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APPENDICES

1.0 INTRODUCTION

At the request of Mrs S Fenton (client), Australian Geotechnical Pty Ltd (AG) prepared this Geotechnical Report in relation to a proposed residential dwelling at 762 – 764 Forest Road, Peakhurst, NSW, 2210 (the site). This geotechnical investigation report is prepared for Development Application (DA) submission and also provide geotechnical design parameters and recommendations regarding the proposed development.

This Geotechnical Investigation Report is intended to provide assessment regarding site description, geology, existing ground conditions, geotechnical design input parameters, construction management of temporary excavations, earthworks and site drainage. In view of the above, the purposes of this report are to provide:

- Geotechnical subsurface conditions and groundwater (if applicable).
- Site Classification in accordance with AS 2870 – *"Residential Slabs and Footings".*
- Geotechnical comments, recommendations and design input parameters for the detailed engineering design, construction approach, construction review and implementation of the risk management actions.

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

- Review of the geological map;
- Obtain Dial Before You Dig (DBYD) plans;
- Drilling of one (1) borehole at the site to bedrock refusal;
- Carry out four (4) Dynamic Cone Penetrometer Tests (DCP) to evaluate shallow allowable bearing pressures (ABP);
- Prepare a geotechnical investigation report summarising the findings of the geotechnical investigation and provide recommendations for the proposed development.

To assist in the preparation of this Geotechnical Report, AG was supplied with the following documents relating to the proposed site development:

- Set of Architectural Drawings prepared by Innovate Architects Pty Ltd, job number 2699 dated October 2020.

These documents were used to illustrate the proposed development site layout, inferred geological conditions and geotechnical issues.

2.0 SITE DETAILS

The following information, presented in Table 1, describes the site.

Table 1: Summary of Site Details

Site Address	762 – 764 Forest Road, Peakhurst, NSW, 2210
Client	Mrs S Fenton
Council Area	Georges River Council

Based on the provided drawings, it is understood that the development will comprise demolition of the north-west portion of the existing structure to allow for construction of a two storey hostel development with basement carparking. Proposed excavation depths of approximately 3.0m from existing ground levels are planned to allow construction of the basement carparking area to RL 55.0m, with locally deeper excavations anticipated for footings, service trenches and pool. Based on the drawings provided, this office understands the following is planned for the site:

- A two storey hostel development covers the northern and eastern portions of site.
- Access to the basement car carparking will be via a new ramp at the north eastern corner of the site
- Associated drainage, excavations, retaining walls, gym, pool services and landscaping.

The proposed basement car parking area will be constructed in close proximity to existing dwelling, 3m from the northern boundary, 3m from the south-western boundary and 1.68m from the eastern boundary.

2.1 Geology and Soil Landscapes

Figure 1 - Regional Geology

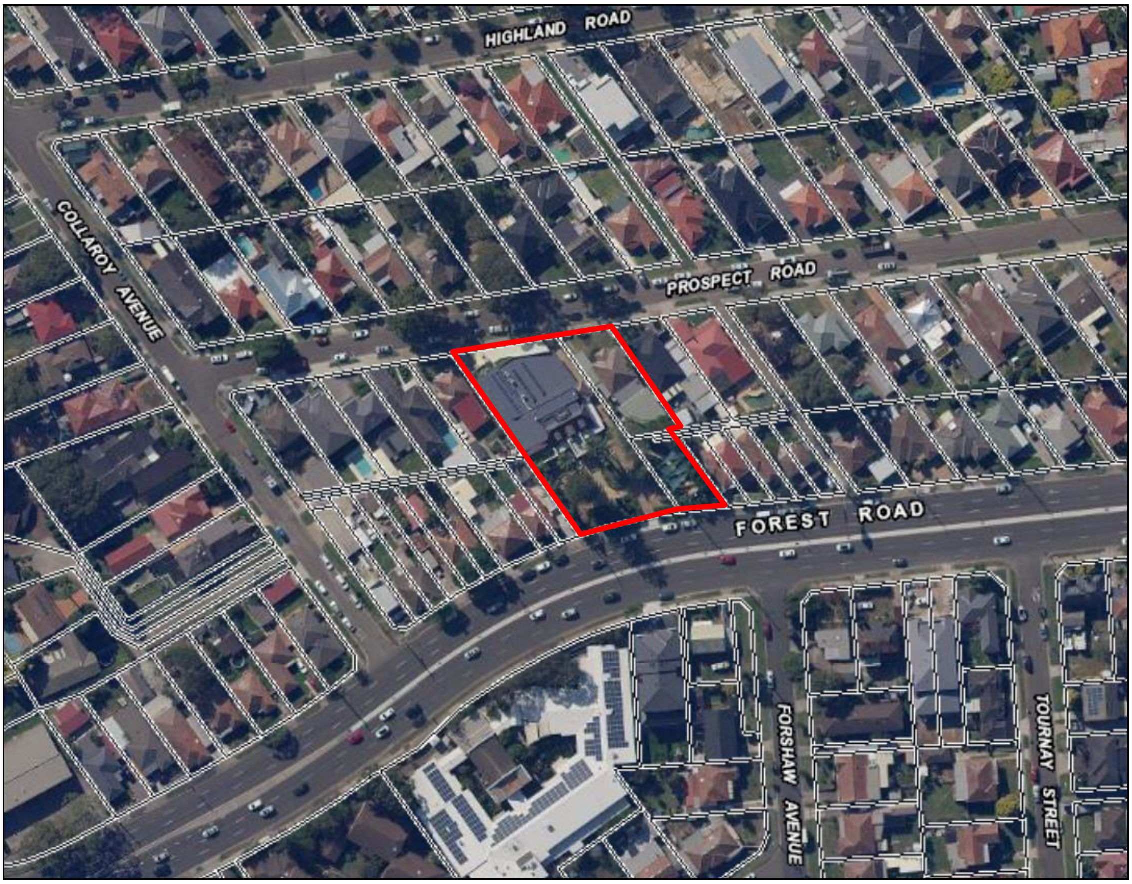
The 1:100,000 scale Geological Series Map of the Sydney region indicates that the subject site is underlain by the Ashfield Shale (Rwa) of the Wianamatta Group, described as *'black to dark grey shale and laminate.*

