Douglas Partners
Geotechnics / Environment / Groundwater

Report on
Geotechnical Investigation of C3 Site

Stage 2 - Midtown
Herring Road, Macquarie Park

Prepared for
Frasers Property Ivanhoe Pty Ltd

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

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1. Introduction

This revised report presents the results of a geotechnical investigation, undertaken by Douglas Partners Pty Ltd (DP) for the C3 site at the proposed Midtown development (Stage 2) at Herring Road, Macquarie Park. Midtown is located at the former Ivanhoe Estate Social Housing precinct. The investigation was commissioned by Chris Koukoutaris of Frasers Property Ivanhoe Pty Ltd (Frasers) and was undertaken in accordance with the Consulting Services Agreement dated 26 April 2021 and a subsequent variation. Revision 2 has been issued due to the update of Figure 1, only.

The C3 investigation was undertaken in conjunction with geotechnical investigation for the C2 and C4 sites, which together comprise the Stage 2 area, although the detailed results of those investigations will be reported separately. This revised report has been prepared following the completion of supplementary groundwater wells and permeability testing at the C3 and C4 sites in May and June 2021, the relevant results of which have been incorporated into this report.

The investigation also follows previous geotechnical investigation of the greater Midtown site in 2017, and groundwater monitoring from 2017 to 2018. The geotechnical investigation report for the greater Midtown site was updated in 2018 following the completion of that stage of groundwater monitoring.

A high-rise residential development is proposed at the C3 site. The aim of the investigation was to assess the subsurface soil, rock and groundwater conditions at the site, in order to provide geotechnical comment relevant to the proposed development on:

- Excavation conditions, including excavatability, excavation stability, shoring and batters;
- Groundwater conditions; and
- Foundations.

The investigation included the drilling of six boreholes in or immediately adjacent to the C3 basement area, and installation of selected standpipes. Two of the boreholes (Bores 117 and 118) and some groundwater monitoring standpipes (ie. wells) were requested as a variation to the original scope of work. The details of the field work are presented in this report, together with comments and recommendations on the items listed above.

2. Proposed Development

The proposed C3 development is for a residential high-rise building including a ground floor retail area. Basement car parking is proposed, with basement excavation extending to the site boundaries. Lowermost basement floor levels of RL 40.0 to RL 39.3 are proposed across most of the basement footprint, stepping up to RL 42.1 at the north-eastern side of the building footprint.

It is understood that the development of the C3 block is likely to be undertaken following the completion of the adjacent roads and services, but in conjunction with the proposed park at the neighbouring C2 site, that will adjoin the north-western frontage. Therefore, while shoring is expected to be required to support the other boundaries, an 'open cut' may be possible along the north-western frontage, using temporary batters or benches beyond the C3 site boundaries.

3. Background

In September 2015 the Ivanhoe Estate was rezoned by the Department of Planning and Environment as part of the Macquarie University Station (Herring Road) Priority Precinct, to transform the area into a vibrant centre that benefits from the available transport infrastructure and the precinct's proximity to jobs, retail and education opportunities within the Macquarie Park corridor. The new community will be known as Midtown MacPark, or "Midtown".

Douglas Partners Pty Ltd undertook investigation for the greater Ivanhoe Estate (now Midtown) site, in 2017, and undertook groundwater monitoring at 6 bores from November 2017 to June 2018. The detailed results were reported in the following DP Reports:

- 86043.01.R.001.Rev1, Preliminary Geotechnical Investigation of Ivanhoe Estate, dated 30 July 2018, including several boreholes drilled in the general vicinity, but outside of the C3 site in 2017, and revised in 2018 with summary data relating to groundwater monitoring; and
- 86043.01.R.005.Rev0, Groundwater Monitoring, dated 30 July 2018.

Relevant results from those previous investigations have been referenced in the current report.

Since that time, demolition of the previous residences has been completed, and earthworks have commenced for the development of infrastructure, roads and public areas at Midtown. These works have necessarily destroyed several of the previous groundwater monitoring standpipes or wells. While attempts were made during the current field work to locate possible remaining standpipes (at Bores 10, 12 and 13) near Shrimpton's Creek, these bores appear to have been either destroyed or obscured by overgrowth or temporary construction measures such as fencing and sedimentation controls.

The investigation for the C3 site was undertaken in conjunction with investigations for the C2 and C4 sites, which together comprise the Stage 2 works. Reference is made in this report to the relevant results of those investigations, particularly with respect to standpipes and groundwater levels. The detailed results of those investigations, however, will be separately reported in the following DP Reports:

- 86043.06.R.001, Geotechnical Investigation of the C2 site; and
- 86043.06.R.003, Geotechnical Investigation of the C4 site.

Dataloggers have been installed at four standpipes in the Stage 2 area, with results to be reported separately, on completion of monitoring.

4. Site Description

The greater Midtown site is in Macquarie Park near the corner of Epping Road and Herring Road, within the Ryde Local Government Area. The site occupies an area of approximately 8.2 hectares. The approximate location of the proposed C3 development, with respect to other Stage 2 sites and the greater Midtown area, is shown in Figure 1.

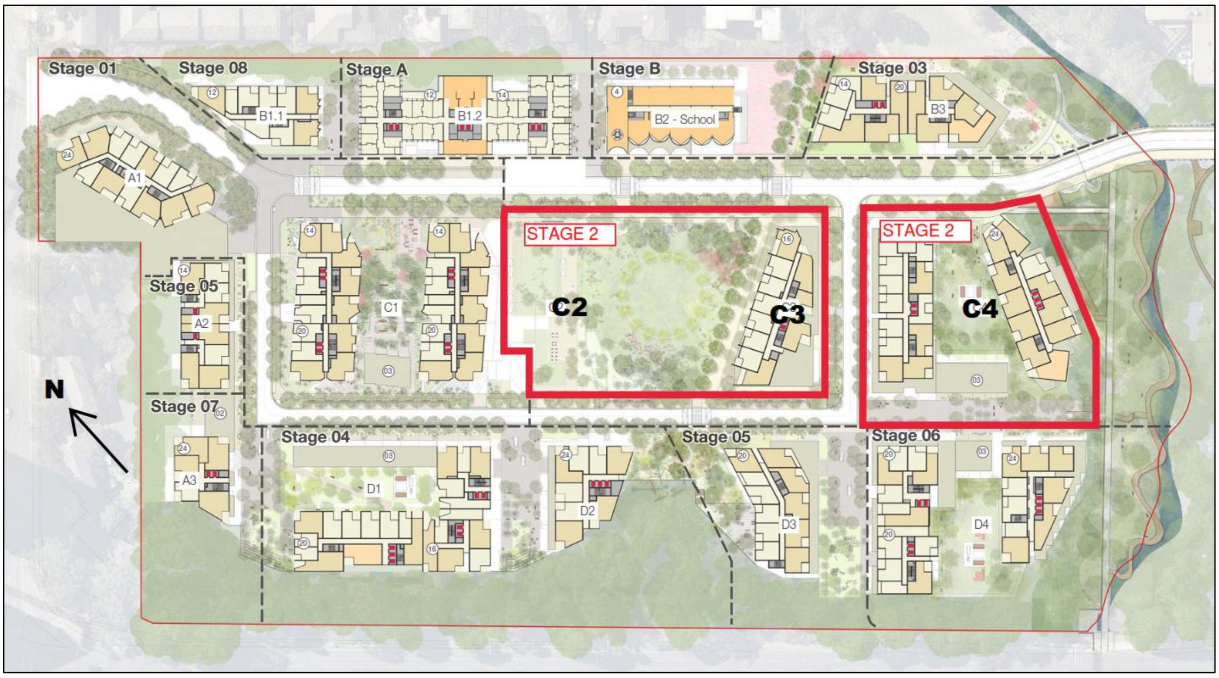


Figure 1: Location of the Stage 2 development areas (red), relative to the greater Midtown site (provided by Client).

Topographically, the Midtown site is located on a sideslope, with ground surface levels falling from approximately RL 71 near Herring Road, to approximately RL 42 at Shrimpton's Creek, at the south-eastern boundary.

Ground surface levels at the C3 development area typically fall from approximately RL 53 to RL 49, towards the east, though local variation was also present due to earthworks for haul roads, sedimentation controls (including swales and a sedimentation basin), and due to temporary stockpiles. While the typical ground surface levels within the C3 site, are similar to those prior to earthworks at the site, these levels were elevated relative to swales excavated at the north-east and south-west of the site, as part of sedimentation control measures for the Midtown earthworks (see also Figure 1).

5. Published Data

Reference to the regional mapping indicates the following at the C3 site:

- The Sydney Soils Landscape Series Sheet indicates that the site is underlain by the residual Lucas Heights soil landscape. These soils typically comprise sandy clay and clayey sand soils developed from Mittagong Formation and Hawkesbury Sandstone;

- The Sydney Geology Series Sheet indicates that the site is underlain by Hawkesbury Sandstone, near the boundary with Ashfield Shale; and
- The site is in an area of no known risk of coastal Acid Sulfate soils and is outside of the Salinity Potential in Western Sydney mapping.

The results of past and present field work indicate that ground conditions are consistent with the mapping of residual soils over Hawkesbury Sandstone, though a layer of fill is generally present, overlying the residual soil.

Reference to the WaterNSW data on registered boreholes indicates that groundwater bores in the vicinity of the Midtown site are relatively distant from the site but that the results are broadly consistent with the previous groundwater monitoring at the greater Midtown site.

6. Field Work

6.1 Field Work Methods

The field work for the current geotechnical investigation of the C3 site comprised 6 deep, small-diameter boreholes (Bore 103 to 106, and 117 to 118), drilled with a truck-mounted (Explora) drilling rig under the supervision of a geotechnical engineer. The boreholes were drilled using auger or rotary drilling methods to the bedrock surfaces, then continued by NMLC (50 mm diameter) diamond core drilling methods into the underlying bedrock. Sampling and identification of strata was undertaken from the cuttings returned by the auger blade, supplemented by disturbed sampling of soils by Standard Penetration Tests, and by logging of the retrieved rock core. Point load strength index tests were also undertaken on the recovered rock core at typical intervals of 1.0 m. The bores were taken to depths of between 13.8 m and 17.1 m.

Initially, groundwater monitoring wells or standpipes were installed in two of the boreholes; Bore 103 and 106. A further two standpipes were installed at 104A and 118A, adjacent to, and subsequent to, the corresponding investigation bores. Fill works at the 118A site had apparently raised ground levels by approximately 0.4 m between drilling of the original geotechnical bore and installation of the well at 118A.

The wells were installed by drilling or reaming of the boreholes with a PCD bit, with screen lengths within the bedrock backfilled with a gravel pack, then with a bentonite seal above the screened length. Where the original cored borehole was taken to greater depth, any cored length below the standpipe screened interval was sealed by bentonite. Spoil (ie cuttings) was used to backfill the standpipe above the bentonite to near ground surface level, and the standpipe was finished at ground surface with a Gatic cover, concreted in place. The bentonite seal is intended to isolate surface water inflow and shallow ‘perched’ groundwater flows from the screened length of the borehole.

Following the installation of the standpipes, they were purged by pumping to remove drilling fluid from the standpipe. A follow-up visit was then undertaken to obtain a groundwater level (following stabilisation of the water levels after purging) and to perform falling or rising head permeability tests, except at Bore 103, where the standpipe was destroyed by site operations after purging, but prior to measurements being taken. The standpipe construction is summarised in Table 1.

Table 1: Summary of Standpipe Construction in C3 Area

Bore	104A	106	118A
Ground Level (RL)	51.7	49.5	50
Backfill Interval (m)	0-10.5	0-7.0	0-3.0
Bentonite Seal Interval (m)	10.5-11.5	7.0-7.5	3.0-4.0
Gravel Interval (m)	11.5-13.5	7.5-11.0	4.0-6.1
Blank PVC Interval (m)	0-12.0	0-8.0	0.0-4.5
Screened PVC Interval (m)	12.0-13.5	8.0-11.0	4.5-6.1

The field work was undertaken in conjunction with investigations for the nearby C2 and C4 sites, which included drilling using similar small-diameter boreholes, in similar geology and the installation of additional standpipes both upslope and downslope of the C3 site. The standpipes in the broader Stage 2 development area are summarised in Table D2, in Appendix D.

Further details on the methods and procedures employed in the investigation are presented in the notes in Appendix A of this report.

Test locations and ground surface levels at test locations were determined relative to Australian Height Datum (AHD) by high precision differential GPS equipment, as per the previous test locations.

The locations of the bores are shown in Drawing 301, in Appendix B, together with other boreholes drilled nearby during the current and previous investigations.

6.2 Field Work Results

The detailed results of the field work for the C3 site are given in Appendix C of this report, together with relevant notes on classification terms, symbols and abbreviations, and rock core photographs. The results of point load strength index ($I_{s(50)}$) tests are included at the relevant depths on the borehole logs.

The results of the current field work may be broadly summarised as follows:

- **Fill** – variable fill, including concrete, gravelly sand and re-worked natural clay, of apparently variable compaction, to depths of 0.2 m to 1.4 m; underlain by
- **Sandy Clay and Clayey Sand** – residual soil, typically stiff and very stiff or dense, to depths of 0.2 m to 1.4 m; underlain by
- **Sandstone** – variable, fractured, very low to medium strength, including extremely low strength and high strength bands, to depths of 2.4 m to 6 m; becoming

  fractured to slightly fractured, low and medium strength, with variable weathering to depths of 5.7 m to 7 m (absent in some locations); underlain by

  slightly fractured to unbroken, medium and high strength with occasional very high strength bands, variably weathered to 13.8 m to depths of more than 16.0 m, then fresh.

No groundwater was observed whilst augering at the borehole locations.

The results of the groundwater measurements from the current investigation are summarised together with previous groundwater measurements in the vicinity of the Stage 2 development area of the Midtown site, in Table D1, in Appendix D. The results generally show that groundwater levels fall from the upper, north-western part of the site, towards Shrimpton's Creek at the south-east, from approximately RL 45.6 at Bore 106 to RL 41.6 at Bore 104A. Groundwater levels measured in a higher level standpipe at Bore 118A, suggest that a 'stacked' groundwater level may be present following periods of heavy rainfall, with higher standing water levels in some shallow wells compared to wells with a deeper screen interval. A standing water level of RL 44.6 was obtained at Bore 118A, which while within the range indicated by other boreholes, is considered relatively high given the position of the borehole.

Rising or falling head permeability tests were undertaken at the intact standpipes in the C3 and C4 areas. The results of the permeability tests are summarised in Appendix C, together with the base calculations associated with the falling or rising head permeability calculations. The results at Bore 114A could not be readily assessed due to the combination of standing water level and well geometry. Hydraulic conductivities of $1.5\text{x}10^{-8}$ m/s to $4.2\text{x}10^{-6}$ m/s were estimated from the tests in boreholes in the C3 area, which is considered to be relatively consistent with results obtained in the broader Stage 2 area. These values are considered to be relatively consistent with the sandstone encountered over the screened lengths, if slightly high, though noting that the higher permeability results were associated with closer fracture spacing.

The results of the field work were generally consistent with the results of previous investigations, although higher groundwater levels were indicated by the current investigation when compared to interpolated levels from previous investigations.

7. Comments

7.1 Geotechnical and Hydrogeological Model

7.1.1 Geotechnical Model

An interpreted geological model has been developed for the C3 site, based on the results of current and previous field work. The model is summarised in Table 2.

Table 2: Simplified Geotechnical Model

Unit	Summary	Typical Description
1	Fill	Variable fill, including gravelly sand and apparently re-worked natural clay soils, to typical depths of 0.5 m to 1.0 m, but likely to be deeper, particularly in areas of stockpiles, recent earthworks and past services
2	Residual Soil	Stiff to very stiff sandy clay and clayey sand, with trace iron-indurated bands, often grading to hard clay and dense clayey sand (extremely weathered sandstone), to depths of 0.2 m to 3.2 m at test locations, though absent at some locations.
3a	Sandstone – Variable	Typically very low to low strength, but with extremely low (soil strength), medium and high strength bands, highly weathered ,typically fractured to highly fractured sandstone

Unit	Summary	Typical Description
3b	Sandstone – Low and Medium Strength	Typically low and medium strength, highly to slightly weathered, fractured and slightly fractured sandstone with some highly fractured fractured bands. This layer is only distinct at the upslope side of the site, and is apparently absent due to deeper weathering at the downslope side of the site.
3c	Sandstone – Medium and High Strength	Typically medium and high strength, moderately weathered to fresh, slightly fractured with some fractured and unbroken lengths. This unit includes significant beds of high strength sandstone at some boreholes, but has been distinguished from Unit 3d by weathering.
3d	Sandstone – High Strength	Typically high strength, fresh, slightly fractured to unbroken, includes a very high strength band at Bore 103

The above interpreted units are shown in relation to the C3 boreholes and site levels on the Interpreted Geotechnical Cross-Sections presented on Drawings 302 to 304, in Appendix B. (Note the change in scale for Drawing 304). It should be noted that the subsurface profile is accurate only at the borehole locations, and that substantial variation can occur in between and away from the boreholes. The interpreted geotechnical boundaries are for illustrative purposes and should not be relied upon.

Previous investigation by DP in the general vicinity of the site have also indicated the presence of dykes and thrust faults, which are considered likely to be encountered at the greater Midtown site, though investigations to date have not confirmed their presence. They are nonetheless considered a possible presence at the C3 area.

The following information also informs the geotechnical model for the site:

- **Dykes** – Dykes may be present on this site. Dykes have been identified by previous DP experience on sites to the north-east of the site and in the geological mapping north-west of the site. Both of these dykes may project to near the site, but given that dykes may “step” or “fork” in plan, they may potentially intersect the subject site.

  Dykes in Sydney are typically near-vertical, planar features that may change in thickness, become discontinuous and/or step in plan. Common dyke widths in Sydney range from less than 1 m to approximately 6 m. They are typically completely weathered basalt or dolerite (clay) near surface and are usually weathered and weaker than the surrounding rock to significant depth. The rock adjacent to the dyke can also be highly fractured, variable or abnormally high strength due to the heat and pressure effects of the intrusion. Higher permeability and greater water seepage is also often observed within and on either side of the dyke material.

- **Thrust Faults –** Thrust faulting, often associated with dykes, have been previously identified on nearby sites. A photograph showing the subsurface profile exposed by bulk excavation at a recently developed site to the north-west of the greater Midtown site, is included in Figure 2.

Figure 2: Back thrusts in an excavation wall at a nearby site

These features are of limited lateral extent and may be present but remain undetected by even significant geotechnical investigation. If encountered, the precise influence and treatment (if required) of dykes and thrust faults are often only determined at construction stage, when their presence, extent and orientation with respect to the works can be more reliably assessed.

7.1.2 Hydrogeological Model

The hydrogeology at the C3 site, in the depth of interest, can be characterised by the following:

- Ephemeral, 'perched' groundwater, or seepage, expected to occur within the upper fill and along the top of rock following periods of rainfall or due to human influences such as stormwater runoff and irrigation. Some ephemeral seepage may also migrate through defects within the rock;
- A transient, 'stacked' groundwater level within the upper sandstone, developing after heavy rainfall and responsive to weather variations; and
- Long-term groundwater levels, at depth, within the sandstone. These water levels are expected to respond to both climatic and weather variations, which would be expected to be reflected by natural fluctuations in groundwater levels.

Within the bedrock, groundwater flows would be concentrated along defects within the rock such as joints and bedding planes. Iron-staining of the existing joints are suggestive of past groundwater passage, and greater water ingress would be expected through such joints.

The existing and past standpipes were installed with bentonite seals to limit the influence of the 'perched' seepage through soil on the standpipe measurements.

Interpreted Cross-Sections A-A' to C-C' (Drawings 302 to 304) in Appendix B show the measured standing water levels at standpipe locations with respect to recent measurements up and downslope of the C3 site, and the interpreted groundwater tables. The model is consistent with broader groundwater measurements at the site, which have generally indicated levels that fall towards Shrimpton's Creek (see Drawing 304, and also Table D1, in Appendix D).

Within the Stage 2 area, higher standing water levels were generally obtained from standpipes with relatively shallow screen depths within the sandstone, compared to wells installed at greater depth (eg shallow well at 118A vs deep well at 104, see Drawing 303). It is noted, however, that these relatively shallow water level measurements appear to fall relatively rapidly when follow-up readings were undertaken following periods of no significant rainfall (eg refer data for 109A, 111A, in Table D1 in Appendix D), and so are considered to reflect the transient 'stacked' groundwater level, likely due to a low permeability aquitard (or aquitards) below the shallower screen, such as a thick underlying sandstone bed with limited defects, and the horizontal to vertical permeability contrast expected within Hawkesbury Sandstone.

For the deep groundwater table, natural groundwater fluctuations in the order of 1.5 m are suggested by the comparison of previous water level monitoring at standpipes at the (now destroyed) Bore 07 and recent measurements in the standpipe at the nearby Bore 101. Both of these standpipes are upslope of the C3 area but in an area of expected similar hydrogeology, with recent groundwater levels being at the upper end of the measured range, approximately 1 m above previous monitored levels.

7.2 Excavation

The proposed basement floor levels are between approximately RL 39.3 and RL 42.1. Excavation of approximately 0.5 m below these levels are anticipated for bulk excavation levels, although these have not been confirmed.

Based on the existing information, excavation of up to approximately 10.5 m to 14 m (for a basement floor level at RL 39.3), is anticipated.

Reference to the results of the geotechnical investigation indicates that the excavation will extend through fill and natural soils (Units 1 and 2) and into sandstone bedrock. Within the sandstone, excavation is expected to proceed through variable strength (Unit 3a), then through generally low and medium strength (Unit 3b) into medium and high strength sandstone (Unit 3c). This may include excavation through significant beds of unbroken, high strength sandstone.

