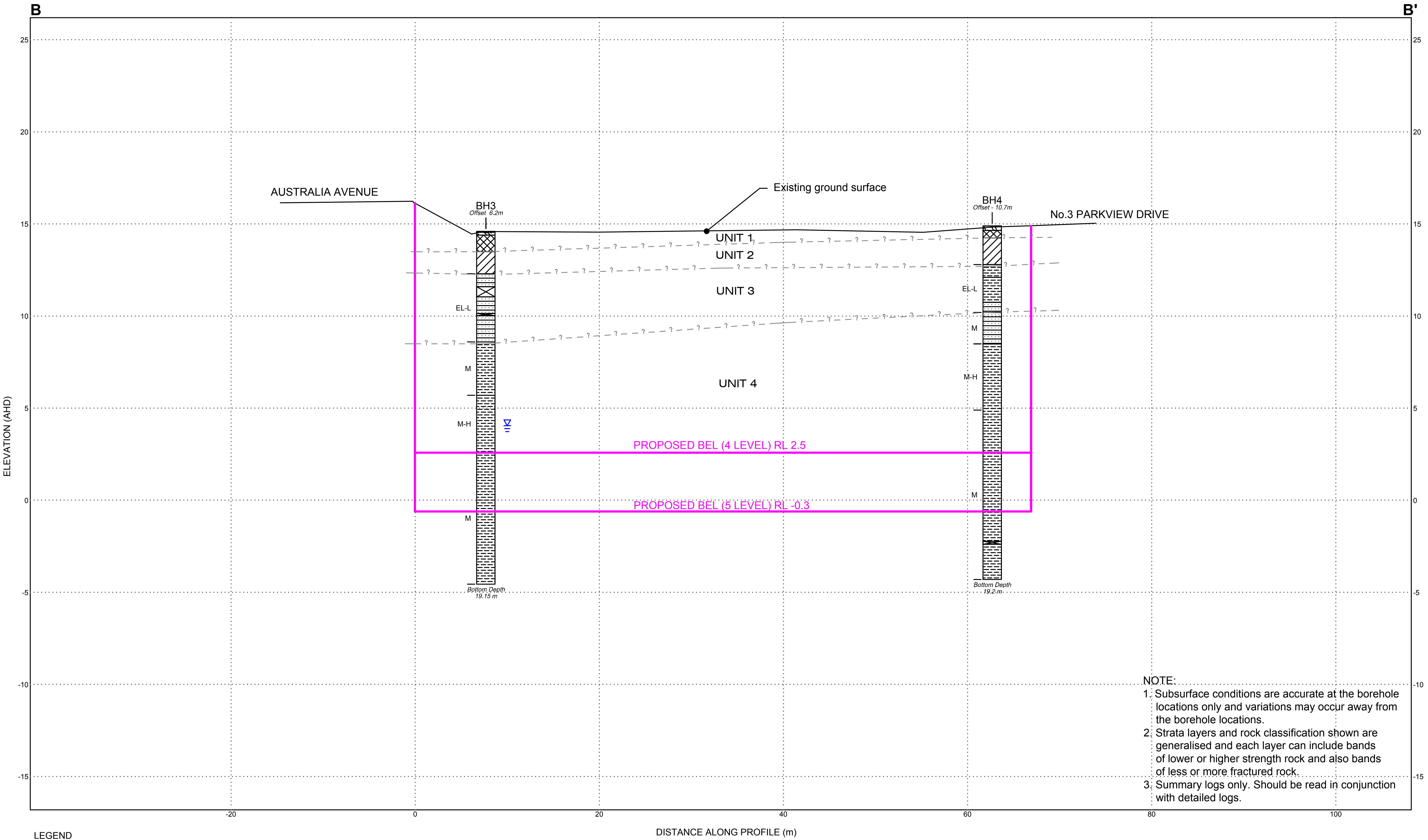
Vertical Exaggeration = 2.0

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Ecove Pty Ltd
OFFICE: Sydney
DRAWN BY: CE
SCALE: 1:400 (H) 1:200 (V) @ A3
DATE: 17.09.2018

TITLE: **Geotechnical Cross-section A-A'**
Carpark P6D (Site 2)
Olympic Park, SYDNEY

PROJECT No: 86411.00
DRAWING No: 2
REVISION: 0



NOTE:
1. Subsurface conditions are accurate at the borehole locations only and variations may occur away from the borehole locations.
2. Strata layers and rock classification shown are generalised and each layer can include bands of lower or higher strength rock and also bands of less or more fractured rock.
3. Summary logs only. Should be read in conjunction with detailed logs.

LEGEND

- ☒ Core Loss
- ■ Asphaltic Concrete
- ▨ Clay
- ▩ Filling
- ▤ Laminite
- ▒ Roadbase
- ▤ Shale

ROCK STRENGTH

EL	Extremely low strength
VL	Very low strength
L	Low strength
M	Medium strength
H	High strength

UNIT 1 - Filling
UNIT 2 - Residual soil
UNIT 3 - Extremely low strength to low strength Laminite/ Shale
UNIT 4 - Medium strength or stronger Laminite/ Shale

TESTS / OTHER
N - Standard penetration test value
⊻ - Water level
PP - Pocket penetrometer test value

0 8
Horizontal Scale (metres)

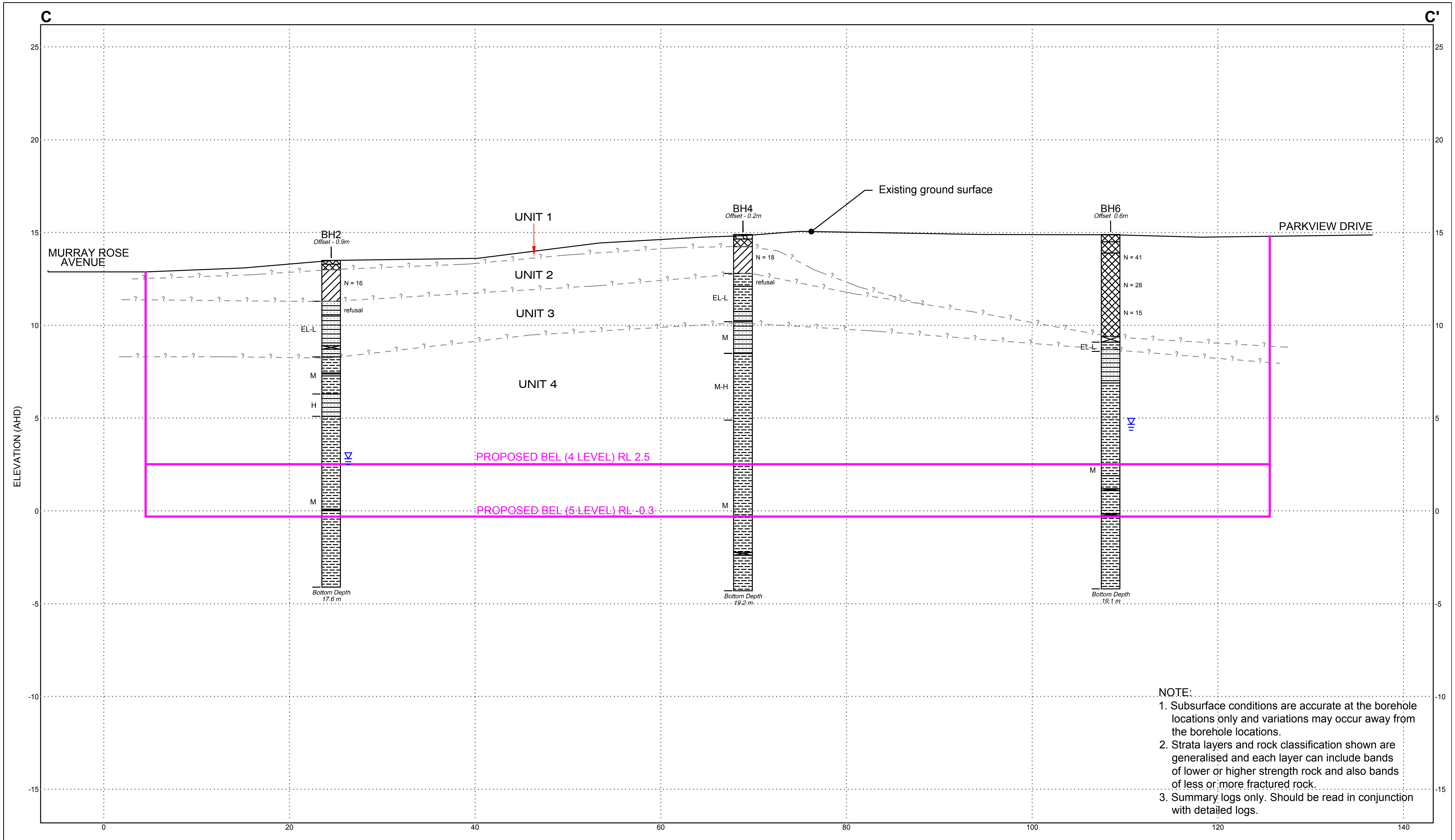
Vertical Exaggeration = 2.0

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Ecove Pty Ltd
OFFICE: Sydney
DRAWN BY: CE
SCALE: 1:400 (H) 1:200 (V) @ A3
DATE: 17.09.2018

TITLE: **Geotechnical Cross-section B-B'**
Carpark P6D (Site 2)
Olympic Park, SYDNEY

PROJECT No: 86411.00
DRAWING No: 3
REVISION: 0



NOTE:
1. Subsurface conditions are accurate at the borehole locations only and variations may occur away from the borehole locations.
2. Strata layers and rock classification shown are generalised and each layer can include bands of lower or higher strength rock and also bands of less or more fractured rock.
3. Summary logs only. Should be read in conjunction with detailed logs.

LEGEND

☒ Core Loss
■ Asphaltic Concrete
▨ Clay
☒ Filling
☰ Laminite
▒ Roadbase
☰ Shale

ROCK STRENGTH

EL	Extremely low strength
VL	Very low strength
L	Low strength
M	Medium strength
H	High strength

UNIT 1 - Filling
UNIT 2 - Residual soil
UNIT 3 - Extremely low strength to low strength Laminite/ Shale
UNIT 4 - Medium strength or stronger Laminite/ Shale

TESTS / OTHER
N - Standard penetration test value
⊻ - Water level
PP - Pocket penetrometer test value

0 8
Horizontal Scale (metres)
Vertical Exaggeration = 2.0

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Ecove Pty Ltd		TITLE: **Geotechnical Cross-section C-C'**	PROJECT No: 86411.00
OFFICE: Sydney	DRAWN BY: CE	**Carpark P6D**	DRAWING No: 4
SCALE: 1:400 (H) 1:200 (V) @ A3	DATE: 17.09.2018	**Olympic Park, SYDNEY**	REVISION: 0

Appendix C

Borehole Logs and Core Photographs

Sampling Methods

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Sampling

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

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

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing it to obtain a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Test Pits

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

Large Diameter Augers

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

Continuous Spiral Flight Augers

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

Non-core Rotary Drilling

The borehole is advanced using a rotary bit, with water or drilling mud being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from the rate of penetration. Where drilling mud is used this can mask the cuttings and reliable identification is only possible from separate sampling such as SPTs.

Continuous Core Drilling

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

Standard Penetration Tests

Standard penetration tests (SPT) are used as a means of estimating the density or strength of soils and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, Methods of Testing Soils for Engineering Purposes - Test 6.3.1.

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150 mm of, say, 4, 6 and 7 as:

  4,6,7
  N=13

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

  15, 30/40 mm

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

Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests

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

- Perth sand penetrometer - a 16 mm diameter flat ended rod is driven using a 9 kg hammer dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands and is mainly used in granular soils and filling.
- Cone penetrometer - a 16 mm diameter rod with a 20 mm diameter cone end is driven using a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). This test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various road authorities.

Soil Descriptions

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Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard AS 1726-1993, Geotechnical Site Investigations Code. In general, the descriptions include strength or density, colour, structure, soil or rock type and inclusions.

Soil Types

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

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

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

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

The proportions of secondary constituents of soils are described as:

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

Definitions of grading terms used are:

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

Cohesive Soils

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

Description	Abbreviation	Undrained shear strength (kPa)
Very soft	vs	<12
Soft	s	12 - 25
Firm	f	25 - 50
Stiff	st	50 - 100
Very stiff	vst	100 - 200
Hard	h	>200

Cohesionless Soils

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

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

Soil Descriptions

Soil Origin

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

- Residual soil - derived from in-situ weathering of the underlying rock;
- Transported soils - formed somewhere else and transported by nature to the site; or
- Filling - moved by man.

Transported soils may be further subdivided into:

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

Rock Descriptions

Rock Strength

Rock strength is defined by the Point Load Strength Index ($Is_{(50)}$) and refers to the strength of the rock substance and not the strength of the overall rock mass, which may be considerably weaker due to defects. The test procedure is described by Australian Standard 4133.4.1 - 2007. The terms used to describe rock strength are as follows:

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

* Assumes a ratio of 20:1 for UCS to $Is_{(50)}$. It should be noted that the UCS to $Is_{(50)}$ ratio varies significantly for different rock types and specific ratios should be determined for each site.

