Douglas Partners
Geotechnics / Environment / Groundwater

Report on
Geotechnical Investigation

Proposed Santa Sophia Catholic College
Precinct E.5 - Red Gables Road, Box Hill

Prepared for
Catholic Education Diocese of Parramatta

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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.

	Signature	Date
Author		29 March 2019
Reviewer	pp MJT	29 March 2019

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Report on Geotechnical Investigation
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1. Introduction

This geotechnical investigation has been prepared by Douglas Partners Pty Ltd (DP) on behalf of the Catholic Education Diocese of Parramatta c/TSA Management Pty Ltd (the Applicant).

It accompanies an Environmental Impact Statement (EIS) in support of State Significant Development Application (SSD 18_9772) for the new Santa Sophia Catholic College on the corner of Fontana Drive and the future road ‘B’, between Red Gables Road and Fontana Drive, in Box Hill North (the site).

The new school will cater for approximately 1,920 primary and secondary school students, inclusive of a 60 student Catholic Early Learning Centre. The school will have 130 full-time equivalent staff. The proposal seeks consent for approximately 15,000 sqm of floor space across a part five and part six storey building. The building will present as three main hubs connected by terraced courtyards and garden spaces.

The school will include:

- Catholic Early learning centre for 60 students;
- General Learning Spaces for years Kindergarten to 12;
- Community Hub – knowledge centre and cafe;
- Creative Hub – art and applied science;
- Performance Hub – multipurpose hall and music, dance and drama spaces;
- Professional Hub – administrative space;
- Research Hub – science and fitness;
- Associated site landscaping and open space including a fence and sporting facilities;
- Bus drop off from Fontana Drive;
- Pick-up and drop-off zone from future road ‘B’;
- Pedestrian access points from Red Gables Road north, Fontana Drive and future road ‘B’;
- Staff parking for 110 vehicles provided off site in an adjacent location;
- Short term parking for pick up and drop off for Catholic Early Learning Centre from Red Gables Road; and
- Digital and non-digital signage to the school.

The purpose of this geotechnical investigation was to to provide information on subsurface conditions for planning and design of excavations, retaining structures and foundations.

The investigation included the drilling of six boreholes, the installation and monitoring of two groundwater wells and laboratory testing of selected samples. The details of the field work are presented in this report, together with comments and recommendations on the issues listed above.

2. Background

JK Geotechnics Pty Ltd have previously carried out a geotechnical investigation for the Proposed Gables Town Centre Development, which includes this site, with the results detailed in a report dated 29 March 2018 (Report No. 31134P(TC)rpt) Revision 1. This report has been reviewed and relevant results have been included within this report.

3. Site Description

The site is located on the northern side of Red Gables Road, Box Hill, approximately mid-way between Boundary and Janpieter Roads. It is an irregular shaped area of 10,000 m^2 with maximum north-south and east – west dimensions of approximately 350 m and 330 m respectively.

The site is covered by grassed paddocks that appear to have been previously used for agricultural purposes. An irrigation system is set-up over the area in an attempt to dewater dams to the north of the site.

The site is located at the crest of a hill with site levels falling to the north and west from this crest at gradients of up to 5°.

The site is bounded by grassed paddocks on all sides. It is located about 100 m north of Red Gables Road and 200 m to 300 m south of two dams that are located at the bottom of the hill.

4. Regional Geography

Reference to the Penrith 1:100 000 scale Geological Series Sheet indicates that the site is predominantly underlain by Ashfield Shale of the Wianamatta Group of Triassic age. The site is located near Hawkesbury Sandstone which is mapped at lower elevations to the north and east. The boundary of these two geological unites is often marked by a thin (typically less than 6 m) transitional unit known as the Mittagong Formation.

The Ashfield Shale typically comprises dark grey to black shale, siltstone and laminite which weathers to a residual clay profile of medium to high plasticity and is sometimes of significant depth. The Hawkesbury Sandstone comprises massive and cross-bedded quartz sandstone with a few shale interbeds. The Mittagong Formation contains quartz sandstone similar to, but finer grained than the underlying Hawkesbury Sandstone, with common micro-crossbedding and laminations, but rare large scale crossbeds as found in the Hawkesbury Sandstone.

The field work confirmed the presence of the Ashfield Shale overlying the Mittagong Formation and then Hawkesbury Sandstone.

5. Field Work Methods

The field work was undertaken between 10 December 2018 and 12 December 2018 and involved the following:

- A walkover inspection by a senior geotechnical engineer.
- The drilling of six boreholes (Bores 101 to 106), using a truck-mounted drill rigs, to depths of 5.3 m to 7.7 m which is generally to a depth below the bulk excavation level of the development. The boreholes were drilled with solid flight augers and rotary methods to depths of 1.5 m to 3.0 m. Bores were then cased and extended into the underlying bedrock using NMLC coring methods.
- Standard penetration tests (SPTs) carried out at regular depth intervals during auger drilling of the boreholes to assess in situ soil strength and subsoil consistency.
- Sampling of soils to assist in logging and to provide specimens for laboratory testing.
- Installation and subsequent monitoring of a groundwater monitoring well in Bores 103 and 105. Water within the standpipe was bailed out (or purged) shortly after installation (12 December 2018). Measurement of the groundwater level was carried out on 19 December 2018, 16 and 18 January 2019.

The ground surface levels were determined by survey using a differential global positioning system (DGPS) accurate to 0.1 m. The borehole locations are shown on Drawing 1 in Appendix B.

6. Field Work Results

The detailed borehole logs are provided in Appendix C. Notes defining classification methods and terms used to describe the soils and rocks are provided in Appendix C. The subsurface conditions encountered on the site can be described as:

- TOPSOIL: - typically silty clay or clayey silt with some vegetation and rootlets to depths ranging between 50 mm and 150 mm; overlying,
- NATURAL CLAY: - Stiff to hard, orange and red brown mottled grey silty clay with some ironstone gravel to depths of 0.8 m to 2.8 m; overlying,
- WEATHERED SHALE: - extremely low to very low strength, extremely to moderately weathered, highly fractured to fractured, grey shale with some ironstone banding in all boreholes; overlying,
- SHALE - Low and medium strength, highly to moderately weathered, fractured, grey shale with some high strength ironstone banding and extremely weathered seams in all boreholes except Boreholes 103; overlying,

- LAMINITE - Low, medium and high strength, moderately weathered to slightly weathered, highly fractured to slightly fractured, grey and dark grey interbedded shale and fine to medium grained sandstone with some ironstone banding. This layer is typically less than 1.5 m thick; overlying,

- SANDSTONE - Medium and high strength, slightly weathered to fresh, slightly fractured, yellow brown, light grey and grey, medium and coarse grained sandstone with trace shale laminations

A summary of the depths and reduced levels of the various strata is provided in Table 1.

Table 1: Summary of Material Strata Levels and Rock Classifications

Bore No.	Surface RL	Top of Natural Clays		Top of EL to VL Strength Shale		Top of L and M Strength Shale		Top of L, M and H Strength Shale and Laminite		Top of M and H Strength Sandstone	
		D (m)	RL (m)	D (m)	RL (m)	D (m)	RL (m)	D (m)	RL (m)	D (m)	RL (m)
101	40.1	0.08	40.0	1.4	38.7	4.0	36.1	5.0	35.1	5.8	34.3
102	40.1	0.1	40.0	0.8	39.3	3.1	37.0	3.5	36.1	5.1	35.1
103	37.5	0.1	37.4	1.4	36.1	-	-	1.8	35.7	3.2	34.3
104	38.4	0.1	38.3	1.1	37.3	2.5	35.9	3.1	35.3	3.6	34.8
105	39.3	0.1	39.2	0.7	38.6	3.0	36.4	3.0	36.4	4.5	34.9
106	40.8	0.1	40.7	2.8	38.0	3.3	37.5	5.6	35.2	6.1	34.7

Note: D = Depth below ground surface level
RL = Reduced Level in m relative to Australian Height Datum
EL = Extremely Low, VL = Very Low, L = Low, M = Medium and H = High

No free groundwater was encountered during auger drilling. The use of water during rotary and NMLC coring precluded the measurement of groundwater. Backfilling of the boreholes at the completion of drilling also precluded long-term monitoring of the groundwater levels.

The results of groundwater level measurements in the standpipes are summarised in Table 2.

Table 2: Results of Groundwater Level Measurements in Standpipes

Test Borehole Location	Surface RL (mAHD)	Standpipe Measurements – Water Level					
		19 December 2018		16 January 2019		18 January 2019	
		Depth (m)	RL (mAHD)	Depth (m)	RL (mAHD)	Depth (m)	RL (mAHD)
BH103	37.5	1.5	37.0	1.5	37.0	1.5	37.0
BH105	39.3	0.5	38.8	0.7	38.6	0.7	38.6

It is noted that irrigation of the paddocks occurred during the period of groundwater monitoring.

7. Laboratory Testing

7.1 Aggressivity Testing

Selected samples collected from the boreholes were tested in the laboratory to determine the pH, sulfate and chloride ion concentrations as well as the electrical conductivity to assess the aggressivity potential of the soil. The detailed results are given in Appendix D and are summarised in Table 3 below.

Table 3: Results of Chemical Testing

Bore	Material	Sample Depth (m)	pH	Chloride Ion (mg/kg)	Sulfate Ion (mg/kg)	Electrical Conductivity (µS/cm)
BH101	Filling	1.0	4.6	54	34	100
BH103	Silty Clay	1.0	4.8	63	58	100
BH106	Silty Clay	1.0	5.4	20	51	59

Note: All samples mixed at a ratio of 1(soil):5(water) prior to testing.

The results of aggressivity testing, when compared with Tables 6.4.2 (C) and 6.5.2 (C) in AS 2159-2009 “Piling: Design and Installation”, indicates that an exposure classification of ‘mildly aggressive’ is appropriate for subsurface concrete elements and ‘non-aggressive’ is appropriate of buried steel elements (e.g. pipes).

7.2 Rock Strength Classification

Point Load Strength Index (Is_{50}) testing was carried out on selected rock core specimens. The results of the tests are given on the borehole logs at the appropriate depths. Figure 1 below shows the range of Is_{50} results at the various depths (shown as Reduced Levels relative to AHD).

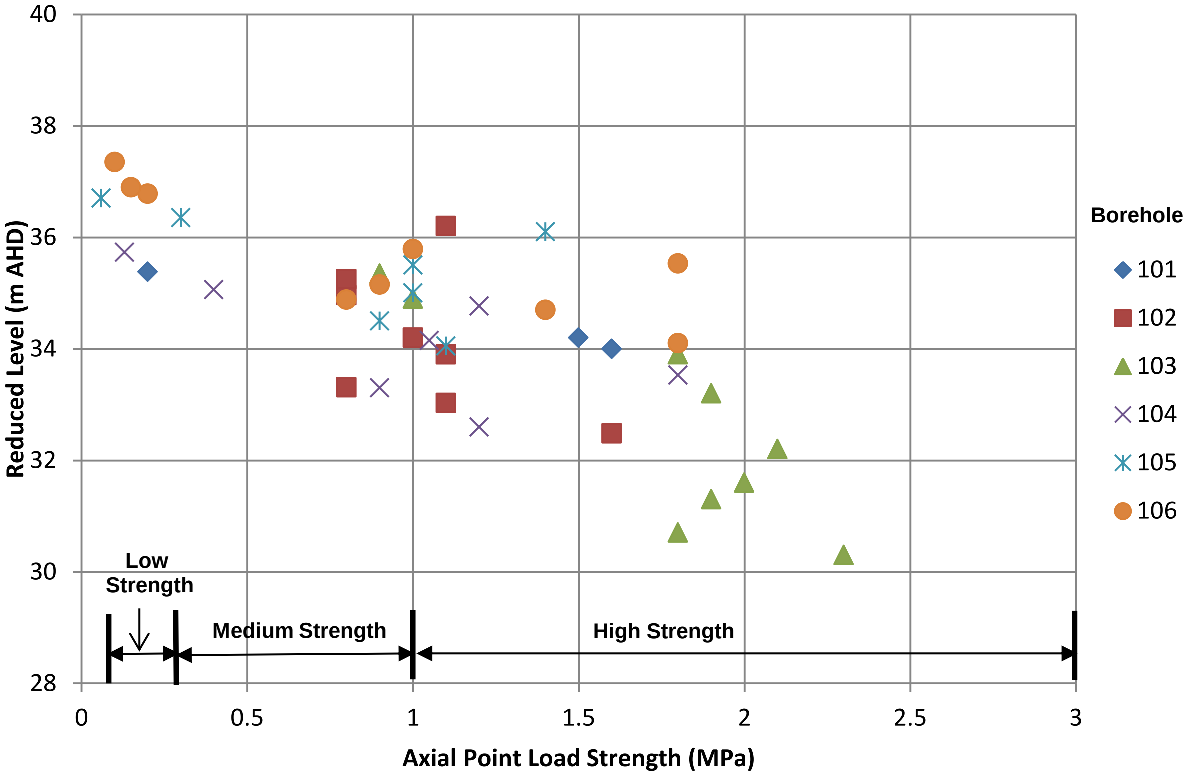

Figure 1: Results of Axial Point Load Tests

8. Proposed Development

It is understood that the site is being considered for the construction of a proposed college. At the time of this report, details were not available to DP of the proposed building and facilities layout; however, the following is anticipated:

- Finished Floor Levels will be at RL 35.3 on the lower northern side of the site and RL 39.3 m on the higher southern side of the site. This will generally require bulk excavation to depths of up to 5 m.
- Several buildings, ranging from two to five storeys will be constructed as part of the school development.
- Recreational areas will be included in the development.

