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Geotechnical Investigation Report

for

Proposed Redevelopment – Marist College North Shore

at

Marist College North Shore

Prepared for

Sydney Catholic Schools c/o

Carmichael Tompkins Property Group

23 October 2020

Report No: 9625.1-GR-1-1

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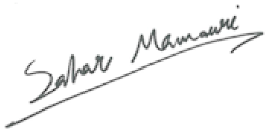
Signature		
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1. INTRODUCTION

This report presents the results of a geotechnical investigation undertaken by Alliance Geotechnical Pty Ltd (AG) for Carmichael Tompkins Property Group (Client) for a proposed development at Marist Catholic College North Shore (the site). The investigation was undertaken in accordance with the scope of works outlined in AG's proposal, Estimate No. 4022, dated 15th September 2020.

AG have previously undertaken a detailed site investigation (DSI) for this site in 2019 (Report No. 9625-ER-1-1, dated 29 November 2019).

Based on the project brief and preliminary architectural drawings provided by CTPG, it is understood that the proposed development comprises the demolition of the existing buildings to be replaced with new school facilities.

The purpose of this report is to provide recommendations regarding:

- Geotechnical subsurface conditions and groundwater;
- Suitable footings system and competent foundation depth;
- Geotechnical design parameters for deep foundations;
- Excavations and vibration management;
- Temporary shoring system and retaining wall design parameters;
- Soil aggressivity in relation to concrete and steel;
- Earthquake site soil class assessment.

In order to achieve the project objectives, the following scope of work was carried out for the geotechnical investigation:

- Review of the geological maps and the provided architectural drawings;
- Site walkover;
- Drilling of five (5) boreholes with two (2) boreholes drilled to a depth of up to 6.0m and three (3) boreholes to a minimum of 5m below the top of bedrock.
- Standard Penetration Tests (SPTs) at 1.5m depth intervals in soil in boreholes;
- Laboratory tests on the recovered soil and rock samples.

1.1. Proposed Development

The client supplied AG with architectural plans pertaining to the proposed development. AG understands the development consists of the following stages of construction:

- **Stage 1** – the year 7/8 project – completed;
- **Stage 2** –Demolition of hall lobby and anteroom, construction of specialist classrooms and Carlow Street main building which is a five-storey building over a partial underground basement;
- **Stage 3** – Precinct works. Demolition Miller Street block, Presbytery to Parish refurbishment, refit Ron Dyer Building, construct Precinct Pavilion and Canteen which are two-storey buildings;

- **Stage 4** – Miller Street Development. Construct new auditorium, function space, childcare centre and commercial lettable space which is a five-storey building over a partial underground basement;
- **Stage 5** – Complete landscaping works on site;
- **Stage 6** – Sports hall and tech rooms. Sports hall re-purpose, Tech and applied science refurb and admin refurb.
- **Stage 7** –Fit out of childcare.

Referring to the preliminary architectural drawings prepped by WMK Architecture, dated 2 September 2020. The proposed five-storey buildings located at the north eastern corner of the site have a single basement level. The basement finished floor level is at RL 81m AHD. The Precinct Pavilion and Canteen are double storey buildings with no underground level.

2. SITE DESCRIPTION AND REGIONAL GEOLOGY

2.1. Site Location and Description

The site is known as Marist College North Shore, located at 270 Miller Street, North Sydney, NSW and occupies an area of 4000 m^2. The school is bound to the north east by Carlow Street, to the east by Miller Street and to the south west by Ridge Street. To the west it is bound by Ridge Street and residential houses. The site location in relation to the surrounding features is shown in Figure 1.