Degree of Weathering

The degree of weathering of rock is classified as follows:

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

Degree of Fracturing

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

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

Rock Descriptions

Rock Quality Designation

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

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

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

Stratification Spacing

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

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

Symbols & Abbreviations

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
$\underline{\nabla}$	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

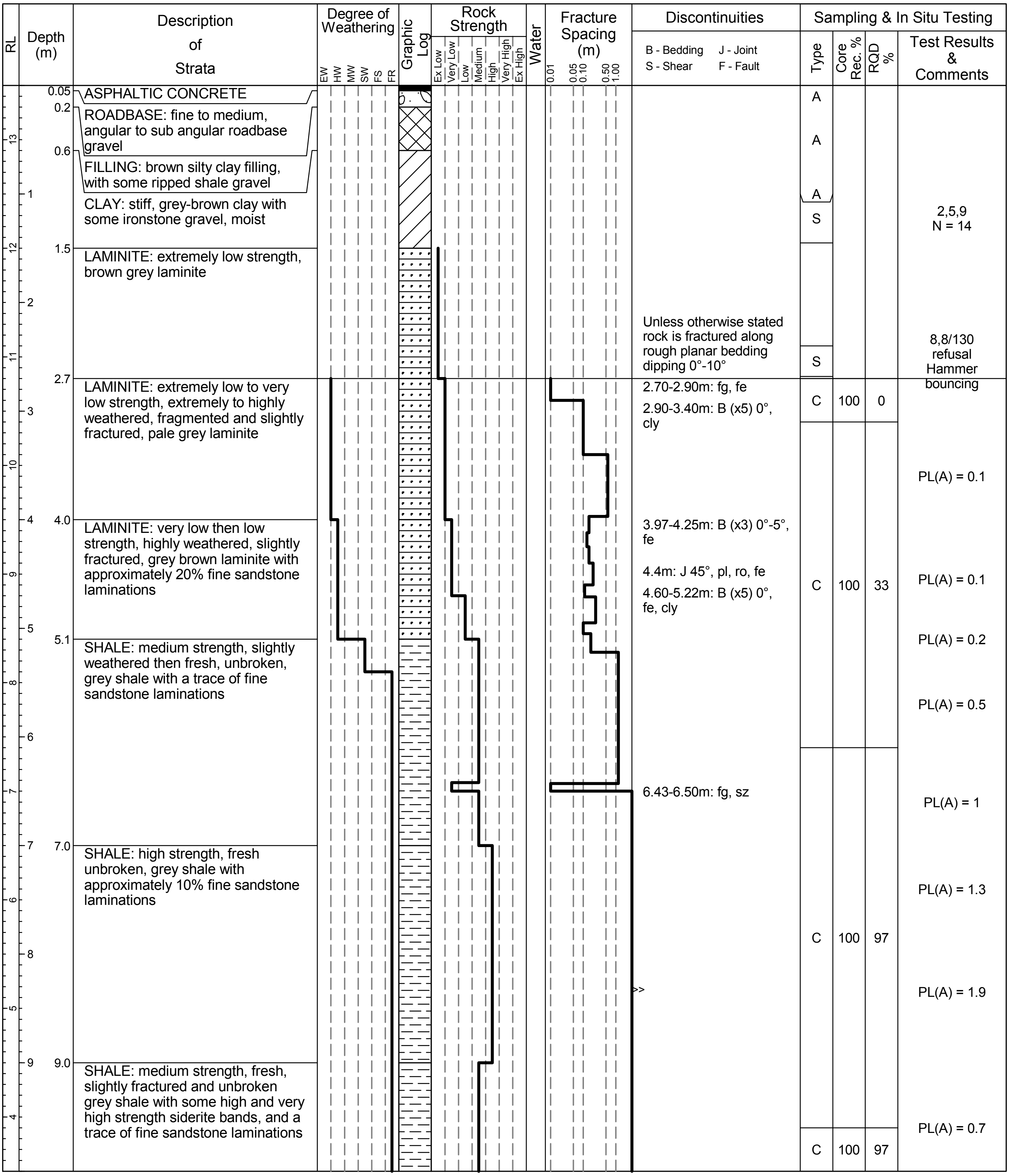
░	Granite
░	Dolerite, basalt, andesite
░	Dacite, epidote
□	Tuff, breccia
□	Porphyry

BOREHOLE LOG

CLIENT: Ecove Pty Ltd
PROJECT: Carpark P6D
LOCATION: Olympic Park, Sydney

SURFACE LEVEL: 13.5 AHD
EASTING: 321634.3
NORTHING: 6253297.4
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 86411.00
DATE: 28/8/2018
SHEET 1 OF 2



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
0.05	ASPHALTIC CONCRETE		A			
0.2	ROADBASE: fine to medium, angular to sub angular roadbase gravel		A			
0.6	FILLING: brown silty clay filling, with some ripped shale gravel		A			
	CLAY: stiff, grey-brown clay with some ironstone gravel, moist		S			2,5,9 N = 14
1.5	LAMINITE: extremely low strength, brown grey laminite	Unless otherwise stated rock is fractured along rough planar bedding dipping 0°-10°	S			8,8/130 refusal Hammer bouncing
2.7	LAMINITE: extremely low to very low strength, extremely to highly weathered, fragmented and slightly fractured, pale grey laminite	2.70-2.90m: fg, fe 2.90-3.40m: B (x5) 0°, cly	C	100	0	PL(A) = 0.1
4.0	LAMINITE: very low then low strength, highly weathered, slightly fractured, grey brown laminite with approximately 20% fine sandstone laminations	3.97-4.25m: B (x3) 0°-5°, fe 4.4m: J 45°, pl, ro, fe 4.60-5.22m: B (x5) 0°, fe, cly	C	100	33	PL(A) = 0.1 PL(A) = 0.2
5.1	SHALE: medium strength, slightly weathered then fresh, unbroken, grey shale with a trace of fine sandstone laminations	6.43-6.50m: fg, sz				PL(A) = 0.5 PL(A) = 1
7.0	SHALE: high strength, fresh unbroken, grey shale with approximately 10% fine sandstone laminations		C	100	97	PL(A) = 1.3 PL(A) = 1.9
9.0	SHALE: medium strength, fresh, slightly fractured and unbroken grey shale with some high and very high strength siderite bands, and a trace of fine sandstone laminations		C	100	97	PL(A) = 0.7

RIG: Scout 2 **DRILLER:** SS **LOGGED:** CE/SI **CASING:** HW to 2.5m
TYPE OF BORING: Solid flight auger (TC-bit) to 2.5m, Rotary (water) to 2.7m, NMLC-coring to 18.05m
WATER OBSERVATIONS: No free ground water observed whilst augering
REMARKS: 10% water loss at 10.15m

SAMPLING & IN SITU TESTING LEGEND

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

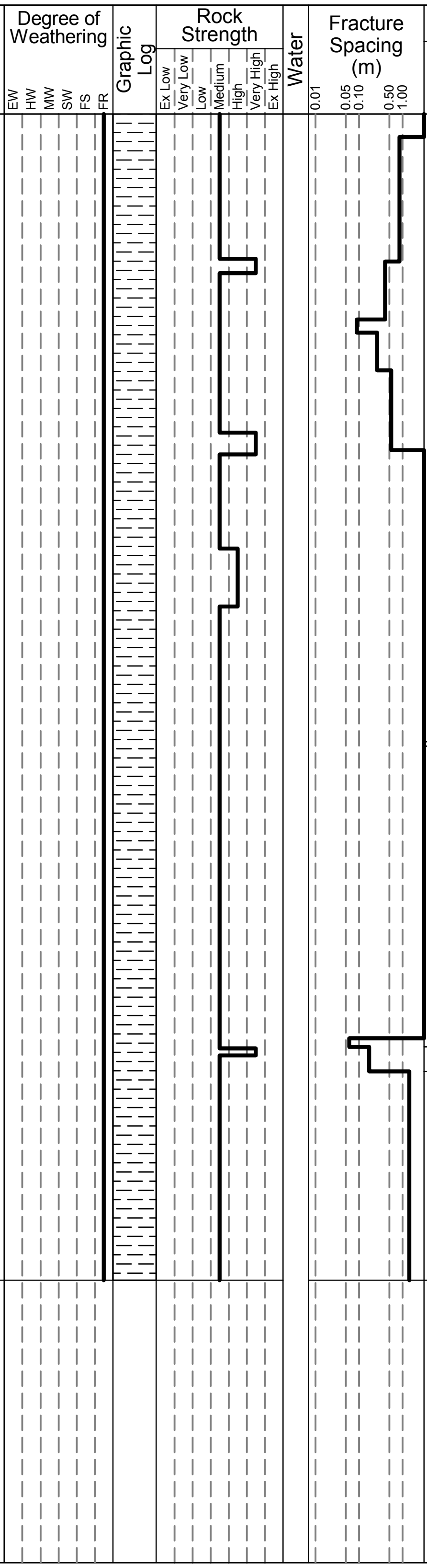
Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Ecove Pty Ltd
PROJECT: Carpark P6D
LOCATION: Olympic Park, Sydney

SURFACE LEVEL: 13.5 AHD
EASTING: 321634.3
NORTHING: 6253297.4
DIP/AZIMUTH: 90°/--

BORE No: 1
PROJECT No: 86411.00
DATE: 28/8/2018
SHEET 2 OF 2



RL	Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
		SHALE: medium strength, fresh, slightly fractured and unbroken grey shale with some high and very high strength siderite bands, and a trace of fine sandstone laminations *(continued)*					PL(A) = 0.6
			10.16m: J 45°, pl, sm, cln				
3	11		11.02-11.15m: J 70°, pl, sm, cln	C	100	97	
2			11.42m: J 45°, pl, sm, sz 30mm				PL(A) = 0.5
	12		11.77 & 12.43m: B 0°, fg, 5-10mm				
							PL(A) = 6.6
1							PL(A) = 0.6
	13						PL(A) = 0.3
0				C	100	100	
	14						PL(A) = 0.6
-1							
	15						
-2							PL(A) = 0.6
	16						PL(A) = 0.4
-3			16.38m: B 0°, cly 10mm 16.44m: J sv, un, ro, cln 16.61m: J sv, pl, ro, cln	C	100	98	
	17						PL(A) = 0.7
-4							
	18 18.05						PL(A) = 0.5
	18.05	Bore discontinued at 18.05m					
-5							
	19						
-6							

RIG: Scout 2 **DRILLER:** SS **LOGGED:** CE/SI **CASING:** HW to 2.5m

TYPE OF BORING: Solid flight auger (TC-bit) to 2.5m, Rotary (water) to 2.7m, NMLC-coring to 18.05m

WATER OBSERVATIONS: No free ground water observed whilst augering

REMARKS: 10% water loss at 10.15m

SAMPLING & IN SITU TESTING LEGEND

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

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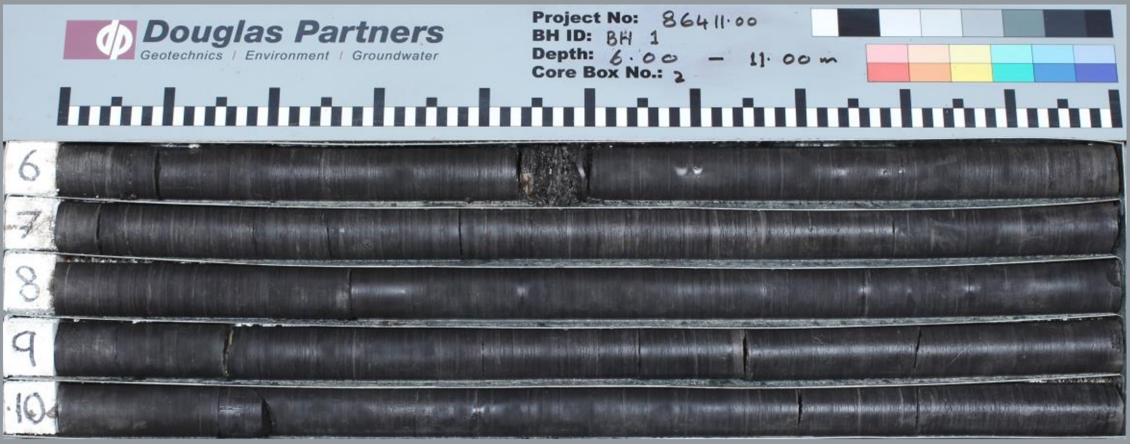
BORE: 1 PROJECT: SYDNEY OLYMPIC PARK AUGUST 2018