The geotechnical report is required by the Secretary's Environmental Assessment Requirements (SEARs) for SSD 18_9772. Table 4 identifies the relevant SEARs requirement/s and corresponding reference/s within this report.

Table 4: SEARs and Relevant Reference

SEARs Item	Report Reference
Geotechnical Reports	Plans and Documents

9. Comments

9.1 Geotechnical Model

The geotechnical model for the site can be considered to comprise several units as follows, in increasing depth order:

Unit 1	-	A thin layer of topsoil overlying natural clays to depths up to 2.8 m.
Unit 2	-	Extremely low and very low strength, highly weathered shale to depths ranging from 2.3 m to 4.0 m.
Unit 3	-	Ashfield Shale - low and medium strength, highly weathered to slightly weathered, fractured, grey shale. Ashfield Shale is not expected in the lower elevations on-site (Note its absence from Borehole 103).
Unit 4	-	Mittagong Formation – low, medium and high strength laminite (interbedded shale and fine grained sandstone).
Unit 5	-	Hawkesbury Sandstone – medium and high strength, medium to coarse grained sandstone.

Cross-sections showing the ground profile at borehole locations from the investigation and inferred geological unit boundaries between them is provided in Drawings 2 to 4, Appendix B.

The groundwater appears to be at a level within the shale. It is likely that groundwater seepage flows will occur within the upper weathered shale profiles. Groundwater levels are likely to fluctuate, particularly after wet weather.

9.2 Excavation Conditions

Bulk excavation to RL 34.5 m for the proposed basement will encounter Geological Units 1 to 5.

Excavation within the Unit 1 soils and the Unit 2 weathered rock should be readily achievable by bulldozer blade or an excavator with bucket attachment. Some light to medium ripping assistance or the careful use of rock hammers, grinders or rock saws may be required for layers of higher strength ironstone within Unit 3 rock.

Any excavation within Units 3, 4 and 5 will probably require medium to heavy rock breaking equipment. Medium strength rock is expected to have an unconfined compressive strength (UCS) of 6 – 20 MPa, high strength rock is expected to have a UCS of 20 – 60 MPa. Low productivity during excavation should be expected with such materials. Rock breaking equipment will generally cause noise and vibrations that could be disturbing to neighbours.

All excavated materials will need to be disposed in accordance with current EPA policies. Under the Waste Avoidance and Resource Recovery Act (NSW EPA, 2001) a waste/fill receiving site must be satisfied that materials received meet the environmental criteria for proposed land use. This includes filling and virgin excavated natural materials (VENM), such as may be removed from site. 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 is anticipated that there may be some seepage of groundwater into the excavation. Such seepage will need to be collected during construction by the judicious placement of drainage sumps and by intermittent pumping or gravity discharge. At this stage, it is not possible to estimate the likely extent and rate of seepage although it is anticipated from the extent of fracturing in the rock that it should be readily handled by sump and pump measures. It is suggested that monitoring of flow during the early phases of excavation below the groundwater table be undertaken to assess long term drainage requirements.

Noise and vibration will be associated with the excavation of bedrock materials. Further comments regarding vibrations are included in Section 9.4.

9.3 Excavation Support

9.3.1 General

Vertical excavations in Units 1 to 4 will not be stable for any extended period of time due to either the low shear strength of the soils/weathered rock (Units 1 to 2) or the high degree of fracturing of the shale and laminite (Units 3 and 4).

The sidewalls of the basement excavation will therefore require temporary shoring support during excavation and permanent retaining wall support as part of the final construction. The following methods of support are recommended:

- **Temporary Batters (for excavations up to 3 m)** - Temporary batter could be used at the sides of the excavation to a depth of up to 3 m, but will only where space permits. The temporary batters will allow block retaining walls, or similar, to be constructed in front of the batter. Further details regarding batter slopes are provided in Section 9.3.2.
- **Soldier pile/infill panel wall system (for excavations greater than 3 m)** - for excavations greater than 3 m depth, where batters cannot be provided, it is suggested that the excavation be supported by temporary shoring and permanent retaining walls such as a soldier pile/infill panel wall system. The soldier piles would generally be spaced at about 2 m to 3 m centres and should be founded at least two pile diameters below the lowest excavation level (both bulk and detailed) adjacent to the pile location. Soldier piles typically involve either bored piles or continuous flight auger (CFA) piles.

  At the completion of the each excavation lift, reinforced shotcrete infill panels should be constructed. At no stage should progressive vertical excavation proceed beyond 2 m without infill panel support being constructed. It is possible that adverse jointing may give rise to unstable wedges and thus cause localised or even major instability in the exposed material. Regular inspections by a geotechnical professional following each progressive lift of excavation would be prudent to determine if any further stabilisation measures are required.

  Strip drains should be installed behind the shotcrete of the soldier pile/infill panel wall system to facilitate drainage and prevent build-up of water pressures behind the shoring.
- **Continuous pile wall (for excavations greater than 3 m)** – for retaining walls requiring greater stiffness then consideration could be given to installing a continuous pile wall. A continuous pile

wall involves the installation of either bored or CFA piles immediately adjacent to each other to provide a continuous pile wall.

The presence of medium and high strength rock will require a drilling rig with sufficient torque capacity to drill through these layers. The drilling contractor should confirm that the proposed drill rig is of sufficient size and capacity to be able to confidently drill through these medium and high strength layers.

9.3.2 Temporary Batters

During bulk excavation, the maximum unprotected batter slopes in Table 5 are recommended for the temporary battering of internal excavations of up to 3 m depth. Deeper excavation should incorporate benches or flatter batters.

Table 5: Temporary Batter Slopes

Material Description	Batter Slope (H:V)
Natural Soils (Unit 1)	1.5:1
Weathered Rock (Unit 2)	1:1[1]
Low, Medium and High Strength Shale and Laminite (Units 3 and 4)	0.75:1[1]

Note: 1. Subject to geotechnical inspection every 2 m drop of excavation to determine if flatter batters or stabilisation measures are required.

9.3.3 Design of Lateral Support

The design of retaining walls should take due account of both lateral earth pressures and surcharges acting on the walls.

The earth pressure coefficients and bulk unit weights in Table 6 are suggested for the design of a single anchored/propped wall using a triangular pressure distribution.

Table 6: Design Parameters for Retaining Structures

Strata	Earth Pressure Coefficients			
	Bulk Unit Weight, (kN/m^3)	'Active' Permanent K_a	'At Rest' Temporary K_o	Passive*
Residual Soils (Units 1)	20	0.35	0.5	NA
Weathered shale (Unit 2)	22	0.25	0.3	NA
Low, medium and high strength shale and laminite (Units 3 and 4)	23	0.1	0.2	1000 kPa
Medium and High strength sandstone (Unit 5)	24	NA	NA	6000 kPa

Note: *Only applicable below bulk excavation level.

The active earth pressure coefficient, $K_{a,}$ to be used for estimating soil pressures in Table 5 is for a flexible wall allowing some lateral or outward "tilting" movement. Where it is necessary to limit movement, it is suggested that the wall be designed for K_0 (lateral earth pressure coefficients "at rest") conditions in combination with an analytical approach that considers the excavation and propping or anchoring sequence.

The passive pressures provided in Table 5 are ultimate and an appropriate factor of safety should be used to limit movement.

The design for lateral earth pressures for a multi-anchored wall system may be based on a uniform rectangular earth pressure distribution. The following earth pressure distributions are considered appropriate:

- Units 1 and 2 = 4H (where H= height to be retained)
- Units 1 and 2 = 8H (where lateral movements are to be limited)
- Units 3 and 4 = 2H
- Units 3 and 4 = 4H (where lateral movements are to be limited)

The design of temporary and permanent support will need to consider the possibility that 45° joints in the shale and laminite (Units 3 and 4) will daylight near the base of the excavation leading to large wedges of rock requiring support by the temporary and permanent retaining structures. Sufficient anchoring of the shoring wall should be undertaken to prevent movements along 45° joints, even though there is a low probability that a joint would run the full length and height of the excavation. It is suggested that design be carried out such that the support system has a factor of safety of 1.2 against the ultimate sliding force along the most unfavourable 45° joint.

The support system would typically comprise anchors spaced over the rock face. These anchors should have their bond lengths behind the projected 45° line from the bulk excavation level and should provide sufficient force to resist the movement of a wedge of rock projected at 45° from just below the anchor to the ground surface. The frictional resistance of the wedge along the joint may be calculated assuming an angle of friction of 20°. Regular rock-face inspections will be required during excavation to determine whether the assumed factor of safety is adequate. Additional anchors may be required to increase the factor of safety if large wedges are observed during excavation.

Wall design using the parameters given in Table 5 assume the following:

- A level surface behind the top of the excavation;
- Retaining walls will need to allow for hydrostatic pressures from the ground surface level if drainage is not installed or maintained;
- Construction traffic and other surcharge loadings (e.g. stacked materials) are not applied at the crest of the retaining walls, for a distance of say 5 m behind the wall/shoring (otherwise the resultant additional lateral loads need to be considered);
- Passive resistance may be developed in Units 4 or 5 from one pile diameter below the bulk excavation level or below the base of any adjacent localised excavation. The passive pressures calculated are ultimate values to which an appropriate factor of safety (say 3) should be incorporated so as to limit the movement that otherwise is required to develop full passive pressure.

The final or detailed design of retaining walls is normally undertaken using interactive computer programs such as WALLAP, PLAXIS or FLAC, which can take due regard of soil-structure interaction during the progressive stages of wall construction, anchoring and bulk excavation.

9.3.4 Ground Anchors

Temporary ground anchors will be required for the lateral restraint of most boundary shoring walls greater than 3 m height (unless soil nails are used) until such time that the walls are permanently strutted by the building floor slabs. The anchors should preferably have their bond length within weathered (or stronger) rock.

Suggested allowable bond stresses for the design of temporary ground anchors for the support of piled wall systems are given in Table 7.

Table 7: Bond Stresses for Anchor Design

Material Description	Ultimate Bond Stress (kPa)
Weathered shale (Unit 2)	100
Low and medium and high strength shale and laminite (Unit 3 and 4)	300
Medium and High strength sandstone (Unit 5)	1000

Ground anchors should be designed to have a free length that extends beyond an imaginary line drawn upwards at an angle of 45° from the toe of the wall. The minimum free length should be 3 m. After installation, each anchor should be proof loaded to 125 % of the design working load and locked-off at about 80 % of the working load. Periodic checks should be carried out during the construction phase to ensure that the lock-off load is maintained and not lost due to creep effects or other causes. The above parameters are based on the assumption that the anchor holes are clean and thoroughly flushed and that the grouting and other installation procedures carried out carefully and in accordance with normal good anchoring practice. The successful anchoring contractor should be required to demonstrate that design bond values are achievable with the proposed anchor construction methods.

If required, permanent ground anchors will require appropriate corrosion protection, anticipated to include grouting and sheathing, to maintain the integrity of the anchor for its design life.

Approval should be sought from the adjacent property owners, where anchors extend below neighbouring properties, roads or public access areas.