Materials in Units 1, 2 and 3a are likely to be readily excavated using conventional earthmoving equipment (e.g. bulldozers and hydraulic excavators, with some rock hammering of stronger bands within the variable sandstone). Medium and high strength sandstone (Unit 3c) is likely to require excavation by ripping tynes mounted on large bulldozers (eg D12 or larger), large rock hammers, rock saws and milling heads. Productivity would slow if very high strength bands (e.g. as encountered at Bore 103, though in Unit 3d, below the depth of excavation) are encountered.

Excavation into the typically fractured low and medium strength sandstone of Unit 3b may also require these heavier excavation methods to maintain productivity, although some limited excavation may be possible using conventional earthmoving equipment, depending on the thickness and continuity of medium and higher strength bands within the unit, and defects within the rock.

The excavatability of the medium and high strength (Unit 3c) bedrock will be governed by the defects within the rock mass. Based on the rock cores, the rock in this unit frequently includes bed spacings of more than 1 m, although more fractured zones are also present. In general, the excavation of high strength sandstone (which is a significant proportion of the Unit 3c sandstone), is likely to be difficult and slow, with low productivity and high hammer/tyne wear expected.

7.2.1 Vibrations

Significant vibrations are anticipated during excavation within low to high strength bedrock. Excavation methods may therefore be limited by acceptable vibration levels, particularly if the new services installed in the adjacent roads are sensitive to vibrations. At this stage, no buildings are within 50 m of the site, but depending on the staging of other site works, consideration may also need to be given to other structures, particularly if they are occupied at the time of the works. Acceptable vibration levels should therefore be confirmed with the asset owners prior to excavation.

The limit may need to be adjusted to reflect the asset requirements, response of neighbouring structures during excavation and vibration dosage once the neighbouring building is occupied.

A vibration trial may be required to size equipment at the commencement of excavation into rock. The trial may indicate that minimum offset distances are required from vibration-sensitive assets for the preferred plant, or that alternative excavation methods or equipment are required.

Where a vibration trial indicates that the equipment may potentially exceed vibration levels, or where buildings or occupants are otherwise sensitive to vibration levels, consideration could be given to continuous vibration monitoring during the works. These monitors may be set up to activate a flashing ‘alarm’ light, or send text messages, if pre-set vibration levels are exceeded during the work.

7.2.2 Batters

Batters or excavation support will be required for excavations through soil and extremely low to very low strength sandstone, and also for fractured low and medium strength rock (i.e. Units 1, 2, 3a and 3b),

Preliminary safe batter slopes are provided in Table 3, for batter slopes no greater than 3 m in height, with horizontal ground beyond the crest and below the toe, no deflection sensitive structures or services above the crest, no surcharges above the crest and no seepage from the face.

Table 3: Preliminary Safe Batter Slopes for Batter Slopes ≤ 3 m Height

Unit	Material	Maximum Temporary Safe Batter Slope (Horizontal:Vertical)
1	Fill	2:1
2	Residual Soil	1.5:1
3a	Sandstone – Variable	1:1
3b	Sandstone – Low and Medium Strength	0.5:1
3c	Sandstone – Medium and High Strength	Vertical

Such batters are only currently anticipated in the temporary case along the north-western side of the excavation, due to the expected prior construction of services in the adjacent road reserves.

Batters higher than 3 m, steeper batters, or batters subject to surcharges behind the crest (within an exclusion zone equal to the height of the batter, extending back from the crest), adjacent sloping ground or seepage would generally require more detailed geotechnical assessment. Along the north-western site boundary, for example, batters from current ground levels to the base of Unit 3b, would exceed the 3 m batter height, and may encounter some water seepage at the base of the batter slope (see Drawing 302, Interpreted Geotechnical Cross-Section A-A'). These conditions would require specific analysis but would also be dependent on the site levels and operations within the adjacent C3 park during the C2 excavation works.

All batter slopes should be subject to inspection by an experienced geotechnical professional at maximum 1.5 m drops. Flatter or steeper slopes may be required, depending on the results of assessment. Protection for the face of the batter slope may also be required to reduce the risk of loose materials falling into the excavation below.

Within the medium and high strength sandstone (Unit 3c) the rock is likely to be able to be cut vertically and stand unsupported, even for cut depths greater than 3 m, but subject to regular defect and localised stability assessment by an experienced geotechnical professional, at drops no greater than 1.5 m. This may indicate that additional local support (e.g. bolts or anchors) and/or shotcrete is required due to adverse jointing or other defects.

7.2.3 Waste Classification

All excavated materials will need to be disposed of in accordance with the provisions of the current legislation and guidelines including the Waste Classification Guidelines (EPA, 2014). This includes fill and natural materials that may be removed from the site.

7.3 Shoring/Retaining Walls

7.3.1 General

Shoring will be required where the rock strength or condition is unsuitable for vertical excavation, and conditions are unsuitable for batters (eg inadequate space). Shoring is therefore anticipated along all boundaries, except if and where acceptable batters may be formed in the adjacent site to the north-west. Shoring may still be required along part, or all, of the north-western boundary, depending on the adjoining ground and possibly groundwater levels.

Soldier pile shoring walls are considered suitable for this site, with walls taken down through the Unit 1, 2, 3a and 3b material to socket in or bear on at least medium strength, slightly fractured sandstone (ie Unit 3c) with infill shotcrete panels constructed between the piles as excavation proceeds. Typical soldier pile spacings at 2 m to 2.5 m are likely to be suitable for the support of the natural clay soils and weathered rock above the groundwater table.

Bored, concrete piles would be suitable for the construction of shoring piles at this site, although casing may be required for drilling through fill and possibly soil materials, to prevent side wall material falling into the pile excavation. A heavy-duty, high torque drilling rig is likely to be required to obtain significant socket (i.e. embedment) into medium and high strength sandstone, as is expected at this site, particularly given the medium and high strength bands present in some areas in the Unit 3b material. DP note that while some significant bands of medium strength materials are present in the Unit 3b material (e.g. at Bore 103), the investigation results suggest that these layers are fractured to slightly fractured, with some relatively steep defects, and that the medium strength bands are relatively discontinuous across the site.

Given the depth of excavation, anchors would generally be required to provide temporary lateral support to the shoring wall, with final support provided by the basement structure.

Inspections are recommended during the pile excavation to allow for geotechnical assessment of the foundation material, deepening of the piles where necessary, and advance notice of areas where poorer ground conditions are present. Inspections of the exposed rock face between soldier piles during excavation is also recommended at 1.5 m drops, prior to placement of mesh and shotcrete, to allow assessment of possible steep joints or defects which might require additional support.

If encountered, the presence of dykes or thrust faulting may result in locally poorer rock conditions, which may lead to additional support being required in some areas of the site. Detailed investigation and/or careful monitoring and inspection of ground conditions during excavation (including for soldier piles) would generally be appropriate to ensure that support is taken down to an appropriate depth in any affected areas. It is not likely to be practical to assess the presence of dykes in advance, unless a dyke location and orientation is determined during an earlier stage of works at the site.

7.3.2 Shoring Design

For a shoring wall supported by multiple rows of anchors or props, preliminary design may be based on a uniform rectangular earth pressure distribution of 4H (where H is the wall height in metres, and pressure is in kPa), provided that deflections are not a concern. Where walls are constructed close to existing deflection-sensitive structures or utilities, a pressure of between 6H and 8H should be considered, depending on the sensitivity of the utilities and the soil profile to be retained. Higher pressures would be appropriate where batters (ie sloping ground) are present above the wall, or where concentrated loads are proposed behind the wall, either during construction (eg plant) or in the permanent case (eg elevated garden beds or roads).

The detailed design of shoring/retaining walls is nowadays normally undertaken using software that can account for the soil-structure interaction during the progressive excavation and support installation sequence (eg Wallap, Flac, Plaxis.)

Allowance should be made for the provision of drainage behind retaining structures, or alternatively the walls should be designed for full hydrostatic pressures. Appropriate drainage (eg strip or core drains) should be included to prevent hydrostatic water levels rising above the design hydrostatic level of the shoring/retaining wall design.

For piled wall systems terminating above the bulk excavation level it may be necessary, depending on the design of wall restraint, to install ‘toe bolts’ or anchors at the base of each pile for stability purposes.

7.3.3 Anchor Design

The preliminary design of anchors may be based on the bond strengths indicated in Table 4.

Table 4: Parameters for Preliminary Anchor Design

Material	Ultimate Bond Strength
Variable sandstone (Unit 3a)	100 kPa
Low and medium strength sandstone (Unit 3b)	300 kPa

Material	Ultimate Bond Strength
Medium and High strength Sandstone (Unit 3c)	1000 kPa

The above values assume that the anchor holes are adequately cleaned and free of clay smear. It would be appropriate for these values to be confirmed by the anchoring contractor based on their specific installation methods and experience, and for the rock conditions encountered during anchor installation at the site. Pull-out tests may be appropriate if higher bond values are to be adopted.

After installation, all temporary anchors should be proof loaded to 125% of the nominal working load, then locked off at 70% of the working load. For anchors supporting any structures on the boundaries, lock off values should be 90% of the working load. Checks should also be made at regular intervals to ensure that load is maintained in anchors and not lost due to creep effects.

While it is expected that the adjacent sites will be under the control of the developer at the time of construction, appropriate permissions from adjacent landowners would be required if support measures (eg anchors) are proposed across site boundaries. Anchors should also be de-stressed following the provision of permanent lateral support by the basement structure.

7.3.4 Shoring Wall and Excavation Movement

Typical horizontal movements in the order of 0.15% of the wall height would be expected for a well-constructed and designed, high stiffness shoring wall (ie with multiple rows of anchors), but depending on the excavation and support sequence and support provided. For a 6 m high shoring wall, this corresponds to approximately 10 mm movement.

In addition to retaining wall movements, basement excavations into medium and high strength sandstone bedrock may result in lateral movement of the sandstone faces due to stress relief effects. Release of these stresses may cause horizontal movements along the rock bedding surfaces and defects, with estimated movements of between 0.5 mm and 2 mm per metre depth of excavation into medium and high strength sandstone, at the midpoint of the excavation. It is not practical to provide restraint against stress-relief movements, and appropriate allowance should instead be made for such movements in construction and planning.

Survey monitoring of the excavation and retaining walls would generally be appropriate to assess movement of any shoring walls during excavation, particularly where any deflection-sensitive structures or services are present behind the walls.

7.4 Groundwater and Dewatering

7.4.1 Groundwater Inflows

As can be seen from Drawing 304 in Appendix B, the proposed basement floor levels are below the measured groundwater levels, within bedrock. Some groundwater inflow or seepage is expected to occur through defects within the rock (eg bedding planes and joints). Inflow is expected to be greater in sandstone where fracturing is more closely spaced, or where existing iron-staining is present, suggesting past groundwater flows. Groundwater flow into the excavation through Units 3a, 3b and 3c are expected.

As noted in Section 7.1.2, comparison between past monitoring and recent measurements in the sideslope area suggest that the current groundwater levels are elevated compared to past monitoring results, possibly due to the prolonged period of wet weather earlier this year. Groundwater inflows through the Unit 3a and 3b sandstone, as suggested by the recent groundwater level measurements, may therefore only occur following periods of wetter weather, or during years of relatively wet weather.

The results of permeability testing indicated hydraulic conductivities ranging from 1.5x10^{-8} m/s to 4.2x10^{-6} m/s in the C3 area, with other results in the Stage 2 area falling within this range. This range is generally considered to be consistent with expected permeabilities in Hawkesbury Sandstone, though noting that the upper permeabilities are relatively high compared to typical values, but are nonetheless within previous DP experience in similar ground conditions.

The test results in the Stage 2 area did not indicate a strong correlation between hydraulic conductivity and the units of the geotechnical model, as can be seen in Figure 3, although the variability in hydraulic conductivity does appear to reduce in the underlying, Unit 3D materials.

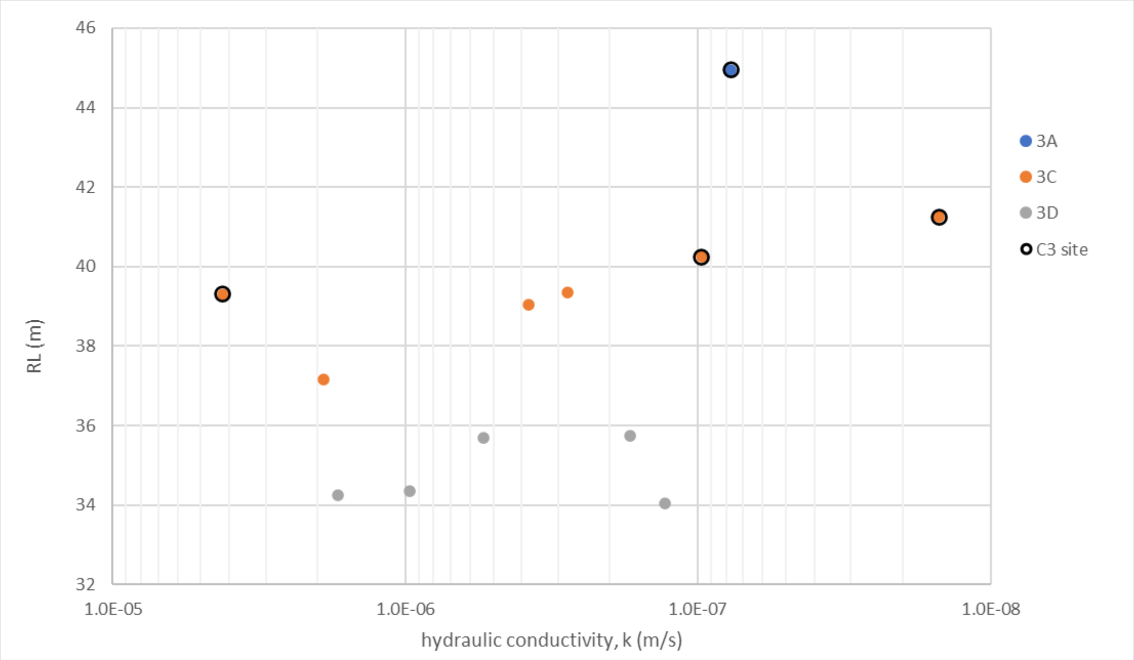


Figure 3: Summary of results of Hydraulic Conductivity (k) for Stage 2 area permeability tests, with respect to the Geotechnical Model Units 3A, 3C and 3D.

In considering these results, it is noted that the gravel pack (and screened length) interval was often located at depths where the rock core indicated higher fracture spacings, in order to capture data from expected higher permeability zones.

Estimates of medium to long-term groundwater inflow to a drained C3 basement excavation, have been separately analysed by Seep/W and reported in DP Memo 86043.06.R.004, dated 18 June 2021. The analysis, suggests likely groundwater inflows of approximately 2 ML/year into the C3 basement excavation. Higher inflows would be expected immediately following initial excavation, as stored water is lost to the excavation and groundwater levels around the basement stabilise, with inflow levels expected to stabilise to long-term typical inflows in approximately one year after excavation. Fluctuations in groundwater inflow will still occur following periods of rainfall.

While not identified by the current investigation, dykes or thrust faults may be associated with significantly increased permeabilities, relative to those considered in the inflow estimates, if encountered during excavation. While initial inflows from the defects would be significantly higher, their medium and long-term influence on inflows will depend on their continuity and connectivity to defects within and beyond the site.

7.4.2 Management of Groundwater Seepage

Based on the above inflow estimates, it is considered that a drained basement would be technically feasible for the C3 basement, with manageable water inflows expected for a robust permanent basement drainage system. As discussed in the following section, however, current government regulations should be considered, as well as the long-term costs of maintaining such a drainage/pumping system and any levies or costs associated with groundwater treatment (if required) and disposal.

It is understood that a Water Access Licence exemption would apply to the excavation of the building and for ongoing dewatering of the basement, subject to monitoring requirements, if groundwater inflows are less than 3 ML/year. The inflow estimates suggest that groundwater inflows to a drained basement are likely to be less than 3 ML/year but may possibly exceed these values.

Options for management of short and long-term groundwater inflows may therefore include the following:

1. A watertight, ‘tanked’ basement would reduce or remove risks associated with obtaining licenses, and ongoing management and maintenance of a drained basement.
2. Design and construction of a drained basement based on the expected inflows of less than 3 ML/year. This would require a commitment to construction-stage (and possibly longer-term) grouting, if required, in areas of higher inflow to manage the groundwater inflows to avoid exceedances. Approvals, monitoring and reporting of groundwater inflows will still be required. If elevated groundwater inflows are not effectively managed by grouting, then dewatering, excavation and construction may need to cease until management works are effective, or until Option 1 or 3 can be implemented.
3. Design and construction of a drained basement allowing for groundwater inflows to potentially exceed 3 ML/year. This would require that groundwater take is approved and properly accounted for under a Water Access Licence. This would involve an ongoing commitment to the costs associated with obtaining and maintaining the Water Access Licence, potentially for the life of the building, and would be subject to licensing and approval. Obtaining the necessary entitlements is a separate process to DA and early discussions with the Natural Resources Access Regulator (NRAR) would be key to confirm that a license (and therefore a drained basement) is achievable. This option could allow groundwater inflows to revert to an exemption in the long term, if subsequent management works (eg grouting) can reduce groundwater inflows to less than 3 ML/year.

These options may be controlled by the associated approvals and licenses that are required prior to dewatering, rather than by DA approvals and therefore may be limited by the regulator. Early discussions with the regulator will be important to confirm that the adopted approach will be accepted.

During excavation, from a practical perspective, groundwater seepage into the basement excavation is likely to be readily managed using ‘sump-and-pump’ methods, in the temporary case, complemented by grouting if excessive local inflows occur. This is consistent with DP experience with other deep excavations near the subject site. As the C3 site is part of the state significant “Ivanhoe Estate” redevelopment, it is understood that a Water Supply Works Approval will not be required at this site, subject to assessment and the Conditions of Consent,

Further information may be required to support the assessment, such as the current groundwater monitoring program.

The selection of an appropriate strategy for basement design should therefore include consideration of the regulatory risks (ie whether or not the necessary approvals and licenses can be obtained, or Conditions of Consent become too onerous), construction stage risks (eg excessive costs or delays due to grouting and groundwater management, and dewatering or design changes), long-term risks (eg cost of ongoing groundwater management/licenses), and geotechnical risks (eg presence of a high-permeability defect at the base of the excavation), as well as the known costs of design and construction.

Excavation of the basement would largely involve excavation in sandstone in the usual manner. Targeted grouting of bedding planes and joint swarms below the groundwater table may be appropriate to limit groundwater inflows into the basement to facilitate temporary management of groundwater. Grouting for groundwater management may only be economical where significant groundwater inflows are relatively localised, and of limited permeability, as grouting of large areas or where significant inflow is occurring can be costly and time consuming.

If a tanked basement design is selected, this would involve the construction of a waterproof basement floor and walls, to reduce or prevent groundwater inflows into the basement. Given that deep groundwater fluctuations in the order of 1.5 m have been observed, it is recommended that allowance be made for potential deep groundwater level rises of at least a further 1.0 m above the highest measured deep (long-term) water values, (ie to a design level of RL 45.6, based on current data), for the tanked basement design. This is expected to also cater for the anticipated groundwater level increase of less than 0.5 m anticipated on the upslope side of the basement due to the damming effect of the basement. This should be confirmed by groundwater modelling and analysis, based on the proposed tanking design, noting that excessive groundwater increases may require drainage around the outside of the tanked basement.

Seepage above the level of (partial) basement tanking may still occur due to higher, transient groundwater levels, particularly following periods of wet weather, and as such the basement design should allow for drainage of any groundwater seepage above the level of the tanked basement design, such as by a series of relief drains at the design level of tanking. For a tanked design based on the above recommendations, such seepage is expected to be below the 3 ML/year threshold, but would still require monitoring and reporting of this seepage ‘take’. Alternatively, the basement may be designed as fully tanked (i.e. waterproof walls to the ground surface), to effectively eliminate even short-term seepage into the basement. Any tanked (or partially tanked) basement design must also consider uplift forces that may arise.

Seepage is likely to be iron-rich and a precipitate (gelatinous ‘sludge’) may develop within drains over time, which could cause ‘clogging’ and blockage of drainage lines and pumps. Allowance should be made for future maintenance to clear such material from drainage lines and from pump fixtures.

It is noted that, given the relatively low permeability of the sandstone, any dewatering activities are expected to only cause drawdown to a relatively short distance from the C3 basement. Given that the groundwater levels are within bedrock, dewatering activities are not expected to create any risk of ground surface settlement, or have any influence on acid sulphate soils.

7.5 Foundations

The excavation for the C3 basement will extend into medium and high strength sandstone, and shallow foundations are therefore expected to be adopted to support the building loads.

Preliminary rock classification of the sandstone below RL 40 at the subject bores has been undertaken for foundation performance based on Pells et al (1998) and summarised in Table 5. These classifications are for foundation performance, only, and accordingly the rock 'strength' has been downgraded due to defects. A 1.0 m plan footing dimension has been assumed to perform the classification.

Table 5: Sandstone Foundation Classification at Bore Locations Below RL41

Sandstone Class	RL at Bore					
	103	104	105	106	117	118
III/IV	41.0	39.7	41.0 to 38.0 or below 36.2	41.0, but not below RL38.5	41.0	41.0
II/III	41.0	38.4	41.0 to 38.0 or below 36.2	41.0, but not below RL38.5	39.7, but not below RL39.1	41.0, but not below RL37.6
I/II	41.0 to 39.1 or below 38.5	38.4	41.0 to 38.0 or below 36.2	41.0, but not below RL38.5	NA	40.7, but not below RL37.6

Note: The classification is based on an interval of rock below the foundation level, with the interval dependent on the plan dimension of the footings.

As can be observed in the above table, a range of allowable bearing pressures may be adopted, though higher classifications may be more difficult to achieve on site, and so require additional excavation and/or re-design during the construction stage, depending on local conditions.

Maximum allowable bearing pressures for the design of shallow foundations founded on sandstone below bulk excavation level are provided in Table 6.

