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Report on
Geotechnical Investigation

Proposed Residential Development
29, 31 & 35 Reddall Street
Manly

Prepared for
Reddall Street Pty Ltd

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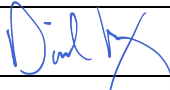
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Report on Geotechnical Investigation

Proposed Residential Development

29, 31 & 35 Reddall Street, Manly

1. Introduction

This report presents the results of a geotechnical investigation undertaken for the design and construction of a proposed residential development at 29, 31 & 35 Reddall Street, Manly.

The investigation was commissioned by Mr Steve Donnellan of Reddall Street Pty Ltd. The investigation was undertaken in accordance with Douglas Partners Pty Ltd's (DP) proposal 207028.00.P.001.Rev0 dated 6 September 2021.

It is understood that the proposed development will comprise a two-storey residential development comprising five dwellings with two separated single level basements. DP was commissioned to carry out a geotechnical investigation to provide information on the subsurface conditions for the design of excavations and footings.

The investigation included the drilling of five boreholes (Bores 1 to 5) using portable equipment and geotechnical inspection of the site. Details of the investigation and the results obtained are given within this report, together with comments and recommendations relating to design and construction practices.

The following documents were provided to DP for information:

- Architectural design drawings DA00 to DA12 by Wolski Coppin Architecture (Project 22020 dated 25-1-23);
- Site survey drawing by Intrax (Reference 122511_SUR_DE) Rev 4 dated 16-6-21); and
- Geotechnical site investigation report 30375vrpt by JK Geotechnics (dated 15-4-19).

2. Site Description & Geology

Colour photographs 1 to 6 in Appendix A depict the site at the time of the field work. For the purposes of site descriptions within this report, site east is assumed to be the direction downslope and perpendicular from Reddall Street.

The site for the proposed development comprises three adjacent residential lots located on the lower (eastern) side of Reddall Street. The site is near trapezoid shaped with major site dimensions of approximately 50 m by 40 m. Total cross fall over the site towards the east is in the order of 8 m (i.e. RL 22 to RL 14 AHD), with a resultant average slope angle in the order of 10°.

The site is bounded to the east by a downslope residential lot (95 Bower Street), to the north by a Council reserve, and to the south by College Street.

Existing developments on the site comprise one and two storey brick and tile dwellings with associated concrete or paved driveways, garages and external decking. An in-ground swimming pool is located within the rear (eastern) yard on 29 Reddall Street.

Away from existing structures, the lots are typically grassed or over-grown with shrubs and weeds. There are several mature trees scattered across the lots. Various timber, concrete block or sandstone flagging retaining walls to around 1 m height separate terraced areas on each lot.

Reference to the Sydney 1:100 000 Geological Series Sheets indicates that the site is underlain by Hawkesbury Sandstone of Triassic age.

Hawkesbury Sandstone typically comprises medium to coarse grained sandstone with very minor shale and laminite lenses. Bedrock exposed within the outcrops either on or beside the site, and recovered from the boreholes, is consistent with Hawkesbury Sandstone (refer to Photos 2, 4 and 5).

3. Previous Investigation

Previous investigation at the site by JK Geotechnics included a desk-top study assessment and the hand-auger drilling to refusal of two bores (Bores JK1 and JK3) in the rear yards of 35 and 31 Reddall Street respectively. Bore JK1 reached refusal in sandstone bedrock at 0.9 m depth and Bore JK3 refused in silty sand fill with ash and sandstone gravel at 0.4 m depth.

The approximate location of Bores JK1 and JK3 are indicated on Drawing 1 in Appendix A.

4. Field Work Methods

The current field work comprised geological mapping by a senior engineering geologist between 29 November and 1 December 2021, in conjunction with the drilling of five bores (Bores 1 to 5) at the locations indicated on Drawing 1 in Appendix A. Dynamic cone penetrometer tests were conducted beside four of the bore locations (DCP 2 to 5) to provide information on the strength of the overburden soils.

The bores were initially drilled with a 100 mm diameter hand auger to refusal on the top of the bedrock surface at depths ranging from 0.2 m to 1.25 m, and then temporarily lined with PVC casing. The bores were then advanced into the bedrock to depths ranging from around 6.6 m to 7.8 m (being between 2 m to 5 m below the proposed level of basement excavation) using NMLC (50 mm diameter) diamond core methods fitted to hand operated Pro-line equipment (refer to Photos 1 and 3).

The Pro-line equipment was manually carried onto the site and was powered by hydraulic lines running from a support vehicle parked off-site. Water used to flush the bore whilst drilling was continually recirculated through a mud tank, with excess drilling water dispersed on the site at the conclusion of the field work.

The field work was carried out under the direction of a geotechnical engineer who also logged the bores and undertook the DCP testing.

The coordinates and surface level for the borehole locations were determined using a differential Global Positioning System (dGPS) receiver, which has an accuracy of +/- 0.1 m. Coordinates are in GDA94/MGA Zone 56 format (Geocentric Datum of Australia 1994 base with Map Grid of Australia projection) and surface level is relative to Australian Height Datum (AHD).

5. Field Work Results

5.1 Walkover Geological Inspection

There was no evidence of distress in the walls of the existing residential structures on the site which could be attributed to significant previous slope or footing movements. Cracking in concrete paths and steps surrounding the residential structures is probably due to gradual consolidation of underlying filling, tree root growth or slow downhill soil creep.

A concrete block garden wall alongside the College Street boundary displays a distinct outward lean, probably due to nearby trees pushing it over (refer to Photo 4).

Sandstone bedrock exposures were noted on and beside the site and are indicated on Drawing 1. Areas of seepage were noted across Reddall Street, most probably due to shallow groundwater flow across the surface of the bedrock (refer to Photo 6).

The residence on 95 Bower Street has been constructed on a bench, cut approximately 1.5 m to 2 m into the slope below the downslope boundary of the subject site. DP understands that the owner of 95 Bower Street has reported that some stormwater or groundwater seepage has previously occurred onto their property from the subject site.

5.2 Subsurface Conditions

Subsurface conditions encountered at the borehole locations are given in the detailed logs in Appendix B and should be read in conjunction with the notes defining classification methods and descriptive terms. The succession of strata is broadly summarised below:

- **FILL or NATURAL SOILS** – surficial silty sand fill or natural silty sand (topsoil) and sand with some rootlets, extending to between 0.3 m and 1.25 m depth (absent in Bore 1 where bedrock was directly overlain by a concrete slab) overlying;
- **SANDSTONE BEDROCK** – typically medium to high strength and ranging from highly weathered to fresh, initially very low to low strength in Bore 4. The sandstone bedrock encountered by the bore was typically slightly fractured, with most defects ranging from relatively low angle bedding planes to high angle joints. The sandstone bedrock extended to the termination depth of the bores with the exception of Bores 3 and 5 which encountered a layer of;
- **LAMINITE** – an approximately 2 m thick, very low to low strength, highly weathered, interlaminated sandstone and siltstone lying at around 4 m depth in Bores 3 and 5. Both of these bores were extended by around 3 m to determine the thickness of the laminite layer.