9.4 Vibrations

During excavation it will be necessary to use appropriate methods and equipment to keep ground vibrations within acceptable limits. The standards detailed in the Appendix E are considered appropriate for management of ground vibrations.

Provisional Allowed Vibration Limit

From current information it is considered that the structures adjacent to the site can withstand vibration levels higher than those required to maintain the comfort of their occupants. A human comfort criterion is therefore indicated and the peak particle velocity in any direction i (PPVi), is proposed as the control parameter. It is recommended that a Provisional Allowed Vibration Limit of 8 mm/sec PPVi be set during normal working hours, at foundation level of the potentially affected building/s.

Excavation Plant

DP maintains a database of vibration trial results which can provide guidance for the selection of plant. Trial data is dependent on site conditions and equipment, hence actual vibration levels may differ from predictions and a specific trial is recommended at the commencement of rock excavation. The database suggests that buffer distances within the ranges shown in Table 8 should be maintained between excavation plant and adjacent buildings. These estimates should be examined in relation to the distances between adjacent buildings and the proposed excavation footprint, in order to select suitable plant.

Table 8: Approximate Buffer Distances for Excavation Plant

Excavation Plant	Minimum Buffer Distance	
	(from trial maxima)[1]	(from trial averages)
Provisional Allowed Vibration Limit:	8 mm/s PPVi	
Likely equivalent maximum Vector Sum PPV	11 mm/s VSPPV	
Ripper on 20 t Excavator	2.5 m	0.9 m
Rock Hammer < 500 kg Operating Weight	5.6 m	2.2 m
Rock Hammer 501 – 1000 kg Operating Weight	6.3 m	2.6 m
Rock Hammer 1001 – 2000 kg Operating Weight	9.7 m	4.3 m
Rock Hammer >2000 kg Operating Weight	6.2 m	4.3 m

Note: 1. Smaller distances may be determined from individual trials, as indicated by those from trial averages.

9.5 Foundations

Footing loads for the structure are assumed to be up to 10,000 kN (ultimate) for the multi-storey buildings.

It is anticipated that both the low and medium strength Ashfield Shale (Unit 3), the low, medium and high strength laminite of the Mittagong Formation (Unit 4) and Hawkesbury Sandstone (Unit 5) will generally be exposed at the various bulk excavation levels. It is recommended that all footing loads for each building be transferred to a consistent stratum to achieve uniform founding conditions so as to avoid potential differential settlement across the building. A combination of shallow foundations and piles are therefore recommended over the basement area to uniformly found on the same rock layer.
The design of shallow or pile footings, for axial compression loading, may be based on the maximum Limit State Design or Working Stress parameters given in Table 9.

Table 9: Maximum Foundation Design Parameters

Unit	Serviceability Design Values		Limit State Design Values		Elastic Modulus (MPa)
	Allowable End Bearing Pressure (kPa)	Allowable Shaft Adhesion (kPa)	Ultimate End Bearing Pressure (kPa)	Shaft Adhesion (kPa)	
Low and medium Strength Shale (Unit 3)	1000	100	3000	150	100
Medium and high strength shale and laminite (Unit 4)$^{1\ \&\ 2}$	3500	350	30000	600	1000
Medium and High strength sandstone (Unit 5)^{2}	6000	600	50000	1200	2000

Note:
1. Spoon testing of shallow footing will need to be carried out at least 30% of shallow footings across the site.
2. Increased design parameters may be appropriate following additional investigation of these units.

It should be noted that the serviceability design values" given in Table 8 are based on a 'limiting settlement' of 1 % of the footing width.

The design of footings is usually governed by settlement performance using serviceability rather than the Limit State design values. The Serviceability limit could be assessed, for normal 'static' load cases, using the appropriate elastic modulus value given in Table 9.

Where shallow footings are located close to known sub vertical excavations in rock it may be necessary to downgrade the applied bearing pressure. The entire base should be below an imaginary 'influence line' projected upwards at 45° from the base of the subject excavation. Such situations should be reviewed by the designer on a case-by-case basis.

The foundation design parameters presented in Table 9 assume that the shallow or pile footings are clean at the base and free of loose debris prior to concrete placement.

Over the designated 'socket length' the sidewalls of bored piles should be clean and free of clay 'smear'. Also, the sidewalls should meet the minimum roughness category of "R2" (defined as grooves of 1 to 4 mm depth and width greater than 2 mm, at a spacing of 50 mm to 200 mm) in Pells et.al (1998). A 'grooving' or 'roughening' tool may be required to achieve this criterion.

All footings should be inspected by an experienced geotechnical professional to check the adequacy of the foundation material. Spoon testing of a third of shallow footings founded in Unit 5 sandstone is required.

9.6 Seismic Design

In accordance with Part 4 of the Structural design actions Standard, AS1170.4 – 2007, the site is assessed to have a Site Sub-Soil Class of “C_e”.

9.7 Floor Slabs

The ground floor slab at the lowest level of the basement is expected to be used for carparking and hence will probably only be lightly loaded. The base of the excavation will generally expose bedrock which will provide adequate support for a slab-on-grade. The final surface should be trimmed and scraped clean of debris etc. prior to pouring concrete.

A gravel layer should be provided beneath the floor slab and should slope towards the sump pit to allow sub-floor drainage. Given the high iron content of the shale, seepage is expected to result in the formation of an iron-rich ‘gelatinous’ precipitate over the long-term, that can lead to the blockage of drains and can cause problems for pumps. Adequate provision for access and maintenance of pumps and drains should be incorporated into the design.

9.8 Subgrade Preparation

Where fill is to be place, beneath floor slabs of buildings or recreation areas, the following subgrade preparation measures are recommended:

- Remove all topsoil and filling materials.
- Proof roll the exposed surface using a minimum 12 tonne smooth drum roller in non-vibration mode. The surface should be rolled a minimum of six times with the last two passes observed by an experienced geotechnical engineer to detect any ‘soft spots’.
- Any heaving materials identified during proof rolling should be treated as directed by the geotechnical engineer.
- Any new filling should be placed in layers of 250 mm maximum loose thickness and compacted to a dry density ratio between 98% and 102% relative to Standard compaction with moisture contents maintained within 2% of Standard optimum moisture content.
- Rockfill won from the excavation will general be suitable to re-use as fill up to the subgrade level of floor slabs provided it is broken down to a well-graded material with a maximum particle size of 100 mm. The rockfill won from Units 4 and 5 will generally be more difficult to breakdown than the rockfill won from Units 1 and 3.
- Density testing of the filling should be carried out in accordance with AS3798 “Guidelines for Earthworks for Commercial and Residential Developments”.

Drainage measures should be included within all earthworks operations carried out on site.

10. Limitations

DP has prepared this report for this project at Red Gables Road, Box Hill in accordance with DP's proposal NWS180100 dated 27 November 2018 and acceptance received from Mr Kenny Lim of TSA Management on behalf of the Catholic Education Diocese of Parramatta dated 4 December 2018. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of Catholic Education Diocese of Parramatta, and their agents 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.

This report must be read in conjunction with all of the attached notes 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 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.

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

DP. Any such risk assessment would, however, be necessarily restricted to the geotechnical components set out in this report and to their application by the project designers to project design, construction, maintenance and demolition.

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Appendix A

About This Report

About this Report

Introduction

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

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

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

Borehole and Test Pit Logs

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

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

Groundwater

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

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

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

Reports

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

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

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

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

About this Report

Site Anomalies

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

Information for Contractual Purposes

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

Site Inspection

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

Sampling Methods

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Sampling

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

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

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

Test Pits

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

Large Diameter Augers

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

Continuous Spiral Flight Augers

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

Non-core Rotary Drilling

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

Continuous Core Drilling

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

Standard Penetration Tests

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

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

The test results are reported in the following form.

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

  4,6,7
  N=13

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

  15, 30/40 mm

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

Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests

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

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

Description and Classification Methods

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

Soil Types

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

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

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

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

The proportions of secondary constituents of soils are described as:

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

Definitions of grading terms used are:

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

Cohesive Soils

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

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

Cohesionless Soils

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

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

Soil Origin

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

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

Transported soils may be further subdivided into:

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

Rock Descriptions

Rock Strength

Rock strength is defined by the Point Load Strength Index ($Is_{(50)}$) and refers to the strength of the rock substance and not the strength of the overall rock mass, which may be considerably weaker due to defects. The test procedure is described by Australian Standard 4133.4.1 - 1993. The terms used to describe rock strength are as follows:

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

* Assumes a ratio of 20:1 for UCS to $Is_{(50)}$

Degree of Weathering

The degree of weathering of rock is classified as follows:

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

Degree of Fracturing

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

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

Rock Descriptions

Rock Quality Designation

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

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

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

Stratification Spacing

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

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

Symbols & Abbreviations

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Introduction

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

Drilling or Excavation Methods

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

Water

▷	Water seep
$\underline{\triangledown}$	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

July 2010

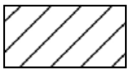
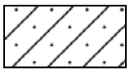
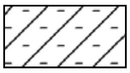
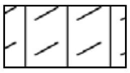
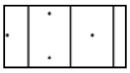
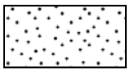
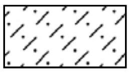
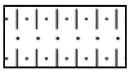
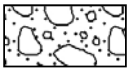
Symbols & Abbreviations

Graphic Symbols for Soil and Rock

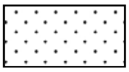
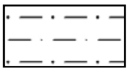
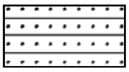
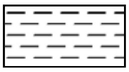
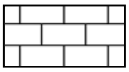
General

■	Asphalt
	Road base
	Concrete
	Filling

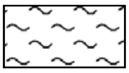
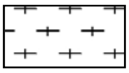
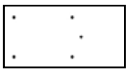
Soils

	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

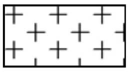
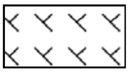
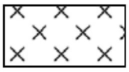
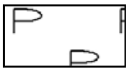
Sedimentary Rocks

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

Metamorphic Rocks

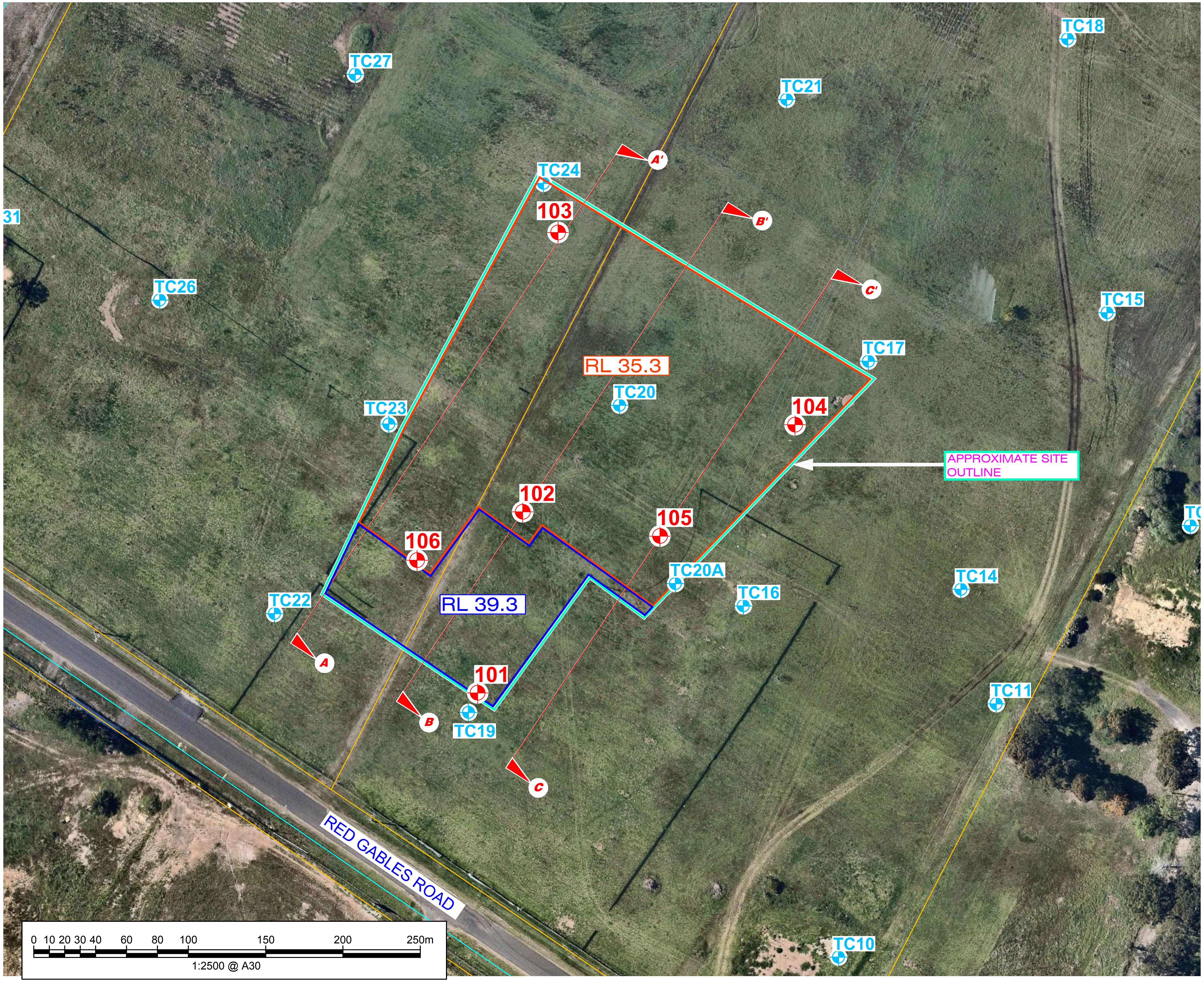
	Slate, phyllite, schist
	Gneiss
	Quartzite