The Soil Landscape Series Sheet 9130gn and 9130bt, Scale 1:100,000, 2002, prepared by the Soil Conservation Service of NSW, indicates that the site is located at the border of the Blacktown and Glenorie landscape which generally comprises of *'Wianamatta Group Ashfield Shale and Bringelly Shale formations. The Ashfield Shale is comprised of laminite and dark grey shale. Bringelly Shale consists of shale, calcareous claystone, laminite, fine to medium grained lithic-quartz sandstone (Herbert, 1983)'*

2.2 Site Description

Both sites combined are approximately L in shape with an estimated area of 2,160m^2 based on the street-directory website.

Figure 2 - Site Location



As indicated in Figure 2, the site is bounded by:

- Prospect Road to the north-west,
- Forest Road to the south-east,
- Low density residential dwellings to the north-east and south-west,
- Topographically, the site is situated on a relatively flat land parcel, with an average sloping angle of 3% ascending from the southern portion of site to northern edge of the proposed development.

3.0 GEOTECHNICAL INVESTIGATION

The fieldwork was undertaken on 15th December 2020 under the full-time supervision of a Geotechnical Engineer from AG, and included subsurface investigations at 4 locations, using a 4wd mounted 100mm solid flight auger drilling rig and hand equipment. Buried metallic services and utilities within the site boundaries near the proposed test locations were identified by referring to DBYD utility maps. Borehole numbered 1 was drilled to refusal in the weathered shale. Four (4) DCP (Dynamic Cone Penetrometer) tests were across the site to aid in the assessment of in-situ soil conditions. The locations of the boreholes and DCPs are shown in the attached drawing.

3.1 DCP Test Results

The DCP test results provided additional information for an assessment of ABP of underlying soils. The results are presented in Table 2:

Table 2: DCP Testing Result Summary

Test Number:	DCP-1	DCP-2	DCP-3	DCP-4
Material Description:	Silty Clay, brown, with roots			
Test Method:	AS1289.6.3.2			
Testing Start Depth:	Surface Level			
Test Location:	BH01	DCP-2	DCP-3	DCP-4
Depth Tested (m):	Blows Per/100mm			
0 – 0.1	2	2	3	3
0.1 – 0.2	6	4	5	5
0.2 – 0.3	6	4	6	5
0.3 – 0.4	5	4	7	5
0.4 – 0.5	5	5	6	6
0.5 – 0.6	4	3	6	5
0.6 – 0.7	3	4	7	5
0.7 – 0.8	3	4	8	5
0.8 – 0.9	2	5	refusal	refusal
0.9 – 1.0	1	5		
1.0 – 1.1	1	refusal		
1.1 – 1.2	2			
1.2 – 1.3	refusal			

3.2 Soil Profiles

The subsurface conditions observed on site are summarised in Table 3. For a detailed description, refer to attached explanatory notes.

Table 3 - Subsurface Soil Profile

Borehole	Approximate Elevation (RL m)	Borehole depth (m)	Fill[2,4] (m)	Reisdual[1,5] (m)	Bedrock[3,6] (m)
BH01	58.4	1.7	0 – 0.3	0.3-1.3	At 1.3
DCP2	58.4	-	-	-	At 1.1
DCP3	57.9	-	-	-	At 0.9
DCP4	57.8	-	-	-	At 0.9

[1] Estimated soil consistency/strength is based on DCP test results at the testing locations. The potential for weak or softer layers throughout the unit should be considered.
[2] Assumed fill thickness based on DCP blow counts and observations made during the geotechnical investigation. Thickness of the fill layer is expected to vary from those indicated in Table 3.
[3] Inferred bedrock composition, continuity, strength and depth should be confirmed by a geotechnical engineering either prior to construction by additional boreholes and testing, or during construction by inspection.
[4] Soil Horizon Unit 1 - TOPSOIL/FILL: Silty Sand, fine to coarse sand, trace fine gravel, grass roots.
[5] Soil Horizon Unit 2 - RESIDUAL: Silty Clay, medium to high plasticity, brown and grey red mottled, fine to medium iron gravels.
[6] Soil Horizon Unit 3 - SHALE, grey to brown, extremely weathered, extremely low estimated strength.