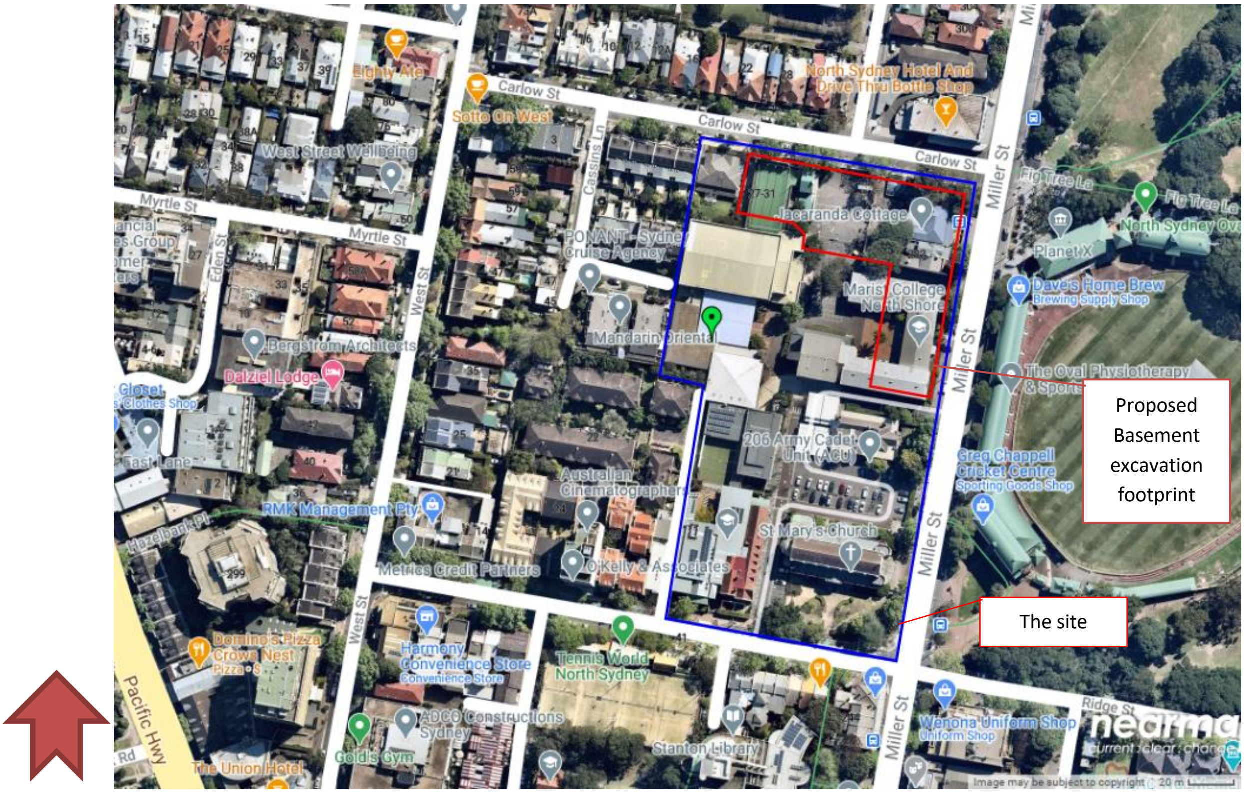

Figure 1- Site Location

At the time of this geotechnical investigation, the site was occupied by two-storey buildings surrounding a central concrete play area. The site slopes gently towards the north, with the local ground surface levels varying between RL 81 m and RL 86 m, relative to Australian height datum (AHD).

The 1:100,000 NSW Department of Mineral Resources Geological Map of the Sydney Region indicates the site is underlain by Ashfield Shale (Rwa). The formation is described as dark-grey to black claystone-siltstone and fine shale-siltstone laminite.

3. GEOTECHNICAL INVESTIGATION

3.1. Methods

This geotechnical site investigation was carried out over two days between 1st October and 2nd October 2020. Selected site photographs taken during the fieldwork are presented in Appendix B.

The investigation comprised the initial scanning of underground utilities and setting out test locations followed by:

- The drilling of five (5) boreholes (BH01 to BH05) up to a maximum depth of 14.6m below the ground surface (bgs);
- Undertaking Standard Penetration Tests (SPTs) in 1.5m depth intervals.

The boreholes were drilled using a drilling rig operated by AG's nominated drilling contractor and were advanced through soil profile by augering with a tungsten carbide (TC) bit. The rock in three boreholes (BH01 to BH03) was recovered by NMLC coring.

The encountered soils were logged by an experienced geotechnical engineer from AG and recovered samples were transported to AG's NATA accredited materials testing laboratory for further testing and storage.

SPTs were undertaken at 1.5m depth intervals in the boreholes to assess the upper soil layers' consistency/density. DCP tests were also carried out to assess the near-surface soil consistency at next to the borehole locations.