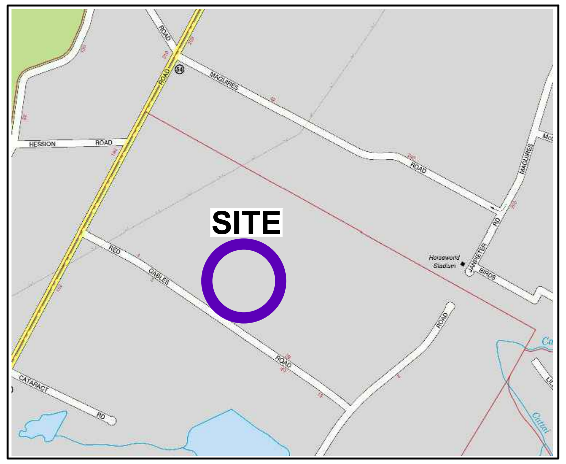
Igneous Rocks

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

Appendix B

Drawings





Locality Plan

NOTE:

1: Base image from Nearmap.com
(Dated 13.11.2018)

2: Test locations are approximate only and
are shown with reference to existing features.

LEGEND

- JK bore location
- DP bore location
- RL 35.3 Finished floor slab level
- A A' Geotechnical Cross Section A-A'

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Catholic Education Diocese of Parramatta

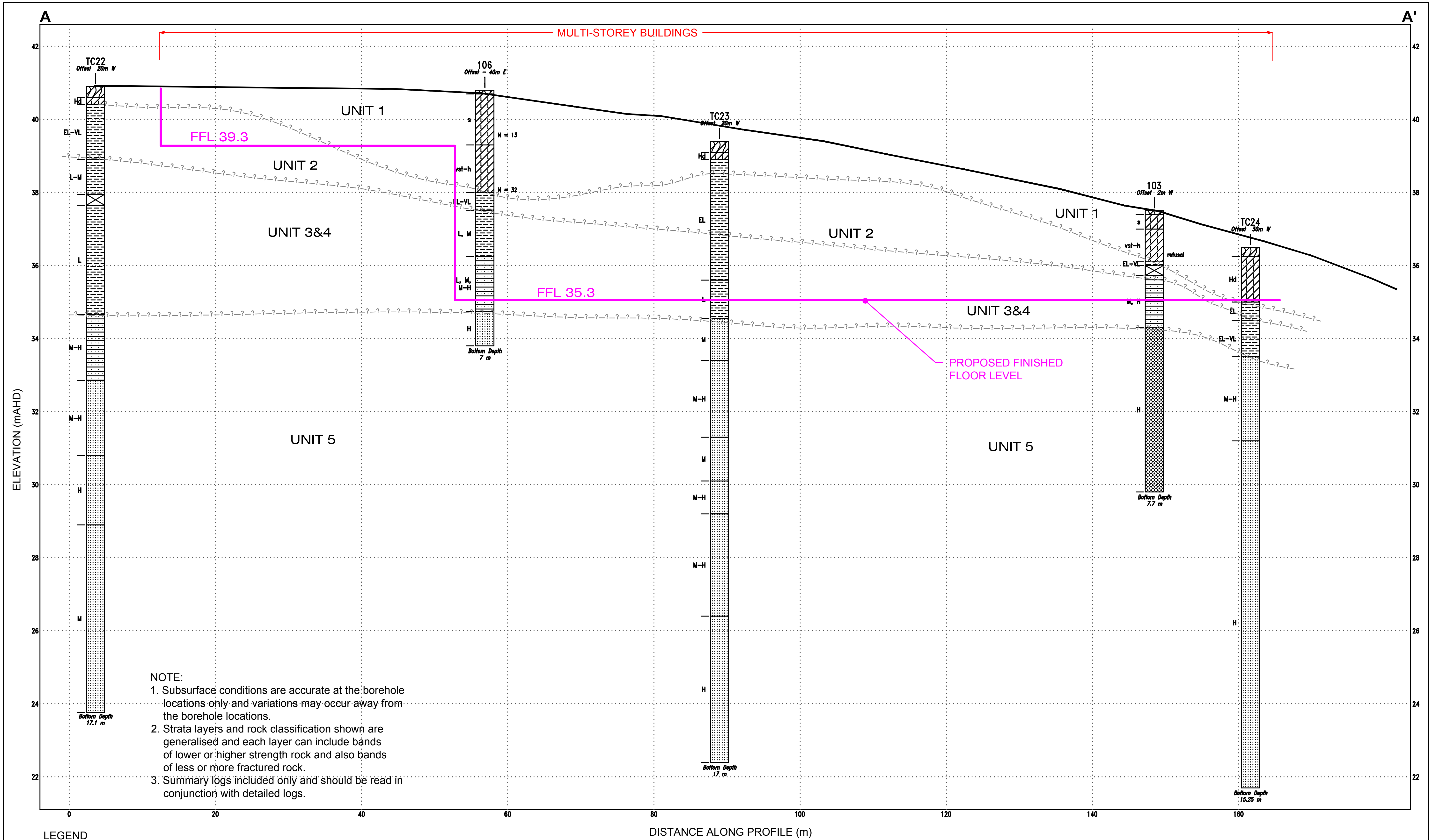
OFFICE: Riverstone
DRAWN BY: PSCH

SCALE: 1:2500 @ A3
DATE: 22.2.2019

TITLE: **Location of Tests Bores**
Proposed Santa Sophia Catholic College
Precinct E.5 - Red Gables Road, BOX HILL

N

PROJECT No: 94526.00
DRAWING No: 1
REVISION: 2



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

LEGEND

☒ Core Loss	▨ Sandstone fine grained
▤ Laminite	▤ Shale
▨ Sandstone	▨ Silty Clay
▩ Sandstone coarse grained	▨ Topsoil

GEOLOGICAL PROFILE UNITS
UNIT 1 - TOPSOIL & NATURAL CLAYS
UNIT 2 - SHALE - Extremely low and very low strength
UNIT 3 - ASHFIELD SHALE - low and medium strength
UNIT 4 - MITTAGONG FORMATION - medium and high strength, fine grained interbedded sandstone and shale
UNIT 5 - HAWKESBURY SANDSTONE - medium and high strength, medium to coarse grained sandstone

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

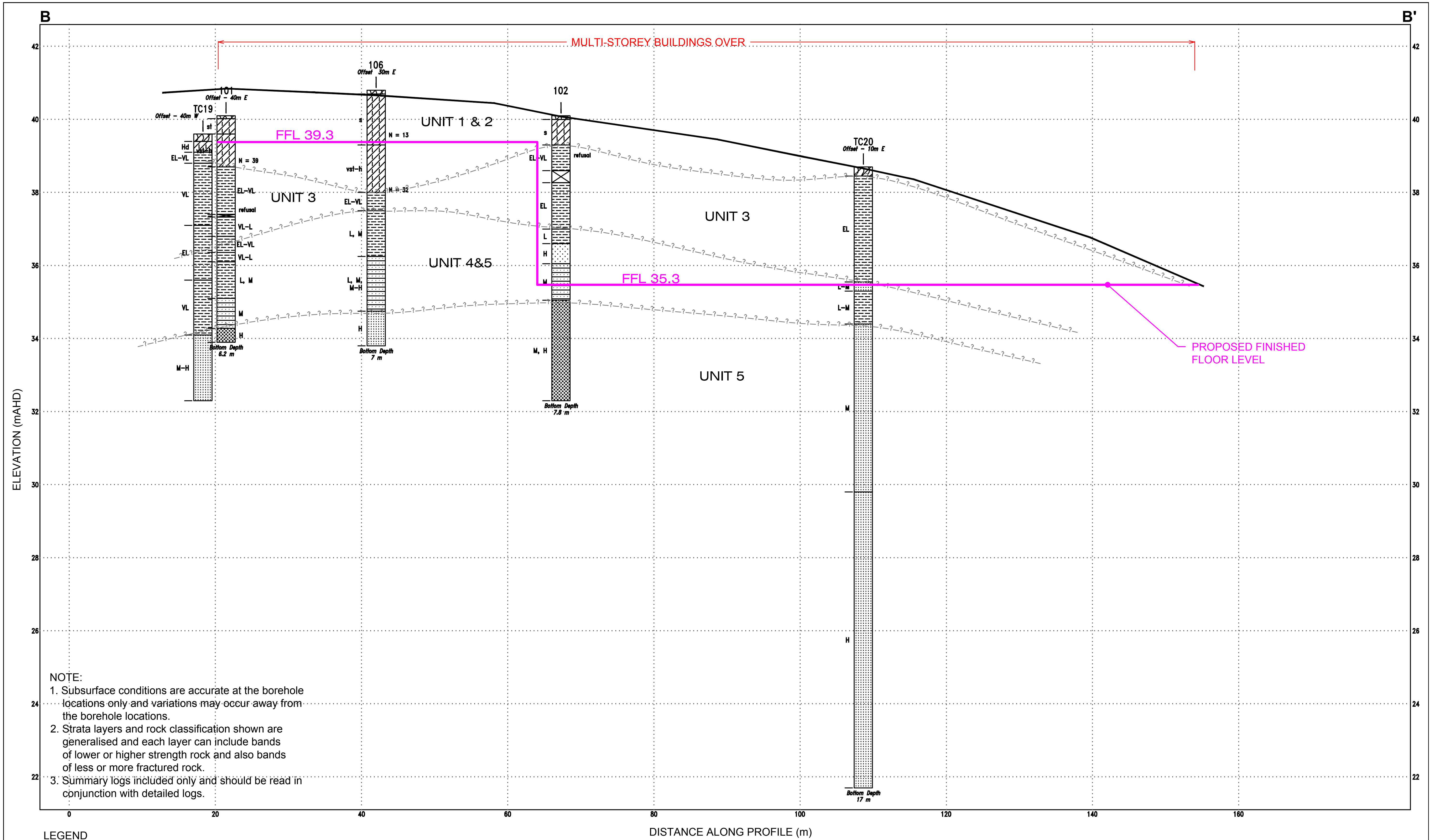
0 10
Horizontal Scale (metres)
Vertical Exaggeration = 5.0

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Catholic Education Diocese of Parramatta
OFFICE: Sydney DRAWN BY: PSCH
SCALE: 1:500 (H) 1:100 (V) @ A3 DATE: 22.02.2019

TITLE: **Inferred Geological Cross-Section A-A'**
Proposed Santa Sophia Catholic College
Precinct E.5 - Red Gables Road, BOX HILL

PROJECT No: 94526.00
DRAWING No: 2
REVISION: 0



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

LEGEND

☒	Core Loss	▨	Sandstone fine grained
▤	Laminite	▤	Shale
▨	Sandstone	▨	Silty Clay
▩	Sandstone coarse grained	▨	Topsoil