Notes:

- Clay seams, defects, and fractured and extremely weathered zones are expected to be present throughout the underlying inferred bedrock, predominately at depths and locations unobserved during the geotechnical investigation.
- Ground conditions are expected to vary across the site, and should be confirmed by a geotechnical engineer, predominately in areas unobserved during the geotechnical investigation.

No groundwater was encountered at the time of our visit; however, some seepage flows are likely from the weathered rock/soil interface following periods of rainfall. Therefore, appropriate drainage systems and free draining backfill should be provided to prevent the build-up of hydrostatic pressures behind all retaining walls.

4.0 RECOMMENDATIONS – EXCAVATIONS

4.1 Batter Slopes

It is anticipated that the proposed basement floor level will be constructed in close proximity to the northern side of the existing dwelling house at 764 Forest Road and the south eastern boundary of no 19 Prospect Road. Excavation will extend to within 1m and 1.68m respectively, therefore will reside within the zone of influence. Temporary or permanent batters may be considered for certain areas of the proposed excavation where sufficient space exists between the proposed excavation walls and adjoining infrastructures. It should be noted that due to the nature of natural soils and weathered bedrock underlying the site, and the potential for elevated groundwater levels within the excavation area, unsupported vertical cuts of the soils carry the potential for slump failure.

Temporary or permanent batter slopes may be considered where sufficient space exists between the excavation walls and adjoining infrastructures, and where the adjacent infrastructures are located outside the “zone of influence” (obtained by drawing a line 45^{o} above horizontal from the base of the proposed basement walls) for the use temporary batter slopes. Table 4 provides maximum recommended slopes for permanent and temporary batters.

Maximum inferred excavation depths are expected to vary within the site from approximately 3.0 (varying throughout) for construction of the proposed development, with locally deeper excavations also anticipated to be required for the proposed building footings and service trenches.

Based on the ground conditions within the site, the total depth of excavation and the extent of the excavation walls to the site boundaries and adjoining infrastructures, it is critical from geotechnical perspective to maintain the stability of the adjacent structures and infrastructures during demolition, excavation and construction. The recommended permanent and temporary (i.e. up to 28 days) batter slopes are presented in Table 4:

Table 4 - Maximum Excavation Batter Slopes

Soil or Rock Material Type	**Maximum Batter Slope (H : V)**	
	Permanent	**Temporary (exposed for up to 28 days)**
Silty Clay (Fill, Colluvium or Residual Soil)	N/A – retaining wall	3 : 1
Shale Bedrock (extremely low to low strength)	2 : 1*	1 : 1*
Shale Bedrock (medium strength)	1 : 1*	0.5 : 1*
Shale Bedrock (high strength, or better)	Semi-Vertical – Vertical*	
* Subject to inspection by a Geotechnical Engineer and carrying out stabilisation works if recommended (shotcrete, rock bolting, etc.). It may be possible to achieve vertical permanent rock excavations (e.g. for the ‘feature excavation’), subject to geotechnical review and implementation of stabilisation works as required to manage the geotechnical risks. If the temporary batter slopes cannot be achieved, then appropriate temporary shoring / excavation support must be provided.		

All batter slopes within the site should remain stable providing all surcharge and construction loads are kept out of the “zone of influence” (obtained by drawing a line 45^{o} above horizontal from the base of the proposed excavation walls) plus an additional 1.0m. A geotechnical engineer or engineering geologist should inspect the batter slopes within the site.

It should be noted that steeper batter slopes may be considered for higher strength (i.e. medium estimated strength, or better) and intact bedrock which may underlie the site, subject to confirmation by a geotechnical engineer during construction by inspection, or by additional borehole drilling and rock strength testing. Consideration should be given to shotcreting and soil nailing where steeper batter slopes are to be used.

Temporary surface protection against erosion should be provided by covering the batter slopes with plastic sheets extending at least 1.5m behind the crest of the cut face or up to the common site boundaries. The sheets should be positioned and fastened to prevent any water infiltration onto or into the batter slopes. Other applicable methods may be adopted for temporary surface protection, and all surface protection should be placed following inspection of the temporary batters by a geotechnical engineer.

An appropriately designed retaining wall by a suitably qualified structural engineer should be implemented and constructed around the proposed excavation perimeter walls following any temporary or permanent batter slopes within the site. All retaining walls should be sufficiently constructed on appropriate bedrock underlying the site, and should take into consideration the lateral earth pressures induced by soil movement along the interface between soils and the underlying bedrock.