The approximate borehole locations are shown on the Borehole Location Plan (Drawing 9625-GR-1-A) provided in Appendix B. The borehole log sheets and core box photographs are provided in Appendix C. These log sheets should be read in conjunction with the attached Explanatory Notes, which explain the terms, abbreviations and symbols used, together with the interpretation and limitation of the logging procedure.

3.2. Results

Reference to the individual borehole log sheets, attached in Appendix C, should be made for a full description of the subsurface conditions encountered at each borehole. Summarised descriptions of the encountered subsurface geotechnical units are provided in Table 1.

The stratigraphy of the site comprises asphalt pavement, 0.1m to 1.1m of clay and gravelly sand fill, which appears to be moderately compacted, overlying residual soils. The residual soils comprise stiff to hard clay overlying bedrock at a depth of 1.8m at the northwestern side (BH02) and 4.5m at the eastern side of the site (BH01).

Table 1 - Summary of Subsurface Profile (Soil and Rock)

Borehole	BH01	BH02	BH03	BH04	BH05
Surface Level (m) *	RL 83.8	RL 82.7	RL 83.4	RL 82.4	RL 81.7
Geotechnical Units	Depth below the ground surface (m)				
Concrete/Fill: Clay/ gravelly sand/silty sand, appears to be moderately compacted	0.0 – 0.1	0.0 – 0.7	0.0 – 0.7	0.0 – 1.1	0.0 – 1.1
Residual Soil: clay, stiff to hard	0.1 – 4.5	0.7 – 1.8	0.7 – 4.0	1.1 – 4.1	1.1 – 3.8
Bedrock: Sandstone, very low strength, extremely weathered (Class V)	-	1.8 – 7.9 (top at RL 80.9)	4.0 – 7.5 (top at RL 79.4)	4.1 – 6.0 (top at RL 78.3)	Below 3.8 (RL 77.9m)
Residual clay interbedded with low to medium to high strength sandstone	4.5 – 6.9 (at top RL 79.3m)	-	7.5 – 8.0 (top at RL 75.9) 10.4 – 13.25 (b) (top at RL 73)	-	-
Bedrock: Sandstone, low strength, highly to moderately weathered (Class IV)	6.9 – 10.2 (a) (at top RL 76.9m)	7.9 – 9.0 (top at RL 74.8)	-	-	-
Sandstone, medium to high strength, moderately weathered (Class III)	10.2 – 13.1 (at top RL 73.6)	9.0 – 12.5 (top at RL 73.7)	8.0 – 10.4 (at top RL 75.4m) 13.25 – 14.6 (at top RL 70.15m)	-	-
Termination depth	13.1 (RL 70.7 m)	12.5 (RL 70.2 m)	14.6 (RL 68.8 m)	6.0 (RL 76.4 m)	3.8 (RL 77.9 m)

(a) 200mm core loss, inferred as a clayey band has been encountered at the depths between 6.5m and 8.7m.

(b) Totally 1500mm core loss, inferred as a clayey band and a clayey seem have been encountered at the depths between 10.4m and 13.25m.

The upper 2m to 6m of the encountered bedrock in BH02 and BH04 was extremely to highly weathered, very low strength sandstone. In BH01, at the upper 3.5m of bedrock, medium to high strength sandstone was underlain by a clayey seam with a thickness of 250mm, followed by a core loss with a thickness of 1100mm, inferred to be a clayey layer. In BH03, the clayey seams (including core loss) thickness was 1500mm which was encountered between a depth of 10.2m and 13.25m.

Residual clay interbedded with low to medium to high strength sandstone and Class V sandstone were underlain by low strength sandstone (assessed as Class IV) extending to a depth of 10.2m in BH01 and 9m in BH02m. The medium to high strength sandstone (assessed as Class III) was intersected below a depth of 8m in BH03, dipping to 10.2m in BH01. It should be noted that Class III sandstone was interbedded with Class V material with a thickness of 2.8m.

Bedrock defects and seams are recorded in the attached logs.

Groundwater seepage was only encountered at a depth of 6m (RL 76.7m AHD) during the drilling of borehole BH02. In the rest of the boreholes, groundwater was not observed during auger drilling and due to the introduction of drilling fluid during the bedrock coring, the groundwater was not detectable.