GEOLOGICAL PROFILE UNITS
UNIT 1 - TOPSOIL & NATURAL CLAYS
UNIT 2 - SHALE - Extremely low and very low strength
UNIT 3 - ASHFIELD SHALE - low and medium strength
UNIT 4 - MITTAGONG FORMATION - medium and high strength, fine grained interbedded sandstone and shale
UNIT 5 - HAWKESBURY SANDSTONE - medium and high strength, medium to coarse grained sandstone

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

0 10
Horizontal Scale (metres)

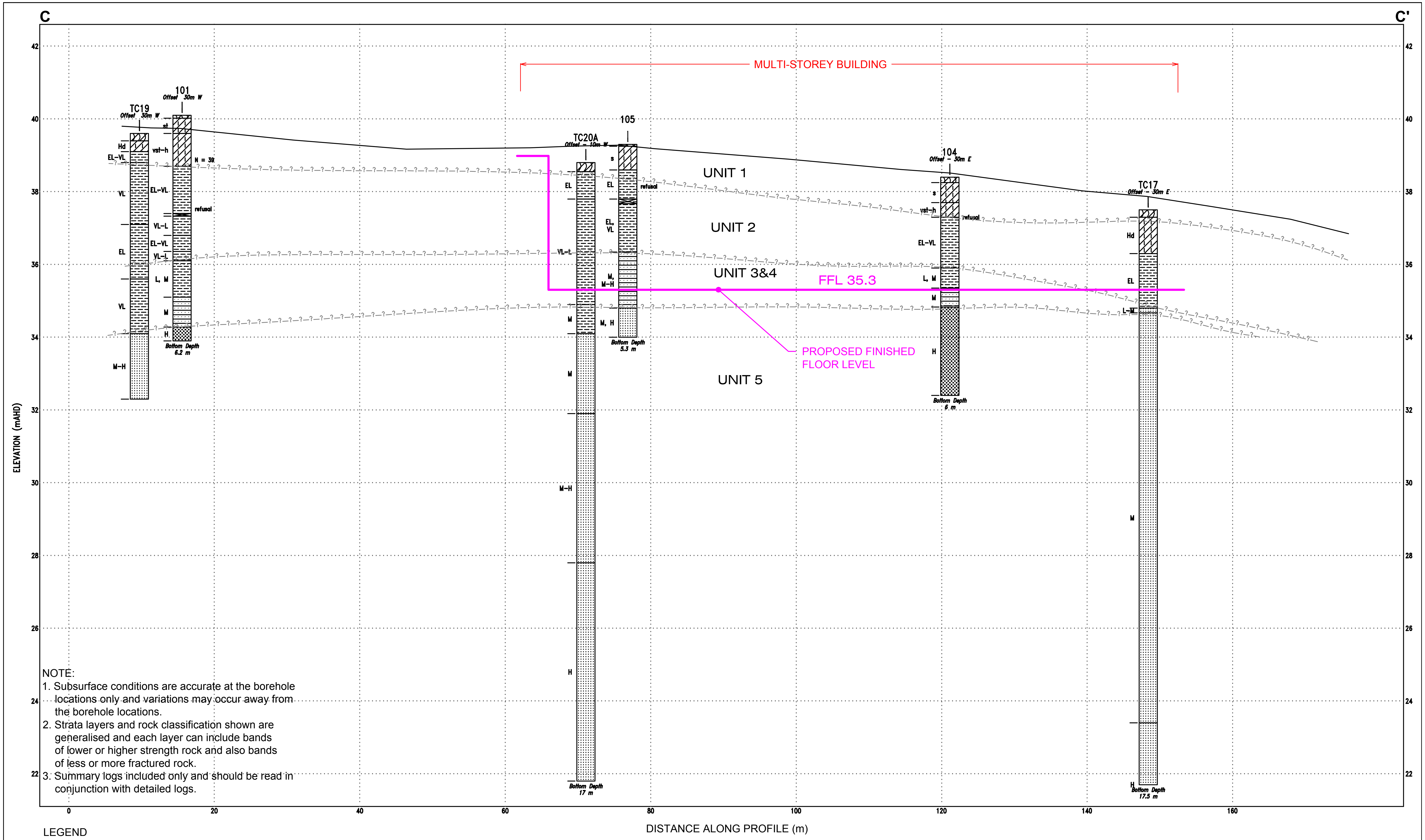
Vertical Exaggeration = 5.0

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Catholic Education Diocese of Parramatta
OFFICE: Sydney
DRAWN BY: PSCH
SCALE: 1:500 (H) 1:100 (V) @ A3
DATE: 22.02.2019

TITLE: **Inferred Geological Cross-Section B-B'**
Proposed Santa Sophia Catholic College
Precinct E.5 - Red Gables Road, BOX HILL

PROJECT No: 94526.00
DRAWING No: 3
REVISION: 0



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

LEGEND

- ☒ Core Loss
- ▤ Laminite
- ▦ Sandstone
- ▩ Sandstone coarse grained
- ▤ Shale
- ▨ Silty Clay
- ▨ Topsoil

GEOLOGICAL PROFILE UNITS

UNIT 1 - TOPSOIL & NATURAL CLAYS
UNIT 2 - SHALE - Extremely low and very low strength
UNIT 3 - ASHFIELD SHALE - low and medium strength
UNIT 4 - MITTAGONG FORMATION - medium and high strength, fine grained interbedded sandstone and shale
UNIT 5 - HAWKESBURY SANDSTONE - medium and high strength, medium to coarse grained sandstone

TESTS / OTHER

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

0 10
Horizontal Scale (metres)
Vertical Exaggeration = 5.0

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Catholic Education Diocese of Parramatta
OFFICE: Sydney
DRAWN BY: PSCH
SCALE: 1:500 (H) 1:100 (V) @ A3
DATE: 22.02.2019

TITLE: **Inferred Geological Cross-Section C - C'**
Proposed Santa Sophia Catholic College
Precinct E.5 - Red Gables Road, BOX HILL

PROJECT No: 94526.00
DRAWING No: 4
REVISION: 0

Appendix C

Results of Field Work

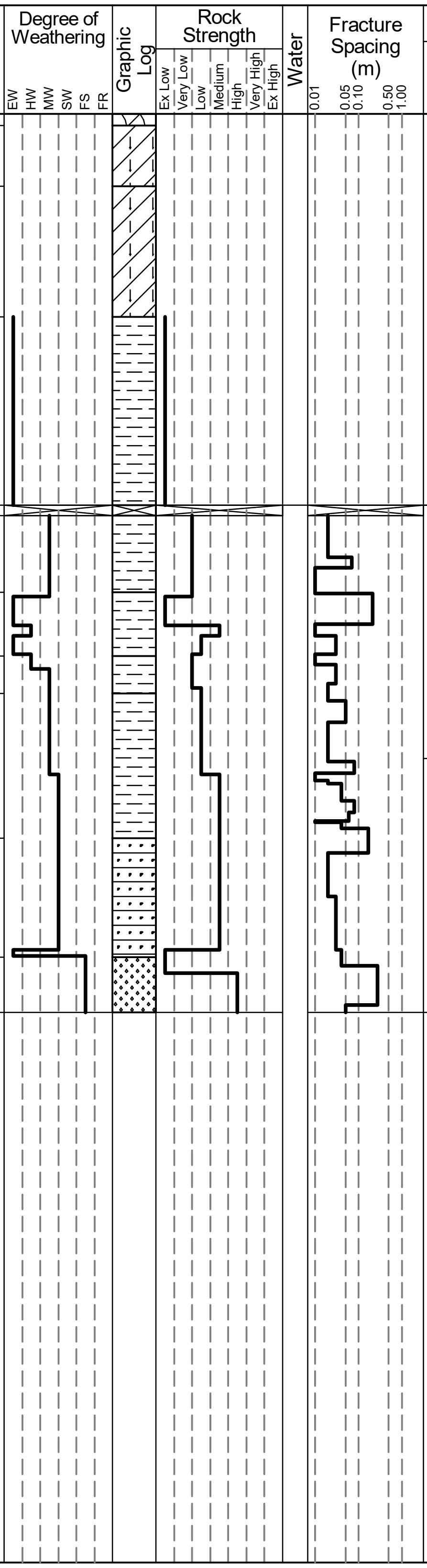
BOREHOLE LOG

CLIENT: Catholic Education Diocese of Parramatta
PROJECT: Proposed Santa Sophia Catholic College
LOCATION: Precinct E.5 - Red Gables Road, Box Hill.

SURFACE LEVEL: 40.1 mAHD
EASTING: 305948.5
NORTHING: 6277437.7
DIP/AZIMUTH: 90°/--

BORE No: 101
PROJECT No: 94526.00
DATE: 10/12/2018
SHEET 1 OF 1

Depth (m)	Description of Strata
0.08	TOPSOIL - brown silty clay with some vegetation, saturated
0.5	SILTY CLAY - stiff, red brown silty clay with some ironstone gravel, moist to wet
1	SILTY CLAY - very stiff to hard, light grey mottled red brown silty clay with some ironstone bands, damp - possibly extremely weathered shale from 1.2 m
1.4	SHALE - extremely low to very low strength, extremely weathered, grey shale with some ironstone banding
2.7 2.77	CORE LOSS
3.3	SHALE - very low to low strength, highly weathered, highly fractured, dark grey shale with some ironstone banding
3.74 4.0	SHALE - extremely low to very low strength, extremely weathered, highly fractured, grey shale with some low and medium strength ironstone gravel
	SHALE - very low strength to low, extremely to moderately weathered, fractured, grey shale with some ironstone gravel
5.0	SHALE - low and medium strength, moderately weathered, fractured, dark grey shale with trace amount of sandstone laminations
5.82	LAMINITE - medium strength, moderately weathered, fractured, dark grey and grey laminite (interbedded shale and sandstone)
6.2	SANDSTONE - high strength, slightly weathered to fresh stained, slightly fractured, yellow brown, medium to coarse grained sandstone with trace amounts of siltstone laminations
	Bore discontinued at 6.2m



Discontinuities

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

Unless otherwise noted, all defects are bedding planes dipping at 0° - 10°

2.7m: CORE LOSS: 70mm

3.07m: J,75°,ro,ir, fe stn

4.23m: J,45°,sm,pl, fe stn

4.45m: J,45°,ro,st, fe stn

5.6m: J,45°,ro,st, fe stn

5.78m: J,40°,ro,st, fe stn

Sampling & In Situ Testing

Type	Core Rec. %	RQD %	Test Results & Comments
A			
A			
A			
S			9,14,25 N = 39
S			30/140 refusal
C	92	0	pp = 400
C	100	22	PL(A) = 0.2 PL(A) = 1.5 PL(A) = 1.6