4.2 Excavation Support System

Where there is insufficient space proposed excavation walls and adjoining infrastructures, or where adjacent infrastructures are located within the “zone of influence” (as outlined in Section 4.1 above), consideration should be given to a suitable retention system such as a soldier pile wall solution sufficiently embedded into appropriate and competent bedrock underlying the site, with concrete infill panels for the support of the excavation and soils.

Closer spaced piles may be required to reduce lateral movements particularly where adjacent infrastructures, such as buildings or pavements which are located near the excavation, and to prevent collapse of loose/soft fill in-situ materials, natural soils and weathered bedrock. Pile spacing should be analysed and designed by the project structural engineer and should consider horizontal pressures due to surcharge loads from adjacent infrastructures (i.e. buildings, road reserves, etc.), and long term loadings.

Battering back of the soils may be required to permit installation of soldier piles and prevent the collapse of soils into the excavation area. This should be monitored by a geotechnical engineer familiar with these site conditions.

The use of a more rigid retention system such as a cast in-situ contiguous pile wall solution should also be considered to reduce the lateral movements and risk of potential damage to adjacent infrastructures (i.e. adjacent road reserves and infrastructures). This option may also be adopted where excessive surcharges are adjacent to the excavation, and to meet acceptable deflection criteria. It should be noted that groundwater inflow may pass through shoring pile gaps during excavation. This may be controlled by the installation of strip drains behind the retention system, connected to the buildings stormwater system. Shotcreting or localised grouting may also be used in weak areas of the retention system, predominately where groundwater seepage and loose/soft soils are visible. Shoring design should take into consideration both short term (during construction) and permanent conditions, along with surcharge loading and footing loads from adjacent infrastructures.

Where groundwater is deemed to be relatively high, and permeability rates are excessive, it is recommended that consideration be given to a contiguous pile wall with strip drains installed behind the piles and shotcreting in weak areas susceptible to groundwater inflow. This should be confirmed by measures discussed in Section 7.0 of this report.

In cases where anchoring is impractical, other temporary support for the adopted shoring system should be considered. This may include the staged excavation and installation of temporary berms or props in front of the retaining wall.

If considered, the shoring wall can be designed using the recommended design parameters provided in Section 4.3. Bulk excavation and foundations (including pile installations) should be supervised, monitored and inspected by a geotechnical engineer, with all structural elements of the development by a structural engineer. Inspections should be considered as "Hold Points" to the project.

4.3 Excavation Support Design Parameters

Excavation pressures acting on the support will depend on a number of factors including external forces from surcharge loading, the stiffness of the support, varying groundwater levels within the site, and the construction sequence of the proposed basement. Therefore, the following parameters may be used for the design of temporary and permanent retaining walls at the subject site:

- A triangular earth pressure distribution may be adopted for derivation of active pressures where a simple support system (i.e. cantilevered wall or propped/anchored wall with only one row of props/anchors are required) is adopted. Cantilevered walls are typically less than 2.5m in height, and should take ensure deflections remain within tolerable limits.
    - Flexible retaining structures (i.e. cantilevered walls or walls with only one row of anchors), should be based on active lateral earth pressure. "At rest" earth pressure coefficient should be considered to limit the horizontal deformation of the retaining structure. Lateral active (or at rest) and passive earth pressures for cantilever walls or walls with only one row of anchors may be determined as follows:

Lateral active or "at rest" earth pressure:

$$P_a = K \gamma H - 2c\sqrt{K}$$

Passive earth pressure:

$$P_p = K_p \gamma H + 2c\sqrt{K_p}$$

- Where lateral deflection exceeds tolerable limits, or where two or more rows of anchors are required, the retention/shoring system should be designed as a braced structure. This more complex support system should utilise advanced numerical analysis tools such as WALLAP or PLAXIS which can ensure deflections in the walls remain within tolerable limits and to model the sequence of anchor installation and excavation. For braced retaining walls, a uniform lateral earth pressure should be adopted as follows:

Active earth pressure:

$$P_a = 0.65\,K\,\gamma\,H$$

Where:

P_a = Active (or at rest) Earth Pressure (kN/m^2)
P_p = Passive Earth Pressure (kN/m^2)
γ = Bulk density (kN/m^3)
K = Coefficient of Earth Pressure (K_a or K_o)
K_p = Coefficient of Passive Earth Pressure
H = Retained height (m)
c = Effective Cohesion (kN/m^2)

- Support systems and retaining structures 'should be designed to withstand hydrostatic pressures, lateral earth pressures and earthquake pressures (if applicable). The applied surcharge loads in their “zone of influence” should also be considered as part of the design, where the “zone of influence” may be obtained by drawing a line 45^o above horizontal from the base of the proposed retailing wall.

Support system designed using the earth pressure approach may be based on the parameters given in Table 5 below for soils and rock horizons underlying the site. Table 5 also provides preliminary coefficients of lateral earth pressure for the soils and rock horizons encountered in the site. These are based on fully drained conditions and that the ground behind the retention walls is horizontal.