The results of probing with the DCP implied that the sandy fill and natural soils overlying the bedrock at the bore locations is typically in a loose to medium dense condition.

5.3 Groundwater Observations

Observation of groundwater levels within the bores was generally obscured by drilling water introduced to flush out cuttings. Standing water in Bore 3 was measured at 2.7 m depth (RL 12.5 m AHD), approximately 20 hours after the completion of drilling, although this observation is also probably influenced by drilling water remaining within the bore.

6. Laboratory Testing

Thirty four representative samples of sandstone or laminite bedrock core from the bores were tested for Axial Point Load Strength Index $Is(50)$, with the results presented in Table 1 below and at the appropriate depths on the borehole logs (refer Appendix B).

Table 1: Summary of Point Load Test Results

Bore	Sample Depth (m)	Material	$Is(50)$ (MPa)	Inferred UCS** (MPa)	Strength
1	0.65	Sandstone	0.5	10	Medium
1	1.5	Sandstone	0.8	16	Medium
1	2.15	Sandstone	1.0	20	Medium to High
1	3.24	Sandstone	0.9	18	Medium
1	4.2	Sandstone	0.4	8	Medium
1	4.97	Sandstone	0.9	18	Medium
1	5.6	Sandstone	1.1	22	High
1	6.4	Sandstone	0.8	16	Medium
2	0.5	Sandstone	0.3	6	Low to Medium
2	1.3	Sandstone	0.6	12	Medium
2	2.1	Sandstone	0.6	12	Medium
2	3.0	Sandstone	0.5	10	Medium
2	4.15	Sandstone	1.2	24	High
2	5.25	Sandstone	1.4	28	High
2	6.15	Sandstone	0.8	16	Medium
3	0.65	Sandstone	0.6	12	Medium
3	1.55	Sandstone	0.8	16	Medium

Bore	Sample Depth (m)	Material	Is(50) (MPa)	Inferred UCS** (MPa)	Strength
3	2.7	Sandstone	1.0	20	Medium to High
3	3.6	Sandstone	0.9	18	Medium
3	4.45	Laminite	0.1	2	Very low to Low
3	5.5	Laminite	0.2	4	Low
3	6.1	Sandstone	0.1	2	Very low to Low
3	7.6	Sandstone	0.9	18	Medium
4	1.45	Sandstone	0.1	2	Very low to Low
4	2.5	Sandstone	0.2	4	Low
4	3.7	Sandstone	0.4	8	Medium
4	4.5	Sandstone	0.5	10	Medium
4	5.65	Sandstone	0.5	10	Medium
5	1.7	Sandstone	0.4	8	Medium
5	2.55	Sandstone	0.5	10	Medium
5	3.65	Sandstone	1.6	32	High
5	4.6	Laminite	0.1	2	Very low to Low
5	5.1	Laminite	0.1	2	Very low to Low
5	6.5	Sandstone	0.3	6	Low to Medium

Note: UCS** assuming a correlation factor with Is(50) of 20:1.

7. Proposed Development

It is understood that the proposed development will comprise a two-storey residential development comprising five dwellings with two separated single level basements (both accessed from College Street). The basement car parking will extend to within around 2 m of the eastern, downslope boundary and to within around 3 m to 6 m of the remaining site boundaries.

Bulk excavation for the proposed basements will range from around 3 m to 4 m depth below existing site levels on the high side and an average of 1 m on the low side of the site.

The footprints of the proposed basement excavations are indicated on Drawing 1 with a typical cross section provided in Drawing 2 (both in Appendix A).

8. Comments

8.1 Geological Model

The results of geotechnical investigations on the site indicate that the sub-surface profile typically comprises medium to high strength Hawkesbury Sandstone bedrock with some low strength layers, generally overlain by sandy fill soils or natural loose to medium dense silty sands and sands. A 2 m thick, very low to low strength laminite layer within the sandstone, which was noted in Bores 3 and 5 below approximately RL 11.2 AHD, possibly extends below the whole site at a similar level.

The soil thickness at the bore locations ranged from negligible to around 1.25 m and is probably thicker behind some of the retaining walls on the site.

Groundwater flow across the site towards the east is expected to be primarily controlled by the presence of defects in the sandstone rock mass, particularly above the less permeable laminite layer. Temporary increased flows would also be likely to occur within the overburden soils and along the surface of the bedrock following rainfall.

An inferred geological cross-section is shown in Drawing 2 in Appendix A, together with the proposed basement level designs.

8.2 Excavation

Excavation for the proposed basements will extend to depths of approximately 3 m to 4 m depth below existing site levels on the high side and an average of 1 m on the low side of the site. The results of the field work indicates that excavation will intersect variable depths of sandy fill or natural soils then medium strength sandstone with some low and high strength layers.

It is expected that bulk excavation of the basements to the easternmost basement will not reach the very low to low strength laminite layer which was encountered below approximately RL 11.2 in Bores 3 and 5.

Based on the results of the bores, few difficulties are foreseen in removing the overburden natural soils and filling with conventional earthmoving equipment. However, excavation of low to medium and high strength sandstone will require excavator mounted rock hammers, ripping hooks, rock saws or milling heads.

On this site the size of rock hammer that may be used will possibly be limited by vibrations generated by the excavation process.

Rock saws or milling heads generate much less vibration than rock hammers but generate substantially more dust. Measures for control of dust generated by rock saws or milling wheels or other excavation techniques will be required.

High horizontal stresses are present in bedrock within the Sydney area. As the excavation depth increases, some of these stresses may be released, which could result in lateral movement of the rock and potentially cause some cracking of the adjacent buildings. Experience in Sydney indicates that lateral movement due to stress relief for an excavation is generally in the range of 0.5 mm to 1 mm,

although occasionally up to 2 mm per metre depth of excavation. In Hawkesbury Sandstone, the movement resulting from stress relief generally occurs across a horizontal distance of up to three times the excavation depth from the excavation boundaries.

Given the distances and differences in levels to residences on adjacent properties, it is not expected that stress relief within the bedrock will be a major issue during bulk excavation on this site.

8.3 Vibrations

The German DIN4150 guidelines for construction vibration indicate that well-constructed residential buildings are generally not adversely affected by vibration levels below a peak particle velocity (ppv) of 15 mm/sec. However, complaints from residents are common for vibration values greater than about 3 mm/sec. While vibrations are only slightly perceptible to humans at about 1 mm/sec, they become strongly perceptible above 3 mm/sec and disturbing above 5 mm/sec.

While it is unlikely that well-constructed buildings will suffer damage with vibration ppv of 15 mm/sec, some minor defects such as cracks through rendering, cornices and skirtings may occur. If the neighbouring buildings have been poorly constructed, then vibration levels less than 8 mm/sec may cause defects to be amplified and damage may be visible. Based on the DIN4150 guidelines for sensitive structures and Australian Standard AS2670.2 for human comfort, it is recommended that the vibration levels at the footing levels of adjacent buildings should be kept to less than 8 mm/sec vector sum peak particle velocity (VSPPV) to minimise damage to the adjacent buildings but as indicated, it is likely that neighbours will be aware of vibration and so should be warned.