2.7 – 6.0m

BORE: 1 PROJECT: SYDNEY OLYMPIC PARK AUGUST 2018



6.0 = 11.0m

BORE: 1 PROJECT: SYDNEY OLYMPIC PARK AUGUST 2018

11.0 – 16.0m

BORE: 1 PROJECT: SYDNEY OLYMPIC PARK AUGUST 2018

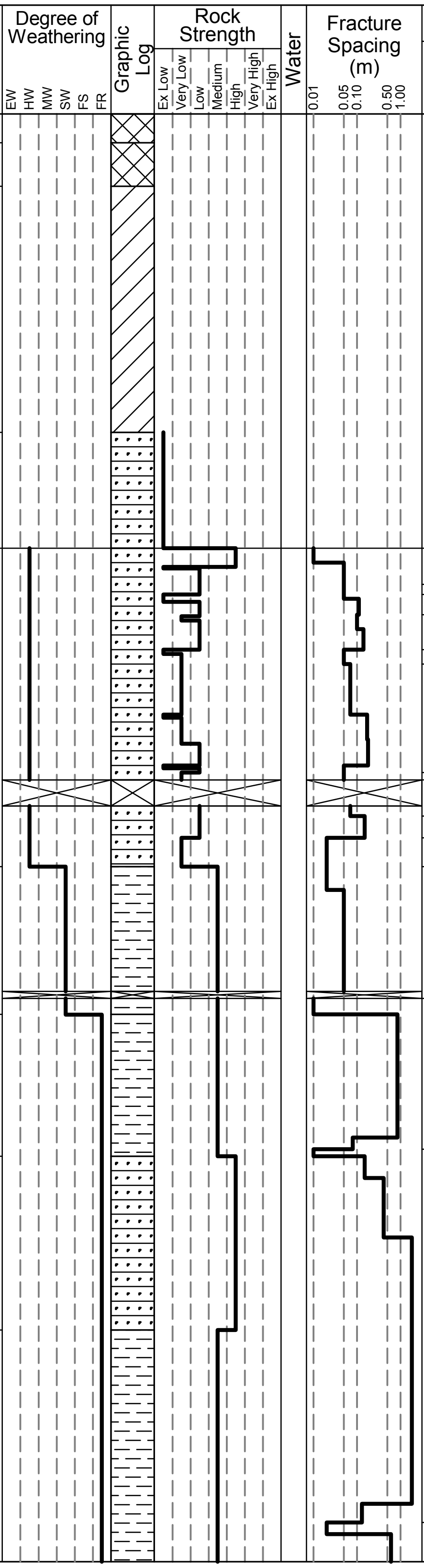
16.0 – 18.05m

BOREHOLE LOG

CLIENT: Ecove Pty Ltd
PROJECT: Carpark P6D
LOCATION: Olympic Park, Sydney

SURFACE LEVEL: 13.5 AHD
EASTING: 321663.8
NORTHING: 6253316.7
DIP/AZIMUTH: 90°/--

BORE No: 2
PROJECT No: 86411.00
DATE: 30/8/2018
SHEET 1 OF 2



RL	Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
		FILLING: grey, medium sand and roadbase gravel (sub-angular to angular) filling, moist		A			
13	0.2						
	0.5	FILLING: grey silty clay filling with a trace of fine sand, moist		A			
	1	CLAY: stiff to very stiff, mottled browand light grey clay, slightly silty with a trace of ironstone gravel, moist		A			
				S			6,7,9 N = 16
12							
	2						
11	2.2	LAMINITE: extremely low strength, light grey laminite	Unless otherwise stated rock is fractured along rough planar bedding dipping 0°-10°	S			15,20/50 refusal
	3						
10	3.0	LAMINITE: very low and low strength, highly wathered, fragmented to fractured, grey-brown laminite with approximately 20% fine sandstone laminations and some extremely low strength bands	3.00-3.10m: fg, fe 3.17m: J 60°, un, ro, cln 3.25m: J sv, pl, ro, fe 3.32-3.40m: Ds 3.46-3.49m: Ds 3.7m: B 0°, cly 15mm 3.80-4.15m: J 70°-80°, un, ro, fe 4.15-4.17m: Cs 4.32m: J 70°, un, ro, fe	C	100	40	PL(A) = 1.2
9	4		4.5m: B 0°, fe, cly 10mm 4.55-4.60m: Ds 4.6m: CORE LOSS: 180mm				PL(A) = 0.2
	4.78		4.85m: J 70°-90°, cu, ro, fe				
	5		5.00-5.20m: J 70°-90°, un, ro, fe 5.20-6.05m: B's 0°, fe, cly	C	88	10	PL(A) = 0.9
8	5.2	SHALE: medium strength, slightly weathered, fractured, grey-brown shale with some fine sandstone laminations	5.9m: J 70°, pl, ro, fe				
	6		6.06m: CORE LOSS: 50mm				
	6.11		6.11-6.22m: fg, fe				
7	6.22	SHALE: medium strength, fresh, slightly fratured grey shale					PL(A) = 0.8
	7		7.06m: B 0°, cly				
6	7.2	LAMINITE: high strength, fresh, slightly fractured then unbroken grey laminite with approximately 20% fine sandstone laminations	7.15-7.20m: fg 7.35m: B 0°, cly, fg 10mm 7.76m: J 85°, pl, ro, cln	C	98	92	PL(A) = 1.2
	8						
5	8.4	SHALE: medium strength, fresh, slightly fractured and unbroken grey shale with some very high strength siderite bands					PL(A) = 1.2
	9						
4			9.6m: J 60°, un, ro, cln 9.73-9.81m: J 80°, un, ro, cln	C	100	92	PL(A) = 0.9 PL(A) = 0.5

RIG: Scout 2 **DRILLER:** SS **LOGGED:** SI **CASING:** HW to 2.5m

TYPE OF BORING: Solid flight auger (TC-bit) to 2.5m, Rotary (water) to 3.0m, NMLC-coring to 17.60m

WATER OBSERVATIONS: No free ground water observed whilst augering

REMARKS: Standpipe installed (screen 5.6-17.6m, gravel 1.0-17.6m, bentonite 0.5-1.0m, backfilled to GL with flush gatic cover)

SAMPLING & IN SITU TESTING LEGEND

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

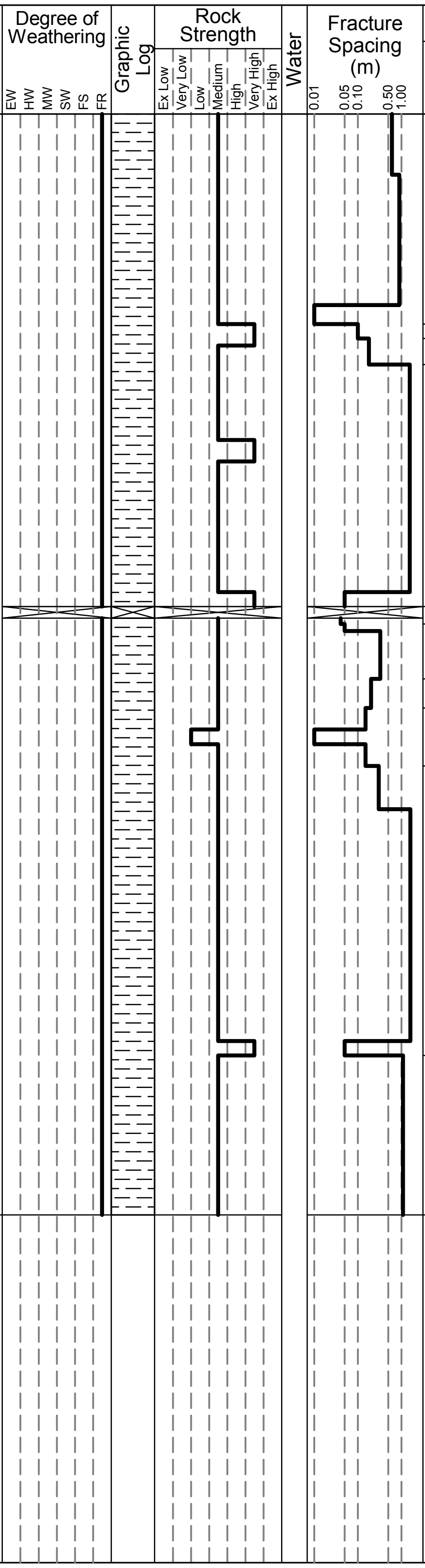
Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Ecove Pty Ltd
PROJECT: Carpark P6D
LOCATION: Olympic Park, Sydney

SURFACE LEVEL: 13.5 AHD
EASTING: 321663.8
NORTHING: 6253316.7
DIP/AZIMUTH: 90°/--

BORE No: 2
PROJECT No: 86411.00
DATE: 30/8/2018
SHEET 2 OF 2



RL	Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
3		SHALE: medium strength, fresh, slightly fractured and unbroken grey shale with some very high strength siderite bands *(continued)*	10.42m: J 70°, un, ro, cln				PL(A) = 0.5
	11			C	100	92	
2			11.32m: J 60°, un, ro, cln 11.45m: J 80°, pl, ro, cln 11.55m: J 70°, un, ro, sz 20mm 11.73m: J 85°, un, ro, cln				PL(A) = 4.3 PL(A) = 0.8
	12						PL(A) = 4.3
1							PL(A) = 0.6
	13						PL(A) = 0.6
0	13.48		13.3m: J 90°, pl, sm, cln 13.4m: CORE LOSS: 80mm 13.52m: J 90°&45°, st, sm, fg 10mm 13.9m: J 45°, pl, sm, cln 14.1m: J 70°, un, ro, cln 14.25-14.35m: fg, cly, Sz 100mm 14.5m: J 70°-90°, cu, ro, cln 14.8m: J 90°, pl, ro, cln	C	97	88	PL(A) = 0.5
	14						
-1							
	15						PL(A) = 1
-2							
	16						PL(A) = 0.9
-3			16.4m: B 0°, cly 10mm 16.5m: siderite band	C	100	97	PL(A) = 6.8 PL(A) = 0.9
	17						
-4							PL(A) = 0.8
	17.6	Bore discontinued at 17.6m					
	18						
-5							
	19						
-6							

RIG: Scout 2 **DRILLER:** SS **LOGGED:** SI **CASING:** HW to 2.5m
TYPE OF BORING: Solid flight auger (TC-bit) to 2.5m, Rotary (water) to 3.0m, NMLC-coring to 17.60m
WATER OBSERVATIONS: No free ground water observed whilst augering
REMARKS: Standpipe installed (screen 5.6-17.6m, gravel 1.0-17.6m, bentonite 0.5-1.0m, backfilled to GL with flush gatic cover)