Table 5 – Soil Parameters for Retaining Wall Design

Material	Fill (Unit 1)	Residual Soils (Unit 2)	Bedrock (Unit 3)[3, 5]		
			Unit 3 EL – VL[5]	Unit 4 L – M[5]	Unit 5 M – H[5,6]
Unit Weight (kN/m3)[4]	17	19	22	22	24
Effective Cohesion c' (kPa)	0	5	25	50	75
Angle of Friction ϕ' (°)	26	24	27	28	30
Earth Pressure Coefficient At Rest Ko[1]	0.56	0.59	0.5	0.5	0.4
Earth Pressure Coefficient Active Ka[2]	0.39	0.42	0.3	0.3	0.25
Earth Pressure Coefficient Passive Kp[2]	2.56	2.37	3.0	3.0	5.0

[1]Earth pressure coefficient at rest (Ko) can be calculated using Jacky's equation.
[2]Earth pressure coefficient of active (Ka) and passive (Kp) can be calculated using Rankine's or Coulomb's equation.
[3]The values for rock assume no defects of adverse dipping is present in the underlying bedrock. All excavation rock faces should be inspected on a regular basis by an experienced engineering geologist or geotechnical engineer.
[4]Above groundwater levels.
[5]Confirmation of the underlying bedrock composition, continuity, strength and depth should be confirmed by additional borehole drilling and rock strength testing, or during construction by a geotechnical engineer.
[6]Preliminary only, and inferred to be present within the site at depth. Inferred estimated bedrock strength is based on observations made during auger penetration resistance at the time of drilling.

Notes:

- For undrained (temporary) clay soils, higher earth pressures (K=1) will apply.
- EL = Extremely Low estimated strength, VL = Very Low estimated strength, L = Low estimated strength, M = Medium estimated strength, H = High estimated strength.

These geotechnical design parameters set out in Table 5 can be verified and/or adjusted by obtaining soil samples from the site for testing at a NATA-registered soils laboratory.

In addition, design of retaining walls should consider the following:

- If piled retaining walls are to provide permanent support to proposed structures, pile sockets in rock may need to be longer to accommodate additional lateral and axial loads. Anchoring may be required for additional lateral support.

- The retaining wall design should also allow for any surcharge loads from adjoining structures, relevant site features and construction loads, these loads should be calculated separately.

* To facilitate the site earthworks, it would be prudent to install a temporary catch drain above the proposed excavation to divert surface run-off away from the building area during construction.

* Static water pressures should be taken into consideration, unless adequate vertical strip drainage is provided behind retaining walls. A hydrostatic pressure distribution could be used for this analysis.

* Appropriate surcharge loading from construction equipment, vehicular traffic and neighbouring structures at finished surface level should be taken into account in the retention design. Surcharge loads on retention structures may be calculated using a rectangular stress block with an earth pressure coefficient of 0.5 applied to surcharge loads at ground surface level.

4.4 Excavation Conditions

Maximum excavation depths of approximately 3.0m (varying throughout) are expected for construction of the proposed development, with locally deeper excavations also anticipated to be required for the proposed building footings and service trenches within the site.

Based on this information and existing ground conditions as encountered during the geotechnical investigation, it is anticipated that excavation will extend through Unit 1 (fill) to Unit 3 (bedrock) inclusive, during excavation of the basement carpark area, as outlined in Table 2 and Table 3 above.

The possibility for encountering higher strength bedrock (i.e. medium estimated strength, or better) should not be precluded during excavation/construction, predominately where deeper excavations are required across the site, and in areas and at depths not observed during the geotechnical investigation, due to the limited investigation carried out within the site.

Particular care will be required to ensure that adjacent buildings and infrastructures (i.e. road reserves, buildings, etc.) are not damaged during demolition, excavation and construction activities (or the like) due to excessive vibrations. Therefore, appropriate excavation and construction methods should be adopted which will limit ground vibrations to limits not exceeding the following maximum Peak Particle Velocity (PPV) for adjacent structures:

* Sensitive and/or historical structures – 2mm/sec
* Residential and/or low rise structures – 5mm/sec
* Unreinforced and/or brick structures – 10mm/sec
* Reinforced and/or steel structures – 25mm/sec
* Commercial and/or industrial buildings – 25mm/sec

Vibrations transmitted by the use of rock hammers are unacceptable and not recommended. To minimise vibration transmission to any adjoining infrastructures, and to ensure vibration limits remain within acceptable limits, rock saw cutting using a conventional excavator with a mounted rock saw (or similar) should be carried out as part of excavation prior to any rock breaking commencing. Although rock hammering is unacceptable and not recommended, if necessary during excavation, it is recommended that hammering be carried out horizontally along pre-cut rock boulders or blocks provided by rock saw cutting, and should remain within limits acceptable. This should be monitored at all times during excavation. The effectiveness of all the above-mentioned approaches must be confirmed by the results of vibration monitoring. The limits of 5mm/sec and 10mm/sec are expected to be achievable if rock breaker equipment or other excavations are restricted to the values indicated in Table 6 below.