RL: 40, 39, 38, 37, 36, 35, 34, 33, 32, 31
Depth (m): 1, 2, 3, 4, 5, 6, 7, 8, 9

RIG: Scout **DRILLER:** Kerney-Ellis **LOGGED:** Boyd **CASING:** HW to 2.5 m HQ to 2.7 m
TYPE OF BORING: 100 mm Spiral Flight Auger to 2.5m, Rotary to 2.7 m, NMLC Coring to 6.2m
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_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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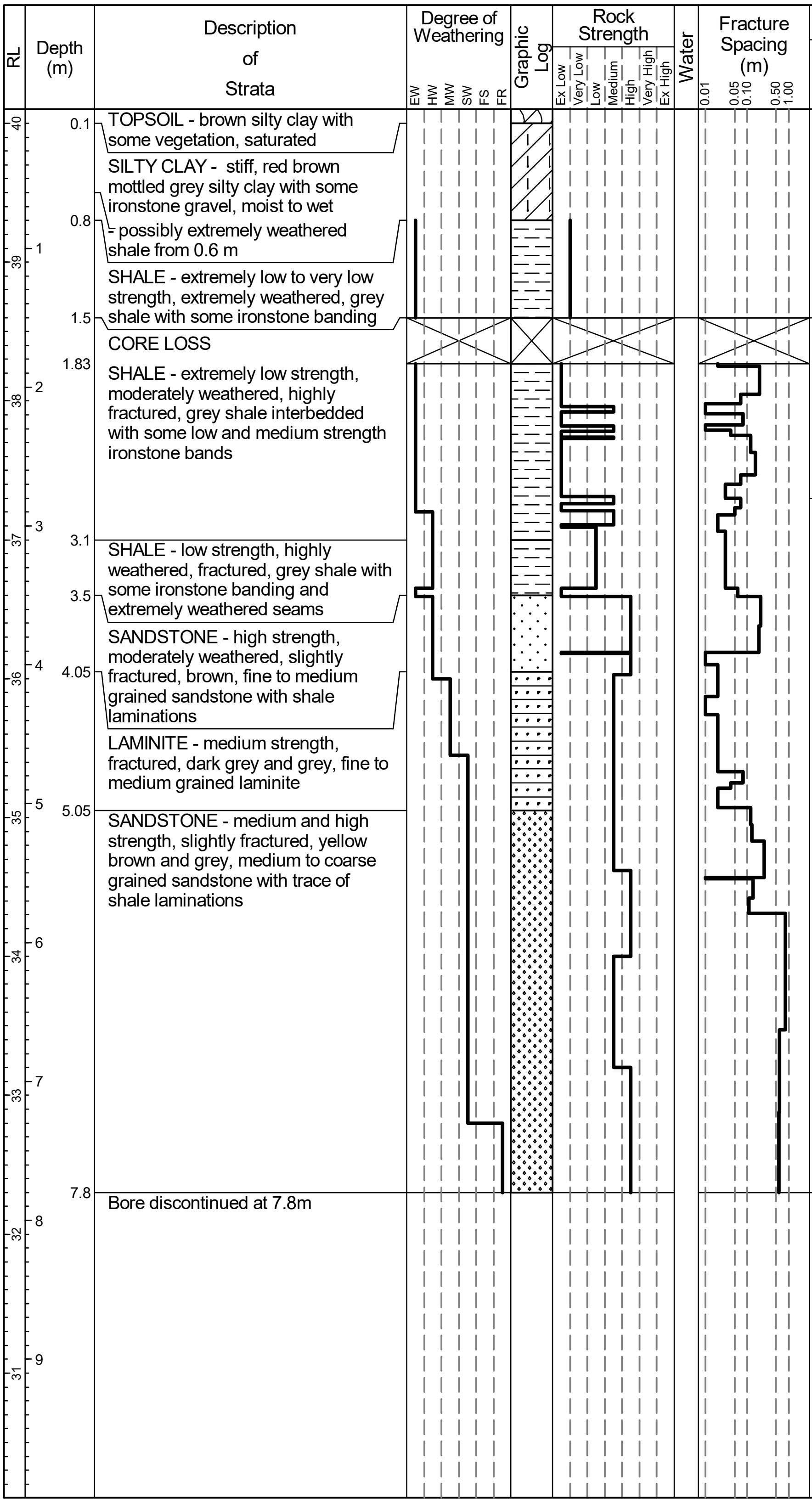
BOREHOLE LOG

CLIENT: Catholic Education Diocese of Parramatta
PROJECT: Proposed Santa Sophia Catholic College
LOCATION: Precinct E.5 - Red Gables Road, Box Hill.

SURFACE LEVEL: 40.1 mAHD
EASTING: 305960
NORTHING: 6277484.7
DIP/AZIMUTH: 90°/--

BORE No: 102
PROJECT No: 94526.00
DATE: 10/12/2018
SHEET 1 OF 1

Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
0.1	TOPSOIL - brown silty clay with some vegetation, saturated		A			
0.8	SILTY CLAY - stiff, red brown mottled grey silty clay with some ironstone gravel, moist to wet - possibly extremely weathered shale from 0.6 m		A A			
1.5	SHALE - extremely low to very low strength, extremely weathered, grey shale with some ironstone banding	Unless otherwise noted, all defects are bedding planes dipping at 0° - 10°	S			30 refusal
1.83	CORE LOSS	1.5m: CORE LOSS: 330mm				
3.1	SHALE - extremely low strength, moderately weathered, highly fractured, grey shale interbedded with some low and medium strength ironstone bands	2.35m: J,45, ro, st, cln 2.7m: J,85,ro, ste, cln 2.8m: J,85,ro, pl, he	C	90	0	pp >400
3.5	SHALE - low strength, highly weathered, fractured, grey shale with some ironstone banding and extremely weathered seams	3.72m: J,85,ro,pl, he	C	100	22	PL(A) = 1.1
4.05	SANDSTONE - high strength, moderately weathered, slightly fractured, brown, fine to medium grained sandstone with shale laminations					
5.05	LAMINITE - medium strength, fractured, dark grey and grey, fine to medium grained laminite					PL(A) = 0.8
7.8	SANDSTONE - medium and high strength, slightly fractured, yellow brown and grey, medium to coarse grained sandstone with trace of shale laminations		C	100	95	PL(A) = 0.8 PL(A) = 1 PL(A) = 1.1 PL(A) = 0.8 PL(A) = 1.1 PL(A) = 1.6
	Bore discontinued at 7.8m					



RIG: Scout **DRILLER:** Kerney-Ellis **LOGGED:** Boyd **CASING:** HW to 1.5 m HQ to 1.5 m
TYPE OF BORING: 100 mm Spiral Flight Auger to 1.5m, NMLC Coring to 7.8m
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_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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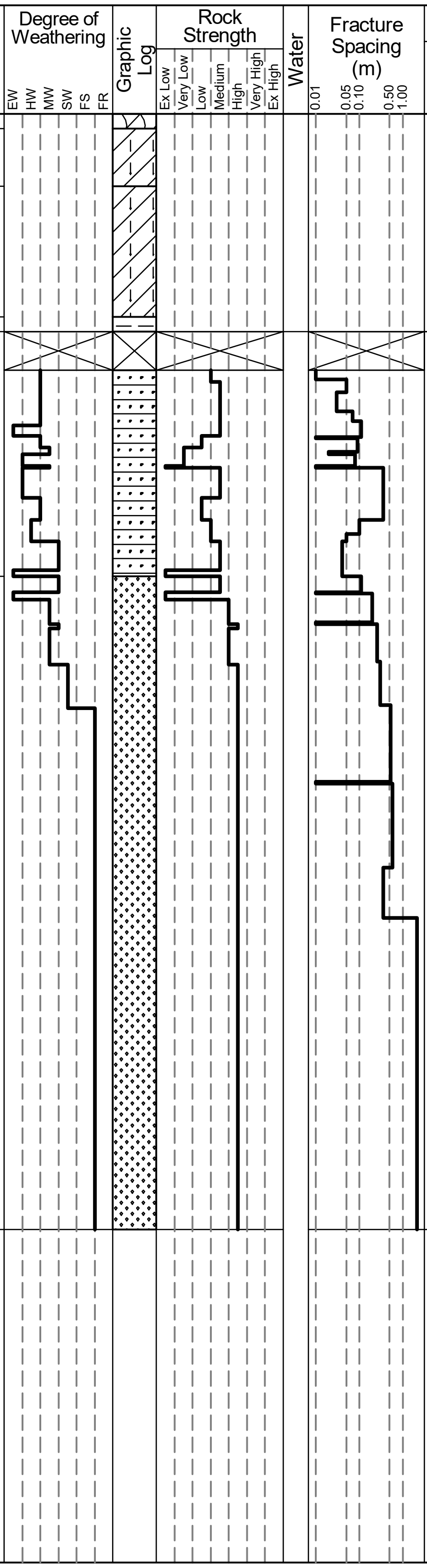
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Catholic Education Diocese of Parramatta
PROJECT: Proposed Santa Sophia Catholic College
LOCATION: Precinct E.5 - Red Gables Road, Box Hill.

SURFACE LEVEL: 37.5 mAHD
EASTING: 305969.1
NORTHING: 6277556.9
DIP/AZIMUTH: 90°/--

BORE No: 103
PROJECT No: 94526.00
DATE: 11/12/2018
SHEET 1 OF 1



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
0.1	TOPSOIL - brown silty clay with some vegetation, saturated		A			
0.5	SILTY CLAY - stiff, red brown silty clay with some ironstone gravel, moist to wet		A			
1.4	SILTY CLAY - very stiff to hard, light grey mottled red brown silty clay with some ironstone bands, damp - possibly extremely weathered shale from 1.2 m	Unless otherwise noted, all defects are bedding planes dipping at 0° - 10°	A S			6,16,30/100 refusal
1.5 1.77	SHALE - extremely low to very low strength, extremely weathered, grey shale with some ironstone banding	1.5m: CORE LOSS: 270mm				
	LAMINITE - medium and high strength, highly to moderately weathered, fractured, moderately weathered, dark grey and grey laminite	2.03m: J,30°,ro, pl, fe stn 2.43m: J,40°,ro, pl, hld 2.8m: J,30°,ro, pl, fe stn	C	87	38	PL(A) = 0.9 PL(A) = 1
3.19	SANDSTONE - high strength, moderately weathered to fresh, slightly fractured, yellow brown and grey, medium to coarse grained sandstone with trace of shale laminations		C	100	90	PL(A) = 1.8 PL(A) = 1.9
			C	100	100	PL(A) = 2.1 PL(A) = 2 PL(A) = 1.9 PL(A) = 1.8 PL(A) = 2.3
7.7	Bore discontinued at 7.7m					

RIG: Scout **DRILLER:** Kerney-Ellis **LOGGED:** Boyd **CASING:** HW to 1.5 m HQ to 1.5 m

TYPE OF BORING: 100 mm Spiral Flight Auger to 1.5m, NMLC Coring to 7.7m

WATER OBSERVATIONS: No free groundwater observed whilst augering

REMARKS: Standpipe installed to 7.7 m. Lower 3 m slotted. Gravel filter. Bentonity plug from 3.7 m to 4.2 m

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

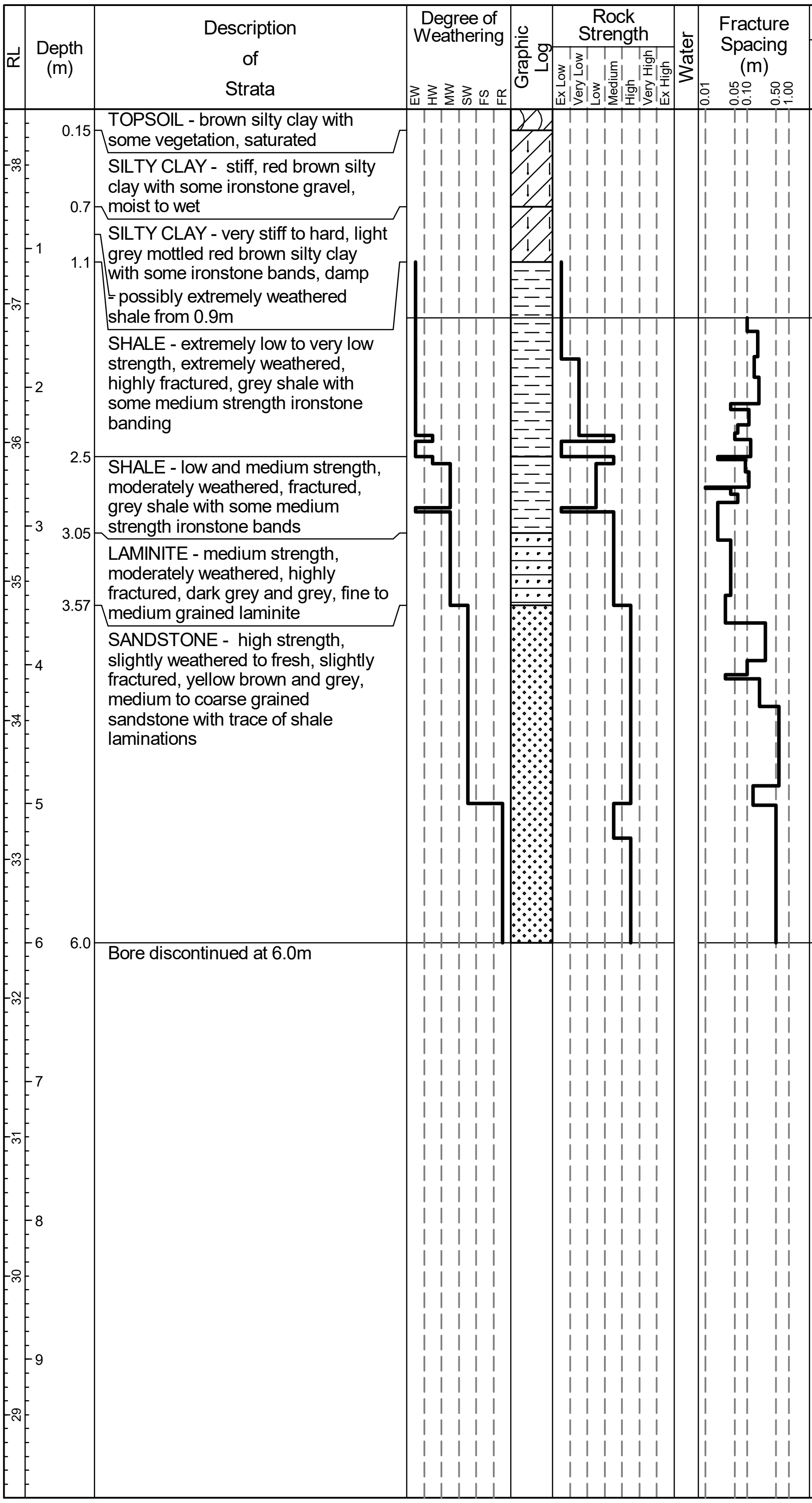
BOREHOLE LOG

CLIENT: Catholic Education Diocese of Parramatta
PROJECT: Proposed Santa Sophia Catholic College
LOCATION: Precinct E.5 - Red Gables Road, Box Hill.