Excavation of low or greater strength sandstone will require the use of pneumatic or hydraulic rock breaking equipment for effective excavation. Ground vibrations generated during excavation works in sandstone bedrock will need to remain within acceptable limits with respect to limiting damage to the adjacent buildings and structures. Vibration arising from rock-sawing, if adopted, would be expected to generally remain within acceptable levels.

Excavation equipment would need to be chosen with a view to limiting vibration at the base of walls/structures on adjacent properties to less than 5 mm/s peak particle velocity (PPV).

Vibration monitoring carried out by Douglas Partners at various excavation sites in Hawkesbury Sandstone around Sydney has indicated the following relationships (Table 2) of peak particle velocity versus distance for various hammer types, milling heads and rock saw attachments. These may be used for initial guidance in plant selection.

Table 2: Approximate Buffer Distances for Selected Plant (Provisional Allowed Limit VSPPV 8 mm/s)

Excavation Plant	Distance from plant by which vibration would attenuate to the Provisional Allowed Limit	
	From DP trial maxima ¹	From DP trial averages
Moving machinery (Excavators/Dozers)	7 m	0.5 m
Rollers (smooth/vibratory/impact)	16 m	3 m
Profilers	1 m	1 m
Trimmers (grinders/milling heads)	2 m	1 m
Rock Saw on Excavator ²	1 m	0.6 m
Jackhammers	3 m	0.9 m
Percussion drilling rigs	1 m	1 m
Auger drilling rigs	0.7 m	0.4 m
Rippers on 6 - 36t Excavators	3 m	1.2 m
Rock Hammer < 500 kg operating weight	7 m	3.0 m
Rock Hammer 501 - 1000 kg operating	8 m	3.3 m
Rock Hammer 1001 - 2000 kg operating	13 m	5.2 m
Rock Hammer > 2000 kg operating weight	7 m	4.9 m

Note:

1. Smaller distances can generally be determined from individual trials, as indicated by those from trial averages;
2. Buffer distances for rock hammers may be reduced by prior saw cutting along, or parallel to, excavation boundaries; and
3. Loading effects from adjacent buildings may reduce vibration levels, often enabling boundary saw cuts with few exceedances;

As the magnitude of vibrations can vary substantially between sites, it is recommended that a vibration trial be undertaken at the commencement of excavation on the site to determine the vibration attenuation characteristics of the site and thus determine the size of equipment that may be used and how close the equipment may approach the adjacent buildings.

To further minimise disturbances of neighbours the operation of this type of equipment should include:

- excavation of soils and loose or rippable sandstone blocks by bucket or single tyne attachments prior to commencement of rock sawing or rock hammering;
- the use of rock sawing wherever possible;
- adjacent saw cuts within zones of massive bedrock should be spaced at distances no greater than about 300 mm as excessive distance between saw cuts could result in excessive vibration when the rock pieces are broken off;
- progressive breakage from open excavated faces;
- selective breakage along open joints where these are present;
- use of rock hammers in short bursts to prevent generation of resonant frequencies;

- orientation of the rock hammer pick away from property boundaries and into the existing open excavation; and
- the movement of large blocks away from the site boundaries prior to breaking up for transport from site.

Depending on the proximity of the excavation footprint to adjacent buildings, vibration monitors could be set up to continuously monitor construction vibrations, with a warning system which can be triggered should vibrations exceed an agreed level.

Excavation works within bedrock at distances from boundary walls or adjacent structures less than the safe distance determined by a vibration trial or where on-site vibration monitoring indicates vibration in excess of 8 mm/s, should be achieved by the use of a milling head, hand operated jack hammer or by rock sawing.

Measures for control of dust generated by a milling head or saw would also be required.

DP recommends that dilapidation (building condition) reports be undertaken on the adjacent residential structures, along with nearby footpaths and roadway prior to commencing demolition and excavation work on the site. The dilapidation reports should document any existing defects so that any potential claims for damage arising during the construction period can be accurately assessed.

8.4 Excavation Support

Any soil around the excavation edges should be temporarily battered back to slopes of 1:1 (V:H) or flatter and covered from the weather to minimise the risk of slumping prior to the construction of permanent retaining walls. Temporary support of the soils may be required if battering within the site boundaries is not possible.

Retaining walls for permanent support of excavations may be designed on the basis of an average bulk unit weight of 20 kN/m³ and 22 kN/m³ for soil and rock, respectively, and a triangular lateral earth pressure distribution based on the earth pressure coefficients given in Table 3.

Table 3: Suggested Lateral Earth Pressures Coefficients

Material	Lateral Earth Pressure Coefficients	
	(Ka)	(Kp)
Fill	0.3	----
Soils and extremely low strength rock	0.3	2 - 3.3
Low to Medium or greater strength rock*	0	2000 kPa (ultimate)

*provided there is no adverse jointing or fracturing of the rock

All surcharge loads, such as building loads from adjacent structures, should be included in the design of retaining walls. Suitable allowance should be made for hydrostatic pressures, batters behind the

slope and surcharges. It is recommended that all retaining structures have free draining backfill with suitable subsoil drainage installed to discharge the water collected in the backfill.

Hawkesbury Sandstone of at least low strength would be expected to stand unsupported in a vertical face, provided there are no adversely oriented joints, faults or other defects. The presence of some 70° to 80° joints in the retrieved bedrock core implies that some steep and/or adversely oriented joints may be intersected within the walls of the basement excavation. Excavation faces should be inspected at maximum 1.5 m drops to assess whether there is any need for rock bolts, anchors or shotcrete protection.

It should be noted that very low strength sandstone (or laminite), if encountered during the bulk excavations, should be protected from long-term weathering by a nominal 80 mm thickness of pinned, mesh-reinforced shotcrete.

If required, anchors can be designed using an allowable bond strength of 350 kPa in medium strength sandstone and 600 kPa in medium to high and high strength sandstone. If temporary bolts or anchors are required to cross property boundaries, written permission from the owners of the property will be required.

It is expected that permanent support for the basement excavation faces will be provided by the final structures.

8.5 Drainage

It is expected that the presence of shallow bedrock will preclude the effective use of stormwater absorption pits on this site. Therefore, it is recommended that all stormwater generated from the new development on the site be piped to the Council system via a system of appropriately sized storage and detention tanks.

In general, groundwater inflow into the proposed basement excavations would be expected to be primarily controlled by the presence of defects in the sandstone rock mass, although temporary increased inflows would be likely to occur along the surface of the bedrock from a perched water table within the overburden soils following rainfall.

It is anticipated that during the excavation there may also be minor seepage from bedding planes or joints within the bedrock. Such seepage should be confined to a temporary sump area and the water allowed to clarify before disposal to the Council's stormwater system.