Table 6 – Operating Restrictions and Vibration Limits for Excavation Equipment

Distance from Adjacent Structure (m)	Maximum Peak Particle Velocity 5 mm/s	
	Equipment	Operating Limit (% of Maximum Capacity)
1.0 to 2.0	hand-operated tools or jack-hammer only	100
2.0 to 5.0	300 kg Rock Hammer	50
5.0 to 10.0	300 kg Rock Hammer or	100
	600 kg Rock Hammer	50

Excavation through Unit 1 to Unit 3 inclusive (softer soils and inferred extremely low to low estimated strength bedrock) should be feasible using conventional earth moving excavators, typically medium to large hydraulic excavators. Smaller sized excavators may encounter difficulty in high strength bands of soils and rocks which may be encountered. Where high strengths bands are encountered, rock breaking or ripping should be allowed for. Removal of the existing pavements and associated infrastructures within the site are also expected to require larger excavators and rock breaking and ripping.

Excavation of medium to higher estimated strength bedrock, which is anticipated to be encountered across the site at depth, would require higher capacity excavators, bulldozers or similar, for effective removal of the rock. This excavation will require the use of heavy ripping and rock breaking equipment or vibratory rock breaking equipment. Furthermore, excavation for the proposed building footings and service trenches may require the use of heavy ripping and rock breaking equipment or vibratory rock breaking equipment, with the possibility of rock saw cutting.

Should rock hammering be used for the excavation in the underlying bedrock, excavation should be carried out away from the adjoining structures, with vibrations transmitted being monitored to maintain vibrations within acceptable limits. Rock saw cutting should be carried out (where required) around the perimeter of the excavation, prior to any rock breaking commencing.

Demolition, excavation and construction activities (or the like) will generate both vibration and noise, predominately whilst being carried out within the underlying bedrock. Vibration control measures should be implemented as part of the excavation process.

A vibration monitoring plan is recommended to be developed to monitor construction activities, and their effects on adjoining infrastructures. A vibration monitoring plan may be carried out attended or unattended. An unattended vibration monitoring must be fitted with alarms in the form of strobe lights, sirens or live alerts sent to the vibration monitoring supervisor, which are activated when the vibration limit is exceeded.

A geotechnical engineer should be contacted immediately if vibrations during construction or in adjacent structures exceed the values outlined above, and work should immediately cease. It is recommended a dilapidation report be carried out prior to any excavation or construction. This should be considered a “Hold Point”.

4.5 Groundwater Control

No groundwater was observed or encountered during augering in borehole numbered BH1, to a maximum depth of approximately 1.7m bgl (RL56.7m AHD).

Following completion of augering in borehole BH1, the borehole was left open to assess groundwater. Groundwater measurements carried out on the 15th December 2020 indicates that BH1 is free of any groundwater at the time of the measurement.

Groundwater which may enter the excavation is expected to be in the form of seepage through the pore spaces between particles of unconsolidated natural soils or through networks of fractures and solution openings in consolidated bedrock. It should be noted that groundwater levels are subject to fluctuate during daily or seasonal factors. Additional groundwater testing and inspections should be carried out prior to construction and design phase of the project, to assess any groundwater inflows throughout the excavation area.

5.0 RECOMMENDATIONS - FOOTINGS

5.1 Site Classification

Footings founded into soil horizon unit 2 will be classified as **Class “H1”**. Based on the geology, natural soil profile as encountered on this limited scope investigation, the site is estimated to have a Characteristic Surface Movement (ys) in the range between **20mm** and **40mm**. Footings and slabs on Bedrock Unit 3 (weathered Sandstone) material may be designed in accordance with AS2870:2011 based on a Site Classification of **Class "A".**

5.2 Footing Design

Following excavation to the proposed basement FFL of RL55.0m AHD, and based on the boreholes carried out, we expect varying ground conditions comprising predominately Unit 3 (inferred Class V Shale), with the potential for Unit 4 (Class V Shale) and Unit 5 (Class III Shale or better) in some areas of the site to be exposed at bulk level excavation.

Based on the proposed development, and assessment of the subsurface conditions, a suitable foundation system comprising a cast in-situ reinforced concrete raft slab is likely to be adopted. The raft foundation should include slab thickening to provide strip and pad footings for the support of the internal walls and columns, respectively.

It should be noted that due to the potential variable bedrock conditions throughout the site (i.e. Class V Shale and inferred Class V bedrock, with the potential for Class III Shale or better in certain areas at bulk level excavation), a conservative allowable bearing pressure of 700kPa should be adopted for the inferred Class IV Shale at bulk level excavation as outlined in Table 7 below. Higher bearing capacities may be justified subject to confirmation by inspection during construction, or by additional borehole drilling and rock strength testing. Bearing capacity and settlement behaviour varies according to foundation depth, shape and dimensions.

Given the potential for variable ground conditions within the site, it is recommended that all foundations are constructed on consistent bedrock throughout the basement FFL to provide uniform support and reduce the potential for differential settlements. This could be attained by strip or pad footings where the suitable bearing capacity is achieved or exposed at bulk level, and pile foundations elsewhere. Reference should be made to the estimated levels of the subsurface conditions outlined in this report, and compared to the final bulk excavation levels across the site.