4. LABORATORY TESTING

Laboratory tests were carried out on selected soil and rock samples collected during the drilling of the boreholes including:

- Atterberg Limit (AS1289 3.1.1 & 3.2.1 & 3.3.1);
- Linear Shrinkage (AS1289 3.4.1);
- Two (2) Soil Aggressivity tests;
- Point Load Index Tests in accordance with AS 4133 4.1.

4.1. Point Load Strength Index (Is_{50}) Testing

The Point Load Strength Index (Is_{50}) tests were undertaken on rock core samples obtained from the boreholes and the results are recorded on the core log sheets presented in Appendix C. The testing was carried out in AG's NATA-registered soil and rock laboratory.

4.2. Atterberg Limits Test and Linear Shrinkage

Atterberg Limits and Linear Shrinkage tests were performed on two soil samples collected from residual soils in BH01 and BH05. The test results are presented in Table 2.

Table 2 - Summary of Atterberg Limits

Borehole No.	Sample Depth (m)	Soil Type	Atterberg Limits (%)			Linear Shrinkage
			LL	PL	PI	%
BH01	1.5 – 2.0	Clay, medium plasticity	40	16	24	11
BH03	1.2m – 1.07m	Clay, high plasticity	70	21	49	17

LL: Liquid Limit PI: Plasticity Index

PL: Plasticity Limit LS: Linear Shrinkage

4.3. Soil Aggressivity Tests

Two soil aggressivity tests were performed for the input to the design of durable concrete and steel materials in contact with the site soils on selected soil samples. Table 3 presents the results of the soil aggressivity tests. The laboratory test certificates are provided in Appendix D.

Table 3 - Aggressivity Test Results

Test	Unit	BH02 Residual clay 2m	BH04 Residual clay 1.2m
Chloride	mg/kg	26	16
pH	--	4.7	4.8
Sulfate (SO_4)	mg/kg (ppm)	46	49
Conductivity	uS/cm	36	31
Resistivity	Ohm.cm	28000	32000
Moisture	%	14	24
Results *	In relation to Concrete	Mild	Non-aggressive
	In relation to Steel	Non-aggressive	Non-aggressive
** assessed in accordance with AS 2159 – 2009, Table 6.4.2 (C) & Table 6.5.2 (C)*			

5. COMMENTS AND RECOMMENDATIONS

5.1. Groundwater and Dewatering

Based on the findings of this investigation, it is not expected to encounter the groundwater table during the proposed excavation works. Groundwater seepage was only encountered in BH02 at a depth of 6m (RL 76.7m AHD) and it is anticipated to occur through the bedrock joints and also at the interface of residual soil and bedrock.

Groundwater seepage tends to fluctuate with seasonal weather patterns. As such, the construction should be planned to manage seepage and surface runoff during basement excavation. It is anticipated that such seepages could be controlled and managed by using sump pumping techniques and that provision is to be allowed in design for a properly designed long term drainage system.

The base of the excavation is anticipated to be founded on residual clay or class V sandstone. Therefore, to provide an appropriate working platform at the base of the excavation, it is recommended to place a layer of compacted single-size gravel with a minimum compacted thickness of 100mm.

During the service life of the building (post-construction), groundwater seepage should be controlled by a properly designed drainage system including a sub-floor drainage system to create a free-draining layer below the base of the basement slab.

5.2. Excavation Conditions and Vibration

Based on the subsurface conditions encountered and summarised in Table 1, bulk excavations are expected to encounter moderately compacted fill material and stiff to hard residual clay to the level of the bulk excavation level.

The bulk excavation level is expected to be founded on residual clay.

Excavations through the overlying soils are expected to be readily achievable using conventional earthworks equipment such as a tracked excavator with tiger toothed bucket. Given that the entire of the basement excavation is expected to be within the soil profile, issues associated with vibration attenuation are unlikely to be experienced during construction.

Generally, the ground vibration Peak Particle Velocity (PPV) should be limited to 5 mm/s at the property boundaries. The maximum 5 mm/s vibration limit is not expected to be exceeded provided that rock breaker equipment and excavation methods are restricted as indicated in Table 4 below.