SURFACE LEVEL: 38.4 mAHD
EASTING: 306030.6
NORTHING: 6277507.2
DIP/AZIMUTH: 90°/--

BORE No: 104
PROJECT No: 94526.00
DATE: 11/12/2018
SHEET 1 OF 1

Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
	TOPSOIL - brown silty clay with some vegetation, saturated		A			
0.15	SILTY CLAY - stiff, red brown silty clay with some ironstone gravel, moist to wet		A			
0.7	SILTY CLAY - very stiff to hard, light grey mottled red brown silty clay with some ironstone bands, damp		A			
1.1	- possibly extremely weathered shale from 0.9m	Unless otherwise noted, all defects are bedding planes dipping at 0° - 10°	S			20,30/50 refusal
	SHALE - extremely low to very low strength, extremely weathered, highly fractured, grey shale with some medium strength ironstone banding	2.12m: J,60°,ro, pl, fe stn	C	90	16	pp >400
2.5	SHALE - low and medium strength, moderately weathered, fractured, grey shale with some medium strength ironstone bands	2.62m: J,35°,ro, pl, fe stn				PL(A) = 0.13
3.05	LAMINITE - medium strength, moderately weathered, highly fractured, dark grey and grey, fine to medium grained laminite					PL(A) = 0.4
3.57	SANDSTONE - high strength, slightly weathered to fresh, slightly fractured, yellow brown and grey, medium to coarse grained sandstone with trace of shale laminations	3.55m: J,30°,ro, pl, fe stn				PL(A) = 1.2
			C	100	99	PL(A) = 1.05 PL(A) = 1.8 PL(A) = 0.9 PL(A) = 1.2
6.0	Bore discontinued at 6.0m					



RIG: Scout **DRILLER:** Kerney-Ellis **LOGGED:** Boyd **CASING:** HW to 1.5 m HQ to 1.5 m
TYPE OF BORING: 100 mm Spiral Flight Auger to 1.5m, NMLC Coring to 6.05m
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_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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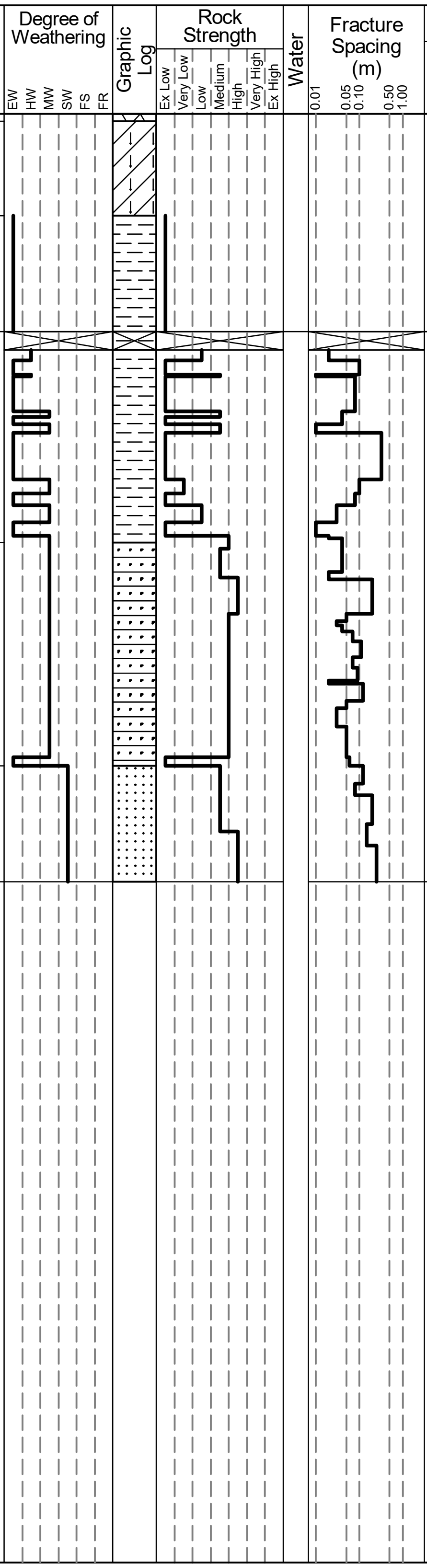
BOREHOLE LOG

CLIENT: Catholic Education Diocese of Parramatta
PROJECT: Proposed Santa Sophia Catholic College
LOCATION: Precinct E.5 - Red Gables Road, Box Hill.

SURFACE LEVEL: 39.3 mAHD
EASTING: 305995.6
NORTHING: 6277478.5
DIP/AZIMUTH: 90°/--

BORE No: 105
PROJECT No: 94526.00
DATE: 12/12/2018
SHEET 1 OF 1

Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
0.05	TOPSOIL - brown silty clay with some vegetation, saturated		A			
0.7	SILTY CLAY - stiff, red brown silty clay with some ironstone gravel, moist to wet - possibly weathered shale from 0.4 m		A A			
1.5	SHALE - extremely low, extremely weathered, grey shale with some very low and low strength ironstone bands	Unless otherwise noted, all defects are bedding planes dipping at 0° - 10°	A S			18,30 refusal
1.63		1.5m: CORE LOSS: 130mm				
2.96	SHALE - extremely low and very low strength, extremely to highly weathered, highly fractured, grey shale with some medium strength ironstone bands		C	91	7	pp = 350 PL(A) = 0.06
4.5	LAMINITE - medium and medium to high strength strength, highly fractured to fractured, dark grey and grey, fine to medium grained laminite with some high strength bands	3.53m: J,45°,ro,st, fe stn	C	100	38	PL(A) = 0.3 PL(A) = 1.4 PL(A) = 1 PL(A) = 1
5.3	SANDSTONE - medium and high strength, slightly fractured, yellow brown and grey, medium to coarse grained sandstone with trace of shale laminations	4.6m: J,25°,ro,pl				PL(A) = 0.9 PL(A) = 1.1
	Bore discontinued at 5.3m					



RIG: Scout **DRILLER:** Kerney-Ellis **LOGGED:** Boyd **CASING:** HW to 1.5 m HQ to 1.5 m
TYPE OF BORING: 100 mm Spiral Flight Auger to 2.5m, NMLC Coring to 5.3m
WATER OBSERVATIONS: No free groundwater observed whilst augering
REMARKS: Standpipr installed to 5.3 m. Lower 3 m slotted. Gravel filter. Bentonity plug from 1.2 m to 1.7 m

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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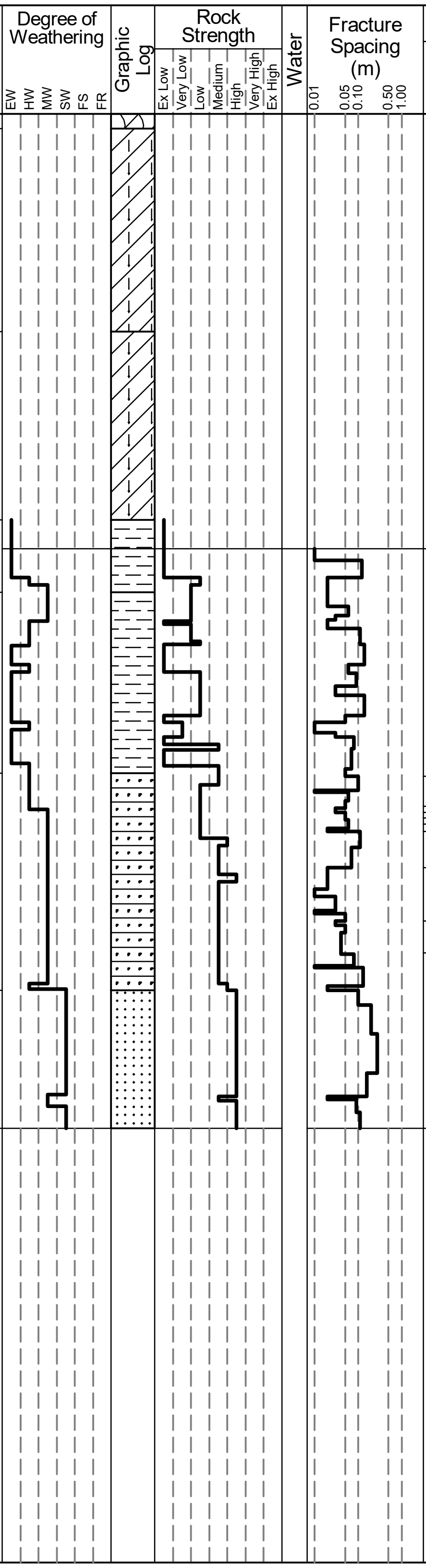
Geotechnics / Environment / Groundwater

BOREHOLE LOG

CLIENT: Catholic Education Diocese of Parramatta
PROJECT: Proposed Santa Sophia Catholic College
LOCATION: Precinct E.5 - Red Gables Road, Box Hill.

SURFACE LEVEL: 40.8 mAHD
EASTING: 305932.7
NORTHING: 6277472.1
DIP/AZIMUTH: 90°/--

BORE No: 106
PROJECT No: 94526.00
DATE: 11 - 12/12/2018
SHEET 1 OF 1



Depth (m)	Description of Strata	Discontinuities B - Bedding J - Joint S - Shear F - Fault	Type	Core Rec. %	RQD %	Test Results & Comments
0.1	TOPSOIL - brown silty clay with some vegetation, saturated		A			
	SILTY CLAY - stiff, red brown silty clay with some ironstone gravel, moist to wet		A A S			6,7,6 N = 13
1.5	SILTY CLAY - very stiff to hard, light grey mottled red brown silty clay with some ironstone bands, damp	Unless otherwise noted, all defects are bedding planes dipping at 0° - 10°	S			5,15,17 N = 32
2.8	SHALE - extremely low to very low strength, extremely weathered, grey shale with some medium strength ironstone banding		C	100	15	pp = 300
3.3	SHALE - low and medium strength, extremely to highly weathered, highly fractured, grey shale with some extremely low strength layers	3.38m: J,40°, ro,st,stn				PL(A) = 0.1 PL(A) = 0.15 PL(A) = 0.2
4.55	LAMINITE - low, medium and medium to high strength, moderately to slightly weathered, highly fractured to fractured, dark grey and grey, fine to medium grained laminite with some high strength bands	4.52m: J,55°, ro,pl,stn 4.57m: J,45°, ro,pl,fe stn 4.78m: J,35°, ro,pl, fe stn 4.82m: J,40°, ro,pl, fe stn 4.86m: J,40°, ro,pl, fe stn 4.9m: J, 60°, ro,pl,fe stn 4.95m: J,85°, ro,st, festn 5.2m: J,75°, ro,st,stn 5.57m: J,30°, ro,st, he 5.79m: J,40°, ro,pl, fe stn				PL(A) = 1 PL(A) = 1.8 PL(A) = 0.9 PL(A) = 0.8
6.05	SANDSTONE - high strength, slightly weathered, slightly fractured, yellow brown and grey, medium to coarse grained sandstone with trace of shale laminations		C	100	55	PL(A) = 1.4 PL(A) = 1.8
7.0	Bore discontinued at 7.0m					