Table 6: Foundation Design Parameters

Sandstone Class	Allowable Bearing Pressure[1,2] (MPa)	Ultimate Bearing Pressure[2,3] (MPa)	Typical Youngs Modulus (MPa)	Minimum Additional Testing / Requirements[4]
III/IV	3.5	15	350	-

Sandstone Class	Allowable Bearing Pressure[1,2] (MPa)	Ultimate Bearing Pressure[2,3] (MPa)	Typical Youngs Modulus (MPa)	Minimum Additional Testing / Requirements[4]
II/III	6	40	900	Spoon testing of 1/3 of footings
I/II	10	100	1500	Additional cored boreholes (e.g. after excavation to basement level), and spoon testing of 1/2 of footings

Note:
1. Allowable pressures assume allowable settlements of less than 1% of the minimum footing plan dimension. Alternative, settlements can be estimated for the proposed load based on the typical Youngs Modulus.
2. All bearing pressures may be limited by defects, subject to inspection of the excavation and possible spoon testing, which may require the bearing pressure to be downgraded. Allowable bearing pressures assume that the bedrock is in a confined state, and that no nearby current or future excavations are present below an imaginary 'influence' line drawn at 1H:1V down from the edge of the footing. Such excavations would require inspection to confirm that adverse jointing is not present. Reduced values of approximately 50% of the value given in Table 6 may also apply.
3. Ultimate values assume settlement of more than 5% to 10% of the minimum footing plan dimension.
4. Geotechnical inspection of all footing excavations is recommended to confirm that the material is consistent with the design requirements; the minimum testing is to provide additional information on defects to confirm that foundation performance is as expected. Additional or lesser testing may be warranted, subject to the results of initial foundation testing and depending on the design bearing pressures.

All foundations should be inspected by a geotechnical professional following excavation and cleaning, to confirm that the foundation material is consistent with the design requirements.

The higher bearing pressures given in Table 6 require the additional testing outlined in that table, and may be associated with a higher risk of inspection 'failures'. This use of a 10 MPa design bearing pressure would require additional cored borehole investigation, which may indicate that the sandstone does not meet the requirements of Class I/II Sandstone.

Spoon testing should be carried out in at least one third of all footings that are designed for an allowable end bearing capacity of more than 3.5 MPa. Spoon testing involves drilling a 50 mm diameter hole below the base of the footing, to a depth of at least 1.5 times the footing width, with the hole left full of water for 24 hours prior to testing to check for the presence of weak/clay bands. If excessive weak seams are detected then the foundation capacity may need to be downgraded, or the footings taken deeper to reach suitable foundation material.

For shoring piles founded in Class 3c materials, but above bulk excavation level, the ultimate bearing capacity will be the unconfined strength of the underlying bedrock, but may be reduced by adverse defects, if present below the foundation. Given these risks, it is suggested that design be based on an ultimate bearing pressure of no greater than 3 MPa, and an 'allowable' bearing pressure of no greater than 1 MPa. The vertical component of any anchors should be considered in the total loads on the pile. The vertical bearing pressure should be reviewed during excavation, prior to vertical loading of the piles.

Should thrust faults or dykes be identified near foundation level then the foundation parameters given in Table 6 may not be achieved, and re-design may be required in the affected area to suit to the conditions encountered.

7.6 Further Investigation and Assessment

Additional investigation and/or assessment may be appropriate, depending on the detailed design and planning decisions for the proposed site, and to support a dewatering management plan, if required. Such works may include:

- Water quality tests to provide information on the chemical composition of groundwater at the site, to support planning for groundwater management and disposal assessment;
- Repeat permeability tests at standpipe locations, to confirm the 'repeatability' of the current test data (particularly if a drained basement is to be adopted); and
- Additional investigation, to reduce the geotechnical risk of excessive inflows to the excavation. This may include inclined boreholes, to provide greater coverage of the site area, and reduce (though not eliminate) the risk of unexpected defects that may cause concentrated seepage inflows to the excavation, and/or 'pilot' excavations to observe inflows to a test pit or similar, excavated to bulk excavation level.

It is noted that data loggers have been installed in Bores 106 and118A (and Bores 114 and 111A, downslope) to monitor groundwater levels. This monitoring is ongoing, and the results of the groundwater monitoring will be reported, separately.

8. References

Pells, P. J., Mostyn, G., & Walker, B. F. (1998). Foundations on Sandstone and Shale in the Sydney Region. *Australian Geomechanics, No 33 Part 3*, 17-29.

9. Limitations

Douglas Partners (DP) has prepared this report for this project at Midtown, Macquarie Park in accordance with the Consultancy Services Agreement dated 26 April 2021, and approved variations. This report is provided for the exclusive use of Frasers Property Ivanhoe Pty Ltd for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

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

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

The assessment of atypical safety hazards arising from this advice is restricted to the geotechnical components set out in this report and based on known project conditions and stated design advice and assumptions. While some recommendations for safe controls may be provided, detailed 'safety in design' assessment is outside the current scope of this report and requires additional project data and assessment.

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 investigation/report did not include the assessment of surface or sub-surface materials or groundwater for contaminants, within or adjacent to the site. Should evidence of fill 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 fill may contain contaminants and hazardous building materials.

Douglas Partners Pty Ltd

Appendix A

About This Report

About this Report

Douglas Partners

Introduction

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

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

Copyright

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

Borehole and Test Pit Logs

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

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

Groundwater

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

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

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

Reports

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

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

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

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

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

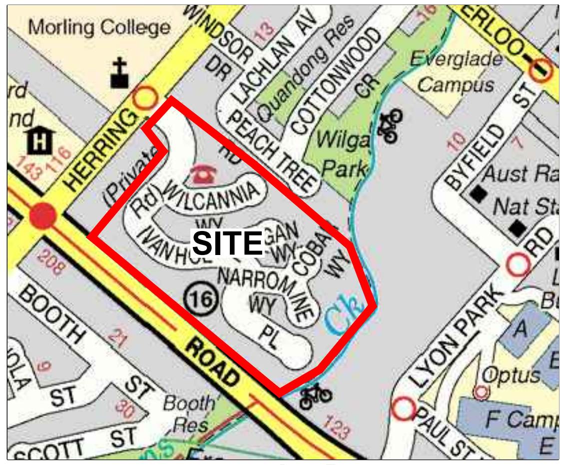
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate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

LEGEND

- Borehole Location (C3 Site)
- Cored Bore Location (2021, Beyond C3 Site)
- Previous (2017) Cored Bore Location (Beyond C3 Site)
- Standpipe Location (Current)
- Standpipe Location (Damaged or Missing)
- C3 Area Boundary
- Site Boundary

NOTE:
1: Base image from MetroMap (Dated 15.04.2021)

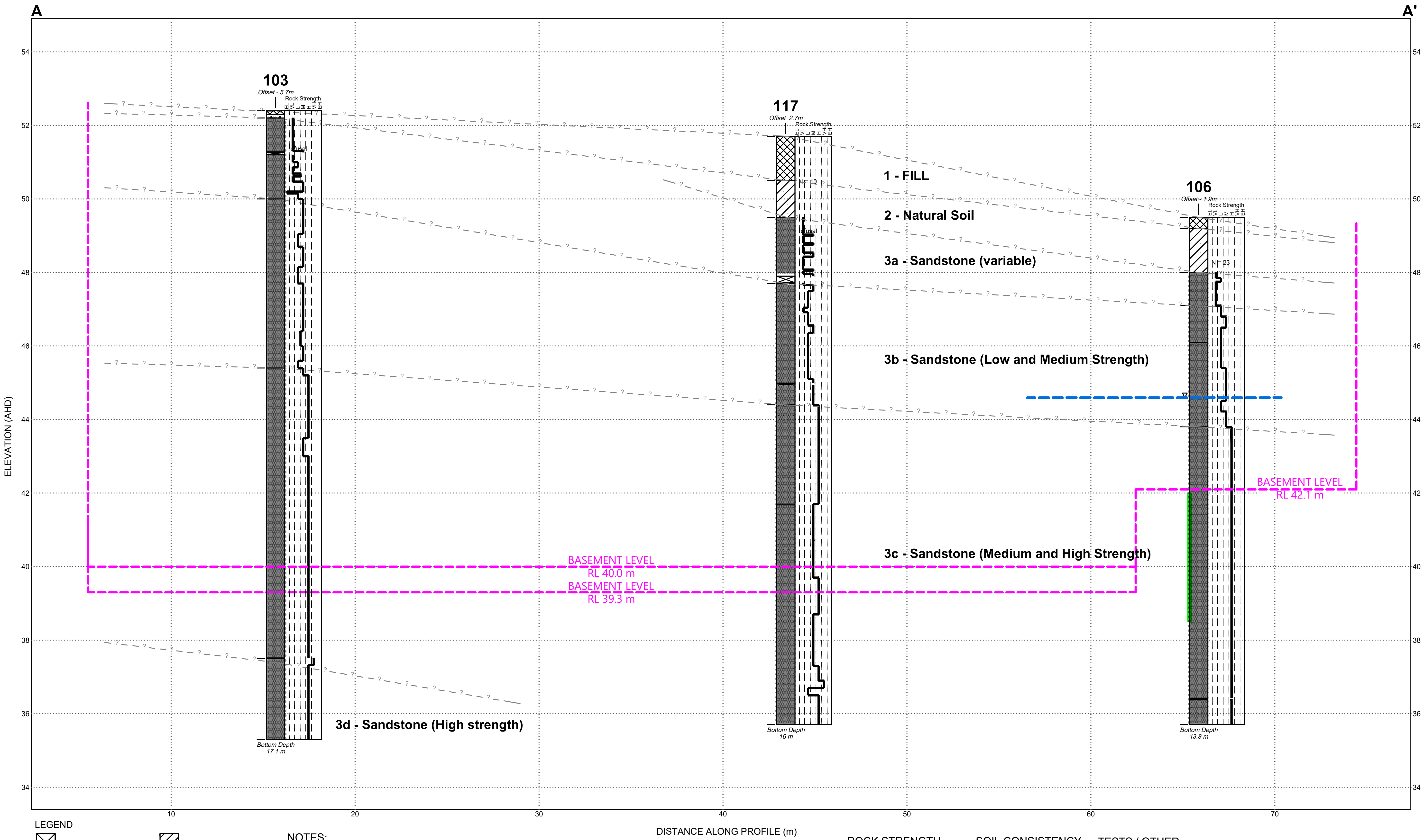
0 10 20 30 40 50 75 100m
1:1000 @ A3

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Frasers Property Ivanhoe Pty Ltd
OFFICE: Sydney
DRAWN BY: PSCH/MG
SCALE: 1:1000 @ A3
DATE: 11.06.2021

TITLE: **Test Location Plan - C3 Site**
Proposed Residential Development
Midtown, Macquarie Park

PROJECT No: 86043.06
DRAWING No: 301
REVISION: 1



LEGEND

☒	Core Loss
△	Concrete
▩	Filling
▦	Sandstone
▨	Sandy Clay

NOTES:
1. Subsurface conditions are accurate at the borehole locations only.
Variations in subsurface conditions may occur between borehole locations.
Interpreted strata boundaries are approximate and should be used as a guide only.
2. Summary logs only and should be read in conjunction with detailed logs.
3. Horizontal and vertical scales are not equal.

ROCK STRENGTH
EL - Extremely Low
VL - Very Low
L - Low
M - Medium
H - High
VH- Very High

SOIL CONSISTENCY
vs - Very Soft
s - Soft
f - Firm
st - Stiff
vst - Very Stiff
h - Hard

TESTS / OTHER
N - Standard penetration test value
— ? — - Interpreted geotechnical boundary
▽ - Water level
– – - Interpreted deep water table
— - Standpipe Gravel Pack Length

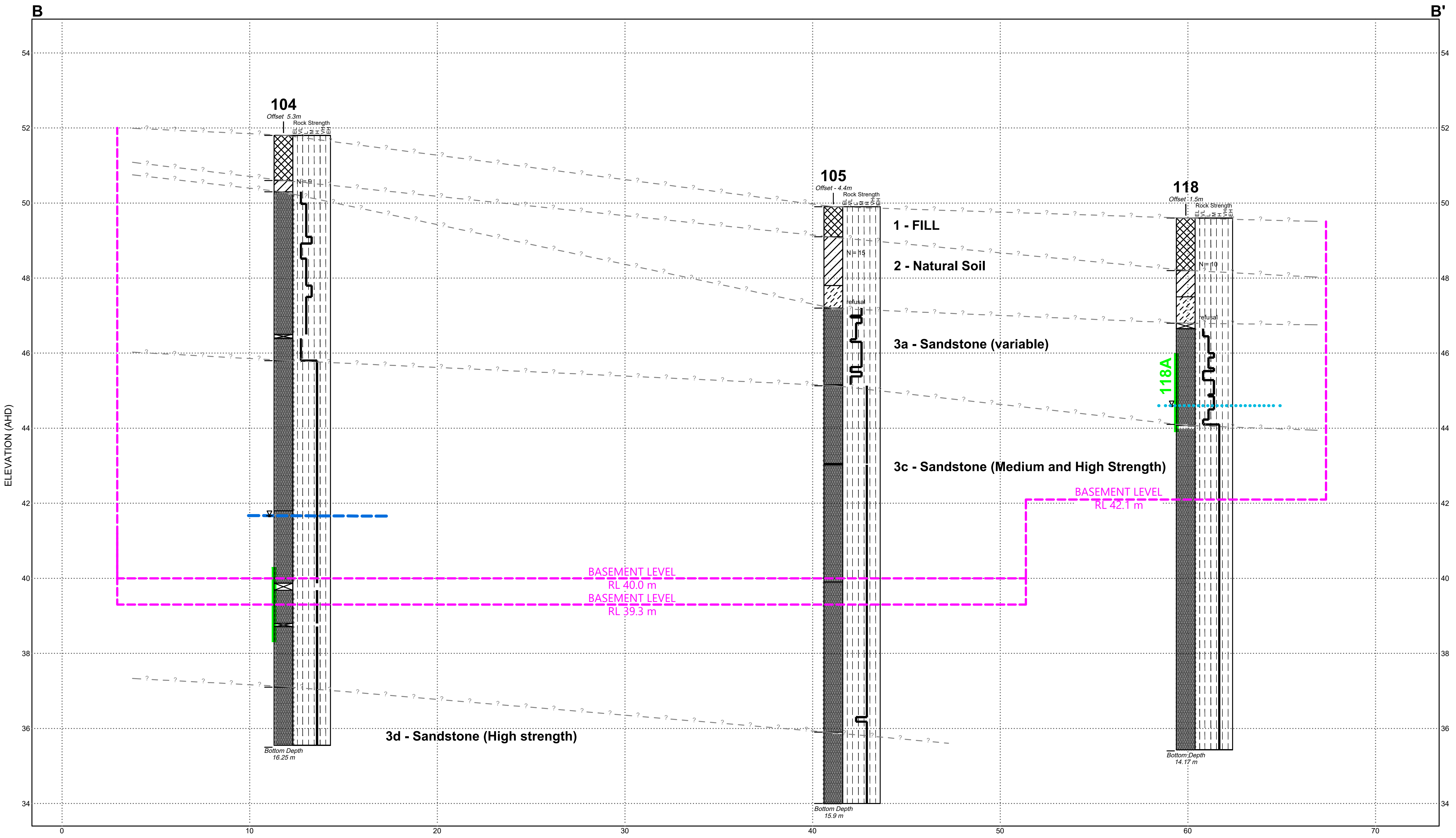
0 4
Horizontal Scale (metres)
Vertical Exaggeration = 2.0

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Frasers Property Ivanhoe Pty Ltd
OFFICE: Sydney
DRAWN BY: SCP
SCALE: 1:200 (H) 1:100 (V) @ A3
DATE: 11.06.2021

TITLE: **Interpreted Geotechnical Cross-Section A-A'**
Proposed Stage 2 Development - Site C3
Midtown, Maquarie Park

PROJECT No: 86043.06
DRAWING No: 302
REVISION: 1



LEGEND

☒ Core Loss

▨ Sandy Clay

▨ Clayey Sand

☒ Filling

☐ Sandstone

NOTES:

1. Subsurface conditions are accurate at the borehole locations only.
   Variations in subsurface conditions may occur between borehole locations.
   Interpreted strata boundaries are approximate and should be used as a guide only.
2. Summary logs only and should be read in conjunction with detailed logs.
3. Horizontal and vertical scales are not equal.

ROCK STRENGTH
EL - Extremely Low
VL - Very Low
L - Low
M - Medium
H - High
VH- Very High

SOIL CONSISTENCY
vs - Very Soft
s - Soft
f - Firm
st - Stiff
vst - Very Stiff
h - Hard

TESTS / OTHER
N - Standard penetration test value
— ? — - Interpreted geotechnical boundary
▽ - Water level
— — - Interpreted deep water table
— - Standpipe Gravel Pack Length
······ - Interpreted transient water table

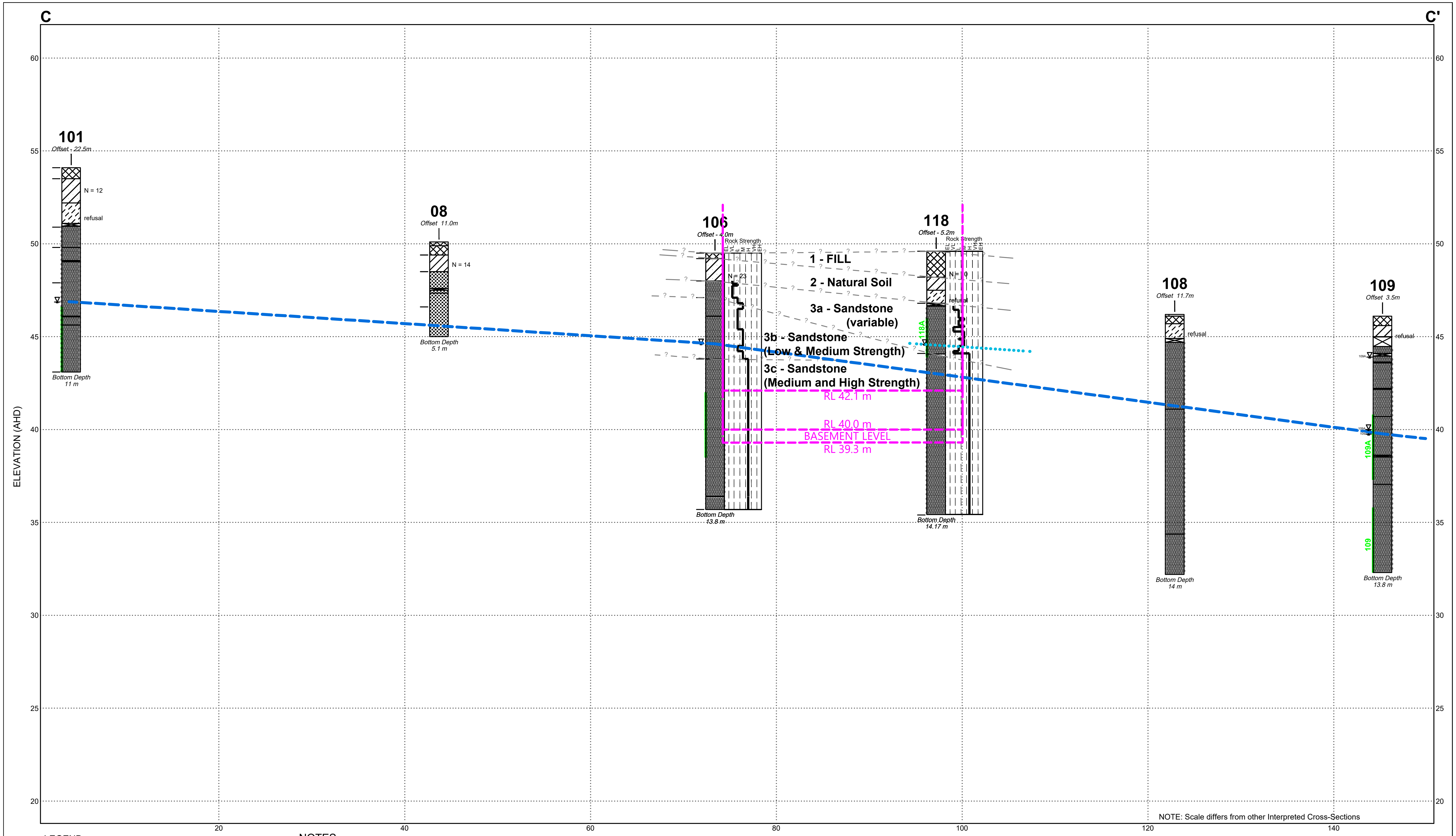
0 4
Horizontal Scale (metres)
Vertical Exaggeration = 2.0

Douglas Partners
Geotechnics | Environment | Groundwater

CLIENT: Frasers Property Ivanhoe Pty Ltd
OFFICE: Sydney
DRAWN BY: SCP
SCALE: 1:200 (H) 1:100 (V) @ A3
DATE: 11.06.2021

TITLE: **Interpreted Geotechnical Cross-Section B-B'**
Proposed Stage 2 Development - Site C3
Midtown, Maquarie Park

PROJECT No: 86043.06
DRAWING No: 303
REVISION: 1



LEGEND

☒	Core Loss	▩	Filling
▤	Bricks	▦	Sandstone
▨	Clay	▩	Sandstone coarse grained
▨	Clayey Sand	▨	Sandy Clay

NOTES:

1. Subsurface conditions are accurate at the borehole locations only.
   Variations in subsurface conditions may occur between borehole locations.
   Interpreted strata boundaries are approximate and should be used as a guide only.
2. Summary logs only and should be read in conjunction with detailed logs.
3. Horizontal and vertical scales are not equal.
4. Bores 01 to 13 were logged based on AS1726-1993, and differences in the simplified logs should be expected between those logs and logging according to current practice, based on AS1726-2017, particularly in the upper, weathered rock profile.