Note that initial inflow into the excavation may vary substantially from long term inflows. Observation of groundwater inflow should be made during the final stages of excavation when the water table has been lowered to near the equilibrium condition, and sandstone faces have been exposed before finalising the drainage design.

As only low flows are anticipated, appropriate long-term groundwater drainage would include sub floor drainage incorporated into the final structure to relieve any long-term seepage flows accumulating below the basement slab. Adequate cross-fall of such drains to a permanent sump should be provided.

Construction of the proposed development and the basements would be expected to capture and divert stormwater and groundwater flows on the site, and generally reduce the volume of water flowing across the downslope boundary and onto 95 Bower Street.

Douglas Partners' experience has been that relatively high soluble iron levels are often present in the groundwater in the Sydney area. Drainage systems need to be constructed taking the presence of iron into account and where possible incorporate access for potential flushing out or rodding of drainage lines. In particular, the use of crushed concrete as a drainage layer is not recommended as concrete in conjunction with oxygen tends to induce the precipitation of iron from groundwater (due to the high pH of concrete), tending to clog the drainage layer with a brown gelatinous iron sludge.

Disposal of pumped groundwater into the stormwater system is dependent on water meeting the EPA quality requirements, including that of less than 0.3 mg/L of soluble iron.

8.6 Foundations

Pad footings founded on sandstone of at least medium strength sandstone are recommended for foundations on this site. Such footings can be designed for an allowable bearing pressure (ABP) of up to 1 MPa.

Note that the design ABP could be increased to 3.5 MPa, subject to inspection and spoon testing (to 1.5 times the minimum plan footing dimension) in the base of all pad footings by a geotechnical engineer or engineering geologist. The spoon testing would be required to confirm that the footings were not immediately underlain by the very low to low strength laminite layer that was encountered by Bores 3 and 5.

If uplift is a consideration, an allowable socket adhesion within the medium to high strength sandstone of 350 kPa is considered appropriate for design when considering compressional or uplift loads for piers. Some slight groundwater inflows could reasonably be expected during construction of pad footings or bored piers, either from across the bedrock surface during wet weather or from isolated bedding planes within the rock mass.

8.7 Inspections During Construction

It is considered that an appropriate level of construction inspections would comprise regular site visits by geotechnical personnel for the duration of excavation and foundation works in order to inspect excavation faces and to confirm the design requirements with respect to founding strata and levels had been met by all footings.

8.8 Waste Classification

It should be noted that under the Waste Minimisation and Management Act, the burden of proof that materials received by a waste fill site meet the environmental criteria for proposed land use rests on the owner of the materials. This includes filling and virgin excavated natural materials, such as will be removed from site.

Excavated materials to be disposed of off-site will require environmental testing to classify the spoil. The type and extent of testing undertaken will depend on the final use or destination of the spoil, and requirements of the receiving site. It should be noted that some non-licensed fill sites, such as those operated by Councils or other bodies may have their own special environmental criteria to be met before admitting any materials.

Note that the site is underlain by sandy fill, residual soils and sandstone bedrock and no issues with respect to acid sulphate soils are expected.

9. Limitations

Douglas Partners Pty Ltd (DP) has prepared this report for this project at 29, 31 and 35 Reddall Street, Manly in accordance with Douglas Partners Pty Ltd.'s (DP) proposal 207028.00.P.001.Rev0 dated 6 September 2021. This report is provided for the exclusive use of Reddall Street Pty Ltd for this project only and for the purposes as described in the report. It should not be used by or be relied upon for other projects or purposes on the same or another site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

The 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.

This report must be read in conjunction with all of the attached notes entitled "About this Report" (in Appendix B) 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.

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

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

The contents of this report do not constitute formal design components such as are required, by the Health and Safety Legislation and Regulations, to be included in a Safety Report specifying the hazards likely to be encountered during construction and the controls required to mitigate risk. This design process requires a risk assessment to be undertaken, with such assessment being dependent upon factors relating to likelihood of occurrence and consequences of damage to property and to life. This, in turn, requires project data and analysis presently beyond the knowledge and project role respectively of DP. DP may be able, however, to assist the client in carrying out a risk assessment of potential hazards contained in the Comments section of this report, as an extension to the current scope of works, if so requested, and provided that suitable additional information is made available to DP. Any such risk assessment would, however, be necessarily restricted to the geotechnical / groundwater components set out in this report and to their application by the project designers to project design, construction, maintenance and demolition.

Douglas Partners Pty Ltd

Appendix A

About this Report
Drawings 1 and 2

About this Report

Douglas Partners



Introduction

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

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

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Borehole and Test Pit Logs

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

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

Groundwater

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

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole is left open;

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

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

Reports

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

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

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

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

About this Report

Site Anomalies

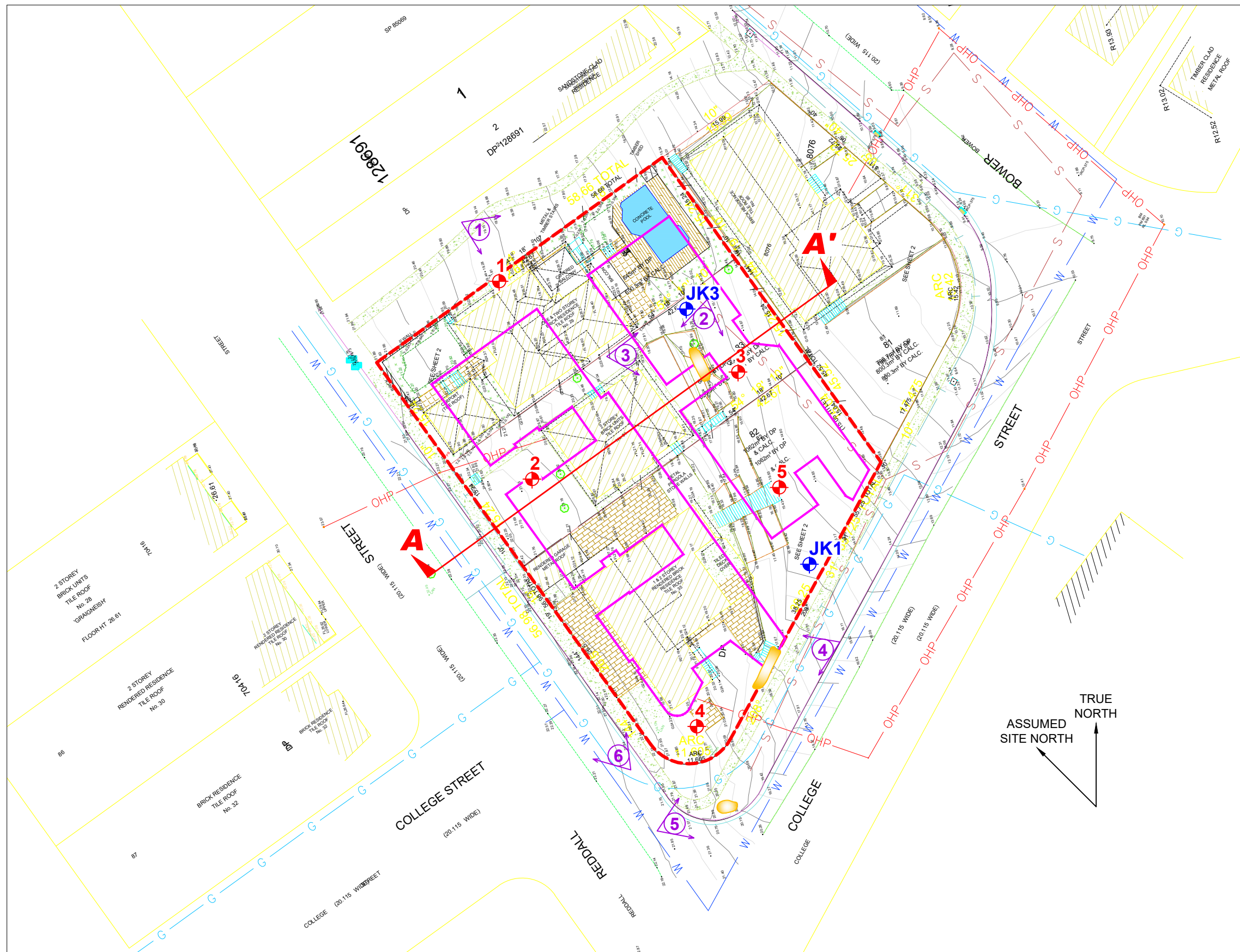
In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, DP requests that it be immediately notified. Most problems are much more readily resolved when conditions are exposed rather than at some later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