Table 4 - Recommendations for Rock Breaking Equipment

Distance from Adjacent Structure (m)	Maximum Peak Particle Velocity 5 mm/s	
	Equipment	Operating Limit (% of Maximum Capacity)
1.5 to 2.5	hand-operated jack-hammer only	100
2.5 to 5.0	300 kg rock hammer	50
5.0 to 10.0	300 kg rock hammer or 600 kg rock hammer	100 50

A dilapidation survey on nearby structures and infrastructure is recommended to be undertaken prior to the commencement of any site excavations. The report should include precise measurements of the existing defects and cracks presented with the relevant photos.

5.3. Excavation Support

The provided preliminary drawings do not include a detailed basement floor level with the setbacks from the site boundary.

Unsupported temporary batter slopes are considered feasible provided the excavations in the soil and Class V sandstone do not extend below the ‘zone of influence’ of any adjacent structures, road and infrastructure (i.e. a 45° line from a 1.5m offset of adjacent structures or infrastructures footing). To assess the feasibility of using batter slopes, the footing level of the adjoining structures and infrastructure, and also surrounding services invert level should be assessed by the designer.

The recommended maximum temporary batter slopes are presented in Table 5.

Table 5- Maximum Recommended Batter Slope Angle

Material	Maximum Batter Slope (H: V)
	Temporary
Fill: moderately compacted clay/ gravelly sand/silty sand	2: 1
Residual Soil: stiff to hard clay	1: 1
Bedrock: Class V Sandstone	1: 2 *

* Subject to inspection by a geotechnical engineer and carrying out remedial works if recommended (shotcrete, rock bolting, etc.).

Where temporary batter slopes are not considered feasible due to the space restriction, the soils and Class V sandstone should be retained by an appropriately designed shoring system.

The shoring system could take the form of a soldier pile wall with reinforced shotcrete infill panels. Weep holes and vertical drains should be provided behind shotcrete to avoid build-up of hydrostatic pressure in the overburden soils and rock mass.

The piles should extend below the base of the excavation by a minimum depth of 500mm. The piled wall can be designed as a cantilevered shoring system or laterally supported by ground anchors.

For the purpose of anchor installation, careful consideration should be given to the adjacent structures/infrastructure particularly, foundation conditions of the adjacent buildings. Along the eastern side, permission should be obtained from TfNSW (formerly RMS) to install anchors below Miller Street.

Where the piles' lateral capacity may be provided by the installation of anchors, the length of the anchors should be specified by the design engineer following undertaking analysis to assess the lateral pressures and stability of the excavation.

The anchoring system should be designed to provide temporary support with long-term lateral support being later transformed on to the permanent structure. Anchors will need to be installed progressively as the excavation proceeds and will require the permission of the adjacent landowners for anchors to be extended into their land. In addition, the adjacent neighbouring footing level and underground service levels in the road reserve must be confirmed prior to finalising anchor design.

Temporary anchors may be designed using the recommended ultimate bond stresses presented in Table 6 below.

Table 6 – Recommended Bond Stresses for Temporary Anchor Design

Description	Ultimate Bond Stress (kPa)
Very low strength sandstone (Class V)	150
Low strength sandstone	300
Medium to high strength sandstone	800

Periodic lift-off checks of installed anchors should be carried out during construction to ensure lock off-load is maintained. It is recommended that the anchors be installed and proof-tested in accordance with the requirements of AS4678-2002 and RMS QA Specification B114.

It is recommended that an experienced geotechnical engineer be engaged to check the design of the excavation support system.

The specific requirements set out above for excavation support at the upper levels and the stability of the shale face should be assessed at no greater than 1.5m vertical cut intervals by an experienced geotechnical engineer as the excavation proceeds.

A geotechnical monitoring program should be prepared to undertake regular monitoring during the construction of the shoring system to check and confirm that deflections and movements are within tolerable limits accepted in design.

The shoring system deflection should be monitored using survey monitoring points for checking against the design and tolerable values. This would be developed as a part of excavation management and monitoring plan. Survey targets should be installed in regular intervals along the length of the shoring wall and be monitored regularly as excavation and construction progress.