Installation of piles may be required where the axial and working loads transmitted through the building walls and columns exceed the bearing pressure of the bedrock exposed at basement FFL. These should be socketed into consistent and appropriate bedrock underlying the site. For cases where resistance against wind loads and lateral loads need to be increased, piles may also be required.

Allowable Bearing Pressures (ABP) for the preliminary structural design of footings are provided in Table 7:

Table 7 – Allowable Bearing Pressures for Footing Design

Unit Type/Material		Maximum Allowable (Serviceability) Values (kPa)		
		End Bearing Pressure[1]	Shaft Adhesion (Compression)	Shaft Adhesion (Tension)
Fill (Unit 1)	N/A	N/A	N/A	N/A
Residual Soils (Unit 2)	N/A	100	N/A	N/A
Bedrock (Unit 3)[2]	EL – VL	700	50	25
	L[3, 4]	1,000	100	50
	M[3, 4]	1,500	150	75

[1]Minimum embedment of 0.4m for shallow foundations and 0.5m for deep foundations.
[2]Confirmation of the underlying bedrock composition, continuity, strength and depth should be confirmed by additional borehole drilling and rock strength testing, or during construction by a geotechnical engineer.
[3]Preliminary only, and inferred to be present within the site at depth. Subject to confirmation by a geotechnical engineer during construction by inspection, or by additional borehole drilling and rock strength testing.
[4]At least Class IV Sandstone, or better. Subject to confirmation by a geotechnical engineer, as discussed in this report.

Notes:

- EL = Extremely Low estimated strength, VL = Very Low estimated strength, L = Low estimated strength, M = Medium estimated strength.
- N/A = Not Applicable. Not recommended for the proposed development.
- The composition, depth, strength and continuity of the underlying bedrock material should be confirmed either prior to construction by further borehole drilling and rock strength testing, or during construction by inspection.
- It is recommended that geotechnical inspections on the foundations are completed by a geotechnical engineer to determine the material and confirm the required bearing capacity has been achieved.

Specific geotechnical advice should be obtained for footing deigns and end bearing capacities, and design of the foundation system (shallow and pile foundations) should be carried out in accordance with AS 2870-2011 and AS 2159-2009.

If footings are designed through soil assessed to be affected by soil creep, the footing design should allow for some lateral soil creep to occur over the building's design life (e.g. nominally 2 mm/year). However, because the majority of the footings are proposed within excavations expected to encounter shale, soil creep forces will most likely not need to be addressed in the footing design.

We recommend that geotechnical inspections of foundations be completed by an experienced geotechnical engineer to determine that the designed socket materials have been reached and the required bearing capacity has been achieved. The geotechnical engineer should also determine any variations between the boreholes carried out and inspected locations. Inspections should be carried out in dewatered foundations for a more accurate examination, and inspections should be carried out under satisfactory WHS requirements. Geotechnical inspections for verification capacities of the foundations should constitute as a "Hold Point".

5.3 Filling

Where filling is required, the following recommended compaction targets should be considered:

- Place horizontal loose layers not more than 300mm thickness over the prepared subgrade.
- Compact to a minimum dry density ratio not less than 98% of the maximum dry density for the building platforms.
- The moisture content during compaction should be maintained at ±2% of the Optimal Moisture
- Content (OMC).
- The upper 150mm of the subgrade should be compacted to a dry density ratio not less than 100% of the maximum dry density.

Any soils which are imported onto the site for the purpose of filling and compaction of the excavated areas should be free of deleterious materials and contamination. The imported soils should also include appropriate validation documentation in accordance with current regulatory authority requirements. The design and construction of earthworks should be carried out in accordance with AS 3798-2007. Inspections of the prepared subgrade should be carried out by a geotechnical engineer, and should include proof rolling as a minimum. These inspections should be established as "Hold Points".

5.4 Subgrade Preparation

The following are general recommendations on subgrade preparation for earthworks, slab on ground constructions and pavements:

- Remove existing fill and topsoil, including all materials which are unsuitable from the site.
- Excavate natural soils and rock.
- Excavated material may be used for engineered fill.
- Rock may be used for subgrade material underlying pavements.
- Any natural soils (predominately clayey soils) exposed at the bulk excavation level should be treated and have a moisture condition of 2% OMC. This should be followed by proof rolling and compaction of the upper 150mm layer.

- Any soft or loose areas should be removed and replaced with engineered or approved fill
- material.
- Any rock exposed at the bulk excavation level should be clear of any delirious materials (and free of loose or softened materials). As a guideline, remove an additional 150mm from the bulk excavation level.
- Ensure the foundations and excavated areas are free of water prior to concrete pouring.
- Areas which show visible heaving under compaction or proof rolling should be excavated at least 300mm and replaced with engineered or approved fill, and compacted to a minimum dry density ratio not less than 98% of the maximum dry density.