SAMPLING & IN SITU TESTING LEGEND

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

Douglas Partners
Geotechnics / Environment / Groundwater

BORE: 2 PROJECT: SYDNEY OLYMPIC PARK AUGUST 2018

3.0 – 7.0m

BORE: 2 PROJECT: SYDNEY OLYMPIC PARK AUGUST 2018

7.0 – 12.0m

BORE: 2 PROJECT: SYDNEY OLYMPIC PARK AUGUST 2018

12.0 – 17.0m

BORE: 2 PROJECT: SYDNEY OLYMPIC PARK AUGUST 2018

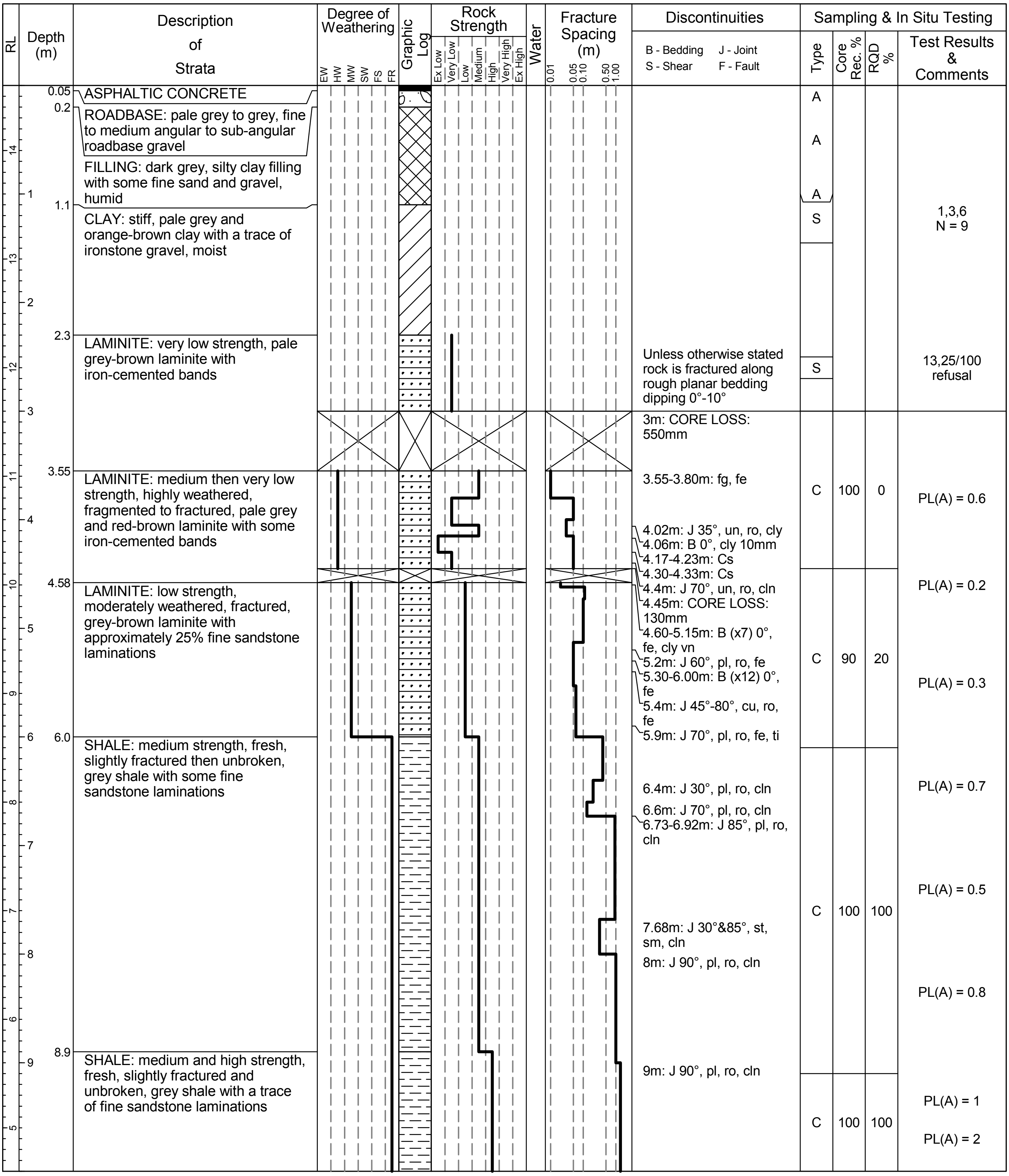
17.0 – 17.6m

BOREHOLE LOG

CLIENT: Ecove Pty Ltd
PROJECT: Carpark P6D
LOCATION: Olympic Park, Sydney

SURFACE LEVEL: 14.6 AHD
EASTING: 321625.4
NORTHING: 6253268.1
DIP/AZIMUTH: 90°/--

BORE No: 3
PROJECT No: 86411.00
DATE: 29/8/2018
SHEET 1 OF 2



Depth (m)	Description of Strata
0.05	ASPHALTIC CONCRETE
0.2	ROADBASE: pale grey to grey, fine to medium angular to sub-angular roadbase gravel
	FILLING: dark grey, silty clay filling with some fine sand and gravel, humid
1.1	CLAY: stiff, pale grey and orange-brown clay with a trace of ironstone gravel, moist
2.3	LAMINITE: very low strength, pale grey-brown laminite with iron-cemented bands
3.55	LAMINITE: medium then very low strength, highly weathered, fragmented to fractured, pale grey and red-brown laminite with some iron-cemented bands
4.58	LAMINITE: low strength, moderately weathered, fractured, grey-brown laminite with approximately 25% fine sandstone laminations
6.0	SHALE: medium strength, fresh, slightly fractured then unbroken, grey shale with some fine sandstone laminations
8.9	SHALE: medium and high strength, fresh, slightly fractured and unbroken, grey shale with a trace of fine sandstone laminations

Discontinuities
Unless otherwise stated rock is fractured along rough planar bedding dipping 0°-10°
3m: CORE LOSS: 550mm
3.55-3.80m: fg, fe
4.02m: J 35°, un, ro, cly
4.06m: B 0°, cly 10mm
4.17-4.23m: Cs
4.30-4.33m: Cs
4.4m: J 70°, un, ro, cln
4.45m: CORE LOSS: 130mm
4.60-5.15m: B (x7) 0°, fe, cly vn
5.2m: J 60°, pl, ro, fe
5.30-6.00m: B (x12) 0°, fe
5.4m: J 45°-80°, cu, ro, fe
5.9m: J 70°, pl, ro, fe, ti
6.4m: J 30°, pl, ro, cln
6.6m: J 70°, pl, ro, cln
6.73-6.92m: J 85°, pl, ro, cln
7.68m: J 30°&85°, st, sm, cln
8m: J 90°, pl, ro, cln
9m: J 90°, pl, ro, cln

Sampling & In Situ Testing			
Type	Core Rec. %	RQD %	Test Results & Comments
A			
A			
A			
S			1,3,6 N = 9
S			13,25/100 refusal
C	100	0	PL(A) = 0.6
C	90	20	PL(A) = 0.2 PL(A) = 0.3
C	100	100	PL(A) = 0.7 PL(A) = 0.5 PL(A) = 0.8
C	100	100	PL(A) = 1 PL(A) = 2

RIG: Scout 2 **DRILLER:** SS **LOGGED:** SI **CASING:** HW to 2.5m

TYPE OF BORING: Solid flight auger (TC-bit) to 2.5m, Rotary (water) to 3.0m, NMLC-coring to 19.15m

WATER OBSERVATIONS: No free ground water observed whilst augering

REMARKS: Standpipe installed (screen: 4.5-19.15m, gravel: 1.0-19.15m, bentonite: 0.5-1.0m, backfill to GL with flush gatic cover)

SAMPLING & IN SITU TESTING LEGEND

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

Douglas Partners
Geotechnics / Environment / Groundwater

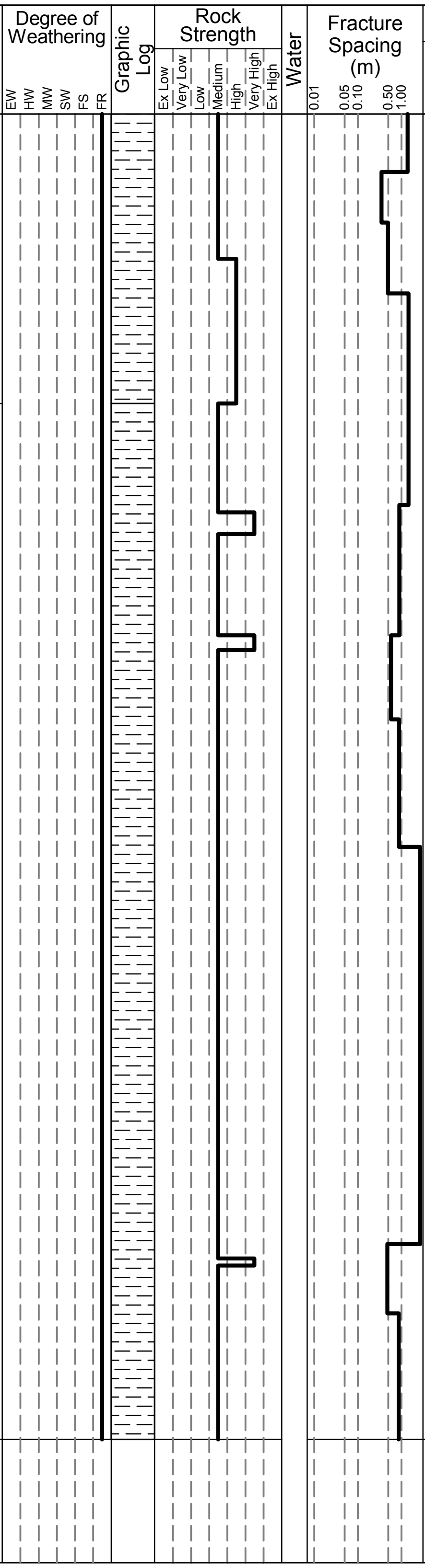
BOREHOLE LOG

CLIENT: Ecove Pty Ltd
PROJECT: Carpark P6D
LOCATION: Olympic Park, Sydney

SURFACE LEVEL: 14.6 AHD
EASTING: 321625.4
NORTHING: 6253268.1
DIP/AZIMUTH: 90°/--

BORE No: 3
PROJECT No: 86411.00
DATE: 29/8/2018
SHEET 2 OF 2

Depth (m)	Description of Strata
	SHALE: medium and high strength, fresh, slightly fractured and unbroken, grey shale with a trace of fine sandstone laminations *(continued)*
12.0	SHALE: medium strength, fresh, slightly fractured and unbroken, grey shale with some very high strength siderite bands
19.15	Bore discontinued at 19.15m



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

- 10.4m: J 45°, pl, ro, cln
- 10.75m: J 60°, pl, ro, cln
- 11.24m: B 10°, ca 1mm
- 12.70-12.90m: J 90°, pl, ro, cln
- 13.60-13.70m: siderite band
- 14.18-14.30m: J 80°, pl, ro, cln
- 15.06m: J 45°-80° cu, he/ti
- 17.8m: J 45°, pl, ro, cln
- 18.28m: J 20°, pl, ro, cln, siderite 20mm

Sampling & In Situ Testing

Type	Core Rec. %	RQD %
C	100	100
C	100	100
C	100	100
C	100	100

Test Results & Comments

- PL(A) = 0.8
- PL(A) = 1
- PL(A) = 0.4
- PL(A) = 3.9
- PL(A) = 0.7
- PL(A) = 3.9
- PL(A) = 0.8
- PL(A) = 0.7
- PL(A) = 0.7
- PL(A) = 0.6
- PL(A) = 9
- PL(A) = 0.8

RIG: Scout 2 **DRILLER:** SS **LOGGED:** SI **CASING:** HW to 2.5m
TYPE OF BORING: Solid flight auger (TC-bit) to 2.5m, Rotary (water) to 3.0m, NMLC-coring to 19.15m
WATER OBSERVATIONS: No free ground water observed whilst augering
REMARKS: Standpipe installed (screen: 4.5-19.15m, gravel: 1.0-19.15m, bentonite: 0.5-1.0m, backfill to GL with flush gatic cover)

SAMPLING & IN SITU TESTING LEGEND

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

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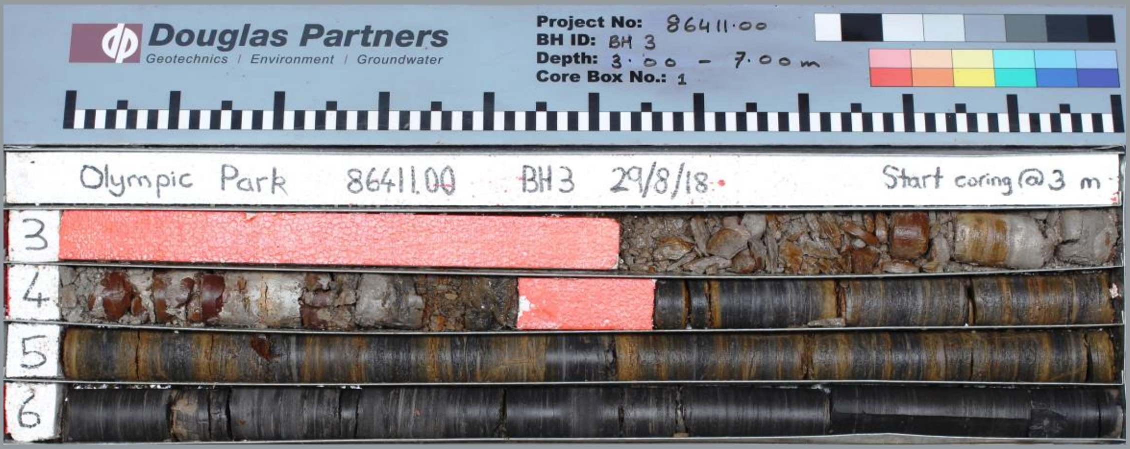
BORE: 3 PROJECT: SYDNEY OLYMPIC PARK AUGUST 2018