Information for Contractual Purposes

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

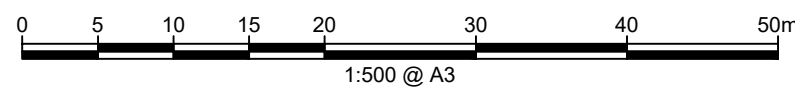
Site Inspection

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



Locality Plan

NOTE:
1: Base Survey Plan from Intrax, Reference No. 122511_SUR_DE, Revision 4
(Dated 16.06.2021)
2: Test locations are approximate only and are shown with reference to existing site features.

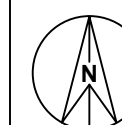


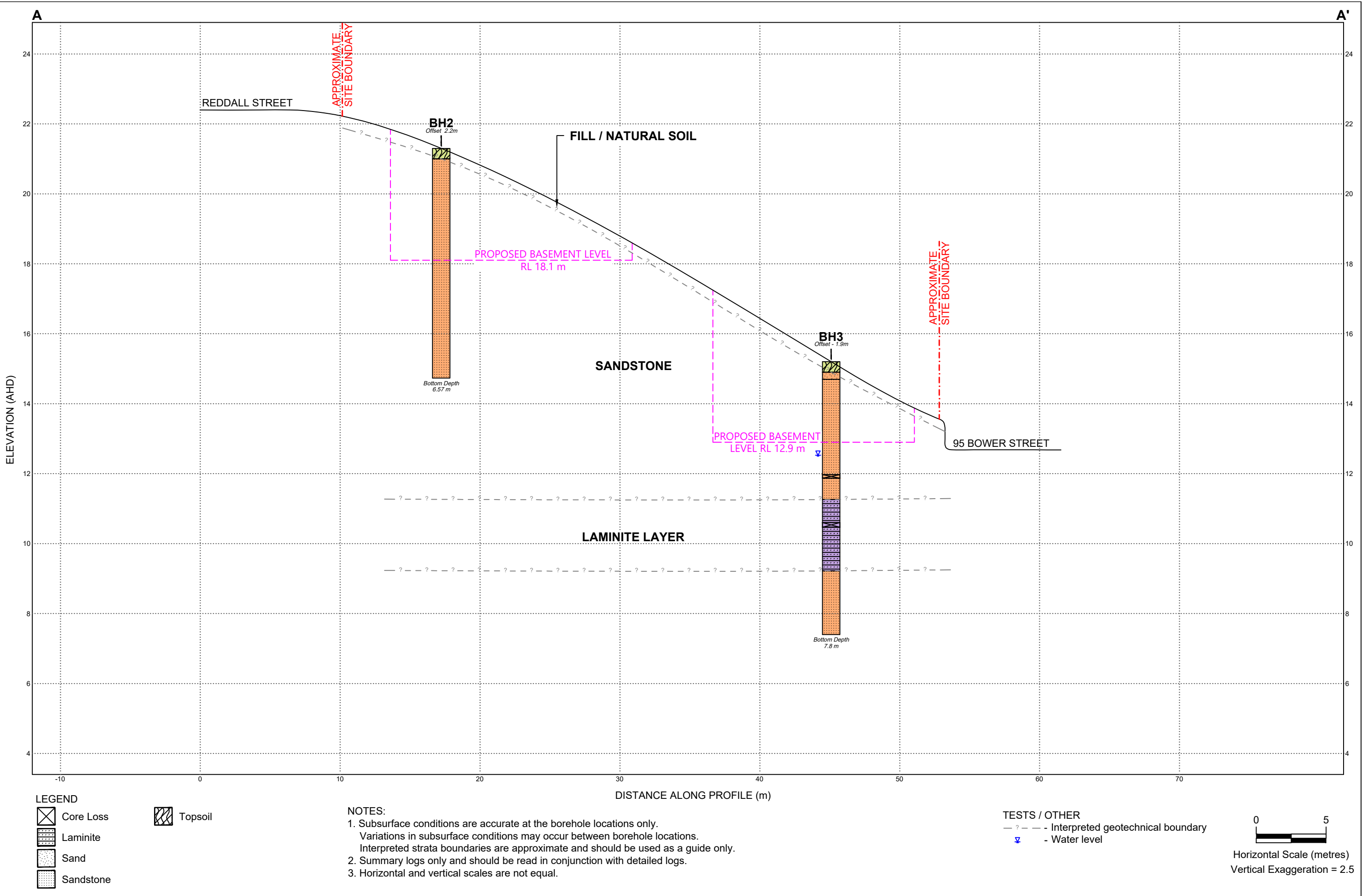
1:500 @ A3

TRUE NORTH
ASSUMED
SITE NORTH

LEGEND

- Approximate Site Boundary
- Approximate Footprint of Proposed Basements
- Borehole Location
- Approximate Previous Borehole Location (JK, 2017)
- Sandstone Bedrock Outcrop
- Photo number with direction of view
- Geological Cross Section





Appendix B

Borehole Logs and Core Photos
Dynamic Cone Penetrometer Result Sheet
Site Photographs



Sampling

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

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

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

Test Pits

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

Large Diameter Augers

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

Continuous Spiral Flight Augers

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

reliability, due to the remoulding, possible mixing or softening of samples by groundwater.