ROCK STRENGTH

EL -	Extremely Low
VL -	Very Low
L -	Low
M -	Medium
H -	High
VH-	Very High

SOIL CONSISTENCY

vs -	Very Soft
s -	Soft
f -	Firm
st -	Stiff
vst -	Very Stiff
h -	Hard

TESTS / OTHER

N	- Standard penetration test value
— ? — —	- Interpreted geotechnical boundary
▽	- Water level
━ ━ ━	- Interpreted deep water table
━━━	- Standpipe Gravel Pack Length
•••••••	- Interpreted transient water table

0 8

Horizontal Scale (metres)

Vertical Exaggeration = 2.0

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Frasers Property Ivanhoe Pty Ltd
OFFICE: Sydney
DRAWN BY: SCP
SCALE: 1:400 (H) 1:200 (V) @ A3
DATE: 11.06.2021

TITLE: **Interpreted Cross-Section C-C'**
Proposed Stage 2 Development - Site C3
Midtown, Maquarie Park

PROJECT No: 86043.06
DRAWING No: 304
REVISION: 1

Appendix C

Results of Field Work

Sampling Methods

Sampling

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

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

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

Test Pits

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

Large Diameter Augers

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

Continuous Spiral Flight Augers

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

Non-core Rotary Drilling

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

Continuous Core Drilling

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

Standard Penetration Tests

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

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

The test results are reported in the following form.

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

  4,6,7
  N=13

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

  15, 30/40 mm

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

Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests

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

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

Soil Descriptions

Description and Classification Methods

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

Soil Types

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

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	19 - 63
Medium gravel	6.7 - 19
Fine gravel	2.36 – 6.7
Coarse sand	0.6 - 2.36
Medium sand	0.21 - 0.6
Fine sand	0.075 - 0.21

Definitions of grading terms used are:

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

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

In fine grained soils (>35% fines)

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

In coarse grained soils (>65% coarse)
- with clays or silts

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

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

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

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

Soil Descriptions

Cohesive Soils

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

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

Cohesionless Soils

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

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

Soil Origin

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

- Residual soil - derived from in-situ weathering of the underlying rock;
- Extremely weathered material – formed from in-situ weathering of geological formations. Has soil strength but retains the structure or fabric of the parent rock;
- Alluvial soil – deposited by streams and rivers;
- Estuarine soil – deposited in coastal estuaries;
- Marine soil – deposited in a marine environment;
- Lacustrine soil – deposited in freshwater lakes;
- Aeolian soil – carried and deposited by wind;
- Colluvial soil – soil and rock debris transported down slopes by gravity;
- Topsoil – mantle of surface soil, often with high levels of organic material.
- Fill – any material which has been moved by man.

Moisture Condition – Coarse Grained Soils

For coarse grained soils the moisture condition should be described by appearance and feel using the following terms:

Dry (D)	Non-cohesive and free-running.
Moist (M)	Soil feels cool, darkened in colour. Soil tends to stick together. Sand forms weak ball but breaks easily.
Wet (W)	Soil feels cool, darkened in colour. Soil tends to stick together, free water forms when handling.

Moisture Condition – Fine Grained Soils

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

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

Rock Descriptions

Rock Strength

Rock strength is defined by the Unconfined Compressive Strength and it 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 Point Load Strength Index $Is_{(50)}$ is commonly used to provide an estimate of the rock strength and site specific correlations should be developed to allow UCS values to be determined. The point load strength test procedure is described by Australian Standard AS4133.4.1-2007. The terms used to describe rock strength are as follows:

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

* 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
Residual Soil	RS	Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.
Extremely weathered	XW	Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible
Highly weathered	HW	The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Moderately weathered	MW	The whole of the rock material is discoloured , usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.
Slightly weathered	SW	Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.
Fresh	FR	No signs of decomposition or staining.
Note: If HW and MW cannot be differentiated use DW (see below)		
Distinctly weathered	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching or may be decreased due to deposition of weathered products in pores.

Rock Descriptions

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 occasional fragments
Fractured	Core lengths of 30-100 mm with occasional shorter and longer sections
Slightly Fractured	Core lengths of 300 mm or longer with occasional sections of 100-300 mm
Unbroken	Core contains very few fractures

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 stronger. 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
▽	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

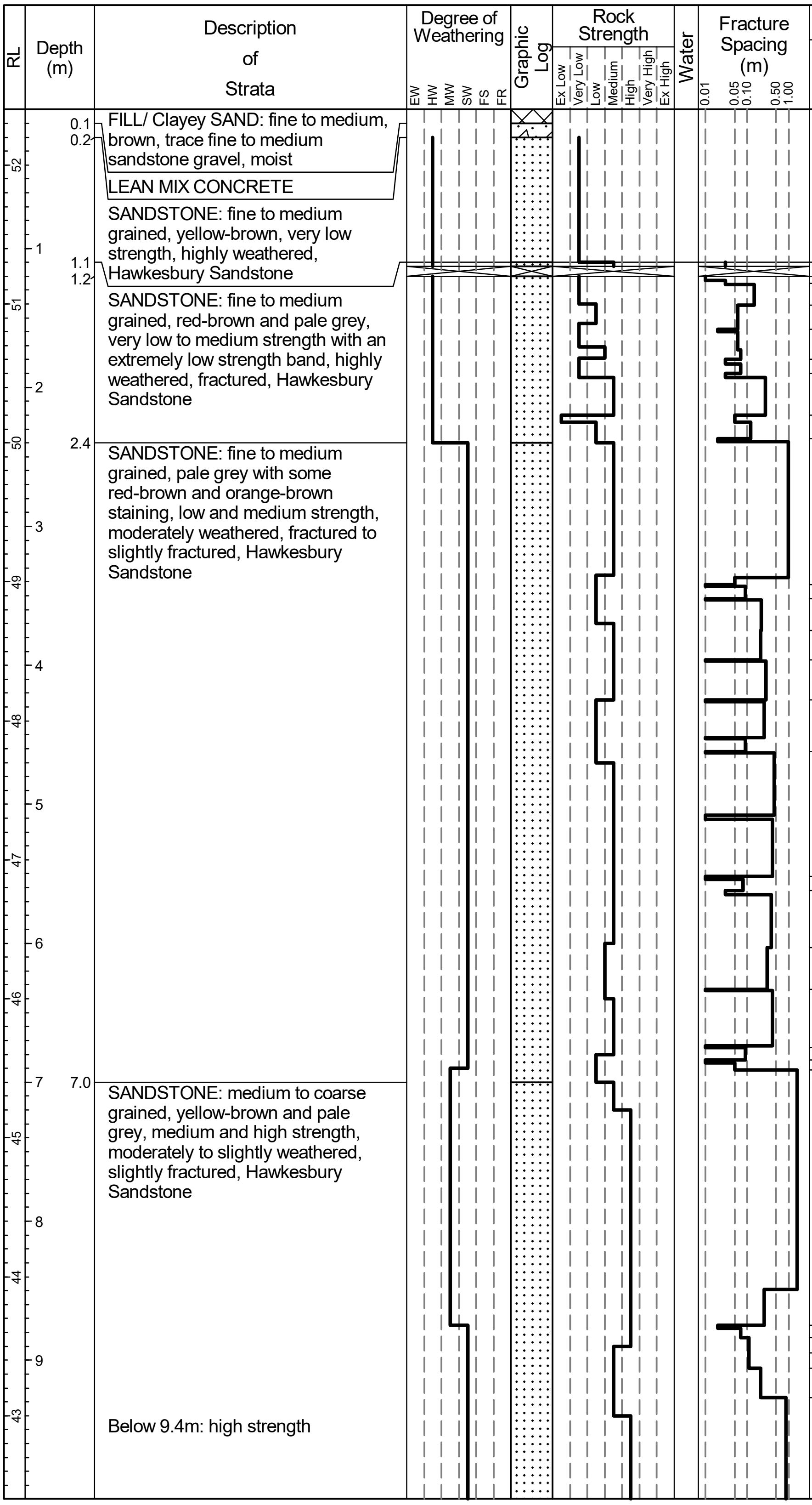
BOREHOLE LOG

CLIENT: Frasers Property Ivanhoe Pty Ltd
PROJECT: Proposed Stage 2 Development
LOCATION: Midtown, Maquarie Park

SURFACE LEVEL: 52.4 AHD
EASTING: 325617.7
NORTHING: 6260365.1
DIP/AZIMUTH: 90°/--

BORE No: 103
PROJECT No: 86043.06
DATE: 28/4/2021
SHEET 1 OF 2

Depth (m)	Description of Strata
0.1	FILL/ Clayey SAND: fine to medium, brown, trace fine to medium sandstone gravel, moist
0.2	LEAN MIX CONCRETE
	SANDSTONE: fine to medium grained, yellow-brown, very low strength, highly weathered, Hawkesbury Sandstone
1.1	
1.2	SANDSTONE: fine to medium grained, red-brown and pale grey, very low to medium strength with an extremely low strength band, highly weathered, fractured, Hawkesbury Sandstone
2.4	SANDSTONE: fine to medium grained, pale grey with some red-brown and orange-brown staining, low and medium strength, moderately weathered, fractured to slightly fractured, Hawkesbury Sandstone
7.0	SANDSTONE: medium to coarse grained, yellow-brown and pale grey, medium and high strength, moderately to slightly weathered, slightly fractured, Hawkesbury Sandstone Below 9.4m: high strength



Discontinuities

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

- 1.13m: CORE LOSS: 70mm
- 1.2m: Fg 30mm
- 1.25m: Cs 30mm
- 1.41-1.73m: B0°-5° (x6) pl, ro, cln & cly vn
- 1.58m: Cs 20mm
- 1.8m: Cs30mm
- 1.9m: Cs 30mm
- 2.2m: Ds 50mm
- 2.37m: Cs 20mm
- 3.37m: B0°, pl, ro, cly 2mm
- 3.42m: Cs 10mm
- 3.52m: Cs 15mm, fe stn
- 3.75m: J30°, pl, ro, cbs vn
- 3.96m: Cs 10mm
- 4.25m: Cs 10mm
- 4.52m: Ds 10mm
- 4.62m: Ds 10mm
- 5.08m: B0°-5° (x2), pl, sm, cly co
- 5.45m: Cs 10mm
- 5.52m: Fg 20mm, cly inf, fe stn
- 5.62m: J45° (x2), pl, ro, fe stn
- 6.03m: J80°, pl, sm, cly vn
- 6.33m: Cs 10mm
- 6.74m: Ds 60mm
- 6.75m: B5°, pl, ro, fe stn
- 6.84m: B0° (x3), pl, ro, fe stn
- 6.91m: J70°-90°, ir, ro, fe stn
- 8.49m: B10°, pl, ro, fe stn
- 8.75m: Cs 20mm
- 8.84m: B0°, pl, ro, fe stn
- 8.95m: B5°, un, ro, cly vn
- 9.06m: B5°, pl, ro, fe stn
- 9.06-9.42m: B0°-10° (x4), pl, ro, fe stn
- 9.27m: B10°, pl, sm, cly 1mm, fe stn

Sampling & In Situ Testing

Type	Core Rec. %	RQD %	Test Results & Comments
C			
A			
S			25/70 mm refusal
C	94	24	PL(A) = 0.17
C	100	90	PL(A) = 0.3 PL(A) = 0.73 PL(A) = 0.36 PL(A) = 0.53
C	100	85	PL(A) = 0.4 PL(A) = 0.53 PL(A) = 1.9
C	100	82	PL(A) = 0.39

RIG: Explora **DRILLER:** JD **LOGGED:** TM **CASING:** HW to 1.0m, HQ to 1.1m

TYPE OF BORING: Solid flight auger (TC-bit) to 1.0m; Rotary to 1.1m; NMLC-Coring to 17.1m; PCD to 11.0 m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Groundwater well installed to 15.0m (screen 15.0-12.0m; blank 12.0-0.0m; gravel 15.0-11.5m; bentonite 11.5-11.0m; backfill to GL; gatic at surface); Coordinates and surface levels obtained from differential GPS

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)

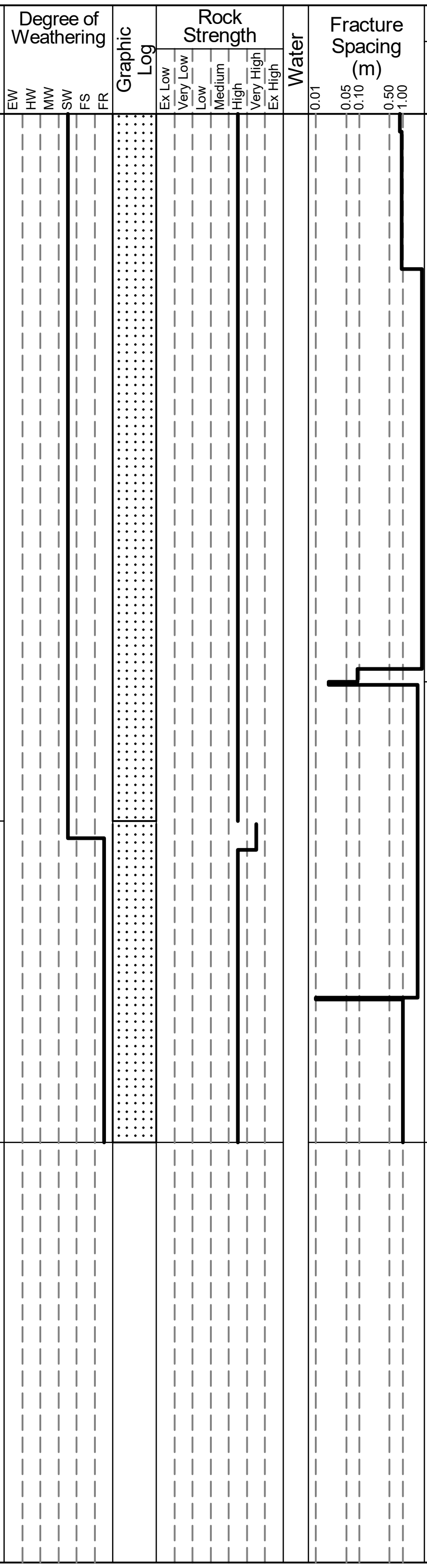
Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Frasers Property Ivanhoe Pty Ltd
PROJECT: Proposed Stage 2 Development
LOCATION: Midtown, Maquarie Park

SURFACE LEVEL: 52.4 AHD
EASTING: 325617.7
NORTHING: 6260365.1
DIP/AZIMUTH: 90°/--

BORE No: 103
PROJECT No: 86043.06
DATE: 28/4/2021
SHEET 2 OF 2



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

Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
	SANDSTONE: medium to coarse grained, yellow-brown and pale grey, medium and high strength, moderately to slightly weathered, slightly fractured, Hawkesbury Sandstone *(continued)*	10.12m: B5°, pl, ro, cly vn	C	100	82	PL(A) = 1.6
11	Below 11.07m: slightly fractured to unbroken	11.07m: B5°, pl, sm, cly co				PL(A) = 1.2
12			C	100	100	PL(A) = 1.6
13						PL(A) = 1.5
14		13.83m: B0°, pl, sm, cly 2mm, fe stn 13.92m: Fg 20mm, fe stn				PL(A) = 2
14.88	SANDSTONE: medium to coarse grained, pale grey, strength, fresh, slightly fractured to unbroken, Hawkesbury Sandstone					PL(A) = 3.5
16		16.1m: Fg 10mm	C	100	99	PL(A) = 1.9
17						PL(A) = 1
17.1	Bore discontinued at 17.1m Target depth reached					

RIG: Explora **DRILLER:** JD **LOGGED:** TM **CASING:** HW to 1.0m, HQ to 1.1m

TYPE OF BORING: Solid flight auger (TC-bit) to 1.0m; Rotary to 1.1m; NMLC-Coring to 17.1m; PCD to 11.0 m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Groundwater well installed to 15.0m (screen 15.0-12.0m; blank 12.0-0.0m; gravel 15.0-11.5m; bentonite 11.5-11.0m; backfill to GL; gatic at surface); Coordinates and surface levels obtained from differential GPS

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: 103 PROJECT: MACQUARIE PARK APRIL 2021

1.10 – 5.00m

BORE: 103 PROJECT: MACQUARIE PARK APRIL 2021

5.00 – 10.00m

BORE: 103 PROJECT: MACQUARIE PARK APRIL 2021

10.00 – 15.00m

BORE: 103 PROJECT: MACQUARIE PARK APRIL 2021

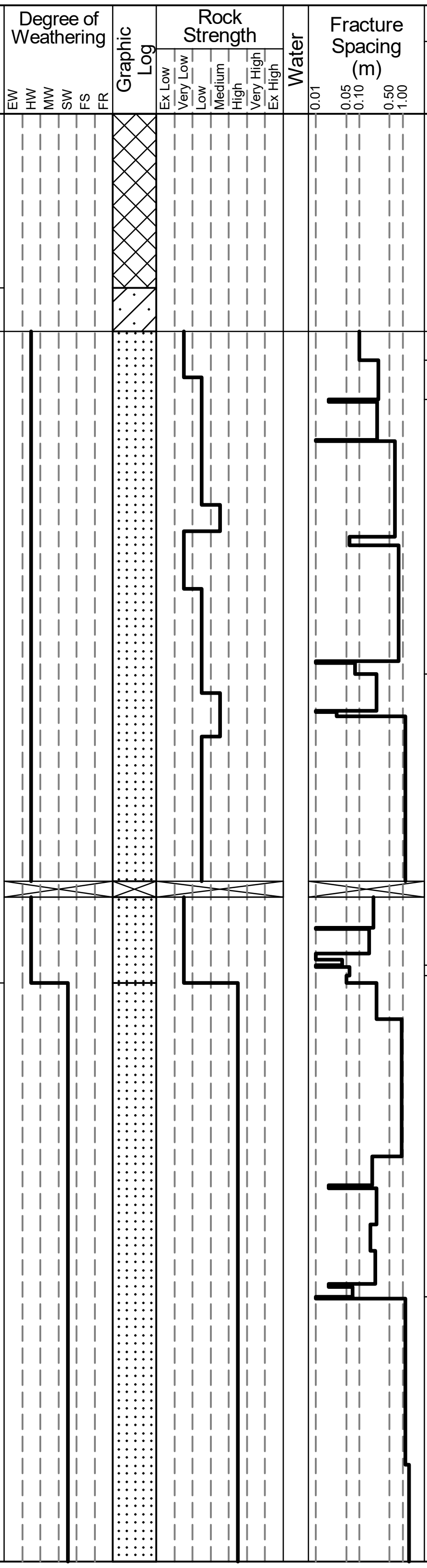
15.00 – 17.10m

BOREHOLE LOG

CLIENT: Frasers Property Ivanhoe Pty Ltd
PROJECT: Proposed Stage 2 Development
LOCATION: Midtown, Maquarie Park

SURFACE LEVEL: 51.8 AHD
EASTING: 325637.8
NORTHING: 6260346.9
DIP/AZIMUTH: 90°/--

BORE No: 104
PROJECT No: 86043.06
DATE: 27/4/2021
SHEET 1 OF 2



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

RL	Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
		FILL/ Sandy CLAY: low to medium plasticity, brown, fine to medium sand, with fine to medium sandstone gravel, w<PL		A			
				A			
51							
	1			A			
	1.2	Sandy CLAY CL-CI: low to medium plasticity, yellow-brown, fine to medium sand, w<PL, stiff, residual		S			3,3,6 N = 9
	1.5	SANDSTONE: fine to medium grained, yellow-brown then pale grey and red-brown, very low to low strength with some medium strength bands, highly weathered, fractured, Hawkesbury Sandstone	1.6m: B0°, pl, ro, cly vn 1.7m: B5°, pl, sm, cly 1mm 1.97m: Cs 20mm				PL(A) = 0.06
50							
	2			C	100	70	PL(A) = 0.08
			2.25m: Cs 10mm				
49							
	3		2.92m: Cs 60mm				PL(A) = 0.19
48							
			3.78m: Cs 10mm 3.87m: B0°, pl, ro, fe stn	C	100	75	
	4		4.12m: B5° (x2), pl, ro, fe stn & cly vn				PL(A) = 0.23
			4.48m: Ds 30mm				
47							
	5						PL(A) = 0.26
	5.41		5.3m: CORE LOSS: 110mm				
			5.62m: Cs 10mm				
46			5.8m: Fg 40mm, cly co				
	6 6.0	SANDSTONE: medium to coarse grained, orange brown and pale grey, high strength, slightly weathered, slightly fractured, Hawkesbury Sandstone	5.88m: Cs 10mm 5.95m: Ds 50mm				PL(A) = 1.1
			6.25m: B5°, pl, ro, fe stn				
45				C	97	65	
	7						PL(A) = 1.8
			7.20-7.67m: B0°-5° (x4), pl, ro, fe stn				
44							
			7.85m: J80°, pl, ro, cln				
	8						PL(A) = 1.1
			8.08m: Cs 20mm 8.17m: Cs 10mm				
43							
	9						PL(A) = 1.5
				C	100	100	
			9.33m: J80°, pl, ro, cln				
42							
	10.0						

RIG: Explora **DRILLER:** JD **LOGGED:** TM **CASING:** HW to 1.0m, HQ to 1.5m

TYPE OF BORING: Solid flight auger (TC-bit) to 1.0m; Rotary to 1.5m; NMLC-Coring to 16.25m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Coordinates and surface levels obtained from differential GPS