RIG: Scout **DRILLER:** Kerney-Ellis **LOGGED:** Boyd **CASING:** HW to 2.5 HQ to 3.0 m

TYPE OF BORING: 100 mm Spiral Flight Auger to 2.5m, Rotary to 3 m, NMLC Coring to 7.65m

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_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

Appendix D

Laboratory Test Results

Douglas Partners
Geotechnics / Environment / Groundwater

CHAIN OF CUSTODY DESPATCH SHEET

Project No: 94526.00
Suburb: Box Hill
To: ~~Lab name~~ Envirolab Services
12 Ashley St Chatswood NSW 2067
Project Name: Proposed College
Order Number
Project Manager: Gavid Boyd
Sampler: Groundtest
Attn: Aileen Hie
Emails: gavin.boyd@douglaspartners.com.au
Phone: 9910 6200
Date Required: Same day ☐ 24 hours ☒ 48 hours ☐ 72 hours ☐ Standard ☐
Email:
Prior Storage: ☐ Esky ☐ Fridge ☒ Shelved Do samples contain 'potential' HBM? Yes ☐ No ☐ (If YES, then handle, transport and store in accordance with FPM HAZID)

Sample ID	Lab ID	Date Sampled	Sample Type	Container Type	Analytes									Notes/preservation
			S - soil W - water	G - glass P - plastic	~~Heavy Metals~~	~~OCP/OPP PCB~~	~~TRH and BTEX~~	~~PAH~~	~~Total Phenols~~	~~Asbestos 500 ml~~	pH, SO4, Cl, EC			
BH101/1.0m	1	10-1-19	S	P							•			
BH103/1.0m	2	10-1-19	S	P							•			
BH106/1.0m	3	10-1-19	S	P							•			
PQL (S) mg/kg												**ANZECC PQLs req'd for all water analytes ☐**		

ENVIROLAB
Envirolab Services
12 Ashley St
Chatswood NSW 2067
Ph: (02) 9910 6200
Job No: 210388
Date Received: 29/01
Time Received: 16:43
Received by: AZ
Temp: [✓ Cool]/Ambient
Cooling: Ice/[✓ Icepack]
Security: [✓ Intact]/Broken/None

PQL = practical quantitation limit. If none given, default to Laboratory Method Detection Limit
Lab Report/Reference No:
Metals to Analyse: 8HM unless specified here:
Total number of samples in container: 3
Relinquished by: Petrina
Transported to laboratory by:
Send Results to: Douglas Partners Pty Ltd
Address:
Phone:
Fax:
Signed: P.Tukely
Received by: Andy Zhang AZ.
Date & Time: 29/01 16:43

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

CERTIFICATE OF ANALYSIS 210388

Client Details

Client	Douglas Partners Pty Ltd
Attention	Gavin Boyd
Address	96 Hermitage Rd, West Ryde, NSW, 2114

Sample Details

Your Reference	**94526.00, Box Hill**
Number of Samples	3 SOIL
Date samples received	29/01/2019
Date completed instructions received	29/01/2019

Analysis Details

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

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

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

Report Details

Date results requested by	30/01/2019
Date of Issue	30/01/2019

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

Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Jacinta Hurst, Laboratory Manager

Misc Inorg - Soil				
Our Reference		210388-1	210388-2	210388-3
Your Reference	UNITS	BH101	BH103	BH106
Depth		1.0	1.0	1.0
Date Sampled		10/01/2019	11/01/2019	11/01/2019
Type of sample		SOIL	SOIL	SOIL
Date prepared	-	30/01/2019	30/01/2019	30/01/2019
Date analysed	-	30/01/2019	30/01/2019	30/01/2019
pH 1:5 soil:water	pH Units	4.6	4.8	5.4
Electrical Conductivity 1:5 soil:water	µS/cm	100	100	59
Chloride, Cl 1:5 soil:water	mg/kg	54	63	20
Sulphate, SO4 1:5 soil:water	mg/kg	34	58	51

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

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			30/01/2019	1	30/01/2019	30/01/2019		30/01/2019	[NT]
Date analysed	-			30/01/2019	1	30/01/2019	30/01/2019		30/01/2019	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	1	4.6	4.5	2	103	[NT]
Electrical Conductivity 1:5 soil:water	µS/cm	1	Inorg-002	<1	1	100	110	10	104	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	54	[NT]		89	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	34	[NT]		82	[NT]

Result Definitions

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

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.
Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.	

Laboratory Acceptance Criteria

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

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

Spikes for Physical and Aggregate Tests are not applicable.

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

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

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

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

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

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

Measurement Uncertainty estimates are available for most tests upon request.

Appendix E

Ground Vibration Notes

Ground Vibration

Ground vibration can be described by measurement of the acceleration, velocity or displacement of the ground particles at one or more locations. Triaxial geophone sensors for example can measure the peak velocities of radial, transverse or vertical particle motion (designated PPVr, PPVt and PPVz respectively and PPVi for any directional component) within selected sample periods and peak velocities can also be determined in the resultant direction of particle motion, from calculations of instantaneous vector sums throughout the sample period. Vector sum velocities are designated VSPPV, or in many cases simply PPV.

There are three aspects of vibration which need to be assessed:

- Effects on structures
- Effects on architectural finishes
- Effects on humans

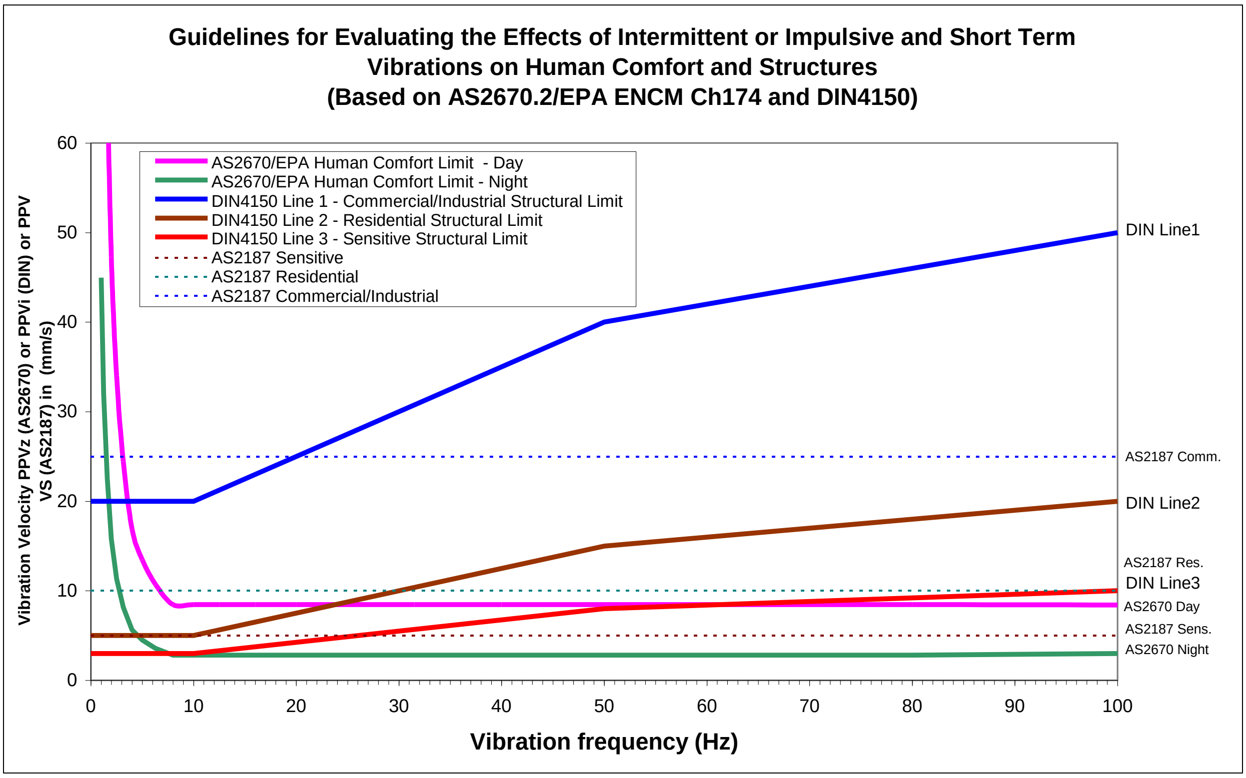
Numerous standards and guidelines exist worldwide which provide a basis for these assessments. Their focus varies from structural damage to human comfort and from transient to intermittent to continuous vibration. Most provide guideline vibration limits for protection against damage or human discomfort, however these limits are not always consistent and application of a particular standard or guideline should be based on the expected type of vibration, the types and conditions of the potentially affected buildings and the potential for discomfort of their occupants.

Both the guideline and the vibration limits should be determined on a case by case basis and the adopted limits (damage and human comfort or the lower of the two) may differ from the guideline values, according to the experience of the vibration consultant, due to the sensitivity of the building or the activities of its occupants. Some applicable guidelines are summarised in the graph on the following page.

Depending on site conditions, proposed works, results of building condition surveys and on-site vibration trials (indicating vibration attenuation rates and dominant vibration frequencies of excavation plant), the standards, guidelines and limits discussed below are considered appropriate for management of ground vibration generated during rock excavation.

Effects on Structures

The German Standard DIN4150-3-1999 “Structural vibration – effects of vibrations on structures”, recommends that ground vibration at foundation level of residential buildings, in good condition bearing on sound rock foundations, be limited to 5 - 15 - 20 mm/s PPVi (at vibration frequencies of 10-50-100 Hz typical of excavation plant), in order to reduce the potential for structural damage. Higher limits (20-40-50 mm/s PPVi) and lower limits (3-8-10 mm/s PPVi) are recommended for commercial/industrial and sensitive buildings respectively. From DP experience where buildings are bearing on loose sand, maximum vibration levels should be significantly reduced to the order of 5 to 7 mm/s VSPPV to reduce the risk of vibration-induced sand densification and settlement.



Effects on Architectural Finishes

It has been found from experience that even with buildings bearing on rock, vibration levels as low as 10 mm/s VSPPV may cause minor defects such as cracks through rendering, cornices and skirtings. Management of vibration may require a lowering of structural damage criteria to this architectural damage criterion, or negotiations with owners of affected buildings.

Effects on Humans

Ground vibration can be strongly perceptible to humans at levels above 2.5 mm/s VSPPV and can be disturbing at levels above 5 mm/s VSPPV. Complaints from residents and building occupants are sometimes received when levels are as low as 1 mm/s VSPPV. The Australian Standard AS2670.2-1990 “Evaluation of human exposure to whole-body vibrations – continuous and shock induced vibrations in buildings (1-80 Hz)” indicates an acceptable day time limit of 8 mm/s PPVz for human comfort. Management of vibration may require a lowering of damage criteria to this human comfort criterion, or negotiations with occupants of affected buildings.

Vibration Dosage

A vibration limit based on a particle velocity allows real time control of excavation using automatic SMS warning systems, or flashing lights attached to vibration monitors. Occasional exceedances (vibration levels exceeding the allowed limit) are not damaging or disturbing and can be allowed but frequent exceedances should be avoided by changes in excavation methods. The difference between occasional and frequent is difficult to gauge on site but can be assessed using recorded vibration data, on the basis of experience or by application of a vibration dosage criterion.

A vibration dosage value (VDV) can be used to assess the effect of intermittent vibration (e.g. from bursts of rock hammering) on humans over a defined period. Acceptable dosages (generally VDVz for vertical vibrations found most disturbing by humans) have been defined for occupants of residential, commercial and industrial buildings ("Assessing Vibration: a technical guideline", Department of Environment and Conservation, 2006). Estimates of VDV (eVDV) can be calculated from recorded vibration data and can be compared with recommended maxima of 0.4, 0.8 and 1.6 $m/s^{1.75}$ for residential, commercial and industrial locations respectively, to assess the need to change excavation methods to restore human comfort.

The vibration dosage guideline does not relate VDV to structural damage however it is considered that if the VDV is acceptable from a human comfort viewpoint, vibration leading to that VDV would be unlikely to cause damage to the corresponding residential, commercial or industrial structure.

Management of vibration may require addition of these vibration dosage criteria to other human comfort or damage criteria, if the frequency of vibration exceedances becomes difficult to assess on site or by experienced-based data review.