Non-core Rotary Drilling

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

Continuous Core Drilling

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

Standard Penetration Tests

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

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

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150 mm of, say, 4, 6 and 7 as:
4,6,7
N=13
- In the case where the test is discontinued before the full penetration depth, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm as:
15, 30/40 mm

Sampling Methods

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

Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests

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

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



Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are 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 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.
<i>Note: If HW and MW cannot be differentiated use DW (see below)</i>		
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:

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

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

Douglas Partners



Introduction

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

Drilling or Excavation Methods

C	Core drilling
R	Rotary drilling
SFA	Spiral flight augers
NMLC	Diamond core - 52 mm dia
NQ	Diamond core - 47 mm dia
HQ	Diamond core - 63 mm dia
PQ	Diamond core - 81 mm dia

Water

▷	Water seep
▽	Water level

Sampling and Testing

A	Auger sample
B	Bulk sample
D	Disturbed sample
E	Environmental sample
U ₅₀	Undisturbed tube sample (50mm)
W	Water sample
pp	Pocket penetrometer (kPa)
PID	Photo ionisation detector
PL	Point load strength Is(50) MPa
S	Standard Penetration Test
V	Shear vane (kPa)

Description of Defects in Rock

The abbreviated descriptions of the defects should be in the following order: Depth, Type, Orientation, Coating, Shape, Roughness and Other. Drilling and handling breaks are not usually included on the logs.

Defect Type

B	Bedding plane
Cs	Clay seam
Cv	Cleavage
Cz	Crushed zone
Ds	Decomposed seam
F	Fault
J	Joint
Lam	Lamination
Pt	Parting
Sz	Sheared Zone
V	Vein

Orientation

The inclination of defects is always measured from the perpendicular to the core axis.

h	horizontal
v	vertical
sh	sub-horizontal
sv	sub-vertical

Coating or Infilling Term

cln	clean
co	coating
he	healed
inf	infilled
stn	stained
ti	tight
vn	veneer

Coating Descriptor

ca	calcite
cbs	carbonaceous
cly	clay
fe	iron oxide
mn	manganese
slt	silty

Shape

cu	curved
ir	irregular
pl	planar
st	stepped
un	undulating

Roughness

po	polished
ro	rough
sl	slickensided
sm	smooth
vr	very rough

Other

fg	fragmented
bnd	band
qtz	quartz

Symbols & Abbreviations

Graphic Symbols for Soil and Rock

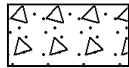
General



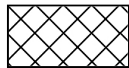
Asphalt



Road base



Concrete



Filling

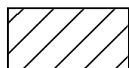
Soils



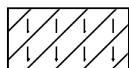
Topsoil



Peat



Clay



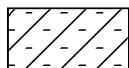
Silty clay



Sandy clay



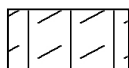
Gravelly clay



Shaly clay



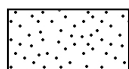
Silt



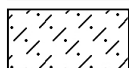
Clayey silt



Sandy silt



Sand



Clayey sand



Silty sand



Gravel



Sandy gravel



Cobbles, boulders



Talus

Sedimentary Rocks



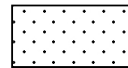
Boulder conglomerate



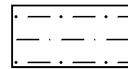
Conglomerate



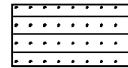
Conglomeratic sandstone



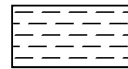
Sandstone



Siltstone



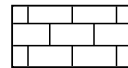
Laminite



Mudstone, claystone, shale

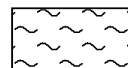


Coal

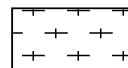


Limestone

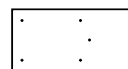
Metamorphic Rocks



Slate, phyllite, schist

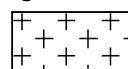


Gneiss

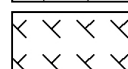


Quartzite

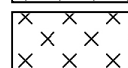
Igneous Rocks



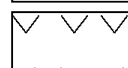
Granite



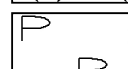
Dolerite, basalt, andesite



Dacite, epidote



Tuff, breccia



Porphyry

ENVIRONMENTAL LOG

Borehole No.
1
1/1

Environmental logs are not to be used for geotechnical purposes

Client: CATHOLIC ARCHDIOCESE OF SYDNEY												
Project: PROPOSED RESIDENTIAL DEVELOPMENT												
Location: 95 BOWER STREET, 31 AND 35 REDDALL STREET, MANLY, NSW												
Job No. E30375K Method: HAND AUGER R.L. Surface: N/A												
Date: 17-5-17 Datum:												
Logged/Checked by: J.D.C./G.F.												
Groundwater Record	SAMPLES			Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	ASS	SAL									
DRY ON COMPLETION					0			FILL: Silty sand, fine to medium grained, dark grey, with root fibres, trace of ash and fine to medium grained sandstone gravel.	M			
					0.5		-	SANDSTONE: fine to medium grained, light grey.	XW			POSSIBLY CRUSHED SANDSTONE FILL
					1			END OF BOREHOLE AT 0.9m				HAND AUGER REFUSAL
					1.5							
					2							
					2.5							
					3							
					3.5							

ENVIRONMENTAL LOG

Borehole No.

3

1/1

Environmental logs are not to be used for geotechnical purposes

Client: CATHOLIC ARCHDIOCESE OF SYDNEY Project: PROPOSED RESIDENTIAL DEVELOPMENT Location: 95 BOWER STREET, 31 AND 35 REDDALL STREET, MANLY, NSW												
Job No. E30375K Date: 17-5-17			Method: HAND AUGER Logged/Checked by: J.D.C./G.F.				R.L. Surface: N/A Datum:					
Groundwater Record	SAMPLES			Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	ASS	ASB									
DRY ON COMPLETION					0			FILL: Silty sand, fine to medium grained, dark grey, with root fibres, trace of ash and fine to medium grained sandstone gravel.	M			
					0.5			END OF BOREHOLE AT 0.4m				HAND AUGER REFUSAL
					1							
					1.5							
					2							
					2.5							
					3							
					3.5							

BOREHOLE LOG

CLIENT: Reddall Street Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 29, 31, 35 Reddall St, Manly