3.0 – 7.0m

BORE: 3 PROJECT: SYDNEY OLYMPIC PARK AUGUST 2018

7.0 – 12.0m

BORE: 3 PROJECT: SYDNEY OLYMPIC PARK AUGUST 2018

12.0 – 17.0m

BORE: 3 PROJECT: SYDNEY OLYMPIC PARK AUGUST 2018

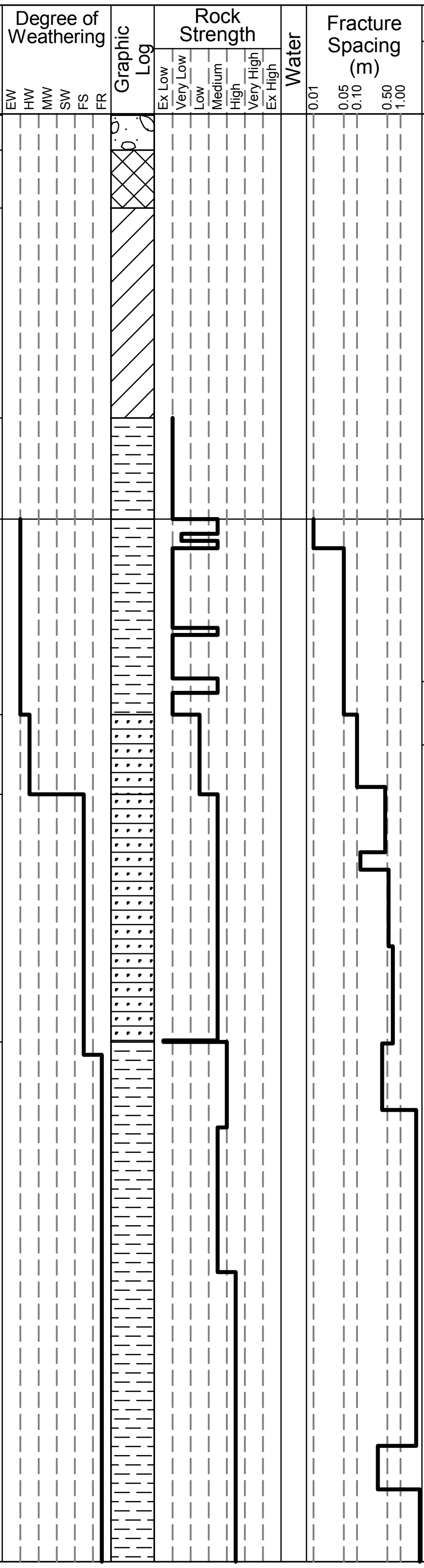
17.0 – 19.15m

BOREHOLE LOG

CLIENT: Ecove Pty Ltd
PROJECT: Carpark P6D
LOCATION: Olympic Park, Sydney

SURFACE LEVEL: 14.9 AHD
EASTING: 321682.4
NORTHING: 6253276.5
DIP/AZIMUTH: 90°/--

BORE No: 4
PROJECT No: 86411.00
DATE: 27/8/2018
SHEET 1 OF 2



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
0.01	ASPHALTIC CONCRETE		A			
0.25	ROADBASE: grey, angular and sub-angular medium roadbase		A			
0.65	FILLING: light grey-brown sandy clay filling, moist					
	CLAY: very stiff, grey-brown clay with some ironstone gravel, moist		A S			5,8,10 N = 18
2.1	SHALE: extremely to very low strength, grey-brown shale	Unless otherwise stated rock is fractured along rough planar bedding dipping 0°-10°	S			25/130 refusal
2.8	SHALE: extremely low to very low strength, extremely to highly weathered, fragmented to fractured, pale grey-brown shale with some medium strength iron-cemented bands	2.80-3.00m: fg, fe, cly 3.00-4.15m: B's 0°-5°, fe, cly, 10-50mm 3.65m: J 80°, un, ro, fe, cly 3.92m: J sv, pl, ro, fe	C	100	56	PL(A) = 0.7
4.15	LAMINITE: low strength, highly to moderately weathered, fractured, grey-brown laminite with approximately 25% fine sandstone laminations	4.15-4.65m: B (x5) 0°, fe, cly co, 1-5mm 4.36m: J 80°, he/fe				PL(A) = 0.2
4.7	LAMINITE: medium strength, fresh stained, slightly fractured, grey-brown laminite with approximately 25% fine sandstone laminations	5.10-5.75m: B (x3) 0°, fe, cly				PL(A) = 0.7
6.41	SHALE: medium and high strength, fresh,slightly fractured and unbroken, dark grey shale with some very high strength siderite bands	6.42m: B 0°, fe, cly, 5mm 6.88m: J 45°-70°, cu, ro, cly	C	100	99	PL(A) = 2.6 PL(A) = 0.8 PL(A) = 1.2 PL(A) = 1.3
		9.20-9.35m: J 80°, pl, ro, cln 9.50-9.80m: J 80°, un, ro, cln	C	100	100	

RIG: Scout 2 **DRILLER:** SS **LOGGED:** CE/SI **CASING:** HW to 2.8m

TYPE OF BORING: Solid flight auger (TC-bit) to 2.5m, Rotary (water) to 2.8m, NMLC-coring to 19.20m

WATER OBSERVATIONS: No free ground water observed whilst augering

REMARKS: 40% water loss from 4.9m

SAMPLING & IN SITU TESTING LEGEND

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

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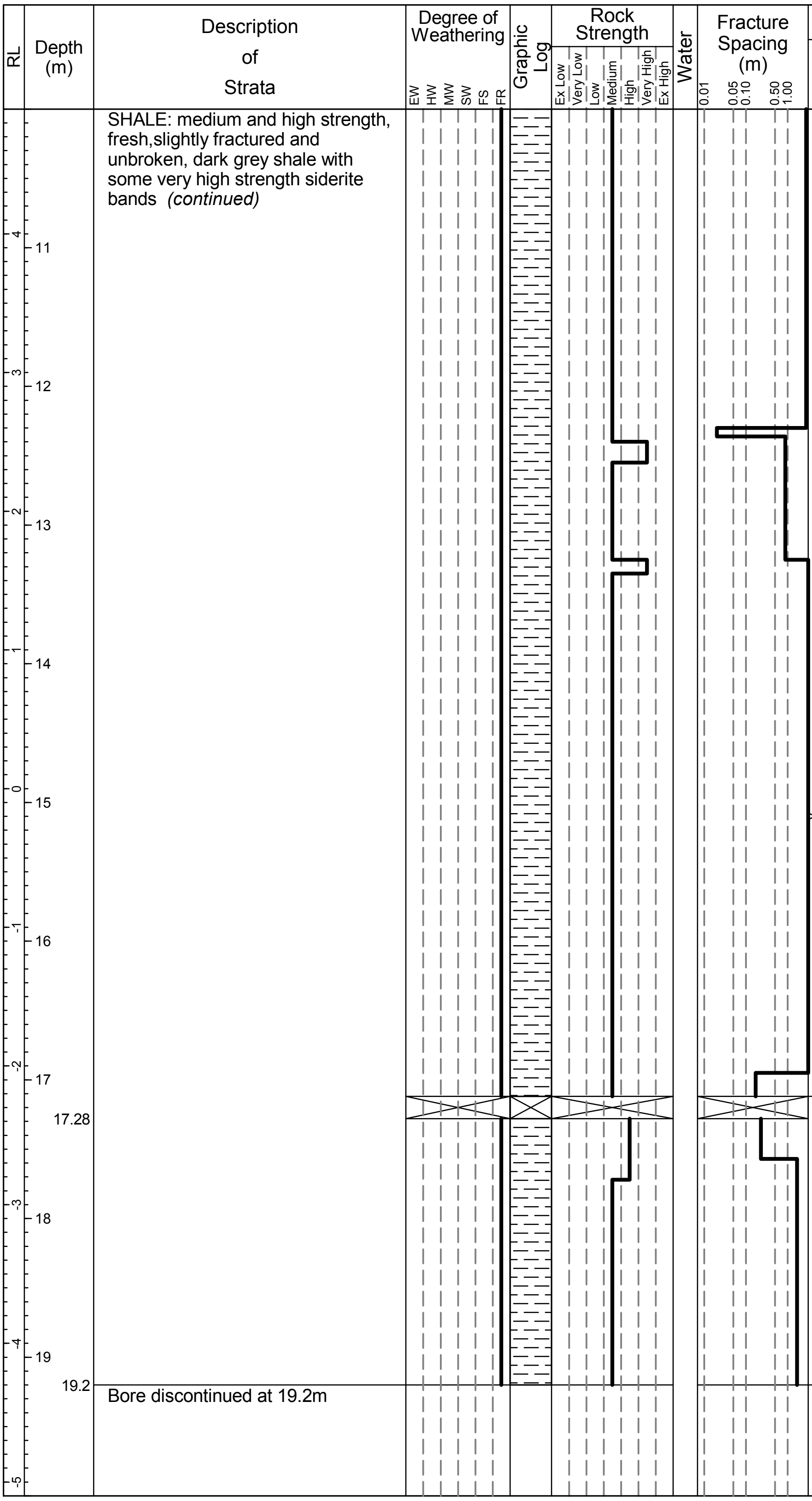
BOREHOLE LOG

CLIENT: Ecove Pty Ltd
PROJECT: Carpark P6D
LOCATION: Olympic Park, Sydney

SURFACE LEVEL: 14.9 AHD
EASTING: 321682.4
NORTHING: 6253276.5
DIP/AZIMUTH: 90°/--

BORE No: 4
PROJECT No: 86411.00
DATE: 27/8/2018
SHEET 2 OF 2

Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
	SHALE: medium and high strength, fresh,slightly fractured and unbroken, dark grey shale with some very high strength siderite bands *(continued)*					PL(A) = 0.7
			C	100	100	
						PL(A) = 0.7
		12.36m: J 60°, pl, sm, cln, sz 40mm				PL(A) = 5.4
						PL(A) = 0.5
		13.25-13.33m: siderite band	C	100	99	PL(A) = 4.4
						PL(A) = 0.9
						PL(A) = 0.6
			C	95	95	
						PL(A) = 0.4
		16.95-17.12m: J 85°, pl, ro, cln				
17.28		17.12m: CORE LOSS: 160mm				PL(A) = 1.2
		17.51m: J 35°, pl, ro, cln				
						PL(A) = 0.7
			C	100	100	
						PL(A) = 0.9
						PL(A) = 0.6
19.2	Bore discontinued at 19.2m					



RIG: Scout 2 **DRILLER:** SS **LOGGED:** CE/SI **CASING:** HW to 2.8m
TYPE OF BORING: Solid flight auger (TC-bit) to 2.5m, Rotary (water) to 2.8m, NMLC-coring to 19.20m
WATER OBSERVATIONS: No free ground water observed whilst augering
REMARKS: 40% water loss from 4.9m

SAMPLING & IN SITU TESTING LEGEND

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

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BORE: 4 PROJECT: SYDNEY OLYMPIC PARK AUGUST 2018

2.8 – 6.0m

BORE: 4 PROJECT: SYDNEY OLYMPIC PARK AUGUST 2018

6.0 -11.0m

BORE: 4 PROJECT: SYDNEY OLYMPIC PARK AUGUST 2018

11.0 – 16.0m

BORE: 4 PROJECT: SYDNEY OLYMPIC PARK AUGUST 2018

16.0 – 19.2m

BOREHOLE LOG

CLIENT: Ecove Pty Ltd
PROJECT: Carpark P6D
LOCATION: Olympic Park, Sydney

SURFACE LEVEL: 16.6 AHD
EASTING: 321644.5
NORTHING: 6253217.3
DIP/AZIMUTH: 90°/--

BORE No: 5
PROJECT No: 86411.00
DATE: 3/9/2018
SHEET 1 OF 3

Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
0.02	ASPHALTIC CONCRETE		A			
	FILLING: variably compacted grey and light grey-brown sand and ripped sandstone gravel filling, damp		A A S			15,19,25/100 refusal
1.5	FILLING: variably compacted grey to grey-brown sandy clay and ripped shale fragments filling, damp to moist	Unless otherwise stated rock is fractured along rough planar bedding dipping 0°-10°	S			13,18,18 N = 36
4.0	FILLING: light grey and brown, ripped shale fragments filling with clay	4m: CORE LOSS: 1400mm	C	33	0	
5.4						
6.4		6.1m: CORE LOSS: 300mm				
6.5	LAMINITE: high strength, fresh, unbroken grey laminite with approximately 20% fine sandstone laminations		C	90	87	PL(A) = 1.8 PL(A) = 1.2 PL(A) = 1.3
9.1	SHALE: medium and high strength, fresh, unbroken grey shale with a trace of fine sandstone laminations	9.1m: J 45°, pl, sm, cln	C	100	100	PL(A) = 0.9

RIG: Scout 2 **DRILLER:** SS **LOGGED:** SI **CASING:** HW to 2.5m
TYPE OF BORING: Solid flight auger (TC-bit) to 2.5m, Rotary (water) to 4.0m, NMLC-coring to 20.85m
WATER OBSERVATIONS: No free ground water observed whilst augering
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_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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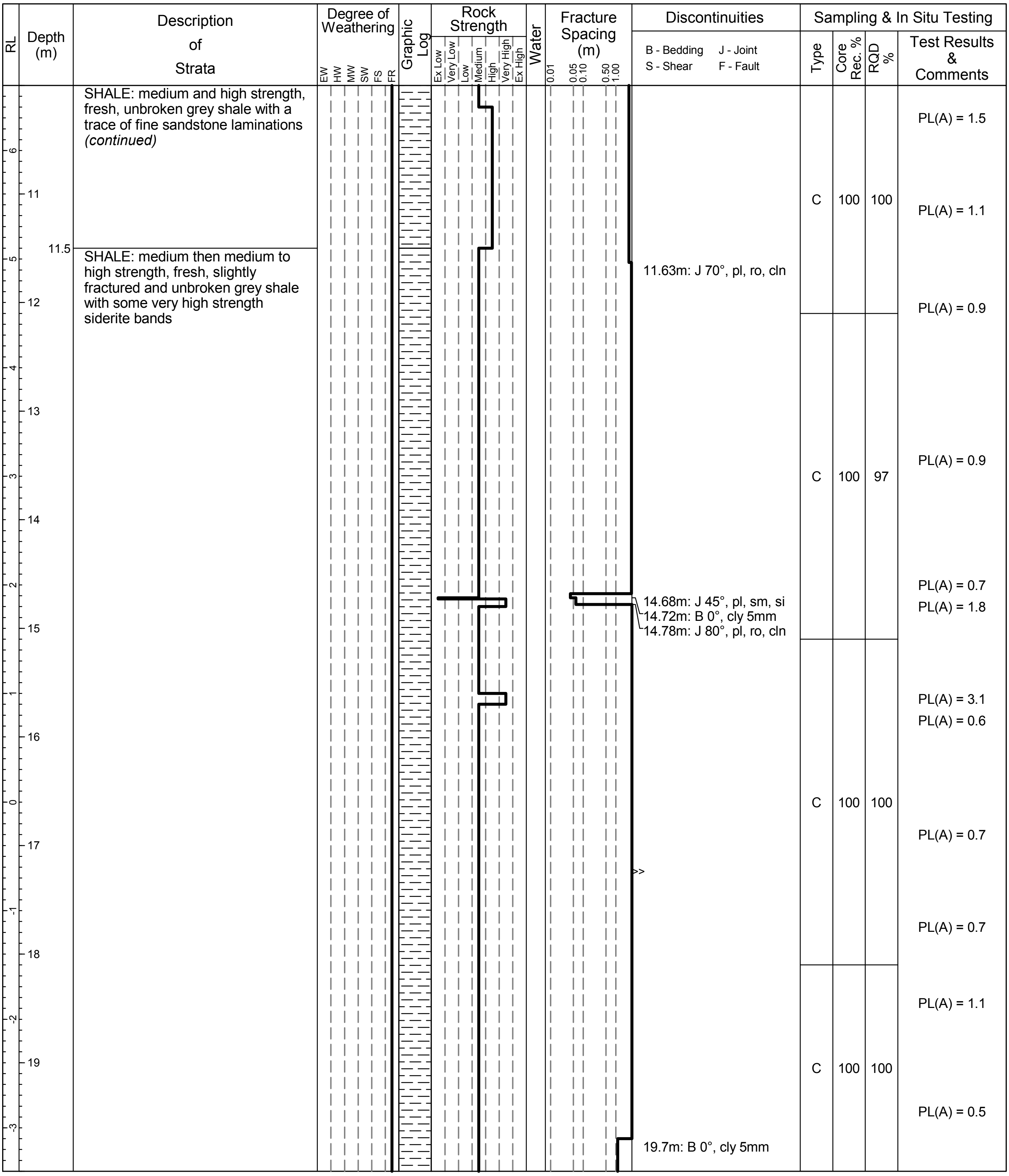
BOREHOLE LOG