6.0 CONDITIONS OF THE RECOMMENDATIONS

- The advice given in this report assumes that the test results are representative of the overall subsurface conditions. However, it should be noted that actual conditions in some parts of the building site may differ from those found in the boreholes. If excavations reveal soil conditions significantly different from those shown in our attached Borehole Log(s), Australian Geotechnical must be consulted and excavations stopped immediately.

- The foundation depths quoted in this report are measured from the surface during our testing and may vary accordingly if any filling or excavation works are carried out. The description of the foundation material for has been provided for its easy recognition over the whole building site.

- Any sketches in this report should be considered as only an approximate pictorial evidence of our work. Therefore, unless otherwise stated, any dimensions or slope information should not be used for any building cost calculations and/or positioning of the building. Dimensions on logs are correct.

7.0 RECOMMENDATIONS FOR FURTHER GEOTECHNICAL INVESTIGATION

It is recommended that following demolition, additional machine drilled boreholes and rock strength testing be carried out across the site, in order to confirm the ground conditions and findings and recommendations presented in this report.

Further investigations are required prior to and during the construction phase of the project. These inspections should include but are not limited to the following;

- A dilapidation survey will be required for the adjoining dwellings, roadways and site features prior to and following the construction phase of the project. These surveys should be carried out by a qualified person.

- Geotechnical inspections of shoring wall piles installations.

- Geotechnical inspections of foundations (shallow and piles) to confirm the preliminary bearing capacities have been achieved.

- All spoil excavated and carted from the development site will require classification in accordance with the Waste Classification Guideline Part 1, prepared by NSW Environment Protection Authority (EPA).

- This type of investigation (as per our commission) is not designed or capable of locating all soil conditions, (which can vary even over short distances). Therefore, it is recommended that the builder or owner engage a suitably qualified person to confirm the soil profile and whether the design parameters outlined within this report are available at footing and foundation level.

- Unsupported excavations, batter slopes and saw cut faces must be inspected every metre of vertical excavation by a qualified person. This is required to assess the adequacy of design requirements outlined within this report, in order to provide additional direction (where required) with regards to the construction of batter slopes and saw cutting.

- Monitoring of any groundwater inflows into the excavation during construction.

- Our investigation could only be carried out in grassed areas. Much of the site was inaccessible with the machine drilling rig, therefore subsurface conditions were inferred using DCP testing equipment. Following demolition of the existing structure, additional boreholes logs should be conducted to determine whether design parameters outlined within this report are adequate.

8.0 LIMITATIONS

Australian Geotechnical Pty Ltd (AG) has based its geotechnical assessment on available information obtained prior and during the site inspection/investigation. The geotechnical assessment and recommendations provided in this report, along with the surface, subsurface and geotechnical conditions are limited to the inspection and test areas during the site inspection/investigation, and then only to the depths investigated at the time the work was carried out. Subsurface conditions can change abruptly, and may occur after AG's field testing has been completed.

It is recommended that if for any reason, the site surface, subsurface and geotechnical conditions (including groundwater conditions) encountered during the site inspection/investigation vary substantially during construction, and from AG's recommendations and conclusions, AG should be contacted immediately for further testing and advice. This may be carried out as necessary, and a review of recommendations and conclusions may be provided at additional fees. AG's advice and accuracy may be limited by undetected variations in ground conditions between sampling locations.

AG does not accept any liability for any varying site conditions which have not been observed, and were out of the inspection or test areas, or accessible during the time of the investigation. This report and any associated information and documentations have been prepared solely for Mrs S Fenton, and any misinterpretations or reliances by third parties of this report shall be at their own risk. Any legal or other liabilities resulting from the use of this report by other parties can not be religated to AG.

This report should be read in full, including all conclusions and recommendations. Consultation should be made to AG for any misundertandings or misinterpretations of this report.

9.0 REFERENCES

- Geological Series Map of the Sydney region, scale 1:100,000
- Pells, P.J., Douglas, D.J., Rodway, B, Thorne, C. And Mcmahon, B.K "Design Loadings for Foundations on Shale and Sandstone in the Sydney Region". Australian Geomechanics Journal, Vol.3 1978.
- Pells, P.J., Mostyn, G and Walker, B.F. "Foundations on Sandstone and Shale in the Sydney Region". Australian Geomechanics Journal, Vol. No. 33, Part 3, Dec 1998.
- AS1726:1993, Geotechnical Site Investigations, Standards Australia.
- AS2159:2009, Piling – Design and Installation, Standards Australia.
- AS2870:2011, Residential Slabs and Footings, Standards Australia.
- AS3798:2007, Guidelines on Earthworks for Commercial and Residential Developments, Standards Australia
- NSW Department of Finance and Service, Spatial Information Viewer, maps.six.nsw.gov.au.

For and on behalf of
Australian Geotechnical Pty Ltd

Reviewed By

N. Smith
Principal

J. Lu
Geotechnical Engineer

APPENDIX A

FIGURES

Figure 3: BOREHOLE LOCATION PLAN

APPENDIX B

IMPORTANT INFORMATION AND BOREHOLE LOGS