SURFACE LEVEL: 19 AHD
EASTING: 341938
NORTHING: 6258532
DIP/AZIMUTH: 90°/-

BORE No: BH1
PROJECT No: 207028.00
DATE: 29/11/2021
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Degree of Weathering					Graphic Log	Rock Strength					Water	Fracture Spacing (m)	Discontinuities		Sampling & In Situ Testing				
			EW	HW	MW	SW	FS		FR	Ex Low	Very Low	Low	Medium			High	Very High	Ex High	B - Bedding S - Shear	J - Joint F - Fault	Type	Core Rec. %
19	0.15	CONCRETE SLAB																C	100		PL(A) = 0.5	
	0.4	SANDSTONE: medium grained, pale grey and yellow brown, cross bedded, low strength, slightly weathered, with subvertical joints at 80°-85°, tight, Hawkesbury Sandstone														0.40-0.60m: J85°, pl, ro, organic matter						
18	1	SANDSTONE: medium to coarse grained, pale grey and brown, trace quartz gravel, medium and high strength, slightly weathered and fresh, slightly fractured and unbroken, Hawkesbury Sandstone														0.95m: B5°, cly vn, ti 1m: B5°, cly co, organic matter 2mm		C	100	97	PL(A) = 0.8	
17	2															2.05m: B5°, cly co, organic matter 5mm 2.18-2.23m: Ds with qtz gravel 2.35m: B5°, cly 10mm 2.4m: J45°, pl, ro, cly 2.42-2.50m: Cs 2.5m: CORE LOSS: 160mm		C	75	40	PL(A) = 1	
16	3															3.45m: B20°, pl, ro, cln 3.55m: J45°, un, ro, cln		C	100	100	PL(A) = 0.9	
15	4															3.93m: J70°, pl, ro, fe					PL(A) = 0.4	
14	5																	C	100	100	PL(A) = 0.9	
13	6																	C	100	100	PL(A) = 1.1	
	6.72	Bore discontinued at 6.72m Target depth reached																				PL(A) = 0.8
12	7																					
11	8																					
10	9																					

RIG: Proline

DRILLER: Tightsite

LOGGED: SI

CASING: HQ to 0.4m

TYPE OF BORING: Diacore (180mm) to 0.4m, NMLC Coring to 6.72m

WATER OBSERVATIONS: Obscured by drilling water

REMARKS:

SAMPLING & IN SITU TESTING LEGEND

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

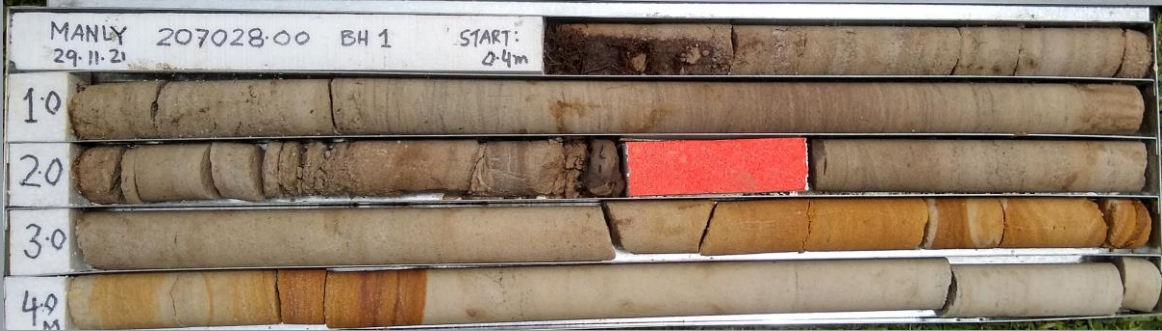
BORE: 1

PROJECT: MANLY

FEB 2022



Project No: 207028.00
BH ID: BH 1
Depth: 0.40 - 5.00 m
Core Box No.: 1/2



0.4-5.00m

BORE: 1

PROJECT: MANLY

FEB 2022



Project No: 207028.00
BH ID: BH 1
Depth: 5.00 - 6.72 m
Core Box No.: 2/2



5.00-6.72m

BOREHOLE LOG

CLIENT: Reddall Street Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 29, 31, 35 Reddall St, Manly

SURFACE LEVEL: 21.3 AHD
EASTING: 341942
NORTHING: 6258508
DIP/AZIMUTH: 90°/-

BORE No: BH2
PROJECT No: 207028.00
DATE: 29/11/2021
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Degree of Weathering					Graphic Log	Rock Strength					Water	Fracture Spacing (m)	Discontinuities		Sampling & In Situ Testing			
			EW	HW	MW	SW	FS		FR	Ex Low	Very Low	Low	Medium			High	Very High	Ex High	B - Bedding S - Shear	J - Joint F - Fault	Type
21	0.3	TOPSOIL/Silty SAND: fine to medium, grey, approximately 30% silt, apparently loose, moist																A			
1		SANDSTONE: medium to coarse grained, pale grey and brown, crossbedded at 10°-20°, medium and high strength, slightly weathered and fresh, slightly fractured and unbroken, Hawkesbury Sandstone																C	100	100	PL(A) = 0.3
20																					PL(A) = 0.6
2																					PL(A) = 0.6
19																					
3																		C	100	78	PL(A) = 0.5
18																					
4																					PL(A) = 1.2
17																		C	100	98	
5																					PL(A) = 1.4
16																					
6																		C	100	98	PL(A) = 0.8
15																					
6.57		Bore discontinued at 6.57m Target depth reached																			
7																					
14																					
8																					
13																					
9																					
12																					

RIG: Proline

DRILLER: Tightsite

LOGGED: SI

CASING: HQ to 0.3m

TYPE OF BORING: Hand Auger to 0.3m, NMLC Coring to 6.57m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS:

SAMPLING & IN SITU TESTING LEGEND

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

BORE: 2

PROJECT: MANLY

FEB 2022



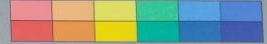
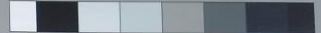
Douglas Partners
Geotechnics | Environment | Groundwater

Project No: 207028-00

BH ID: BH 2

Depth: 0.30 - 5.00 m

Core Box No.: 1/2



MANLY 207028-00 BH2
29-11-21 START: 0.3 M

10

20

30

40

M

0.3-5.00m

BORE: 2

PROJECT: MANLY

FEB 2022



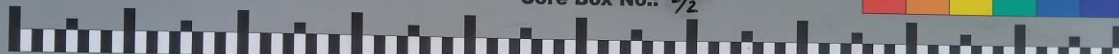
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Project No: 207028-00

BH ID: BH 2

Depth: 5.00 - 6.57 m

Core Box No.: 2/2



50

60

M

BH2 END @ 6.57m

5.00-6.57m

BOREHOLE LOG

CLIENT: Reddall Street Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 29, 31, 35 Reddall St, Manly

SURFACE LEVEL: 15.2 AHD
EASTING: 341967
NORTHING: 6258521
DIP/AZIMUTH: 90°/-

BORE No: BH3
PROJECT No: 207028.00
DATE: 30/11/2021
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Degree of Weathering					Graphic Log	Rock Strength					Water	Fracture Spacing (m)	Discontinuities		Sampling & In Situ Testing			
			EW	HW	MW	SW	FS		FR	Ex Low	Very Low	Low	Medium			High	Very High	Ex High	B - Bedding S - Shear	J - Joint F - Fault	Type
15	0.3	TOPSOIL/Silty SAND: fine to medium, dark grey, approximately 30% silt, trace organics and grass roots, moist																A			
	0.5																	A			
	1	SAND: medium grained, pale grey brown, apparently medium dense, moist, residual																			PL(A) = 0.6
14	1.4	SANDSTONE: medium to coarse grained, pale grey and brown, crossbedded at 10°-25°, medium and high strength, slightly weathered, slightly fractured, Hawkesbury Sandstone																C	100		PL(A) = 0.8
13	2																				PL(A) = 1
	3																	C	94		PL(A) = 0.9
	3.33																				
	3.94	LAMINITE: 80% pale grey and grey siltstone laminated with 20% sandstone, very low and low strength, highly weathered, slightly fractured, Hawkesbury Sandstone																C	85		PL(A) = 0.1
	4.73																				
5	5																	C	100		PL(A) = 0.2
6	5.98	SANDSTONE: fine to medium then medium grained, laminated, very low to low then medium strength, highly than slightly weathered, slightly fractured, Hawkesbury Sandstone																			PL(A) = 0.1
9	9																				
7	7																	C	100	100	
8	7.8	Bore discontinued at 7.8m Target depth reached																			PL(A) = 0.9
8	8																				
9	9																				