CLIENT: Ecove Pty Ltd
PROJECT: Carpark P6D
LOCATION: Olympic Park, Sydney

SURFACE LEVEL: 16.6 AHD
EASTING: 321644.5
NORTHING: 6253217.3
DIP/AZIMUTH: 90°/--

BORE No: 5
PROJECT No: 86411.00
DATE: 3/9/2018
SHEET 2 OF 3



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
	SHALE: medium and high strength, fresh, unbroken grey shale with a trace of fine sandstone laminations *(continued)*		C	100	100	PL(A) = 1.5 PL(A) = 1.1
11.5	SHALE: medium then medium to high strength, fresh, slightly fractured and unbroken grey shale with some very high strength siderite bands	11.63m: J 70°, pl, ro, cln				PL(A) = 0.9
		14.68m: J 45°, pl, sm, si 14.72m: B 0°, cly 5mm 14.78m: J 80°, pl, ro, cln	C	100	97	PL(A) = 0.9 PL(A) = 0.7 PL(A) = 1.8
		>>	C	100	100	PL(A) = 3.1 PL(A) = 0.6 PL(A) = 0.7 PL(A) = 0.7
		19.7m: B 0°, cly 5mm	C	100	100	PL(A) = 1.1 PL(A) = 0.5

RIG: Scout 2 **DRILLER:** SS **LOGGED:** SI **CASING:** HW to 2.5m
TYPE OF BORING: Solid flight auger (TC-bit) to 2.5m, Rotary (water) to 4.0m, NMLC-coring to 20.85m
WATER OBSERVATIONS: No free ground water observed whilst augering
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_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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BOREHOLE LOG

CLIENT: Ecove Pty Ltd
PROJECT: Carpark P6D
LOCATION: Olympic Park, Sydney

SURFACE LEVEL: 16.6 AHD
EASTING: 321644.5
NORTHING: 6253217.3
DIP/AZIMUTH: 90°/--

BORE No: 5
PROJECT No: 86411.00
DATE: 3/9/2018
SHEET 3 OF 3

RL	Depth (m)	Description of Strata	Degree of Weathering EW HW MW SW FS FR	Graphic Log	Rock Strength Ex Low Very Low Low Medium High Very High Ex High	Water	Fracture Spacing (m) 0.01 0.05 0.10 0.50 1.00	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
-4		SHALE: medium then medium to high strength, fresh, slightly fractured and unbroken grey shale with some very high strength siderite bands *(continued)*							C	100	100	PL(A) = 1.1
	20.85	Bore discontinued at 20.85m										
	21											
-5												
	22											
-6												
	23											
-7												
	24											
-8												
	25											
-9												
	26											
-10												
	27											
-11												
	28											
-12												
	29											
-13												

RIG: Scout 2 **DRILLER:** SS **LOGGED:** SI **CASING:** HW to 2.5m
TYPE OF BORING: Solid flight auger (TC-bit) to 2.5m, Rotary (water) to 4.0m, NMLC-coring to 20.85m
WATER OBSERVATIONS: No free ground water observed whilst augering
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_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

BORE: 5 PROJECT: SYDNEY OLYMPIC PARK AUGUST 2018

4.0 – 8.0m

BORE: 5 PROJECT: SYDNEY OLYMPIC PARK AUGUST 2018

8.0 – 13.0m

BORE: 5 **PROJECT: SYDNEY OLYMPIC PARK** **AUGUST 2018**

13.0 – 18.0m

BORE: 5 **PROJECT: SYDNEY OLYMPIC PARK** **AUGUST 2018**

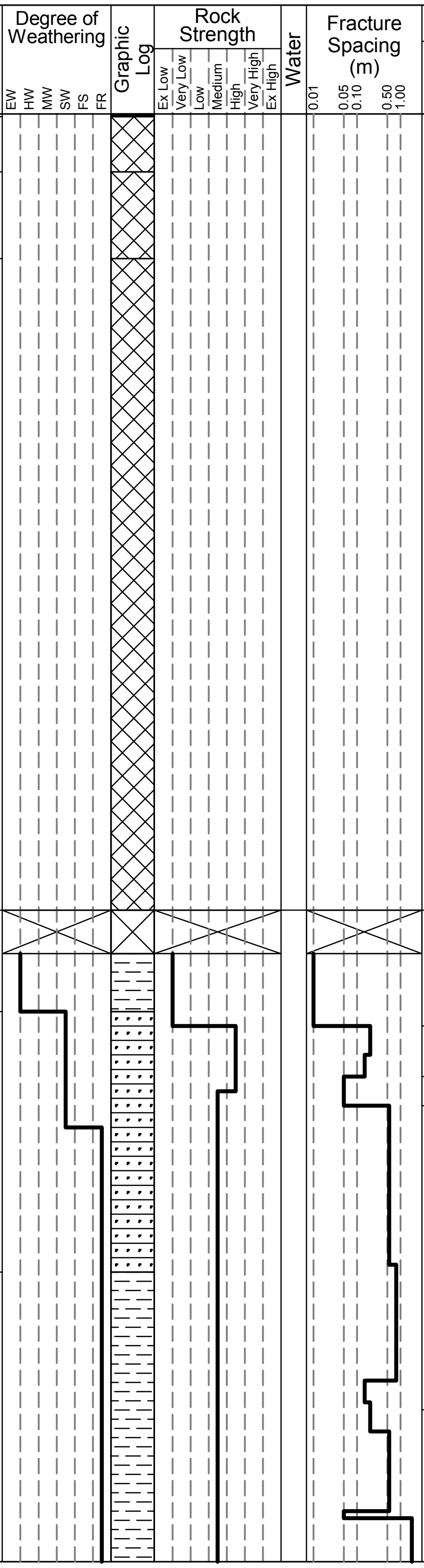
18.0 – 20.85m

BOREHOLE LOG

CLIENT: Ecove Pty Ltd
PROJECT: Carpark P6D
LOCATION: Olympic Park, Sydney

SURFACE LEVEL: 14.9 AHD
EASTING: 321699.1
NORTHING: 6253240.6
DIP/AZIMUTH: 90°/--

BORE No: 6
PROJECT No: 86411.00
DATE: 5/9/2018
SHEET 1 OF 2



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
0.02	ASPHALTIC CONCRETE		A			
0.4	FILLING: grey silty sand filling with ripped sandstone		A			
	FILLING: grey ripped shale filling		A			
1.0	FILLING: variably compacted, light grey to grey and grey-brown silty clay and ripped shale filling		S			6,18,23 N = 41
			S			6,13,15 N = 28
			S			5,7,8 N = 15
5.5 5.8	SHALE: extremely to very low strength, extremely to highly weathered, fragmented, grey-brown shale with some medium strength bands	5.5m: CORE LOSS: 300mm 5.80-6.20m: fg				
6.2	LAMINITE: high then medium strength, slightly weathered then fresh, slightly fractured, grey-brown laminite with approximately 20% fine sandstone laminations	6.2m: J 45°,pl, sm, fe 6.3m: J 45°,pl, sm, fe 6.5m: J 70°&80°, st, sm, cln 6.65-6.85m: B (x4) 0°, fe 6.85-6.94m: J 80°, pl, ro, fe	C	83	42	PL(A) = 1.1 PL(A) = 1.5
		7.50m: siltstone clast				PL(A) = 0.9
8.0	SHALE: medium strength, fresh, slightly fractured and unbroken, grey shale with some fine grained sandstone laminations	7.95m: J 70°&85°, st, ro, cln	C	100	98	PL(A) = 0.6
		8.9m: J 45°, un, ro, cln 8.95m: B 0°, cly co 2mm				
		9.65-9.70m: J (x2) 80°, ti				PL(A) = 0.6

RL: 14, 13, 12, 11, 10, 9, 8, 7, 6, 5
Depth: 1, 2, 3, 4, 5, 6, 7, 8, 9

RIG: Scout 2 **DRILLER:** SS **LOGGED:** SI **CASING:** HW to 2.5m, HQ to 5.5m
TYPE OF BORING: Solid flight auger (TC-bit) to 2.5m, Rotary (water) to 5.5m, NMLC-coring to 19.1m
WATER OBSERVATIONS: No free ground water observed whilst augering
REMARKS: Standpipe installed (screen: 6.0-19.1m, gravel: 5.0-19.1m, bentonite: 4.0-5.0m, backfill to GL with flush gatic cover)

SAMPLING & IN SITU TESTING LEGEND

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

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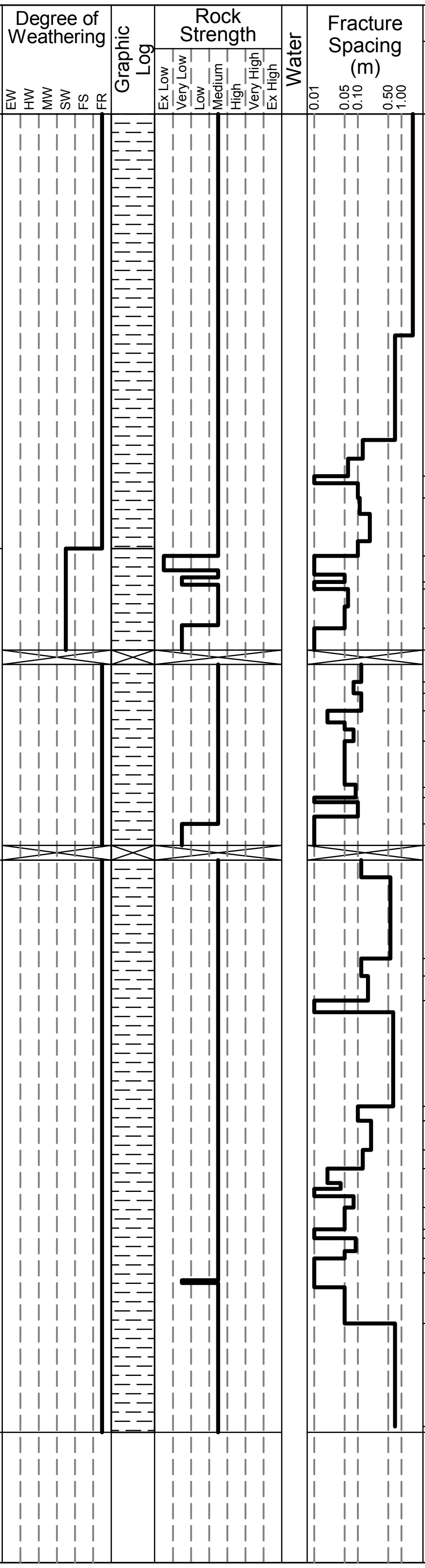
BOREHOLE LOG

CLIENT: Ecove Pty Ltd		**SURFACE LEVEL:** 14.9 AHD	**BORE No:** 6
PROJECT: Carpark P6D		**EASTING:** 321699.1	**PROJECT No:** 86411.00
LOCATION: Olympic Park, Sydney		**NORTHING:** 6253240.6	**DATE:** 5/9/2018
		DIP/AZIMUTH: 90°/--	**SHEET** 2 OF 2



RL	Depth (m)	Description of Strata
4	11	
3	12	
2	13	
	(top)	SHALE: medium strength, fresh, slightly fractured and unbroken, grey shale with some fine grained sandstone laminations *(continued)*
	13.0	SHALE: medium strength, fresh, fragmented to fractured grey shale with some very low strength bands
	13.8	
1	14	
0	15	
	15.15	SHALE: medium strength, fresh, fractured and slightly fractured grey shale with a trace of fine sandstone laminations
-1	16	
-2	17	
-3	18	
-4	19	
	19.1	Bore discontinued at 19.1m
-5		