5.4. Lateral Earth Pressure Coefficients

Earth retaining structures should be designed to withstand the applied lateral pressures of the subsurface soil layers, hydrostatic pressure and live/surcharge loads within the zone of influence of the structure. For the preliminary design of flexible retaining structures, where some lateral movement is acceptable, an 'active' lateral earth pressure coefficient (k_a) is recommended. If it is critical to limit the horizontal deformation of a retaining structure of an earth pressure coefficient 'at rest' (k_o) should be considered. The recommended design parameters as summarised in Table 7.

The below parameters are provided for the retaining structure design. The design groundwater level is to be accounted for with hydrostatic pressure calculations.

Table 7 - Typical Material Properties for Retaining wall Design

Geotechnical Units	c' (kPa)	$\emptyset'$ (degrees)	γ (kN/m³)	K_a	K_p	K_o	E' (MPa)	ϑ'
Fill: Clay/ gravelly sand/silty sand, appears to be moderately compacted	0	24	18	0.42	2.37	0.59	10	0.3
Residual clay, stiff to hard	5	26	18	0.39	2.56	0.56	30	0.3
Very low strength sandstone (Class V)	100	33	22	0.29	3.39	0.46	50	0.3
Low strength sandstone (Class IV)	200	35	23	0.27	3.69	0.43	100	0.3
Medium to high strength sandstone (Class III)	2000	38	24	0.24	4.2	0.38	350	0.25

Legend:
$\emptyset'$: Effective Friction Angle
c': Effective Cohesion
γ : Bulk Unit Weight
K_a: Active earth pressure
K_o: Earth pressure at rest
Kp: Passive earth pressure
E': Elasticity Modulus
ϑ': Poisson's Ratio

The shoring system should be designed adopting a trapezoidal stress distribution if is to be supported by ground anchors. For preliminary design of the anchored walls, supporting areas sensitive to lateral movement, a trapezoidal earth pressure distribution of 0.4 γ H (kPa) should be adopted for the soil profile and Class V sandstone, where H is the retained height in metres. These pressures should be assumed to be uniform over the central 50% of the support system.

The permanent structures or cantilever shoring system should be designed using a triangular earth pressure distribution and the following formula if the retaining system is designed as a cantilever wall:

$$P_h = (\gamma \cdot h + q) \cdot K_a - 2C'\sqrt{K_a} + \textit{Water pressure (if applicable)}$$

where:

P_h = Horizontal active pressure (kN/m^2)
γ = Unit weight of soil (kN/m^3)
K_a = Coefficient of active earth pressure
h = Retained height (m)
q = Surcharge pressure behind retaining wall (kN/m^2)

5.5. Foundations

The bulk excavation level in the proposed five-storey buildings referred to Carlow Street main building and Miller Street development is anticipated to be founded within residual clay. Given the anticipated loads applied by a five-storey building, it is not recommended to adopt shallow pad or raft footings at the basement level. Differential foundation settlement would be expected if the building is founded on a raft footing at the basement level. It is recommended that all structural loads would be taken to the bedrock.

Bored concrete piles are feasible for this project. The design parameters for the foundations are presented in Table 8.

The bedrock quality and bearing capacity are assessed in accordance with the classification presented by Pells et al (1998) for Sydney Sandstone and Shale.

Table 8 – Geotechnical Design Parameters for Deep Foundation

Description	End Bearing Pressure		Shaft Adhesion		Elasticity Modulus (MPa)
	Ultimate (kPa)	Allowable (kPa)	Ultimate (MPa)	Allowable (kPa)	
Residual Soil: clay, stiff to hard (For double storey buildings)	450	150	40	20	30
Very Low strength sandstone (Class V)	2500	800	250	80	50
Low strength sandstone	4500	1500	450	150	100
Medium to high strength sandstone	6000	2500	750	250	350

Particular attention should be given to the residual clay layers interbedded with low to medium to high strength sandstone (e.g. BH03). The above-provided bearing capacities are applicable where a minimum of three pile diameters of the bedrock have been proved below the pile toe. Otherwise, there exists the risk of clayey layer settlement within the zone of influence of the piles.