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

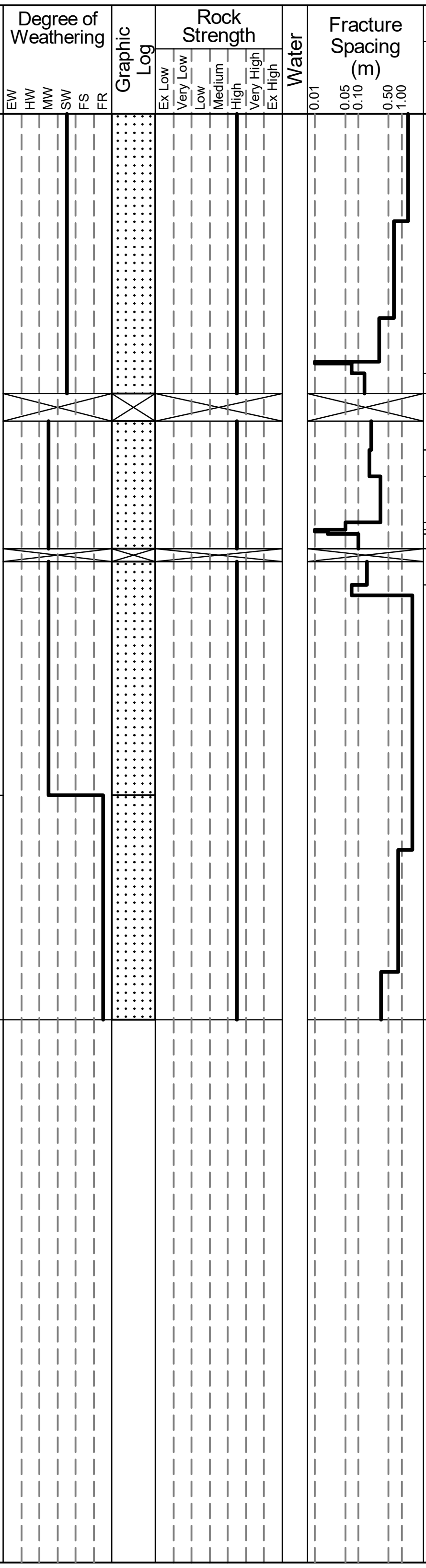
BOREHOLE LOG

CLIENT: Frasers Property Ivanhoe Pty Ltd
PROJECT: Proposed Stage 2 Development
LOCATION: Midtown, Maquarie Park

SURFACE LEVEL: 51.8 AHD
EASTING: 325637.8
NORTHING: 6260346.9
DIP/AZIMUTH: 90°/--

BORE No: 104
PROJECT No: 86043.06
DATE: 27/4/2021
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
	SANDSTONE: medium to coarse grained, orange brown and pale grey, high strength, slightly weathered, slightly fractured, Hawkesbury Sandstone *(continued)*	10.74m: B5°, pl, ro, cln	C	100	100	PL(A) = 1.6 PL(A) = 1.6
		11.41m: J30°, pl, ro, cly vn 11.71m: Cs 10mm 11.79m: B0°, pl, ro, cly vn 11.93m: CORE LOSS: 190mm	C	80	65	PL(A) = 2.2
12.12	Between 12.12-14.70m: red brown, moderately weathered	12.32m: B0°, pl, ro, fe stn 12.5m: J80°, pl, ro, fe stn 12.82m: B0°, pl, ro, cly vn 12.87m: Cs 10mm 12.9m: Ds 100mm 13m: CORE LOSS: 90mm 13.25m: B0°-5° (x2), pl, ro, fe stn	C	95	83	PL(A) = 1.6 PL(A) = 2.9
13.09						
14.7	SANDSTONE: medium to coarse grained, pale grey, high strength, fresh, slightly fractured to unbroken, Hawkesbury Sandstone	15.08m: B0°, pl, ro, cln 15.92m: B0°, pl ro, cly vn	C	100	100	PL(A) = 1.7 PL(A) = 1.7
16.25	Bore discontinued at 16.25m Target depth reached					



RIG: Explora **DRILLER:** JD **LOGGED:** TM **CASING:** HW to 1.0m, HQ to 1.5m
TYPE OF BORING: Solid flight auger (TC-bit) to 1.0m; Rotary to 1.5m; NMLC-Coring to 16.25m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: Coordinates and surface levels obtained from differential GPS

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: 104 PROJECT: MACQUARIE PARK APRIL 2021

1.50 – 6.00m

BORE: 104 PROJECT: MACQUARIE PARK APRIL 2021

6.00 – 11.00m

BORE: 104 PROJECT: MACQUARIE PARK APRIL 2021

11.00 – 16.00m

BORE: 104 PROJECT: MACQUARIE PARK APRIL 2021

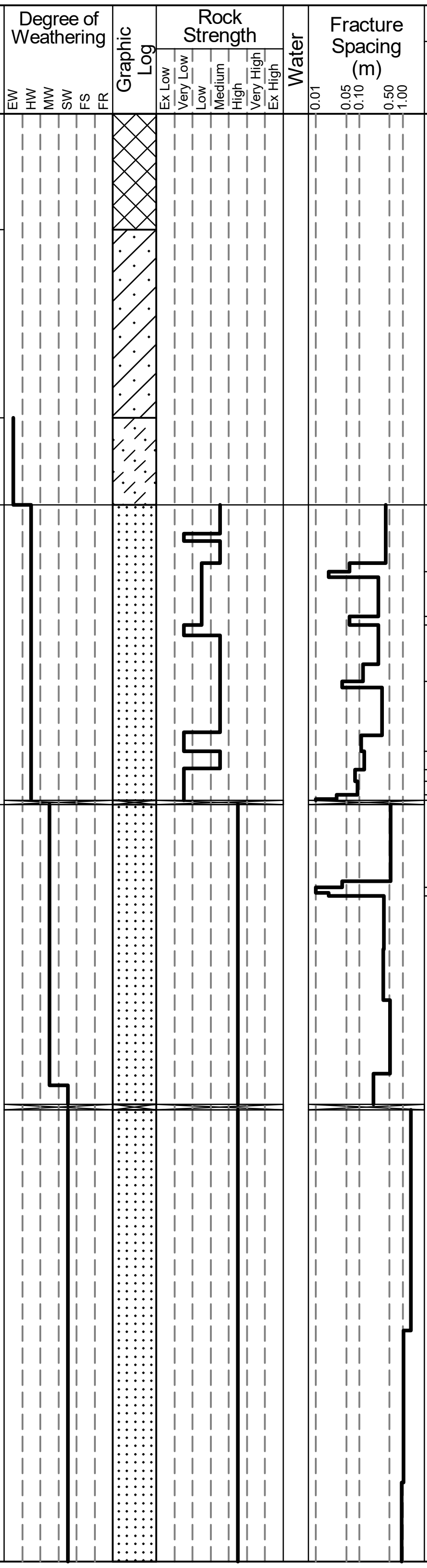
16.00 – 16.25m

BOREHOLE LOG

CLIENT: Frasers Property Ivanhoe Pty Ltd
PROJECT: Proposed Stage 2 Development
LOCATION: Midtown, Maquarie Park

SURFACE LEVEL: 49.9 AHD
EASTING: 325665.6
NORTHING: 6260360.3
DIP/AZIMUTH: 90°/--

BORE No: 105
PROJECT No: 86043.06
DATE: 24/4/2021
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
		FILL/ SAND: fine to medium, dark brown, with clay, moist		A A			
49 48	0.8 1 2	Sandy CLAY CI: medium plasticity, yellow-brown, fine to medium sand, w<PL, stiff, residual		A S A			4,5,10 N = 15
	2.1	Clayey SAND SC: fine to medium sand, pale grey and red-brown, moist, very dense, extremely weathered Hawkesbury Sandstone		S			25/70 mm refusal
47 46	2.7 3 4	SANDSTONE: fine to medium, pale grey and red-brown, very low to medium strength, highly weathered, fractured, Hawkesbury Sandstone	3.1m: J50°, pl, cly co 3.16m: J50°-60° (x3), pl, ro, cln 3.47m: B10°, pl, ro, cln 3.53m: B5°, pl, ro, fe stn 3.8m: B0°, pl, ro, cly vn 3.92m: Cs 40mm 4.29m: B5°, pl, ro, cln 4.4m: B10°, pl, ro, fe stn 4.53m: B5°, pl, ro, fe stn 4.61m: J50°, pl, ro, cln 4.7m: Cs 30mm 4.74m: CORE LOSS: 30mm	C	100	75	PL(A) = 1.7 PL(A) = 0.38
45 44	4.77 5 6	SANDSTONE: medium to coarse grained, orange-brown and pale grey, high strength, moderately to slightly weathered, slightly fractured, Hawkesbury Sandstone	5.3m: B0°, pl, ro, fe stn 5.34m: Cs 10mm (x2) 5.4m: J80°, pl, ro cln 5.77m: B0°, pl, ro cln 6.12m: B0°, pl, ro, cln 6.63m: B0°, pl ro, fe stn	C	98	90	PL(A) = 0.75 PL(A) = 1.4
43 42 41 40	6.88 7 8 9 10.0		6.84m: CORE LOSS: 40mm 8.4m: B0°, pl, sm, cly vn 9.45m: B0°, pl, ro, cly vn	C C	99 100	99 100	PL(A) = 1.3 PL(A) = 1 PL(A) = 1 PL(A) = 2.1

RIG: Explora **DRILLER:** JD **LOGGED:** TM **CASING:** HW to 2.5m, HQ to 2.7m

TYPE OF BORING: Solid flight auger (TC-bit) to 2.5m; Rotary to 2.7m; NMLC-Coring to 15.9m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Coordinates and surface levels obtained from differential GPS

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)

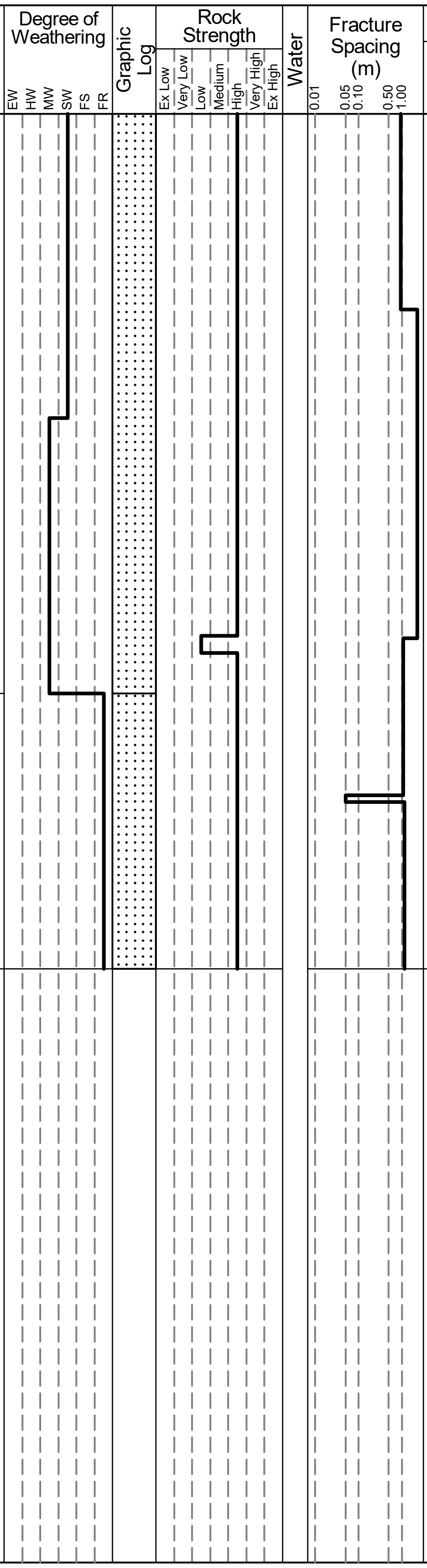
Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Frasers Property Ivanhoe Pty Ltd
PROJECT: Proposed Stage 2 Development
LOCATION: Midtown, Maquarie Park

SURFACE LEVEL: 49.9 AHD
EASTING: 325665.6
NORTHING: 6260360.3
DIP/AZIMUTH: 90°/--

BORE No: 105
PROJECT No: 86043.06
DATE: 24/4/2021
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
		SANDSTONE: medium to coarse grained, orange-brown and pale grey, high strength, moderately to slightly weathered, slightly fractured to unbroken, Hawkesbury Sandstone *(continued)*	10.4m: B10°, pl, ro, cly vn				
39	11						PL(A) = 1.8
			11.35m: B5°, pl, ro, cln	C	100	100	
38	12						PL(A) = 2.3
37	13						PL(A) = 2.6
		At 13.62m: band of low strength siltstone	13.62m: B0°, pl, ro, fe stn				
36	14						PL(A) = 1.4
	14.0	SANDSTONE: medium to coarse grained, pale grey, high strength, fresh, slightly fractured to unbroken, Hawkesbury Sandstone		C	100	99	
			14.7m: B0°-5° (x2) pl, ro, cly vn				
35	15						PL(A) = 1.5
34	15.9						PL(A) = 1.6
	16	Bore discontinued at 15.9m Target depth reached					
33	17						
32	18						
31	19						
30							

RIG: Explora **DRILLER:** JD **LOGGED:** TM **CASING:** HW to 2.5m, HQ to 2.7m
TYPE OF BORING: Solid flight auger (TC-bit) to 2.5m; Rotary to 2.7m; NMLC-Coring to 15.9m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: Coordinates and surface levels obtained from differential GPS

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: 105 PROJECT: MACQUARIE PARK APRIL 2021

2.70 – 7.00m

BORE: 105 PROJECT: MACQUARIE PARK APRIL 2021

7.00 – 12.00m

BORE: 105 PROJECT: MACQUARIE PARK APRIL 2021

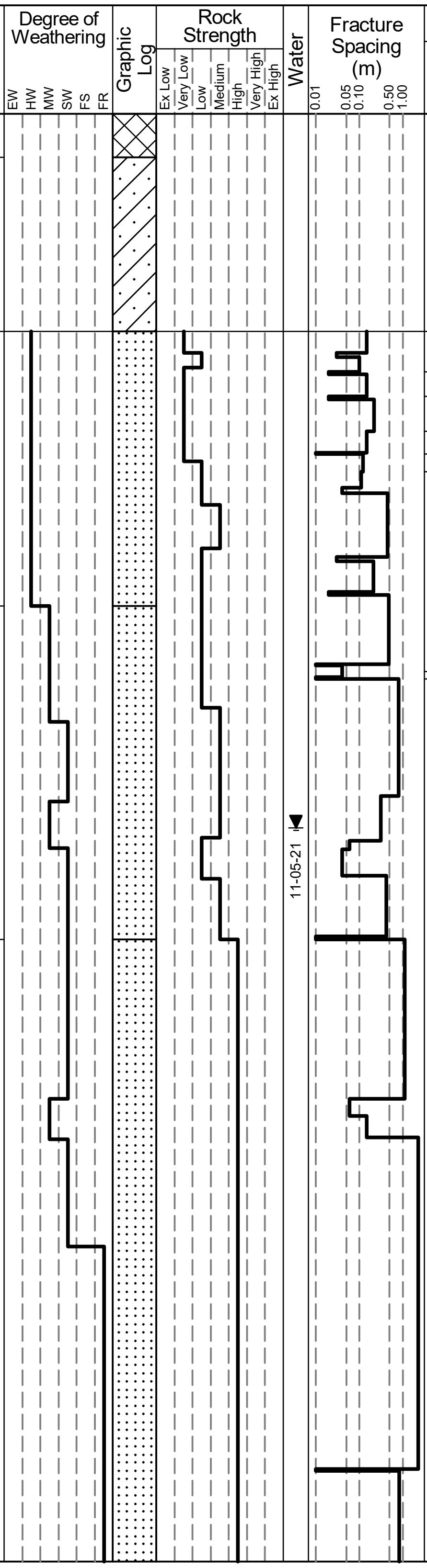
12.00 – 15.90m

BOREHOLE LOG

CLIENT: Frasers Property Ivanhoe Pty Ltd
PROJECT: Proposed Stage 2 Development
LOCATION: Midtown, Maquarie Park

SURFACE LEVEL: 49.5 AHD
EASTING: 325658.4
NORTHING: 6260394.7
DIP/AZIMUTH: 90°/--

BORE No: 106
PROJECT No: 86043.06
DATE: 28/4/2021
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
		FILL/ Sandy CLAY: low plasticity, brown, trace rootlets and fine to medium igneous gravel, w<PL		A			
49	0.3	Sandy CLAY CL-CI: low to medium plasticity, yellow-brown, fine to medium sand, trace fine to medium sandstone gravel, w<PL, stiff, residual		A			
	1			A S			3,6,17 N = 23
48	1.5	SANDSTONE: fine to medium grained, pale grey and red-brown, very low to medium strength, highly weathered, fractured, Hawkesbury Sandstone	1.5m: Ds 150mm 1.68m: J45°, pl, ro, fe stn 1.78m: Ds 20mm 1.95m: Ds 20mm 2.19m: Ds 150mm 2.35m: J60°, pl, ro, cln 2.47m: B0°-5° (x3), pl, ro, cly vn	C	100	0	PL(A) = 0.09
	2						
47				C	100	70	PL(A) = 0.7
	3		3.06m: Cs 30mm 3.3m: Cs 20mm				
46	3.4	SANDSTONE: fine to medium grained, pale grey, orange-brown and red-brown, low to medium strength, moderately weathered, slightly fractured	3.8m: Cs 10mm 3.85m: Cs 40mm 3.9m: B0°, pl, sm, cly co				PL(A) = 0.16
	4						
45			4.71m: B5°, pl, ro, cly vn				PL(A) = 0.51
	5		5.02m: B0°-10° (x6), pl, sm, cly co & fe stn				
44	5.7	SANDSTONE: medium to coarse grained, red-brown, orange-brown and pale grey, high strength, slightly weathered to fresh, slightly fractured, Hawkesbury Sandstone	5.68m: B10°, pl, ro, fe stn & Fg 20mm	C	100	95	PL(A) = 2.6
	6						
43	7		6.8m: B0° (x4), pl, ro, fe stn	C	100	96	PL(A) = 1.2
42		Below 7.4m: moderately weathered band					PL(A) = 1.1
	8						
41	9						PL(A) = 1.5
40			9.36m: Cs 5mm	C	100	98	PL(A) = 1.6

RIG: Explora **DRILLER:** JD **LOGGED:** TM **CASING:** HW to 1.0m, HQ to 1.5m

TYPE OF BORING: Solid flight auger (TC-bit) to 1.0m; Rotary to 1.5m; NMLC-Coring to 13.8m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Groundwater well installed to 11.0m (screen 11.0-8.0m; blank 8.0-0.0m; gravel 11.0-7.5m; bentonite 7.5-7.0m; backfill to GL; gatic at surface); Coordinates and surface levels obtained from differential GPS

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)

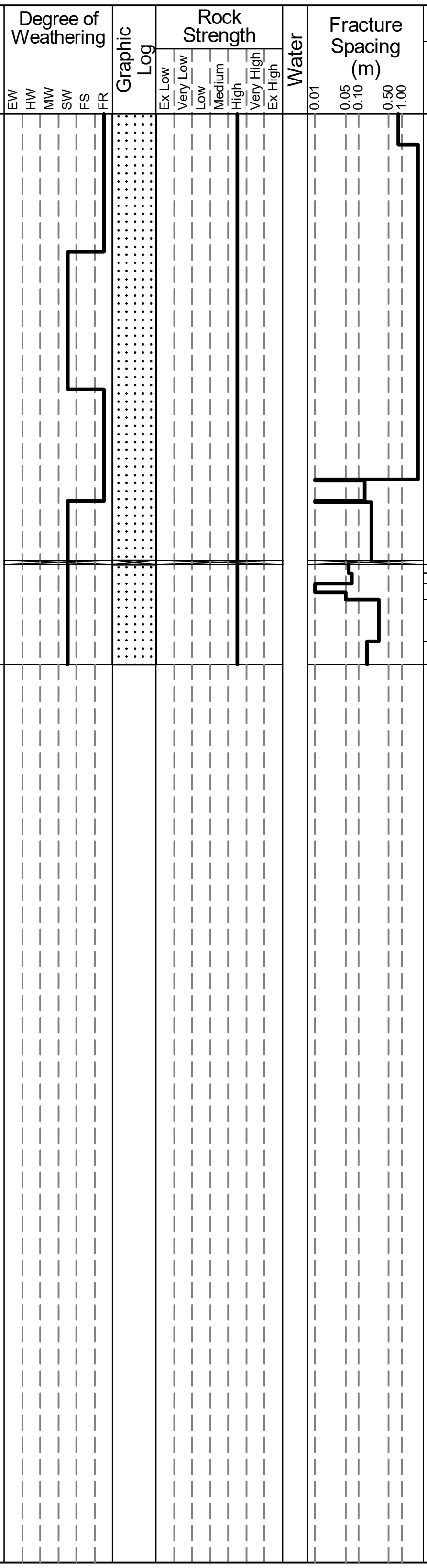
Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Frasers Property Ivanhoe Pty Ltd
PROJECT: Proposed Stage 2 Development
LOCATION: Midtown, Maquarie Park

SURFACE LEVEL: 49.5 AHD
EASTING: 325658.4
NORTHING: 6260394.7
DIP/AZIMUTH: 90°/--

BORE No: 106
PROJECT No: 86043.06
DATE: 28/4/2021
SHEET 2 OF 2



Depth (m)	Description of Strata
	SANDSTONE: medium to coarse grained, red-brown, orange-brown and pale grey, high strength, slightly weathered to fresh, slightly fractured, Hawkesbury Sandstone *(continued)*
13.08	
13.11	
13.8	Bore discontinued at 13.8m Target depth reached

RL: 39, 38, 37, 36, 35, 34, 33, 32, 31, 30
Depth (m): 11, 12, 13, 14, 15, 16, 17, 18, 19

Discontinuities

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

10.21m: B0°, pl, ro, cln
12.52m: Cs 10mm
12.67m: Cs 10mm
12.88m: B0°, pl, ro,cln
13.08m: CORE LOSS: 30mm
13.11m: B0°, pl, ro, fe stn
13.17m: B10°, pl, ro, fe stn
13.24m: Fg 60mm, fe stn
13.35m: J30°, pl,ro, fe stn
13.64m: J30°, pl, ro, cbs co

