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Geotechnical Investigation Report

Proposed Development at
52-54 Powell Street and 125 Parramatta Road,
Homebush NSW 2140

Prepared for

Amna Holdings Pty Ltd

Report No. G386-1 Rev 0

November 2019

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TABLE OF CONTENTS

1.0 INTRODUCTION **4**
2.0 AVAILABLE INFORMATION **5**
3.0 FIELDWORK AND LABORATORY TESTING **5**
4.0 GROUND CONDITION **6**
  4.1 Ground Profile 6
  4.2 Groundwater 6
5.0 DISCUSSIONS AND RECOMMENDATIONS **7**
  5.1 Excavation Conditions 7
  5.2 Vibration Control 7
  5.3 Dilapidation survey 8
  5.4 Underpinning of Existing Structures 8
  5.5 Retaining Walls 8
  5.6 Temporary Anchoring System 9
  5.7 Retaining Walls Design 9
  5.8 Groundwater Management 11
  5.9 Foundations 12
  5.10 Site Earthquake Classification 13
  5.11 Earthwork and Subgrade Preparation 13
  5.12 Road and Martine Services (RMS) Requirement 14
  5.13 Additional Geotechnical Investigation 14
6.0 CONCLUSIONS **14**
LIMITATIONS **15**
REFERENCES **15**

LIST OF TABLES

Table 1: Summary of Ground Profile 6
Table 2: Typical Recommended Peak Particle Velocity 7
Table 3: Allowable Bond Stress for Anchoring 9
Table 4: Retaining Walls Design Parameters 10
Table 5: Coefficient of Lateral Earth Pressure 10
Table 6: Foundation Design Parameters 12

APPENDICES

Appendix A:	Site Plan
Appendix B:	Engineering Borehole Logs
Appendix C:	Photograph of Core Samples
Appendix D:	Results of Laboratory Testing

1.0 INTRODUCTION

Purpose of geotechnical investigation is to assess the site's surface and subsurface conditions and to provide geotechnical recommendations for the design and construction of the proposed development. This report interprets and presents findings of the investigation that was carried out during the fieldwork.

Site Location	52-54 Powell Street and 125 Parramatta Road, Homebush NSW 2140	
Lot/DP	Lot 2 and Lot 3 in DP 130557, and Lot 10 and Lot 11 of Section 23 in DP 477	
Local Council	Strathfield	
Site Area	Approximately 3,025 m^2	
Site Slope	Slope generally < 5° towards north east	
Site Shape	Approximately rectangular	
Existing Structures	1-2 storey commercial type buildings with some open carpark facilities	
Closest Watercourse	Powells Creek located approximately 360m north east to the site	
Special Site Features	No special features were observed during the fieldwork	
Neighbouring Properties	North East	Power Street road reserve and carriageway
	South East	Multi storey mixed used building with potential basement carpark levels
	South West	Parramatta Road reserve and carriageway
	North West	Commercial type property
Geology Map	Sydney 1: 1:100,000 Geological Series Sheet 9029-9130 Edition 1, 1983, by the Geological Survey of New South Wales Department of Minerals and Energy - Sydney	
Regional Geology	Rwa – Ashfield Shale, Wianamatta Group, Triassic age	
Geology Description	Black to dark-grey shale and laminite	
Secondary Geology	Qha - Alluvial soils, Quaternary age, described as silty to peaty sand, silt, and clay. Ferruginous and humic cementation in places. Common shell layers located approximately 250m north east and 450m north west to the site	
Proposed Development	9-storey mixed use building	
Proposed Basements	Two basement levels	
Excavation Depth	Inferred to be approximately 7.0m	

2.0 AVAILABLE INFORMATION

At the time of preparation of this report, following information was provided:

- Architectural drawing project tilted “Homebush Apartments Mixed Use Multi Residential”, referenced project No. 18-084 prepared by Urban Link Pty Ltd and dated October 2019.
- Site Surveyor Plan project tilted “Topographical Survey Plan of No. 52-54 Powell Street, Homebush, being Lot 10 & 11 of Sec. 23, D.P. 477”, reference drawing No. 10420 - 00, prepared by ATS Land & Engineering Surveyors Pty. Ltd. and dated May 2018.
- Site Surveyor Plan project tilted “Topographical Survey Plan of No. 125 Parramatta Road, Homebush, being Lot 2 in DP 130557 & Lot 3 in DP 130557”, reference drawing No. 10419, prepared by ATS Land & Engineering Surveyors Pty. Ltd. and dated May 2018.

3.0 FIELDWORK AND LABORATORY TESTING

Following scope of work was carried out during the fieldwork on site:

- Review of Dial-Before-You-Dig (DBYD) plans and then service locating of drilling locations using an electromagnetic detecting equipment.
- Mechanical drilling of four (4) boreholes, identified as BH1 to BH4 inclusive using a drilling rig.
- Standard Penetration Testing (SPT) within the augering of boreholes.
- Installation of two (2) groundwater piezometers identified as GW1 and GW2 within the boreholes BH1 and BH2 respectively.
- Subsequent standing groundwater level measurement within the installed groundwater well.