The piled foundations should be designed in accordance with AS 2159-2009 Piling – Design and Installation. The geotechnical strength reduction factor (Φgb) should be evaluated by the designer based on the construction method, pile testing method/frequency and other factors provided in Clause 4.3.2 of AS 2159-2009.

Serviceability end bearing pressures are recommended for the design based on limiting the settlement to less than 1% of the minimum pile diameter.

Large settlements (more than 5% of minimum footing dimensions) need to occur in order to mobilise the ultimate end bearing resistance, which could be considered as excessive deflections for the building structure.

Based on the applied structural loads, the proposed double-storey Precinct Pavilion and Canteen buildings may adopt shallow pad footings. An allowable bearing pressure of 100 kPa for pad/strip footings is considered suitable for the residual stiff to hard clay. Alternatively, the structure can be founded on piled/pier footings. The piles are recommended to be designed adopting the bearing capacities provided in Table 8.

5.6. Construction Inspections

Inspections by a suitably qualified and experienced geotechnical engineer or engineering geologist should be undertaken during the basement excavation and pile boring.

The piles will need to be inspected during boring to confirm the soil and rock strength. An experienced geotechnical engineer or engineering geologist should confirm the design socket depths and also confirm that the bases of the piles are clean and free of soft, loose, wet or disturbed soils.

5.7. Earthquake Loading Factors

In accordance with AS1170.4 – 2007, the following factors are considered appropriate:

- Hazard Factor (Z): 0.08
- Site Sub-Soil Class:
  - For structures founded on low strength or better rock: B_e
  - For structures founded on residual clay: C_e

5.8. Further investigation

Due to the limited site access, it was not possible to drill any boreholes at the northeastern corner and western side of the proposed excavation. Also, ground conditions were not confirmed at the location of the proposed double-storey buildings (for Precinct Pavilion and Canteen) due to access constraints. Therefore, it may be considered necessary to undertake further investigations, possibly comprising the drilling of two or three additional boreholes, to confirm the subsurface condition at the northeastern corner, western side of the basement excavation and also at the location of the double-storey buildings.

6. LIMITATIONS

Alliance Geotechnical Pty Ltd (AG) has prepared this report for the site located at 270 Miller Street, North Sydney NSW 2060 in accordance with AG's fee proposal and Terms of Engagement. This geotechnical report has been prepared for Carmichael Tompkins Property Group for this project and for the purposes outlined in this report. This report cannot be relied upon for other projects, other parties on this site or any other site. The comments and recommendations provided in this report are based on the assumption that the geotechnical recommendations contained in this report will be fully complied with during the design and construction of the proposed site development.

The borehole investigation and laboratory testing results provided in this report are indicative of the subsurface conditions at the site only at the specific sampling and testing locations, and to the depths drilled at the time of the investigation. Subsurface conditions can change significantly due to geological and human processes. Where variations in conditions are encountered further geotechnical advice should be sought from AG.