Sampling & In Situ Testing

Type	Core Rec. %	RQD %	Test Results & Comments
			PL(A) = 1.6
C	100	98	
			PL(A) = 2
			PL(A) = 2.7
C	97	70	
			PL(A) = 2.3

RIG: Explora **DRILLER:** JD **LOGGED:** TM **CASING:** HW to 1.0m, HQ to 1.5m

TYPE OF BORING: Solid flight auger (TC-bit) to 1.0m; Rotary to 1.5m; NMLC-Coring to 13.8m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Groundwater well installed to 11.0m (screen 11.0-8.0m; blank 8.0-0.0m; gravel 11.0-7.5m; bentonite 7.5-7.0m; backfill to GL; gatic at surface); Coordinates and surface levels obtained from differential GPS

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: 106 PROJECT: MACQUARIE PARK APRIL 2021

1.50 – 5.00m

BORE: 106 PROJECT: MACQUARIE PARK APRIL 2021

5.00 – 10.00m

BORE: 106 PROJECT: MACQUARIE PARK APRIL 2021

10.00 – 13.80m

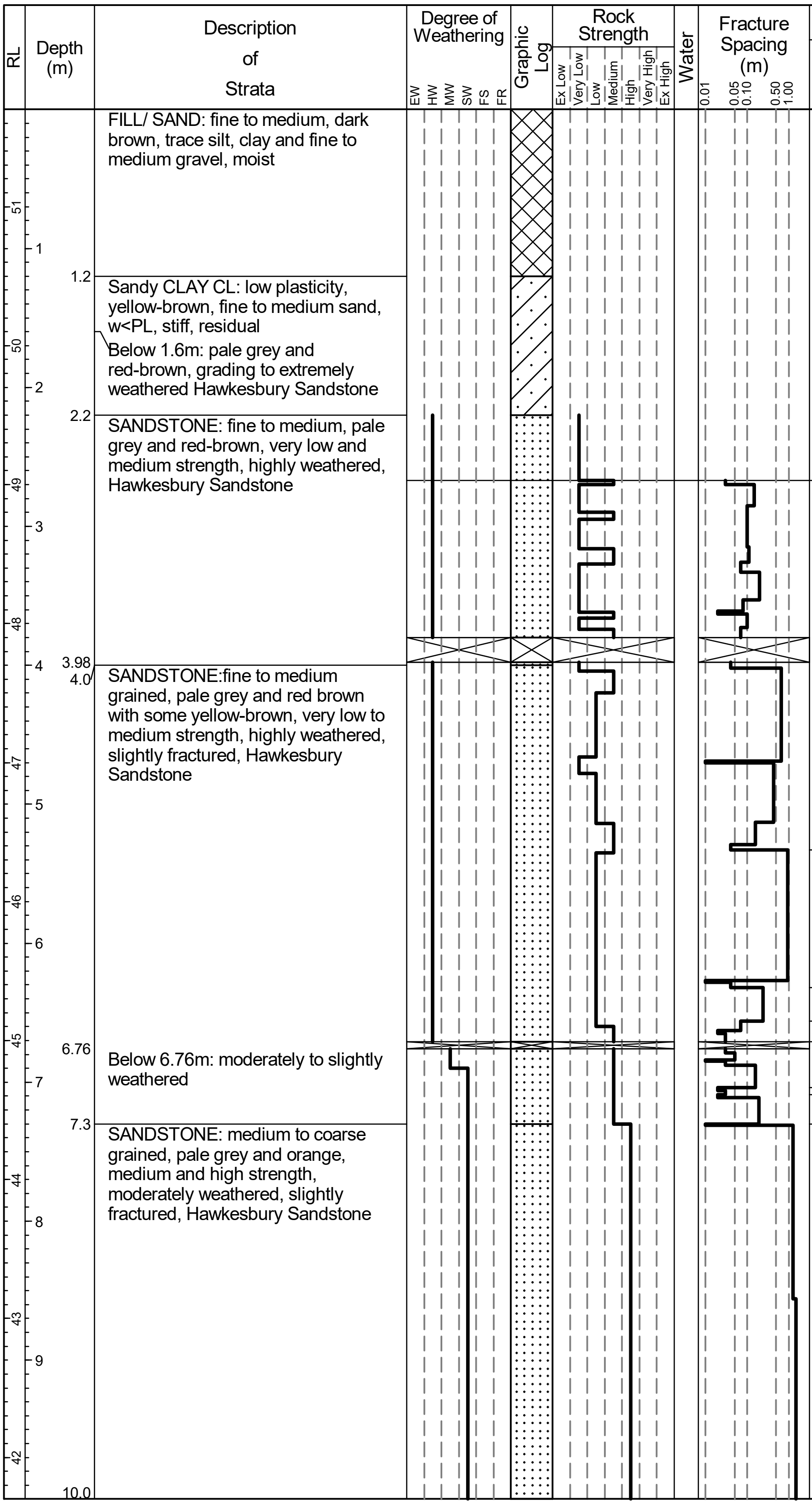
BOREHOLE LOG

CLIENT: Frasers Property Ivanhoe Pty Ltd
PROJECT: Proposed Stage 2 Development
LOCATION: Midtown, Maquarie Park

SURFACE LEVEL: 51.7 AHD
EASTING: 325636.9
NORTHING: 6260386.8
DIP/AZIMUTH: 90°/--

BORE No: 117
PROJECT No: 86043.06
DATE: 21/4/2021
SHEET 1 OF 2

Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Sampling & In Situ Testing Type	Core Rec. %	RQD %	Test Results & Comments
	FILL/ SAND: fine to medium, dark brown, trace silt, clay and fine to medium gravel, moist		A A S			 5,4,6 N = 10
1.2	Sandy CLAY CL: low plasticity, yellow-brown, fine to medium sand, w<PL, stiff, residual Below 1.6m: pale grey and red-brown, grading to extremely weathered Hawkesbury Sandstone		A A			
2.2	SANDSTONE: fine to medium, pale grey and red-brown, very low and medium strength, highly weathered, Hawkesbury Sandstone		S			25/130 mm refusal
		2.70-4.02m: B0°-10° (x12), pl, ro, fe stn	C	100	8	PL(A) = 0.4 PL(A) = 0.91
3.98 4.0	SANDSTONE:fine to medium grained, pale grey and red brown with some yellow-brown, very low to medium strength, highly weathered, slightly fractured, Hawkesbury Sandstone	3.8m: CORE LOSS: 180mm 4.69m: Cs 10mm 5.13m: B5°, pl, ro, cly vn 5.29m: B5°, pl, ro, fe stn 5.33m: J70°, pl, ti	C	89	70	PL(A) = 0.23 PL(A) = 0.16
6.76	Below 6.76m: moderately to slightly weathered	6.27m: Cs 10mm 6.32m: J30°, pl, ro, cly vn 6.56-6.88m: B0°-10° (x9), pl, ro, fe stn & he 6.71m: CORE LOSS: 50mm 7.04m: Ds 20mm 7.09m: Ds 20mm 7.3m: Ds 10mm	C	95	70	PL(A) = 0.4
7.3	SANDSTONE: medium to coarse grained, pale grey and orange, medium and high strength, moderately weathered, slightly fractured, Hawkesbury Sandstone	8.56m: B0°, pl, ro, fe stn	C	100	100	PL(A) = 1.6 PL(A) = 1.3 PL(A) = 1
10.0						



RIG: Explora **DRILLER:** JD **LOGGED:** TM **CASING:** HW to 2.5m, HQ to 2.67m
TYPE OF BORING: Solid flight auger (TC-bit) to 2.5m; Rotary to 2.67m; NMLC-Coring to 16.0m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: Coordinates and surface levels obtained from differential GPS

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)

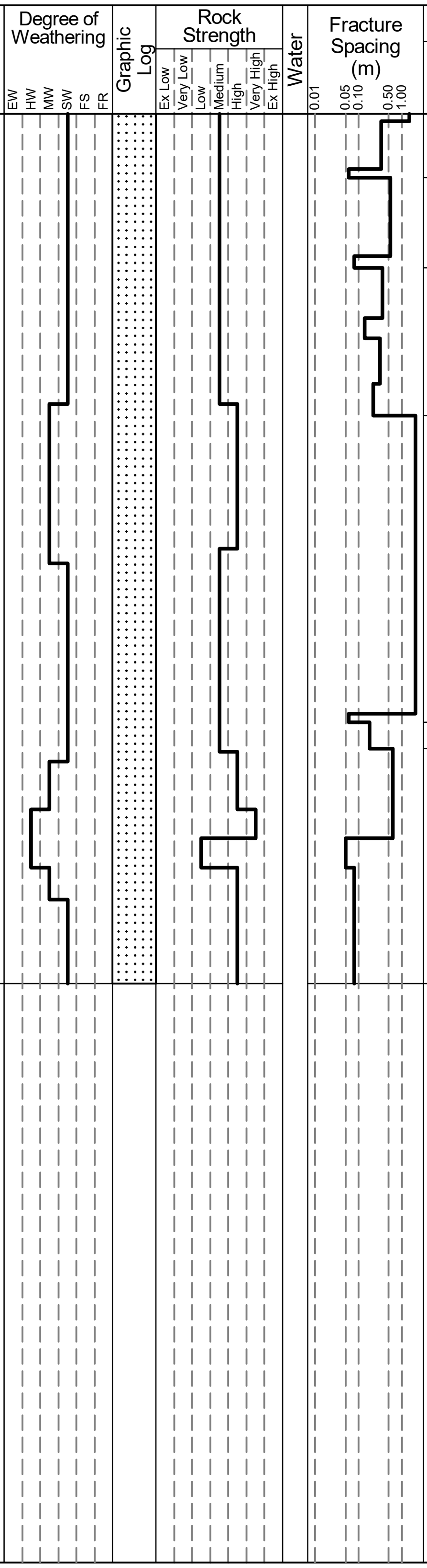
Douglas Partners
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Frasers Property Ivanhoe Pty Ltd
PROJECT: Proposed Stage 2 Development
LOCATION: Midtown, Maquarie Park

SURFACE LEVEL: 51.7 AHD
EASTING: 325636.9
NORTHING: 6260386.8
DIP/AZIMUTH: 90°/--

BORE No: 117
PROJECT No: 86043.06
DATE: 21/4/2021
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
		SANDSTONE: medium to coarse grained, pale grey and yellow-brown, medium to high strength, moderately weathered, slightly fractured, Hawkesbury Sandstone *(continued)*	10.05m: J80°, pl, ro, cln	C	100	95	
41			10.38m: B10°, pl, ro, cly vn 10.44m: B0°, pl, ro, cly vn	C	100	95	PL(A) = 0.56
	11		10.98m: J80°, pl, ro, cln 11.06m: J80°, pl, ro, cln				
40			11.41m: J80°, pl, ro, cln 11.55m: J80°, pl, ro, cln				
	12		11.86m: B10°, pl, sm, cly vn 12.08m: J70°, pl, ro, cln				PL(A) = 0.41
39							PL(A) = 2.2
	13			C	100	97	
38							PL(A) = 0.78
	14		14.14m: B10°, pl, sm, cly 2mm 14.2m: B5°, pl, ro, cly vn 14.38m: B5°, pl, ro, cly vn				
37				C	100	100	
	15	Between 14.8-15.0m: very high strength Between 15.0-15.2m: band of interbedded siltstone clasts and low strength, highly weathered sandstone	15.0-15.2m: J50°-70° (x4), pl, ro, fe stn				PL(A) = 3.2
36				C	100	80	PL(A) = 1.8
	16 16.0	Bore discontinued at 16.0m Target depth reached					
35							
	17						
34							
	18						
33							
	19						
32							

RIG: Explora **DRILLER:** JD **LOGGED:** TM **CASING:** HW to 2.5m, HQ to 2.67m

TYPE OF BORING: Solid flight auger (TC-bit) to 2.5m; Rotary to 2.67m; NMLC-Coring to 16.0m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Coordinates and surface levels obtained from differential GPS

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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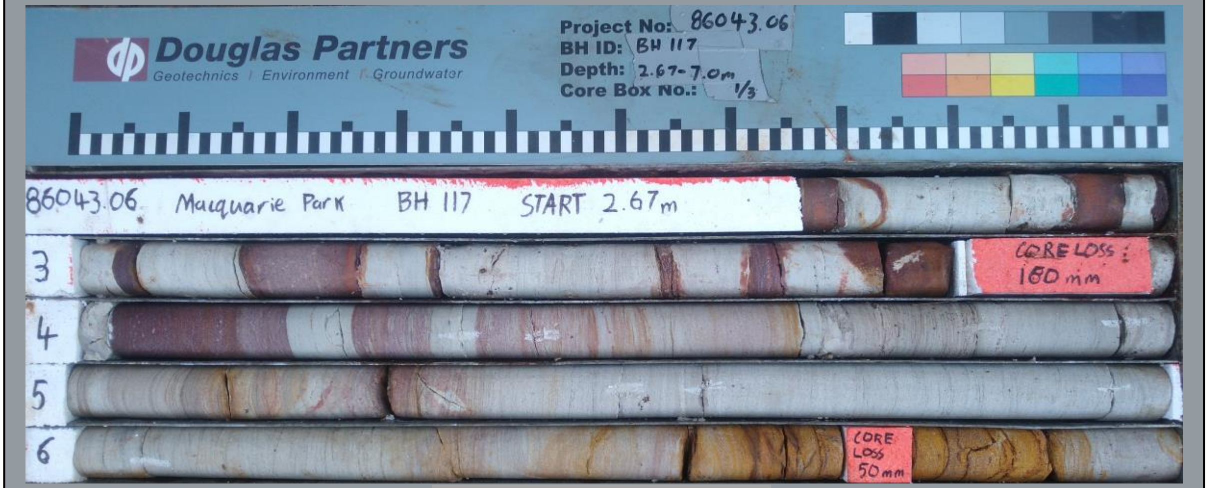
Geotechnics | Environment | Groundwater

BORE: 117 PROJECT: MACQUARIE PARK APRIL 2021



2.67 – 7.00m

BORE: 117 PROJECT: MACQUARIE PARK APRIL 2021

7.00 – 12.00m

BORE: 117 PROJECT: MACQUARIE PARK APRIL 2021

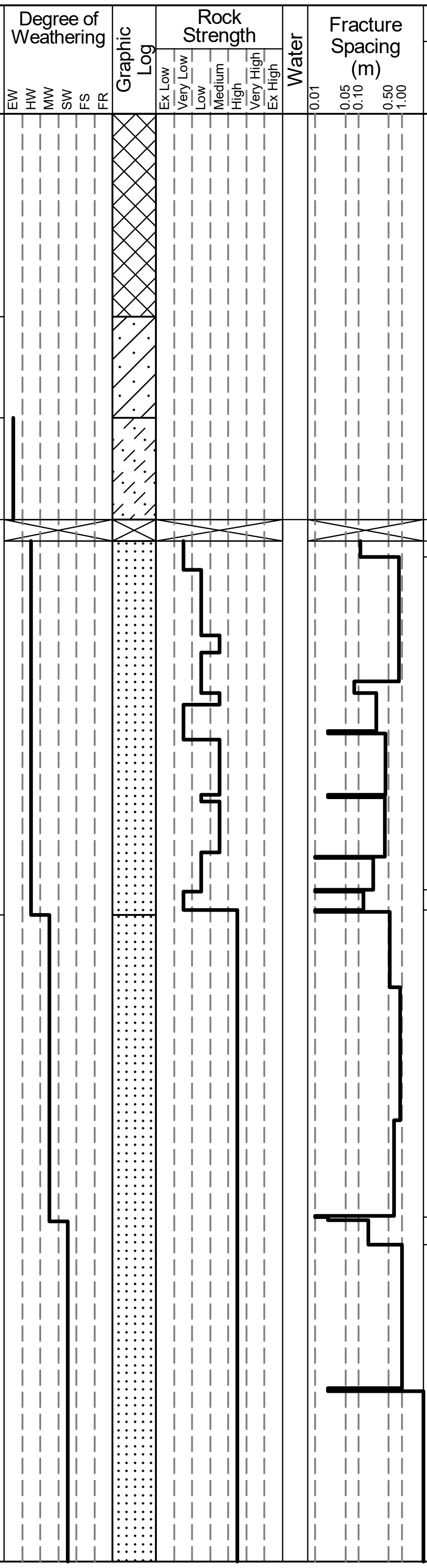
12.00 – 16.00m

BOREHOLE LOG

CLIENT: Frasers Property Ivanhoe Pty Ltd
PROJECT: Proposed Stage 2 Development
LOCATION: Midtown, Maquarie Park

SURFACE LEVEL: 49.6 AHD
EASTING: 325675
NORTHING: 6260377.6
DIP/AZIMUTH: 90°/--

BORE No: 118
PROJECT No: 86043.06
DATE: 27/4/2021
SHEET 1 OF 2



RL	Depth (m)	Description of Strata
49.6	0	FILL/ Sandy CLAY: medium plasticity, dark brown, fine to medium sand, trace rootlets and fine to medium sandstone gravel, w<PL
	1.4	Sandy CLAY CI: medium plasticity, yellow-brown, fine to medium sand, w<PL, stiff, residual
	2.1	Clayey SAND SC: fine to medium, pale grey and red-brown, trace fine to medium ironstone gravel, moist, very dense, extremely weathered Hawkesbury Sandstone
	2.8	
	2.95	SANDSTONE: fine to medium,pale grey and red-brown,very low to low strength, highly weathered, fractured, Hawkesbury Sandstone
	5.53	SANDSTONE: medium to coarse grained, red-brown then orange-brown and pale grey, high strength, moderately weathered to fresh, slightly fractured to unbroken, Hawkesbury Sandstone

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

- 2.8m: CORE LOSS: 150mm
- 2.95m: J90°, pl, ro, cln
- 3.06m: J70°, pl, ro, fe stn
- 3.92m: Ds 80mm
- 4.26m: Cs 20mm
- 4.7m: B10°, pl, ro, fe stn & Ds 20mm
- 5.13m: J60°-70° (x2), pl, ro, cln
- 5.36m: Cs 10mm
- 5.5m: Cs 10mm
- 6.03m: B20°, pl, ro, fe stn
- 6.95m: J80°, pl, ro, cln
- 7.61m: B0°, pl, ro, fe stn
- 7.62m: Cs 20mm
- 7.81m: J70°, pl, ro, cln
- 8.8m: Ds 20mm

Sampling & In Situ Testing

Type	Core Rec. %	RQD %	Test Results & Comments
A			
S			6,5,5 N = 10
S			20,25/130 mm refusal
C	88	45	PL(A) = 0.2 PL(A) = 0.94
C	100	80	PL(A) = 0.23 PL(A) = 1.6
C	100	98	PL(A) = 1.5 PL(A) = 1.1 PL(A) = 1.1
C	100	100	

RIG: Explora **DRILLER:** JD **LOGGED:** TM **CASING:** HW to 2.5m, HQ to 2.8m

TYPE OF BORING: Solid flight auger (TC-bit) to 2.5m; Rotary to 2.8m; NMLC-Coring to 14.17m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Coordinates and surface levels obtained from differential GPS

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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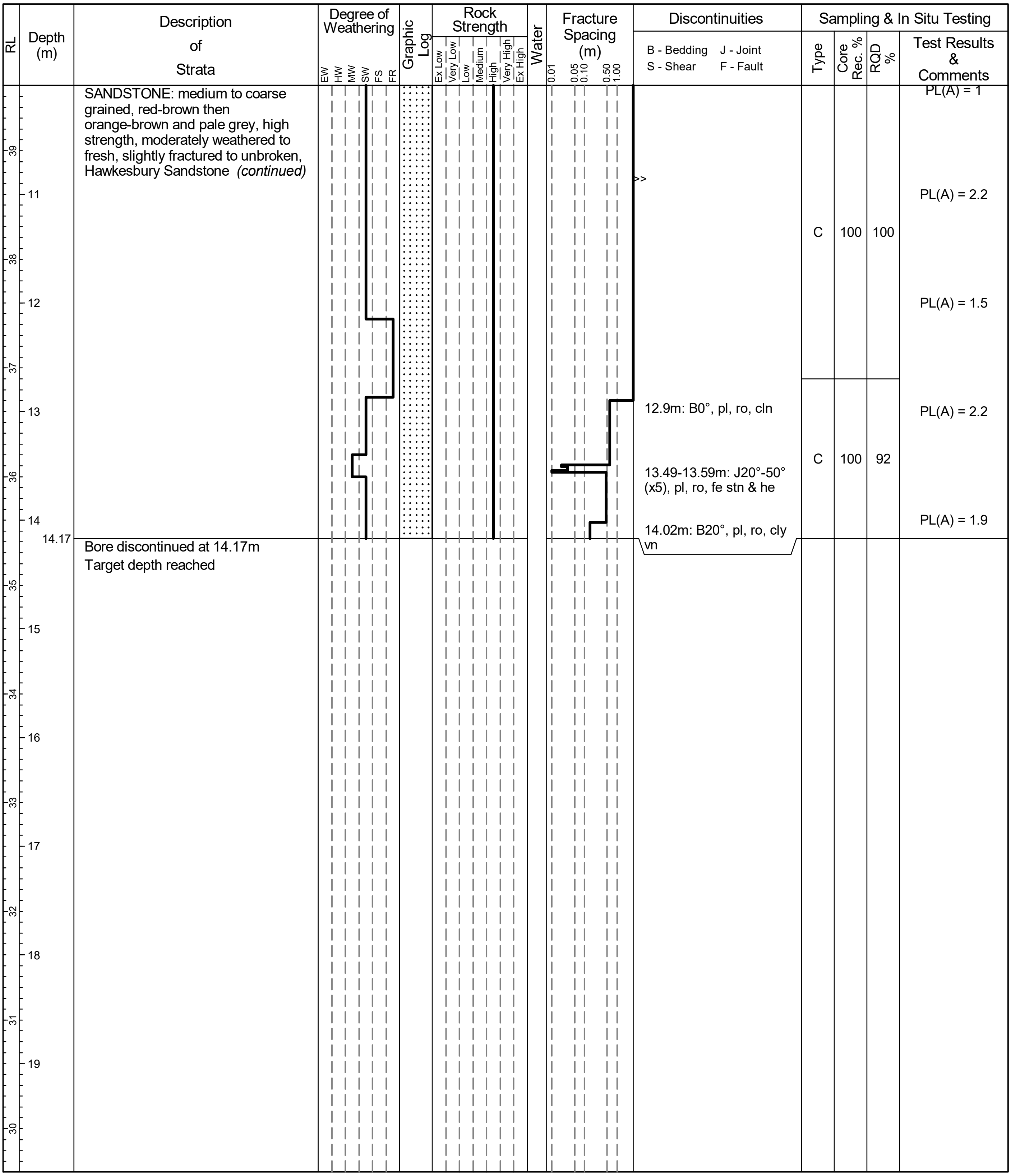
BOREHOLE LOG