The approximate locations of the boreholes, engineering logs of ground conditions and photograph of rock core sample are annexed as Appendix A, B and C respectively.

Laboratory testing of Point Load Index (PLI) was carried out on the recovered rock samples and result is annexed as Appendix D.

4.0 GROUND CONDITION

4.1 Ground Profile

Ground profiles encountered within the borehole and neighbour's borehole are summarised in Table 1. However, reference should be made to engineering logs, photograph of rock core sample and results of PLI tests for design and construction purpose.

Table 1: Summary of Ground Profile

Unit	Details	Depth (m)			
		BH1	BH2	BH3	BH4
Existing Ground Level (RL m AHD)		8.6	6.1	7.7	7.2
Fill	Silty Clay, low to medium plasticity, brown-dark brown, with some fine gravel, moist, firm	0.1 – 1.0	0.1 – 0.4	0.2 – 0.7	0.1 – 0.5
Residual Soils	Silty CLAY, medium to high plasticity, grey-pale brown-red, moist, stiff	1.0 – 3.3	0.4 – 6.0	0.7 – 6.0	0.5 – 5.0
Class V Shale[1]	SHALE, with some clay bands, extremely weathered, extremely low strength, grey-pale brown, moist	3.3 – 9.8	6.0 – 9.0	6.0 – 12.0	5.0 – 13.5

Note: [1] Bedrock was classified in accordance with the research paper of Pells P.J.N, Mostyn G. & Walker B.F. Foundations on Sandstone and Shale in the Sydney Region, Australian Geomechanics Journal, December 1998

4.2 Groundwater

No groundwater seepage was observed during the augering of the boreholes. However, a standing groundwater level was recorded at approximate depth of 4.8m (RL 3.8m AHD) within the installed piezometer GW1 (BH1) on 6th November 2019. The piezometer GW2 (BH2) was dry on that day.

Further, it should be noted groundwater levels within the site may be subject to seasonal fluctuations, rainfall, prevailing weather conditions and also future developments of the areas and land forms.

5.0 DISCUSSIONS AND RECOMMENDATIONS

5.1 Excavation Conditions

Bulk excavation for the proposed development is likely to comprise of fill, residual soils and shale bedrock ranging from extremely low to very low strength. Excavation therefore can be achieved using a conventional earthmoving equipment such as backhoes or tracked excavators. If any stronger bedrock bands encountered within that level and/or for excavation below the very low strength bedrock, a use of the rock breaking and ripping equipment is required. The rock breaking and ripping equipment will generate noise, vibration and dust during the excavation activities.

Prior to commencement of excavation, contractor should refer to the engineering logs, photograph of rock core sample and results of PLI tests to identify the strength of the bedrock. Followed by, assessment should be carried out by a qualified contractor to identify a suitable excavation method for the assessed bedrock materials. The ground profile summarised in Table 1 should be used for design of foundation system only.

5.2 Vibration Control

Vibration Monitoring Plan ('VMP') is recommended to be developed to monitor the potential vibration effects caused by demolition of existing structures and bulk excavation activities, on the neighbouring properties and road carriageways located along the site boundaries. It is recommended that a suitably qualified consultant (noise and vibration) is engaged to prepare a VMP and monitor excavation at and below the low strength bedrock level.

Table 2 summaries the typical recommendable Peak Particle Velocity ('PPV') for different types of the structures based on their sensitises, and inducted vibrations should not be exceeded throughout the construction stage.

Table 2: Typical Recommended Peak Particle Velocity

Type of the Structure	PPV (mm/sec)
Historical or structures in sensitive conditions	2
Residential and low-rise buildings	5
Brick or unreinforced structures in good condition	10
Commercial and industrial buildings or structures of reinforced concrete or steel construction	25

If required, monitoring can be carried out using a suitable vibration monitoring instruction attached with alarm, and appropriate PPV should be selected based on the condition of the subject structure. If vibration on the subject structure is exceed the selected PPV limit, construction activities should cease and the project Geotechnical Engineer should be contacted immediately for review of VMP. Preparation of VMP should constitute as 'Hold Point'.

5.3 Dilapidation survey

Dilapidation survey report on all structures and road carriageways located within the zone of influence (theoretical failure plan) is recommended to be carried out prior to commencement of construction. Preparation of dilapidation survey report should constitute as 'Hold Point'.

5.4 Underpinning of Existing Structures

Regardless of whether a suitable shoring system is adopted or not, structures located within the zone of influence (theoretical failure plan) should be assessed with requirement of underpinning by conducting inspections and assessment of their ground condition.

5.5 Retaining Walls

Temporary batter slops can only be considered in the boundaries, where neighbouring structures and road carriageways are location outside the zone of influence (theoretical failure plan) and sufficient space existed in between the site and excavation boundaries. Elsewhere suitable shoring system should be designed and constructed based on the ground condition recommended in this report.