Discontinuities

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

11.53m: J 70°, pl, ro, cln
12.25-12.50m: J (x4) 35°-45°, pl, sm, cln
12.50-12.55m: J (x4) 45°-65°, pl, sm, cln
12.65m: J 80°, to
12.95m: J 85°, pl, sm, cln
13.05-13.18m: fg, cly, Sz 130mm
13.23m: J 70°, pl, ro, fg 5mm
13.28-13.55m: J (x3) 60°-70°, un, ro, cln
13.55-13.60m: fg, cly Sz 50mm
13.7m: CORE LOSS: 100mm
13.92m: J 45°&80°, st, sm, cln
14m: J 50°, pl, sm, cln
14.12-14.20m: J (x4) 35°-45°, pl, sm, cln
14.33-14.63m: J (x4) 45°-50°, pl, ro, cln
14.65m: J 70°, pl, ro, cln
14.72-14.75m: J 45°, pl, ro, fg 5mm
14.90-15.05m: fg, cly, Sz 100mm
15.05m: CORE LOSS: 100mm
15.83m: J 45°-70°, cu, ro, cln
15.95m: J 60°, pl, sm, cln
16.12-16.20m: fg
16.85m: J (x2) 70°&80°, un, ro, cln
16.95m: J 45°, pl, sm, cln
17.15-17.25m: J 70°&80°, st, sm, cln
17.28-17.30m: J 45°, Sz 20mm
17.55m: J 30°, pl, sm, fg 50mm
17.70-17.76m: fg
17.76m: J 70°, un, ro, cln
17.9m: J 70°, pl, sm, fg 100mm
18.00-18.30m: J (x2) 85°, pl, ro, fg
18.35m: J 85°, pl, sm, cln
19.06m: J 45°, pl, sm, cln

Sampling & In Situ Testing

Type	Core Rec. %	RQD %	Test Results & Comments
C	100	98	PL(A) = 0.8
C	100	94	PL(A) = 0.6 PL(A) = 0.6
C	95	20	PL(A) = 0.6 PL(A) = 0.3
C	90	75	PL(A) = 0.5
C	100	49	PL(A) = 0.6 PL(A) = 0.5
C	100	90	PL(A) = 0.6

RIG: Scout 2 **DRILLER:** SS **LOGGED:** SI **CASING:** HW to 2.5m, HQ to 5.5m

TYPE OF BORING: Solid flight auger (TC-bit) to 2.5m, Rotary (water) to 5.5m, NMLC-coring to 19.1m

WATER OBSERVATIONS: No free ground water observed whilst augering

REMARKS: Standpipe installed (screen: 6.0-19.1m, gravel: 5.0-19.1m, bentonite: 4.0-5.0m, backfill to GL with flush gatic cover)

SAMPLING & IN SITU TESTING LEGEND

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

Douglas Partners
Geotechnics / Environment / Groundwater

BORE: 6 **PROJECT: SYDNEY OLYMPIC PARK** **SEP 2018**

5.5 – 10.0m

BORE: 6 **PROJECT: SYDNEY OLYMPIC PARK** **SEP 2018**

10.0 – 15.0m

BORE: 6 **PROJECT: SYDNEY OLYMPIC PARK** **SEP 2018**

Project No: 86411.00
BH ID: BH6
Depth: 15.00 – 19.10 m
Core Box No.: 3

15.0 – 19.1m

Appendix D

Laboratory Test Results

Material Test Report

Douglas Partners
Geotechnics / Environment / Groundwater
Douglas Partners Pty Ltd
Sydney Laboratory
96 Hermitage Road West Ryde NSW 2114
Phone: (02) 9809 0666
Fax: (02) 9809 0666
Email: mick.gref@douglaspartners.com.au

Accredited for compliance with ISO/IEC 17025 - Testing

NATA
WORLD RECOGNISED ACCREDITATION

Approved Signatory: Mick Gref
Senior Technician
NATA Accredited Laboratory Number: 828

Report Number: **86411.00-1**
Issue Number: 1
Date Issued: 19/09/2018
Client: Ecove Group Pty Ltd
Locked Bag 1451, Meadowbank NSW 2114
Contact: Michael Azar
Project Number: 86411.00
Project Name: Carpark P6D OLYMPIC PARK
Project Location: Carpark P6D (cnr Australia Ave and Park View Dr), Sydney Olympic Park
Work Request: 3746
Sample Number: 18-3746A
Date Sampled: 04/09/2018
Sampling Method: Sampled by Engineering Department
Sample Location: **BH1 (1.0 - 1.45m)**
Material: Clay

Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1)		Min	Max
Sample History	Oven Dried		
Preparation Method	Dry Sieve		
Liquid Limit (%)	57		
Plastic Limit (%)	29		
Plasticity Index (%)	**28**		

Linear Shrinkage (AS1289 3.4.1)		Min	Max
Linear Shrinkage (%)	18.0		
Cracking Crumbling Curling	None		

Material Test Report

Douglas Partners
Geotechnics / Environment / Groundwater
Douglas Partners Pty Ltd
Sydney Laboratory
96 Hermitage Road West Ryde NSW 2114
Phone: (02) 9809 0666
Fax: (02) 9809 0666
Email: mick.gref@douglaspartners.com.au

Accredited for compliance with ISO/IEC 17025 - Testing

NATA
WORLD RECOGNISED ACCREDITATION

Approved Signatory: Mick Gref
Senior Technician
NATA Accredited Laboratory Number: 828

Report Number: **86411.00-1**
Issue Number: 1
Date Issued: 19/09/2018
Client: Ecove Group Pty Ltd
Locked Bag 1451, Meadowbank NSW 2114
Contact: Michael Azar
Project Number: 86411.00
Project Name: Carpark P6D OLYMPIC PARK
Project Location: Carpark P6D (cnr Australia Ave and Park View Dr), Sydney Olympic Park
Work Request: 3746
Sample Number: 18-3746B
Date Sampled: 04/09/2018
Sampling Method: Sampled by Engineering Department
Sample Location: **BH2 (1.0 - 1.45m)**
Material: Clay

Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1)		Min	Max
Sample History	Oven Dried		
Preparation Method	Dry Sieve		
Liquid Limit (%)	57		
Plastic Limit (%)	30		
Plasticity Index (%)	**27**		

Linear Shrinkage (AS1289 3.4.1)		Min	Max
Linear Shrinkage (%)	17.5		
Cracking Crumbling Curling	Curling		

Material Test Report

Douglas Partners
Geotechnics / Environment / Groundwater
Douglas Partners Pty Ltd
Sydney Laboratory
96 Hermitage Road West Ryde NSW 2114
Phone: (02) 9809 0666
Fax: (02) 9809 0666
Email: mick.gref@douglaspartners.com.au

Accredited for compliance with ISO/IEC 17025 - Testing

NATA
WORLD RECOGNISED ACCREDITATION

Approved Signatory: Mick Gref
Senior Technician
NATA Accredited Laboratory Number: 828

Report Number: **86411.00-1**
Issue Number: 1
Date Issued: 19/09/2018
Client: Ecove Group Pty Ltd
Locked Bag 1451, Meadowbank NSW 2114
Contact: Michael Azar
Project Number: 86411.00
Project Name: Carpark P6D OLYMPIC PARK
Project Location: Carpark P6D (cnr Australia Ave and Park View Dr), Sydney Olympic Park
Work Request: 3746
Sample Number: 18-3746C
Date Sampled: 04/09/2018
Sampling Method: Sampled by Engineering Department
Sample Location: **BH3 (1.1 - 1.45m)**
Material: Clay

Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1)		Min	Max
Sample History	Oven Dried		
Preparation Method	Dry Sieve		
Liquid Limit (%)	86		
Plastic Limit (%)	37		
Plasticity Index (%)	**49**		

Linear Shrinkage (AS1289 3.4.1)		Min	Max
Linear Shrinkage (%)	19.5		
Cracking Crumbling Curling	None		

Material Test Report

Report Number: **86411.00-1**
Issue Number: 1
Date Issued: 19/09/2018
Client: Ecove Group Pty Ltd
Locked Bag 1451, Meadowbank NSW 2114
Contact: Michael Azar
Project Number: 86411.00
Project Name: Carpark P6D OLYMPIC PARK
Project Location: Carpark P6D (cnr Australia Ave and Park View Dr), Sydney Olympic Park
Work Request: 3746
Sample Number: 18-3746D
Date Sampled: 04/09/2018
Sampling Method: Sampled by Engineering Department
Sample Location: **BH4 (1.0 - 1.45m)**
Material: Clay

Douglas Partners
Geotechnics / Environment / Groundwater
Douglas Partners Pty Ltd
Sydney Laboratory
96 Hermitage Road West Ryde NSW 2114
Phone: (02) 9809 0666
Fax: (02) 9809 0666
Email: mick.gref@douglaspartners.com.au

Accredited for compliance with ISO/IEC 17025 - Testing

NATA
WORLD RECOGNISED ACCREDITATION

Approved Signatory: Mick Gref
Senior Technician
NATA Accredited Laboratory Number: 828

Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1)		Min	Max
Sample History	Oven Dried		
Preparation Method	Dry Sieve		
Liquid Limit (%)	84		
Plastic Limit (%)	32		
Plasticity Index (%)	**52**		

Linear Shrinkage (AS1289 3.4.1)		Min	Max
Linear Shrinkage (%)	20.5		
Cracking Crumbling Curling	None		

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

CERTIFICATE OF ANALYSIS 200307

Client Details

Client	Douglas Partners Pty Ltd
Attention	Campbell Ellingham
Address	96 Hermitage Rd, West Ryde, NSW, 2114

Sample Details

Your Reference	**86411.00, Carpark P6D Olympic Park**
Number of Samples	4 SOIL
Date samples received	07/09/2018
Date completed instructions received	07/09/2018

Analysis Details

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

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

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

Report Details

Date results requested by	14/09/2018
Date of Issue	12/09/2018

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

Results Approved By
Nick Sarlamis, Inorganics Supervisor

Authorised By

Jacinta Hurst, Laboratory Manager

Soil Aggressivity					
Our Reference		200307-1	200307-2	200307-3	200307-4
Your Reference	UNITS	BH1	BH2	BH3	BH6
Depth		1-1.45	0.4-0.5	2.5-2.75	4.0-4.45
Type of sample		SOIL	SOIL	SOIL	SOIL
pH 1:5 soil:water	pH Units	4.6	5.3	5.1	5.6
Electrical Conductivity 1:5 soil:water	µS/cm	200	150	56	71
Resistivity by calculation	ohm m	50	66	180	140
Chloride, Cl 1:5 soil:water	mg/kg	310	49	21	30
Sulphate, SO4 1:5 soil:water	mg/kg	23	140	44	51

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25°C in accordance with APHA latest edition 2510 and Rayment & Lyons.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25oC in accordance with APHA 22nd ED 2510 and Rayment & Lyons. Resistivity is calculated from Conductivity.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Alternatively determined by colourimetry/turbidity using Discrete Analyer.

Client Reference: 86411.00, Carpark P6D Olympic Park

QUALITY CONTROL: Soil Aggressivity					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	200307-2
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	1	4.6	4.5	2	102	[NT]
Electrical Conductivity 1:5 soil:water	µS/cm	1	Inorg-002	<1	1	200	250	22	103	[NT]
Resistivity by calculation	ohm m	0.1	Inorg-002	<0.1	1	50	39	25	[NT]	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	310	320	3	89	109
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	23	20	14	91	127

Result Definitions

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

Quality Control Definitions

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

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

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

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

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

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

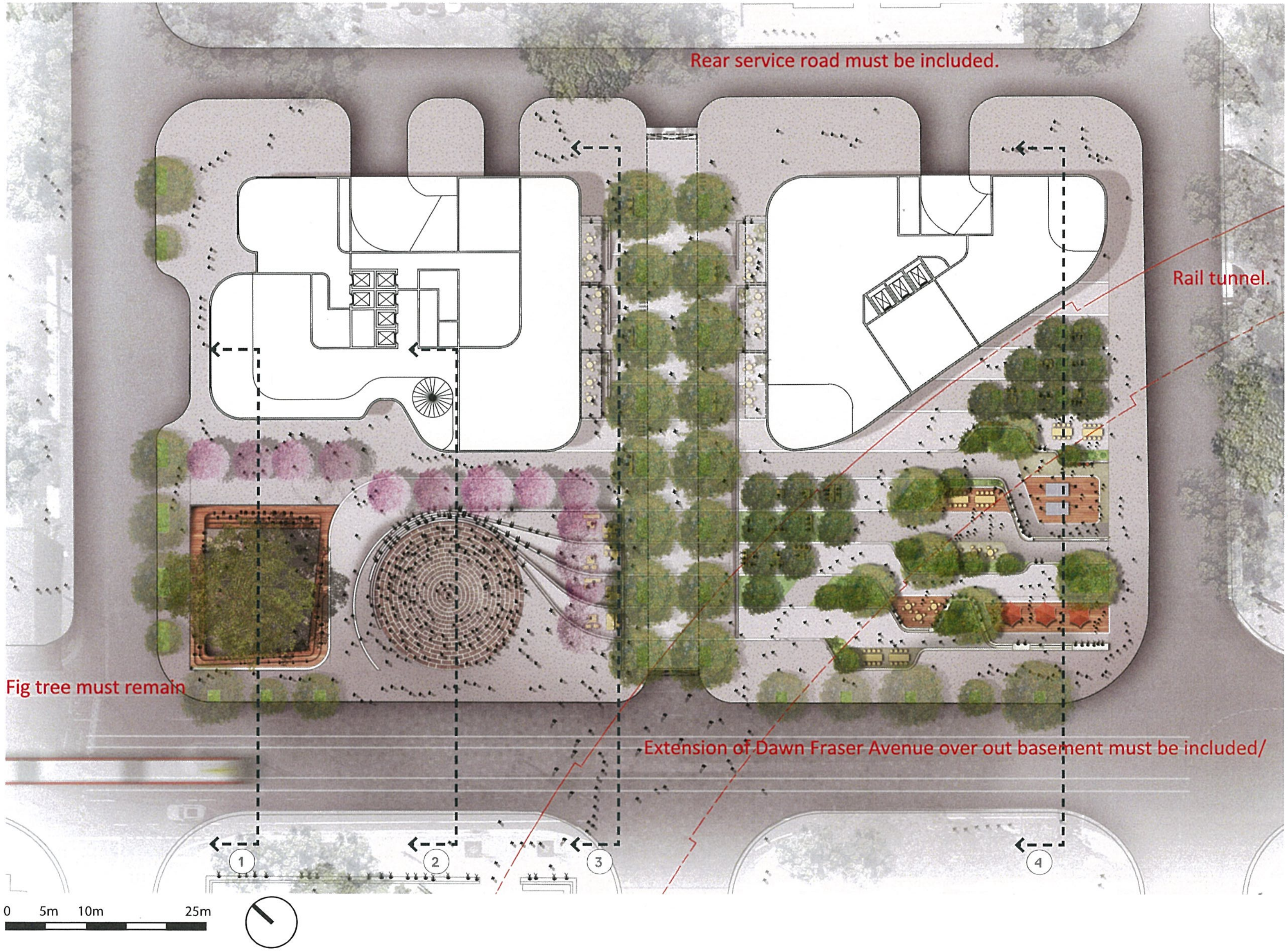
When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

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

Measurement Uncertainty estimates are available for most tests upon request.