CLIENT: Frasers Property Ivanhoe Pty Ltd
PROJECT: Proposed Stage 2 Development
LOCATION: Midtown, Maquarie Park

SURFACE LEVEL: 49.6 AHD
EASTING: 325675
NORTHING: 6260377.6
DIP/AZIMUTH: 90°/--

BORE No: 118
PROJECT No: 86043.06
DATE: 27/4/2021
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
	SANDSTONE: medium to coarse grained, red-brown then orange-brown and pale grey, high strength, moderately weathered to fresh, slightly fractured to unbroken, Hawkesbury Sandstone *(continued)*					PL(A) = 1
11			C	100	100	PL(A) = 2.2
12						PL(A) = 1.5
13		12.9m: B0°, pl, ro, cln	C	100	92	PL(A) = 2.2
		13.49-13.59m: J20°-50° (x5), pl, ro, fe stn & he				
14		14.02m: B20°, pl, ro, cly vn				PL(A) = 1.9
14.17	Bore discontinued at 14.17m Target depth reached					

RIG: Explora **DRILLER:** JD **LOGGED:** TM **CASING:** HW to 2.5m, HQ to 2.8m

TYPE OF BORING: Solid flight auger (TC-bit) to 2.5m; Rotary to 2.8m; NMLC-Coring to 14.17m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Coordinates and surface levels obtained from differential GPS

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: 118 PROJECT: MACQUARIE PARK APRIL 2021

2.80 – 7.00m

BORE: 118 PROJECT: MACQUARIE PARK APRIL 2021

8.00 – 12.00m

BORE: 118 PROJECT: MACQUARIE PARK APRIL 2021

12.00 – 14.17m

Permeability Testing - Rising Head Test Report

Client:	Frasers Property Ivanhoe Pty Ltd	Project No:	86043.06
Project:	Midtown in Macquarie Park	Test date:	11-May-21
Location:	C3 Development Area	Tested by:	TM

Test Location

		Test No.	BH106	
Description:	Standpipe in borehole	Easting:	325658.4	m
Material type:	Sandstone	Northing	6260394.7	m
		Surface Level:	49.5	m AHD

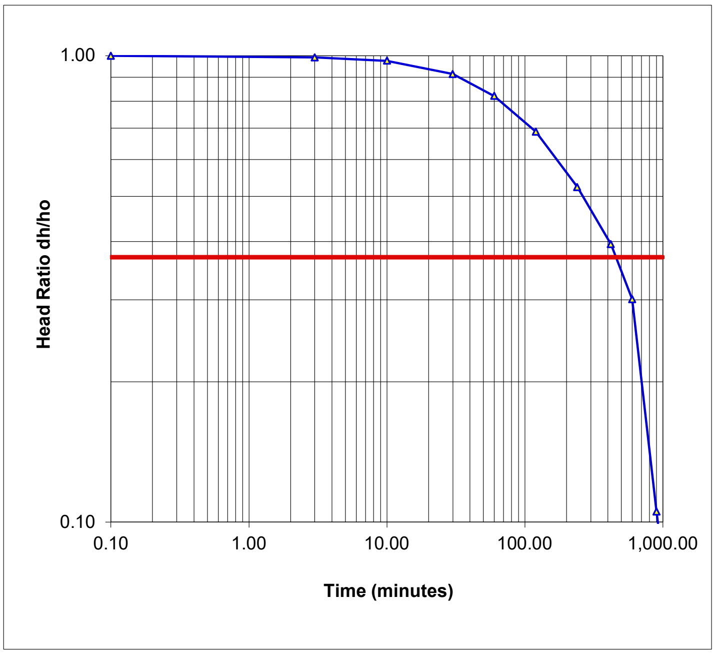
Details of Well Installation

Well casing diameter (2r)	70	mm	Depth to water before test	4.93	m
Well screen diameter (2R)	100	mm	Depth to water at start of test	7.49	m
Length of well screen (Le)	3.5	m			

Comments:

Test Results

Time (min)	Depth (m)	Change in Head δH (m)	δH/Ho
0	7.49	2.56	1.000
3.0	7.47	2.54	0.992
10	7.43	2.50	0.977
30	7.27	2.34	0.914
60	7.03	2.10	0.820
120	6.69	1.76	0.688
240	6.27	1.34	0.523
420	5.94	1.01	0.395
600	5.7	0.77	0.301
900	5.2	0.27	0.105
1000	5.14	0.21	0.082



To = 430 mins
25800 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity	**k =**	**2.9E-08**	m/sec
	=	**0.010**	cm/hour

Permeability Testing - Falling Head Test Report

Client:	Frasers Property Ivanhoe	Project No:	86043.06
Project:	Stage 2 - Midtown	Date:	28-May-21
Location:	Herring Road, Macquarie Park	Tested by:	LS

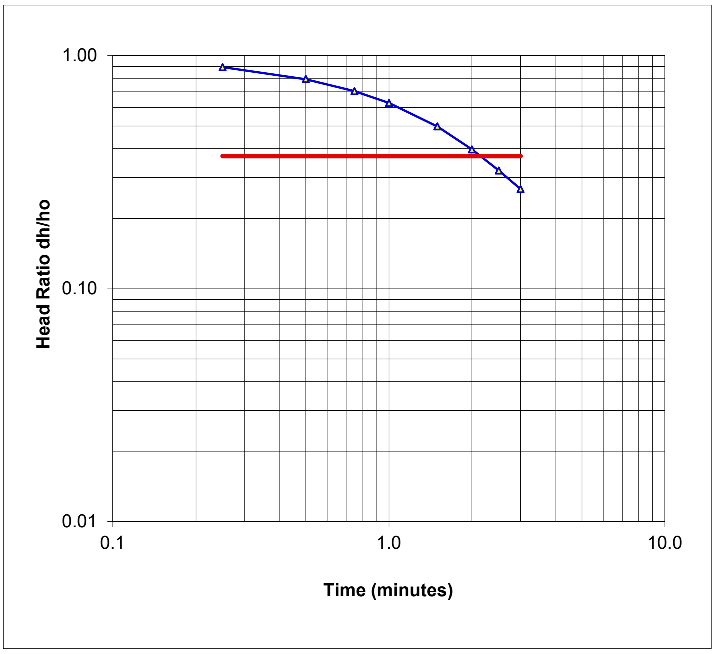
Test Location		**Test No.**	BH104A	
Description:	Unit 3C	Easting:	325637.8	m
Material type:	Sandstone	Northing	6260346.9	m
		Surface Level:	51.8	m AHD

Details of Well Installation

Well casing diameter (2r)	50	mm	Depth to water before test	10.15	m
Well screen diameter (2R)	100	mm	Depth to water at start of test	1.7	m
Length of well screen (Le)	2	m			

Test Results

Time (min)	Depth (m)	Change in Head: δH (m)	δH/Ho
0.0	1.70	8.45	1.000
0.3	2.61	7.54	0.892
0.5	3.45	6.70	0.793
0.8	4.2	5.95	0.704
1.0	4.85	5.30	0.627
1.5	5.95	4.20	0.497
2.0	6.8	3.35	0.396
2.5	7.43	2.72	0.322
3.0	7.89	2.26	0.267



To = 2.3 mins
138 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity	**k =**	**4.2E-06**	m/sec	4.2E-04 cm/sec
	=	**1.504**	cm/hour	

Permeability Testing - Falling Head Test Report

Client:	Frasers Property Ivanhoe	Project No:	86043.06
Project:	Stage 2 - Midtown	Date:	28-May-21
Location:	Herring Road, Macquarie Park	Tested by:	LS

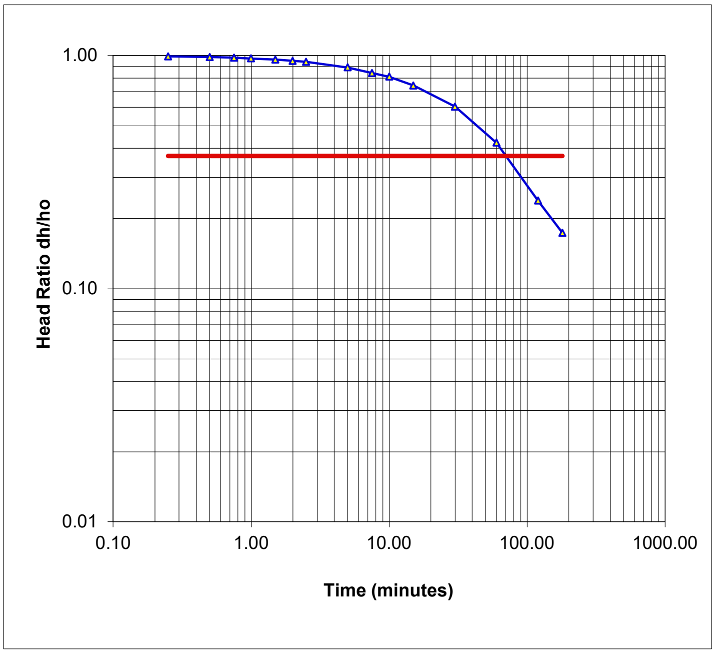
Test Location		**Test No.**	BH106	
Description:	Unit 3C	Easting:	325661.8	m
Material type:	Sandstone	Northing	6260395.3	m
		Surface Level:	49.4	m AHD

Details of Well Installation

Well casing diameter (2r)	50	mm	Depth to water before test	4.98	m
Well screen diameter (2R)	100	mm	Depth to water at start of test	0.37	m
Length of well screen (Le)	3.5	m			

Test Results

Time (min)	Depth (m)	Change in Head: δH (m)	δH/Ho
0.00	0.37	4.61	1.000
0.25	0.41	4.57	0.991
0.50	0.44	4.54	0.985
0.75	0.47	4.51	0.978
1.00	0.5	4.48	0.972
1.50	0.55	4.43	0.961
2.00	0.61	4.37	0.948
2.50	0.65	4.33	0.939
5.00	0.88	4.10	0.889
7.5	1.1	3.88	0.842
10	1.24	3.74	0.811
15	1.55	3.43	0.744
30	2.20	2.78	0.603
60	3.03	1.95	0.423
120	3.88	1.10	0.239
180	4.18	0.80	0.174



To = 65 mins
3900 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity **k = 9.7E-08** m/sec 9.7E-06 cm/sec
= 0.035 cm/hour

Permeability Testing - Falling Head Test Report

Client:	Frasers Property Ivanhoe	Project No:	86043.06
Project:	Stage 2 - Midtown	Date:	28-May-21
Location:	Herring Road, Macquarie Park	Tested by:	LS

Test Location — **Test No.** BH107

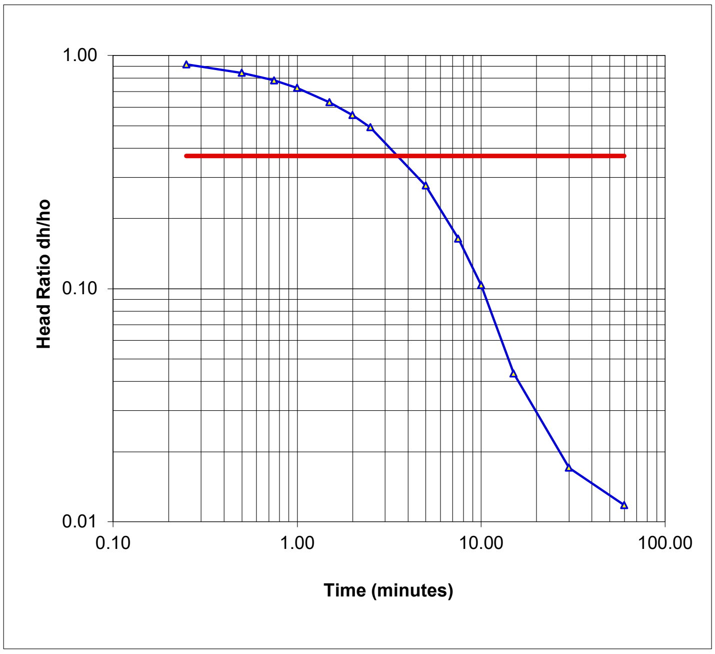
Description:	Unit 3D	Easting:	325658.7	m
Material type:	Sandstone	Northing	6260313.1	m
		Surface Level:	49.7	m AHD

Details of Well Installation

Well casing diameter (2r)	50	mm	Depth to water before test	8.43	m
Well screen diameter (2R)	100	mm	Depth to water at start of test	0.81	m
Length of well screen (Le)	3.5	m			

Test Results

Time (min)	Depth (m)	Change in Head: δH (m)	δH/Ho
0.00	0.81	7.62	1.000
0.25	1.46	6.97	0.915
0.50	2.01	6.42	0.843
0.75	2.47	5.96	0.782
1.00	2.9	5.53	0.726
1.50	3.63	4.80	0.630
2.00	4.2	4.23	0.555
2.50	4.68	3.75	0.492
5.00	6.32	2.11	0.277
7.5	7.18	1.25	0.164
10	7.64	0.79	0.104
15	8.1	0.33	0.043
30	8.30	0.13	0.017
60	8.34	0.09	0.012



To = 3.67 mins
220.2 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity k = **1.7E-06** m/sec — 1.7E-04 cm/sec
= **0.620** cm/hour

Permeability Testing - Falling Head Test Report

Client:	Frasers Property Ivanhoe	Project No:	86043.06
Project:	Stage 2 - Midtown	Date:	28-May-21
Location:	Herring Road, Macquarie Park	Tested by:	LS

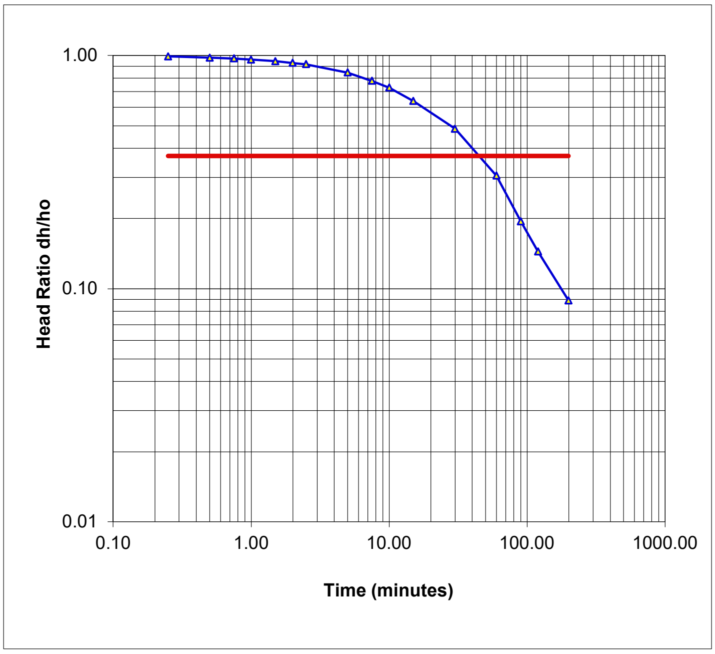
Test Location		**Test No.**	BH109	
Description:	Unit 3D	Easting:	325716	m
Material type:	Sandstone	Northing	6260351.1	m
		Surface Level:	46.1	m AHD

Details of Well Installation

Well casing diameter (2r)	50	mm	Depth to water before test	6.34	m
Well screen diameter (2R)	100	mm	Depth to water at start of test	0.38	m
Length of well screen (Le)	3.5	m			

Test Results

Time (min)	Depth (m)	Change in Head: δH (m)	δH/Ho
0.00	0.38	5.96	1.000
0.25	0.44	5.90	0.990
0.50	0.5	5.84	0.980
0.75	0.55	5.79	0.971
1.00	0.61	5.73	0.961
1.50	0.7	5.64	0.946
2.00	0.8	5.54	0.930
2.50	0.87	5.47	0.918
5.00	1.3	5.04	0.846
7.5	1.7	4.64	0.779
10	2	4.34	0.728
15	2.54	3.80	0.638
30	3.45	2.89	0.485
60	4.52	1.82	0.305
90	5.18	1.16	0.195
120	5.48	0.86	0.144
200	5.81	0.53	0.089



To = 47.8 mins
2868 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity	**k =**	**1.3E-07**	m/sec	1.3E-05 cm/sec
	=	**0.048**	cm/hour	

Permeability Testing - Falling Head Test Report

Client:	Frasers Property Ivanhoe	Project No:	86043.06
Project:	Stage 2 - Midtown	Date:	28-May-21
Location:	Herring Road, Macquarie Park	Tested by:	LS

Test Location

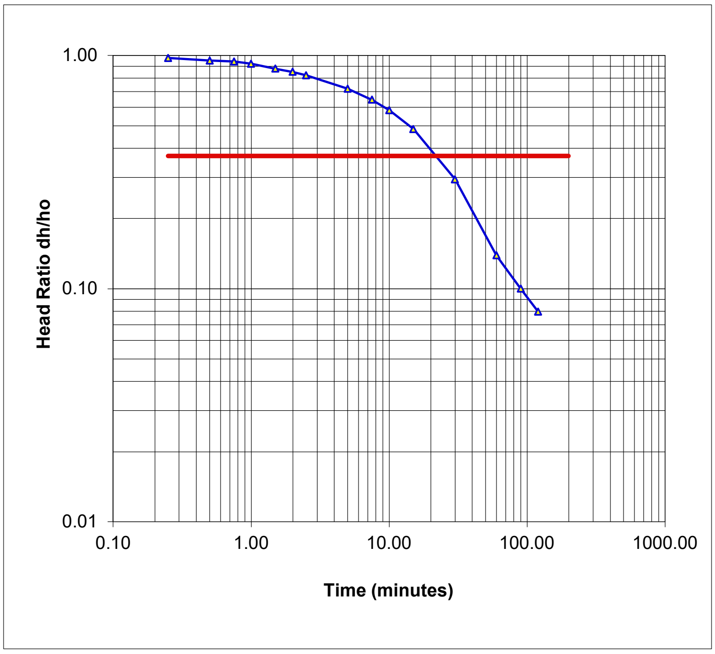
Description:	Unit 3C	**Test No.**	BH109A	
Material type:	Sandstone	Easting:	325716	m
		Northing	6260351.1	m
		Surface Level:	46.1	m AHD

Details of Well Installation

Well casing diameter (2r)	50	mm	Depth to water before test	6.1	m
Well screen diameter (2R)	100	mm	Depth to water at start of test	0.7	m
Length of well screen (Le)	3.5	m			

Test Results

Time (min)	Depth (m)	Change in Head: δH (m)	δH/Ho
0.00	0.70	5.40	1.000
0.25	0.83	5.27	0.976
0.50	0.96	5.14	0.952
0.75	1.01	5.09	0.943
1.00	1.12	4.98	0.922
1.50	1.36	4.74	0.878
2.00	1.51	4.59	0.850
2.50	1.66	4.44	0.822
5.00	2.21	3.89	0.720
7.5	2.61	3.49	0.646
10	2.95	3.15	0.583
15	3.49	2.61	0.483
30	4.51	1.59	0.294
60	5.35	0.75	0.139
90	5.56	0.54	0.100
120	5.67	0.43	0.080
200			



To = 22.7 mins
1362 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity	**k =**	**2.8E-07**	m/sec	2.8E-05 cm/sec
	=	**0.100**	cm/hour	

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Douglas Partners Pty Ltd
ABN 75 053 980 117
www.douglaspartners.com.au

Permeability Testing - Falling Head Test Report

Client: Frasers Property Ivanhoe
Project: Stage 2 - Midtown
Location: Herring Road, Macquarie Park

Project No: 86043.06
Date: 27-May-21
Tested by: LS

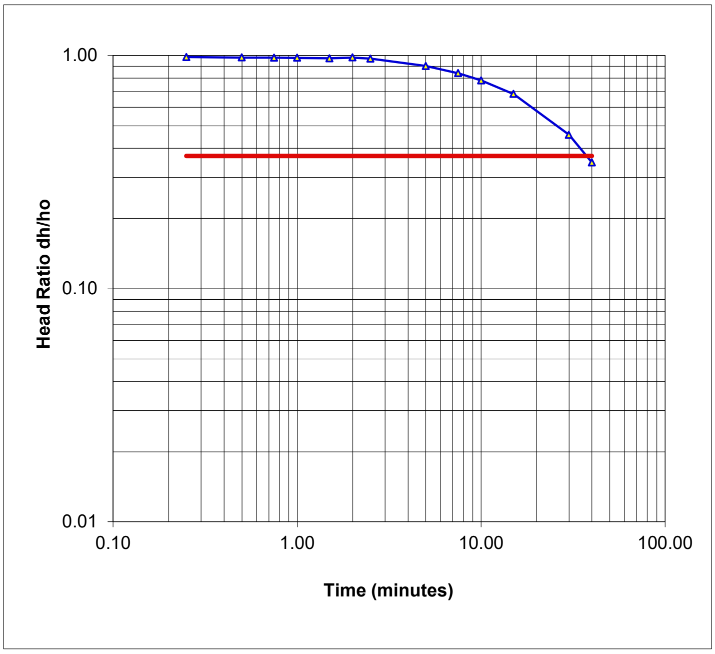
Test Location
Description: Unit 3D
Material type: Sandstone