APPENDIX A – Selected Site Photographs

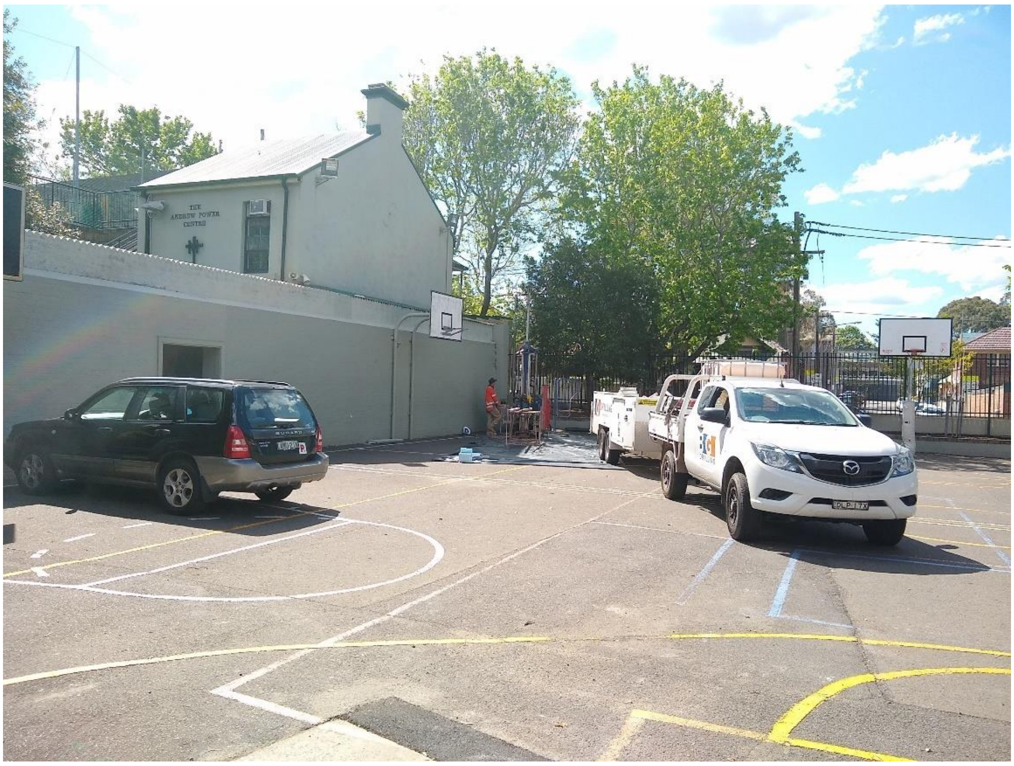

Photo 1 - AG's site investigation – drilling BH02

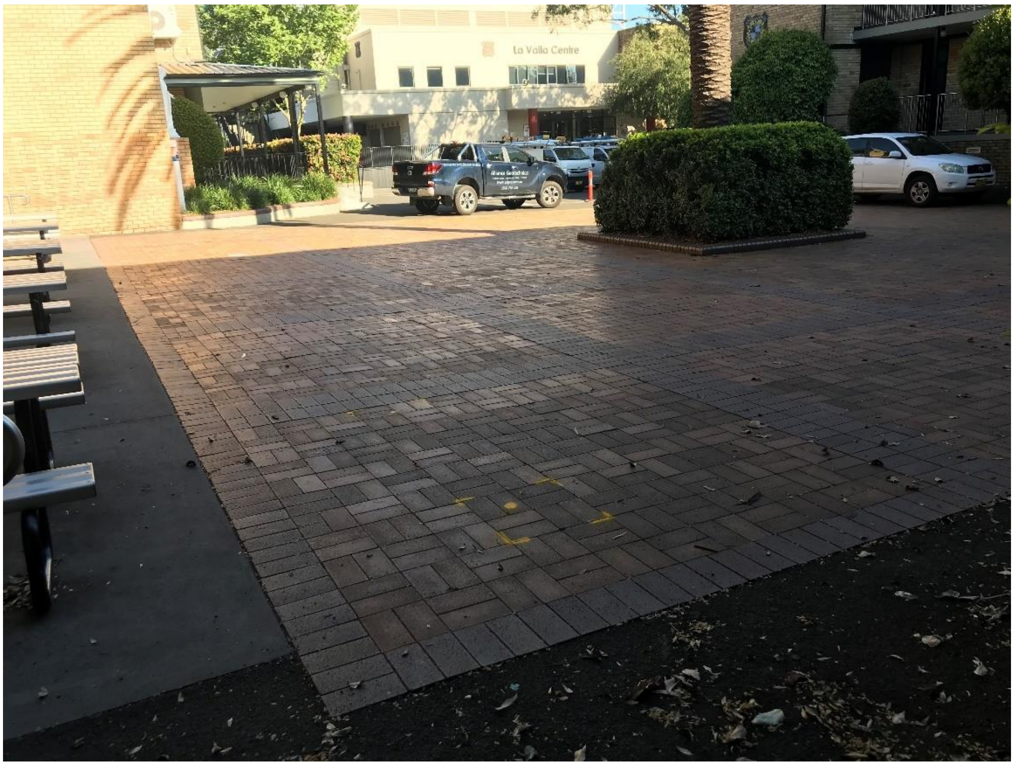

Photo 2 – AG's site investigation – looking south east

APPENDIX B– Borehole Location Plan (Drawing: 9625-GR-1-A)

APPENDIX C –Borehole Logs (BH01 to BH05), Core Box Photos

APPENDIX D – Laboratory Tests Certificate