Appendix E

Supplied Drawings



Rear service road must be included.

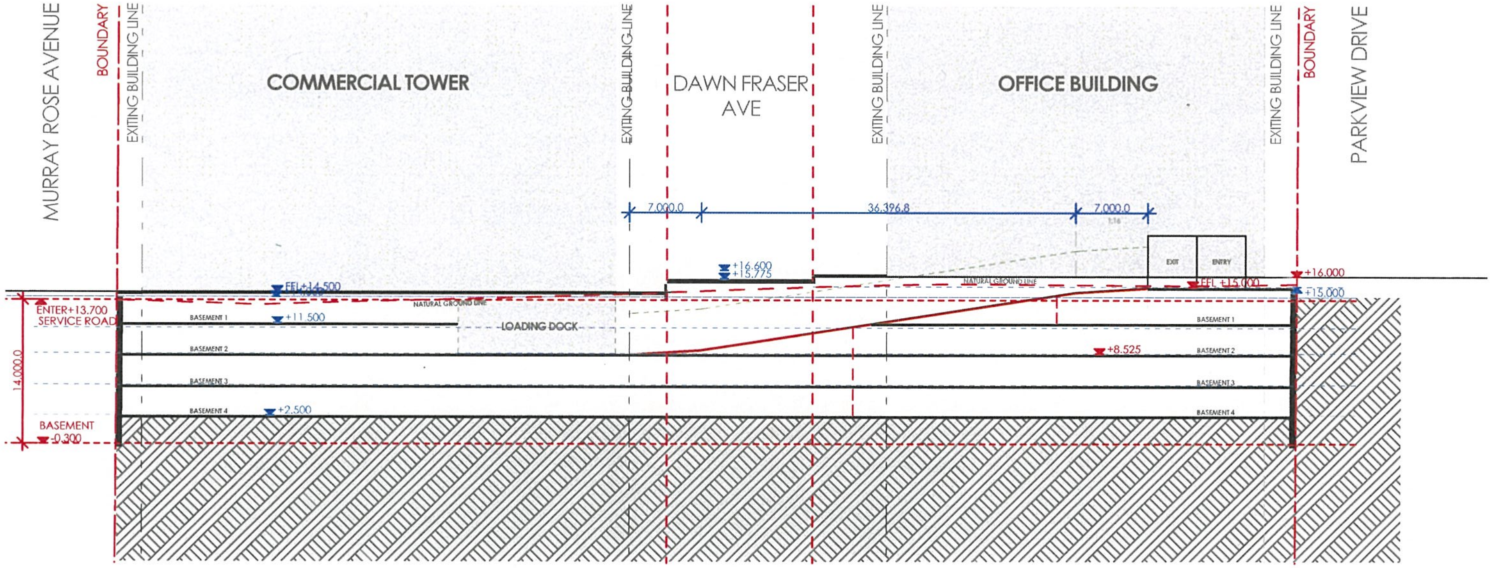
Rail tunnel.

External landscaping to have power and usb plugin points

Fig tree must remain

Extension of Dawn Fraser Avenue over out basement must be included/

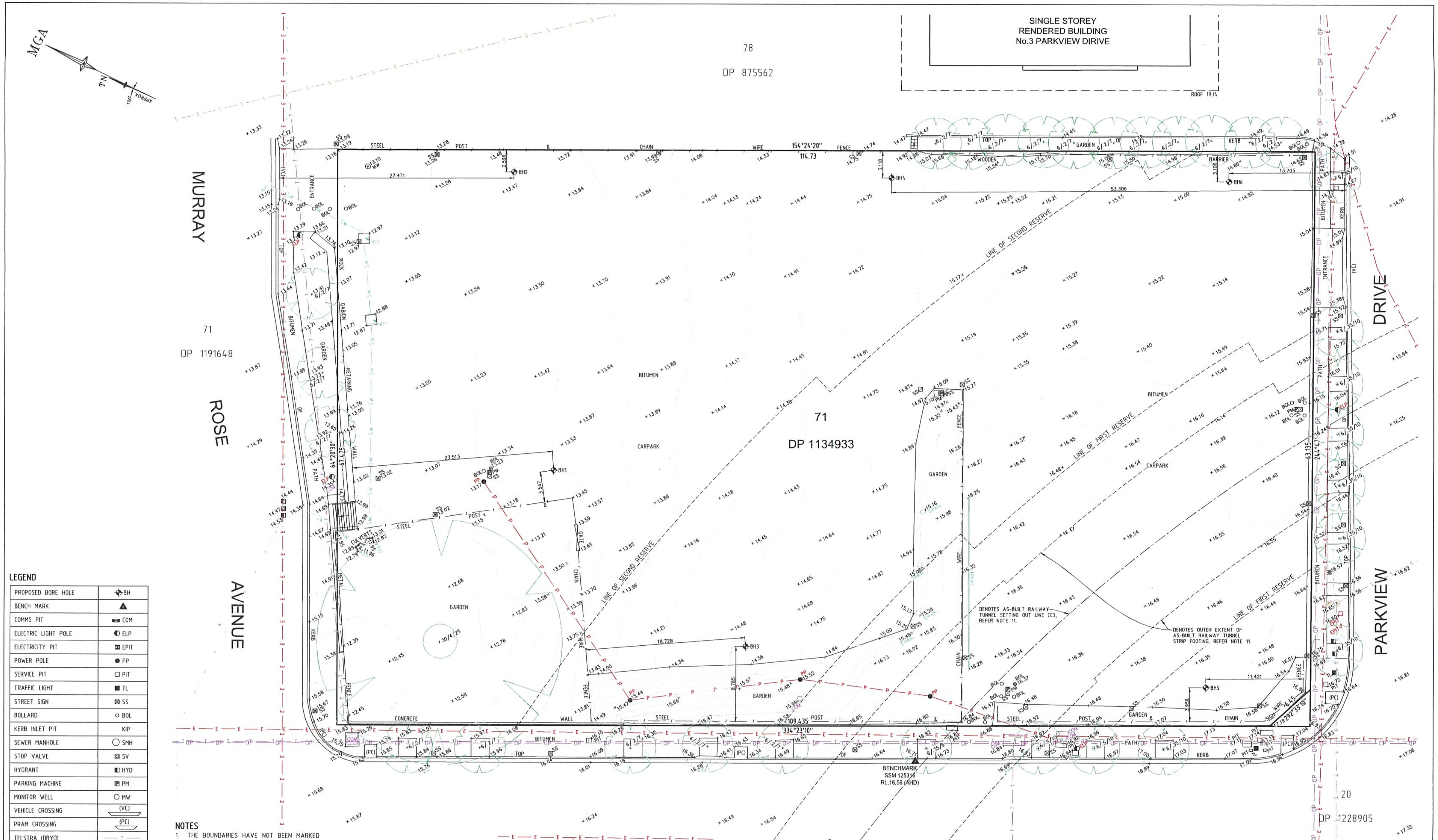
fitzpatrick+partners



SK-59 | SECTION THROUGH RAMP

1:500 @ A3

2A and 2B Sydney Olympic Park |



LEGEND

PROPOSED BORE HOLE	⊕BH
BENCH MARK	▲
COMMS PIT	COM
ELECTRIC LIGHT POLE	◐ ELP
ELECTRICITY PIT	EPIT
POWER POLE	● PP
SERVICE PIT	☐ PIT
TRAFFIC LIGHT	■ TL
STREET SIGN	☒ SS
BOLLARD	○ BOL
KERB INLET PIT	KIP
SEWER MANHOLE	○ SMH
STOP VALVE	☒ SV
HYDRANT	◧ HYD
PARKING MACHINE	PM
MONITOR WELL	○ MW
VEHICLE CROSSING	(VC)
PRAM CROSSING	(PC)
TELSTRA (DBYD)	— T —
OPTUS (DBYD)	— OP —
WATER (DBYD)	—
SEWER (DBYD)	— S —
ELECTRICITY (U'GROUND) (DBYD)	— E —
ELECTRICITY (OVERHEAD)	— P —

DIAL BEFORE YOU DIG
www.1100.com.au

REFER TO NOTES AND LEGEND

0 4 6 8 12 16 20
SCALE 1:200 @ A1

NOTES

1. THE BOUNDARIES HAVE NOT BEEN MARKED
2. ALL AREAS AND DIMENSIONS HAVE BEEN COMPILED FROM PLANS MADE AVAILABLE BY NSW LAND REGISTRY SERVICES AND ARE SUBJECT TO FINAL SURVEY
3. ORIGIN OF LEVELS ON A.H.D. IS TAKEN FROM SSM 125316 R.L. 16.583 (A.H.D.) IN AUSTRALIA AVENUE
4. CONTOUR INTERVAL 0.25 m
5. CONTOURS ARE INDICATIVE ONLY. ONLY SPOT LEVELS SHOULD BE USED FOR CALCULATIONS OF QUANTITIES WITH CAUTION
6. KERB LEVELS ARE TO THE TOP OF KERB UNLESS SHOWN OTHERWISE
7. FLOOR LEVELS SHOWN ARE THRESHOLD LEVELS. NO INVESTIGATION OF INTERNAL FLOOR LEVELS HAS BEEN UNDERTAKEN
8. NO INVESTIGATION OF UNDERGROUND SERVICES HAS BEEN MADE. SERVICES HAVE BEEN PLOTTED FROM RELEVANT AUTHORITIES INFORMATION AND HAVE NOT BEEN SURVEYED. ALL RELEVANT AUTHORITIES SHOULD BE NOTIFIED PRIOR TO ANY EXCAVATION ON OR NEAR THE SITE
9. 8/.4/7 DENOTES TREE SPREAD OF 8m, TRUNK DIAMETER OF 0.4m & APPROX HEIGHT OF 7m
10. BEARINGS SHOWN ARE MGA (MAP GRID OF AUSTRALIA) ADD APPROX. 1°00' FOR TRUE NORTH

11. RAILWAY TUNNEL SETOUT HAS NOT BEEN DERIVED FROM SURVEY. SETOUT IS DERIVED FROM AS-BUILT DOCUMENTATION ONLY. REFER TO THE FOLLOWING DRAWINGS FOR AS-BUILT DOCUMENTATION:
TTWD105b.S01.DWG DATED 21/08/98
TTWD118C.S01.DWG DATED 21/08/98
TTWD132c.DWG DATED 21/08/98
TTWD153d.DWG DATED 21/08/98

Revision	Date	Description	Reference
D	00/00/00	-	00
C	00/00/00	-	00
B	00/00/00	-	00
A	21/08/18	PROPOSED BOREHOLE LOCATIONS	002

THIS IS THE PLAN REFERRED TO IN MY LETTER DATED:

....
Registered Surveyor NSW

LTS
LOCKLEY
Registered Surveyors NSW
www.lts.com.au
Suite 1, Level 1
810 Pacific Highway
Gordon NSW 2072
Locked Bag 5
Gordon NSW 2072
P 1300 587 000
F 02 9499 2760

Client ECOVE PTY LTD
Drawing title
PLAN OF DETAIL AND LEVELS OVER LOT 71 IN DP 1134933
KNOWN AS 2A & 2B HERB ELLIOTT AVENUE, SYDNEY
OLYMPIC PARK

datum AHD
site Area 7711m²
LGA PARRAMATTA
reference number 50357 002DT
scale 1:200 @A1
date of survey 14/06/2018
SHEET 1 OF 3