Test No. BH111
Easting: 325729.6 m
Northing 6260317.2 m
Surface Level: 45.8 m AHD

Details of Well Installation
Well casing diameter (2r) 50 mm
Well screen diameter (2R) 100 mm
Length of well screen (Le) 3.5 m
Depth to water before test 5.95 m
Depth to water at start of test 0.63 m

Test Results

Time (min)	Depth (m)	Change in Head: δH (m)	δH/Ho
0.00	0.63	5.32	1.000
0.25	0.71	5.24	0.985
0.50	0.73	5.22	0.981
0.75	0.74	5.21	0.979
1.00	0.75	5.20	0.977
1.50	0.77	5.18	0.974
2.00	0.73	5.22	0.981
2.50	0.79	5.16	0.970
5.00	1.15	4.80	0.902
7.5	1.48	4.47	0.840
10	1.79	4.16	0.782
15	2.31	3.64	0.684
30	3.52	2.43	0.457
40	4.1	1.85	0.348



To = 37.7 mins
2262 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity **k = 1.7E-07** m/sec 1.7E-05 cm/sec
= 0.060 cm/hour

Permeability Testing - Falling Head Test Report

Client:	Frasers Property Ivanhoe	Project No:	86043.06
Project:	Stage 2 - Midtown	Date:	27-May-21
Location:	Herring Road, Macquarie Park	Tested by:	LS

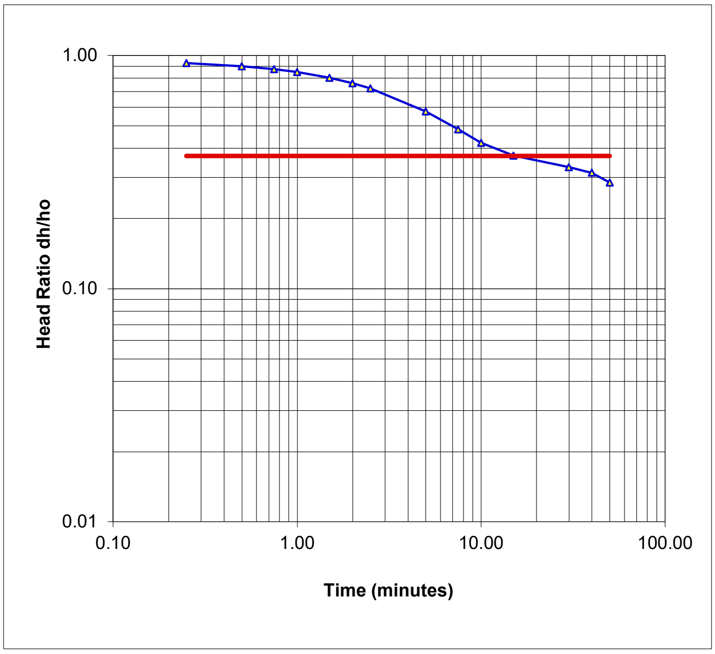
Test Location		**Test No.**	BH111A	
Description:	Unit 3D	Easting:	325729.6	m
Material type:	Sandstone	Northing	6260317.2	m
		Surface Level:	45.8	m AHD

Details of Well Installation

Well casing diameter (2r)	50	mm	Depth to water before test	5.54	m
Well screen diameter (2R)	100	mm	Depth to water at start of test	0	m
Length of well screen (Le)	3.5	m			

Test Results

Time (min)	Depth (m)	Change in Head: δH (m)	δH/Ho
0.00	0.00	5.54	1.000
0.25	0.4	5.14	0.928
0.50	0.56	4.98	0.899
0.75	0.7	4.84	0.874
1.00	0.84	4.70	0.848
1.50	1.1	4.44	0.801
2.00	1.33	4.21	0.760
2.50	1.54	4.00	0.722
5.00	2.35	3.19	0.576
7.5	2.87	2.67	0.482
10	3.2	2.34	0.422
15	3.48	2.06	0.372
30	3.70	1.84	0.332
40	3.8	1.74	0.314
50	3.96	1.58	0.285



To = 16.7 mins
1002 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity	**k =**	**3.8E-07**	m/sec	3.8E-05 cm/sec
	=	**0.136**	cm/hour	

Permeability Testing - Falling Head Test Report

Client: Frasers Property Ivanhoe
Project: Stage 2 - Midtown
Location: Herring Road, Macquarie Park

Project No: 86043.06
Date: 27-May-21
Tested by: LS

Test Location
Description: Unit 3D
Material type: Sandstone

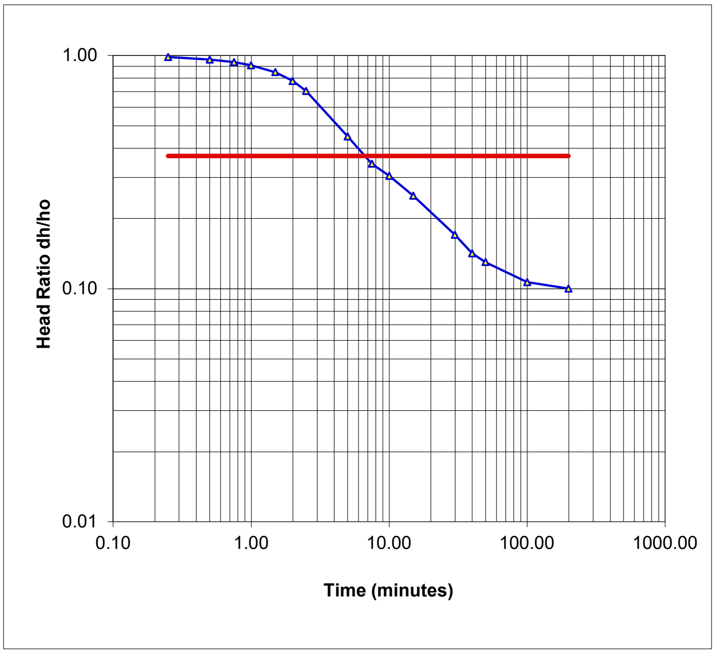
Test No. BH113
Easting: 325701.1 m
Northing 6260273.1 m
Surface Level: 46.9 m AHD

Details of Well Installation
Well casing diameter (2r) 50 mm
Well screen diameter (2R) 100 mm
Length of well screen (Le) 3.5 m

Depth to water before test 6 m
Depth to water at start of test 0 m

Test Results

Time (min)	Depth (m)	Change in Head: δH (m)	δH/Ho
0.00	0.00	6.00	1.000
0.25	0.1	5.90	0.983
0.50	0.24	5.76	0.960
0.75	0.38	5.62	0.937
1.00	0.55	5.45	0.908
1.50	0.91	5.09	0.848
2.00	1.34	4.66	0.777
2.50	1.77	4.23	0.705
5.00	3.3	2.70	0.450
7.5	3.94	2.06	0.343
10	4.17	1.83	0.305
15	4.5	1.50	0.250
30	4.98	1.02	0.170
40	5.15	0.85	0.142
50	5.22	0.78	0.130
100	5.36	0.64	0.107
200	5.4	0.60	0.100



To = 6.6 mins
396 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity k = **9.6E-07** m/sec 9.6E-05 cm/sec
= **0.345** cm/hour

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Permeability Testing - Falling Head Test Report

Client:	Frasers Property Ivanhoe	Project No:	86043.06
Project:	Stage 2 - Midtown	Date:	28-May-21
Location:	Herring Road, Macquarie Park	Tested by:	LS

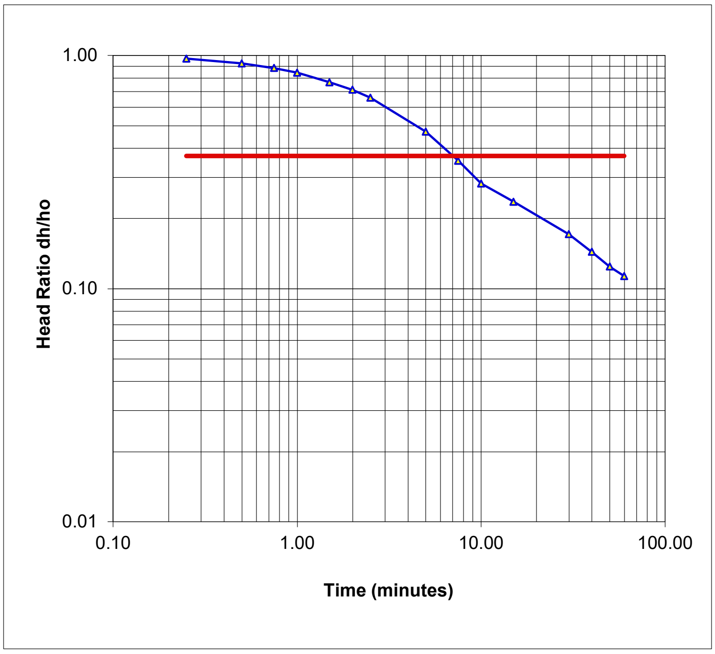
Test Location		**Test No.**	BH114	
Description:	Unit 3D	Easting:	325695.41	m
Material type:	Sandstone	Northing	6260286.6	m
		Surface Level:	47.3	m AHD

Details of Well Installation

Well casing diameter (2r)	50	mm	Depth to water before test	6.19	m
Well screen diameter (2R)	100	mm	Depth to water at start of test	0	m
Length of well screen (Le)	6.6	m			

Test Results

Time (min)	Depth (m)	Change in Head: δH (m)	δH/Ho
0.00	0.00	6.19	1.000
0.25	0.2	5.99	0.968
0.50	0.46	5.73	0.926
0.75	0.72	5.47	0.884
1.00	0.97	5.22	0.843
1.50	1.44	4.75	0.767
2.00	1.78	4.41	0.712
2.50	2.11	4.08	0.659
5.00	3.27	2.92	0.472
7.5	4	2.19	0.354
10	4.44	1.75	0.283
15	4.73	1.46	0.236
30	5.13	1.06	0.171
40	5.3	0.89	0.144
50	5.42	0.77	0.124
60	5.49	0.70	0.113



To = 7.1 mins
426 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity k = **5.4E-07** m/sec 5.4E-05 cm/sec
= **0.195** cm/hour

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Permeability Testing - Falling Head Test Report

Client:	Frasers Property Ivanhoe	Project No:	86043.06
Project:	Stage 2 - Midtown	Date:	27-May-21
Location:	Herring Road, Macquarie Park	Tested by:	LS

Test Location

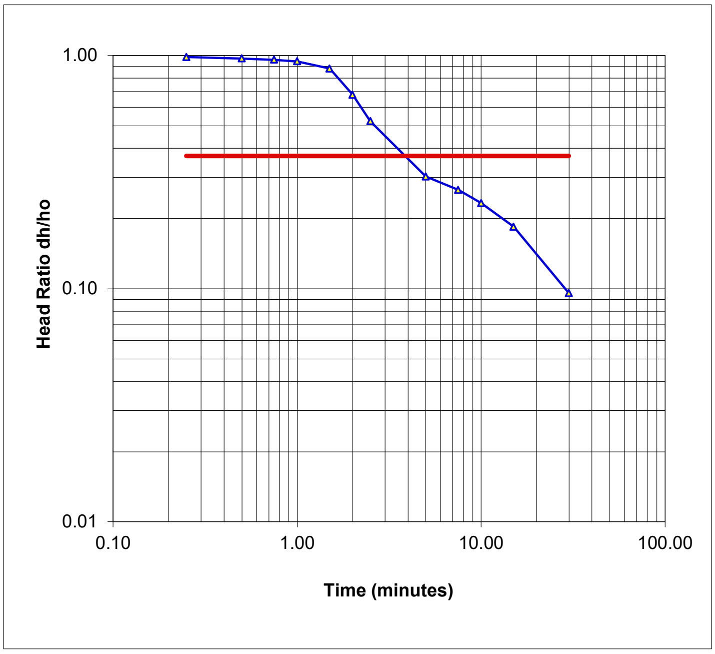
Description:	Unit 3C	**Test No.**	BH115	
Material type:	Sandstone	Easting:	325707.2	m
		Northing	6260312	m
		Surface Level:	46.4	m AHD

Details of Well Installation

Well casing diameter (2r)	50	mm	Depth to water before test	5.73	m
Well screen diameter (2R)	100	mm	Depth to water at start of test	0.41	m
Length of well screen (Le)	3.5	m			

Test Results

Time (min)	Depth (m)	Change in Head: δH (m)	δH/Ho
0.00	0.41	5.32	1.000
0.25	0.49	5.24	0.985
0.50	0.56	5.17	0.972
0.75	0.63	5.10	0.959
1.00	0.71	5.02	0.944
1.50	1.05	4.68	0.880
2.00	2.12	3.61	0.679
2.50	2.95	2.78	0.523
5.00	4.12	1.61	0.303
7.5	4.32	1.41	0.265
10	4.49	1.24	0.233
15	4.75	0.98	0.184
30	5.22	0.51	0.096



To = 3.3 mins
198 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity	**k =**	**1.9E-06**	m/sec	1.9E-04 cm/sec
	=	**0.690**	cm/hour	

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Permeability Testing - Falling Head Test Report

Client:	Frasers Property Ivanhoe	Project No:	86043.06
Project:	Stage 2 - Midtown	Date:	27-May-21
Location:	Herring Road, Macquarie Park	Tested by:	LS

Test Location

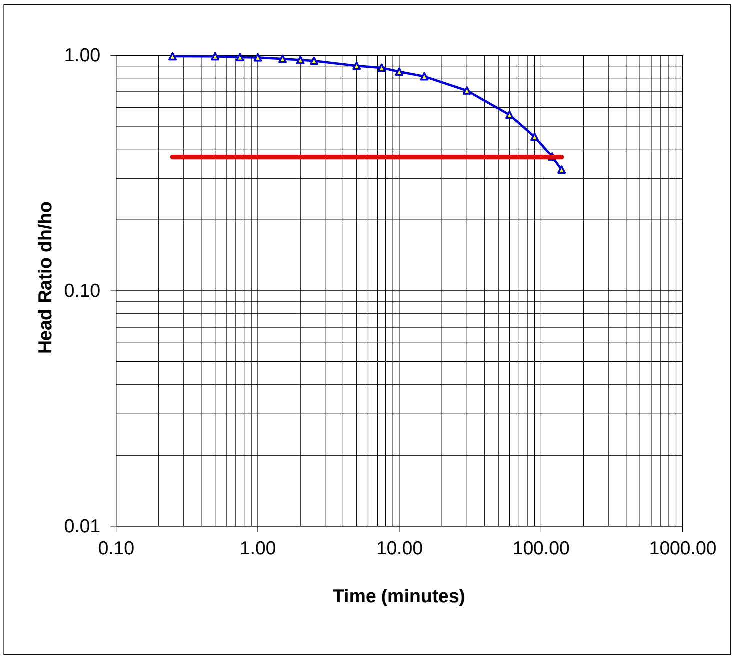
Description:	Unit 3A	**Test No.**	BH118A	
Material type:	Sandstone	Easting:	325675	m
		Northing	6260378	m
		Surface Level:	46.4	m AHD

Details of Well Installation

Well casing diameter (2r)	50	mm	Depth to water before test	5.38	m
Well screen diameter (2R)	100	mm	Depth to water at start of test	0.45	m
Length of well screen (Le)	2.1	m			

Test Results

Time (min)	Depth (m)	Change in Head: dH (m)	dH/Ho
0.00	0.45	4.93	1.000
0.25	0.49	4.89	0.992
0.50	0.5	4.88	0.990
0.75	0.53	4.85	0.984
1.00	0.55	4.83	0.980
1.50	0.62	4.76	0.966
2.00	0.67	4.71	0.955
2.50	0.71	4.67	0.947
5.00	0.93	4.45	0.903
7.5	1.01	4.37	0.886
10	1.17	4.21	0.854
15	1.37	4.01	0.813
30	1.89	3.49	0.708
60	2.62	2.76	0.560
90	3.16	2.22	0.450
120	3.55	1.83	0.371
140	3.77	1.61	0.327



To = 120 mins
7200 secs

Theory: Falling Head Permeability calculated using equation by Hvorslev

$k = [r^2 \ln(Le/R)]/2Le\ To$

where r = radius of casing
R = radius of well screen
Le = length of well screen
To = time taken to rise or fall to 37% of initial change

Hydraulic Conductivity k = **7.7E-08** m/sec 7.7E-06 cm/sec
= **0.028** cm/hour

Appendix D

Summary of Groundwater Measurements

Appendix D - Summary of Groundwater Measurements - Midtown, Macquarie Park

Groundwater level measurements at standpipes in the vicinity of the Stage 2 development area of the Midtown site are summarised in Table D1, below, together with reference to the reports which provide the relevant logs. Standpipe construction details are summarised in Table D2.

Table D1 – Summary of Groundwater Measurements – Stage 2 Midtown, Macquarie Park

Test Location	Ground Surface RL	Depth to Water (m)	Water Level (RL)	Comment	Gravel Interval (m)	Status	Original Report(s)
07	59.1	13.2-13.9	45.2-45.8	Monitoring Period November 2017-June 2018	1.2-21.0	Destroyed	86043.01.R.005.Rev0; 86043.01.R.001.Rev1
10	45.2	4.4-4.9	40.3-40.8	Monitoring Period November 2017-June 2018	2.6-5.6	Missing	86043.01.R.005.Rev0; 86043.01.R.001.Rev1
12	45.2	3.3-4.3	40.8-41.8	Monitoring Period November 2017-June 2018; Responsive to rainfall events	2.3-6.93	Missing	86043.01.R.005.Rev0; 86043.01.R.001.Rev1
13	46.8	4.8-5.3	41.2-42.0	Monitoring Period November 2017-June 2018	1.8-7.0	Missing	86043.01.R.005.Rev0; 86043.01.R.001.Rev1
101	54.1	7.28	46.8	11/05/2021	7.5-11.0	Intact	86043.06.R.001
103	52.4	-	-	No reading obtained before destruction	11.5-15.0	Destroyed	86043.06.R.002
104A	51.7	10.15	41.55	28/05/21	11.5-13.5	Intact	86043.06.R.002
106	49.5	4.93-4.98	44.5-44.6	11&28/05/2021	7.5-11.0	Intact	86043.06.R.002
107	49.7	8.43-8.61	41.1-41.3	28/04/2021 (8.61m), 28/5/21 (8.43m)	13.7-17.2	Intact	86043.06.R.003
109	46.1	6.34-6.4	39.7-39.8	28/04/2021 (6.4m), 28/5/21 (6.34m)	10.3-13.8	Intact	86043.06.R.003
109A	46.1	2.2-6.1	40.0-43.9	17/5/21 (2.2m), 28/5/21 (6.1m); Nested well	5.0-8.5	Intact	86043.06.R.003
111	45.8	4.9-6.0	39.8-40.9	28/4/21 (6.0m), 17/5/21 (4.9m), 27/5/21 (5.95m)	8.3-11.8	Intact	86043.06.R.003
111A	45.8	2.9-5.54	40.3-42.9	17/5/21 (2.9m), 27/5/21 (5.54m); Nested well	5.0-8.5	Intact	86043.06.R.003
113	46.9	6.23-6.0	40.7-40.9	28/04/2021 (6.23m), 27/5/21 (6.0m)	10.8-14.29	Intact	86043.06.R.003

Continued on next page

Table D1 – Summary of Groundwater Measurements – Stage 2 Midtown, Macquarie Park (continued)

Test Location	Ground Surface RL	Depth to Water (m)	Water Level (RL)	Comment	Gravel Interval (m)	Status	Original Report(s)
114	47.3	6.28-6.19	41.0-41.1	28/04/2021 (6.28m), 28/5/21 (6.19m)	8.3-14.92	Intact	86043.06.R.003
114A	47.3	4.06	43.2	28/5/21; Nested well	1.5-4.5	Intact	86043.06.R.003
115	46.4	5.3-5.73	40.7-41.1	17/5/21 (5.3m), 27/5/21 (5.73m);	7.5-11.0	Intact	86043.06.R.003
118A	50.0	5.38	44.6	28/5/21	4.0-6.1	Intact	86043.06.R.002

Table D2 – Summary of Well Construction – Stage 2 Midtown, Macquarie Park

Bore	101	103	104A	106	107	109	109A
Ground Level	54.1	52.4	51.7	49.5	49.7	46.1	46.1
Backfill	0-7.0	0-11.0	0-10.5	0-7.0	0-13.2	0-9.5	0-4.5
Bento	7.0-7.5	11.0-11.5	10.5-11.5	7.0-7.5	13.2-13.7	9.5-10.3	4.5-5.0
Gravel	7.5-11.0	11.5-15.0	11.5-13.5	7.5-11.0	13.7-17.2	10.3-13.8	5.0-8.5
Blank PVC	0-8.0	0-12.0	0-12.0	0-8.0	0-14.2	0-10.8	0-5.5
Slotted PVC	8.0-11.0	12.0-15.0	12.0-13.5	8.0-11.0	14.2-17.2	10.8-13.8	5.5-8.5

Bore	111	111A	113	114	114A	115	118A
Ground Level	45.8	45.8	46.9	47.3	47.3	46.4	50
Backfill	0-7.5	0-4.5	0-10.3	0-7.8	0-0.5	0-7.0	0-3.0
Bento	7.5-8.3	4.5-5.0	10.3-10.8	7.8-8.3	0.5-1.5	7.0-7.5	3.0-4.0
Gravel	8.3-11.8	5.0-8.5	10.8-14.29	8.3-14.92	1.5-4.5	7.5-11.0	4.0-6.1
Blank PVC	0-8.8	0-5.5	0-11.29	0-8.92	0-2.0	0-8.0	0.0-4.5
Slotted PVC	8.8-11.8	5.5-8.5	11.29-14.29	14.92-8.92	2.0-4.5	8.0-11.0	4.5-6.1