RIG: Proline

DRILLER: Tightsite

LOGGED: SI

CASING: HQ to 0.5m

TYPE OF BORING: Hand Auger to 0.5m, NMLC Coring to 7.8m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Standing water level at 2.7m (20 hours after drilling)

SAMPLING & IN SITU TESTING LEGEND

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

BORE: 3

PROJECT: MANLY

FEB 2022



Project No: 207028.00
BH ID: BH3
Depth: 0.50 - 5.00 m
Core Box No.: 1/2



0.5-5.00m

BORE: 3

PROJECT: MANLY

FEB 2022



Project No: 207028.00
BH ID: BH3
Depth: 5.00 - 7.80 m
Core Box No.: 2/2



5.00-7.80m

BOREHOLE LOG

CLIENT: Reddall Street Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 29, 31, 35 Reddall St, Manly

SURFACE LEVEL: 21 AHD
EASTING: 341962
NORTHING: 6258478
DIP/AZIMUTH: 90°/-

BORE No: BH4
PROJECT No: 207028.00
DATE: 1/12/2021
SHEET 1 OF 1

RL	Depth (m)	Description of Strata	Degree of Weathering					Graphic Log	Rock Strength					Water	Fracture Spacing (m)	Discontinuities	Sampling & In Situ Testing				
			EW	HW	MW	SW	FS		FR	Ex Low	Very Low	Low	Medium			High	Very High	Ex High	B - Bedding S - Shear	J - Joint F - Fault	Type
21	0.2	FILL/TOPSOIL/Silty SAND: fine to medium, grey to dark grey, >=30% silt, trace organics and grass rootlets																			
	0.5	FILL/SAND: medium grained, pale brown, trace sandstone boulders, moist																			
20	1.0	FILL/SAND: fine to medium, dark grey, trace silt and plant roots, apparently medium dense, moist																			
	1.35	SANDSTONE: medium grained, pale grey, very low strength, Hawkesbury Sandstone																			
	1.74	SANDSTONE: medium to coarse grained, pale grey brown, low then medium strength, slightly weathered, slightly fractured, Hawkesbury Sandstone																			
19	2																				
	2.25																				
	2.64																				
	2.75																				
18	3																				
	3.3																				
	3.40-3.80																				
17	4																				
	4.20-4.50																				
16	5																				
	5.07-5.10																				
15	6	Bore discontinued at 6.0m Target depth reached																			
14	7																				
13	8																				
12	9																				

RIG: Proline

DRILLER: Tightsite

LOGGED: SI

CASING: HQ to 1.25m

TYPE OF BORING: Hand Auger to 1.2m, NMLC Coring to 6.0m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS:

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

BORE: 4

PROJECT: MANLY

FEB 2022



Project No: 207028.00
BH ID: BH 4
Depth: 1.25 - 6.00 m
Core Box No.: 1/1



1.25-6.00m

BOREHOLE LOG

CLIENT: Reddall Street Pty Ltd
PROJECT: Proposed Residential Development
LOCATION: 29, 31, 35 Reddall St, Manly

SURFACE LEVEL: 15.2 AHD
EASTING: 341972
NORTHING: 6258507
DIP/AZIMUTH: 90°/--

BORE No: BH5
PROJECT No: 207028.00
DATE: 30/11/2021
SHEET 1 OF 1

[illegible]

RIG: Proline

DRILLER: Tightsite

LOGGED: SI

CASING: HQ to 1.21m

TYPE OF BORING: Hand Auger to 1.21m. NMLC Coring to 7.0m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS:

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



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BORE: 5

PROJECT: MANLY

FEB 2022



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Project No: 207028-00

BH ID: BH 5

Depth: 1.25 - 5.00 m

Core Box No.: 1/2



1.25-5.00m

BORE: 5

PROJECT: MANLY

FEB 2022



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Project No: 207028-00

BH ID: BH 5

Depth: 5.00 - 7.00 m

Core Box No.: 2/2



5.00-7.00m

Results of Dynamic Penetrometer Tests

Client Reddall Street Pty Ltd

Project No. 207028.00

Project Proposed Residential Development

Date 1/12/2021

Location 29, 31, 35 Reddall St, Manly

Page No. 1 of 1

Test	DCP2	DCP3	DCP4	DCP5						
Surface Reduced Level (m, AHD)										
Depth (m)										
0 - 0.15	2	1	1	1						
0.15 - 0.30	20/150	1	1	0						
0.30 - 0.45	R	15/100	3	1						
0.45 - 0.60		R	3	4						
0.60 - 0.75			3	6						
0.75 - 0.90			4	6						
0.90 - 1.05			6	15						
1.05 - 1.20			20/100	18/15						
1.20 - 1.35			R	R						
1.35 - 1.50										
1.50 - 1.65										
1.65 - 1.80										
1.80 - 1.95										
1.95 - 2.10										
2.10 - 2.25										
2.25 - 2.40										
2.40 - 2.55										
2.55 - 2.70										
2.70 - 2.85										
2.85 - 3.00										
3.00 - 3.15										
3.15 - 3.30										
3.30 - 3.45										
3.45 - 3.60										

Test Method AS 1289.6.3.2, Cone Penetrometer ☒
 AS 1289.6.3.3, Flat End Penetrometer ☐

Tested By SI
Checked By DEM

Remarks R = Refusal, 25/110 indicates 25 blows for 110 mm penetration
 B = Bouncing E = Excavated D = Discontinued



Photo 1: Drilling Bore 1 on 29 Reddall Street



Photo 2: Sandstone bedrock outcrop on 31 Reddall Street



Photo 3 - Drilling Bore 3 on 31 Reddall Street



Photo 4: Sandstone bedrock and leaning boundary wall beside College Street



Photo 5: Sandstone bedrock outcrop at corner of Reddall & College Streets



Photo 6: Seepage across Reddall Street following extended rainfall

All photos taken November 2021

 Douglas Partners <i>Geotechnics Environment Groundwater</i>	CLIENT: Reddall Street Pty Ltd		TITLE: Site Photographs Geotechnical Assessment - Proposed Residential Development 29, 31 and 35 Reddall Street, Manly	PROJECT No:	207028
	OFFICE: Sydney	DRAWN BY: DEM		PLATE No:	1
	SCALE: NA	DATE: 7 Feb 2023		REVISION:	B