Based on the provided information in Section 2 and ground condition encountered within the boreholes, temporary batter slops may not be considered suitable to all of the excavation boundaries. It is therefore recommended a shoring system consisted of a soldier pile wall system with concrete panels with piles spacing of at least 1.0-1.5m is considered suitable. Detailed retaining wall analysis should be carried out and closer spacing of the piles should be adopted based on the analysis to reduce earth movement and prevent collapse of infill materials. Piles should be socketed into a suitable stratum based on the detailed design and at least 2.0m below the bulk excavation level. Suitable drainage system should be installed behind the retaining wall system and should be discharged into site's stormwater system for long term purpose.

If there is groundwater, soldier pile wall system will cause excessive groundwater inflow into bulk excavation compare to contiguous pile wall systems. It is recommended to conduct a detailed analysis of site's groundwater condition before finalise the shoring wall system. Further it is recommended that suitable shoring wall system should be adapted not only to support the excavation boundaries, but also to limit the groundwater inflow into bulk excavation.

5.6 Temporary Anchoring System

A temporary anchoring system with at least two rows requires to provide a lateral support due to maximum retaining height and significant lateral movement. However, installation of anchoring system within the western excavation boundary is impractical due to existing neighbour's basement floor levels. Therefore, suitable alternative option is recommended below.

Anchors should be extended behind the active zone (theoretical failure plan) to allow an effective bonding with suitable bedrock. The basement floor slabs should be designed and constructed to provided permanent lateral support to the retaining wall system, and anchoring should consider as temporary option for construction stage. If anchoring system is impractical, consideration might be given to temporary support options such as installation of props associated with staged excavation or temporary berms in front of the wall with stage excavation.

Adoptable allowable bond stress for encountered bedrock layers are provided in Table 3 for designing of an anchoring system.

Table 3: Allowable Bond Stress for Anchoring

Ground Profile	Allowable Bond Stress (kPa)
Class V Shale	50

Adopted allowable bond stresses recommended above is typical recommendable bond stress. Therefore, anchors should be installed with bond length of at least 3.0m and proof tested with 1.3 times the design working load before they lock off at working load.

Inspection and monitoring of the construction of shoring piles and temporary anchoring system should be carried out under supervision of the project Geotechnical Engineer and should constitute as 'Hold Points'.

5.7 Retaining Walls Design

The proposed basement cut faces should be supported temporary during construction and in long term using appropriate retaining structures. These retaining structures should be designed to withstand the applied lateral pressures of the soil and rock strata, the existing surcharges in their zone of influence; including existing structures, and construction related activities, and water pressures if exists.

The pressure distribution on cantilever retaining structures may be assumed to be triangular and estimated as follows:

$$\boldsymbol{\rho_h = \gamma k H + q k}$$

Where,

ρ_h = Horizontal pressure (kN/m^2)
γ = Wet density (kN/m^3)
k = Coefficient of earth pressure (k_a or k_o)
H = Retained height (m)
q = Surcharge pressure behind retaining wall (kN/m^2)

Rectangular or trapezoidal pressure distribution may be considered for tied-back retaining system, as recommended in related standards and technical literature.

For the design of flexible retaining structures, where some lateral movement is acceptable, an active earth pressure coefficient is recommended. Should it be critical to limit the horizontal deformation of a retaining structure, use of an earth pressure coefficient at rest should be considered. Recommended parameters for the design of retaining structures are presented in the Table 4.

Table 4: Retaining Walls Design Parameters

Ground Profile	**Unit Weight (kN/m^3)**	**Effective Cohesion c' (kPa)**	**Angle of Friction ϕ(°)**	**Modulus of Elasticity E_{sh} (MPa)**
Fill	17	0	26	8
Residual Soils	20	5	24	15
Class V Shale	22	25	27	75

Preliminary coefficients of lateral earth pressure for the encountered ground profile is provided in Table 5. The coefficients provided are based on horizontal ground surface and fully drained conditions.

Table 5: Coefficient of Lateral Earth Pressure

Ground Profile	**Coefficient of Active Lateral Earth Pressure Ka**	**Coefficient of Active Lateral Earth Pressure at Rest Ko**	**Coefficient of Passive Lateral Earth Pressure Kp**
Fill	0.39	0.56	2.56
Residual Soils	0.42	0.59	2.37
Class V Shale	0.3	0.5	3.0

It should be noted that hydrostatic pressures due to ground water table (if present) and surcharge due to nearby structures (within the zone of influence) should also be taken into the account in the design of the retaining structures.

5.8 Groundwater Management

It should be noted groundwater conditions of a site might change with climate and development variations. Based on the encountered ground conditions, it is anticipated that groundwater is unlikely to be encountered during the bulk excavation.

If groundwater is encountered within the bulk excavation, soldier pile wall system may cause excessive groundwater ingress into the bulk excavation. Therefore, consideration might be given to contiguous pile wall system to control the groundwater ingress. Groundwater ingress in the contiguous pile wall system can be managed by implementing grading the surface and collect the seepage into a sump pits and then remove any excess water.

Whilst, if soldier pile wall system is adopted, then bulk excavation should be carried out in stages with excavation depth not exceeding 2.0m in each stage. Each stage of the bulk excavation should be completed with seal off defects in the bedrock to control the groundwater ingress, installation of the strip drains system and concrete infill panel before continue to next stage.

It is recommended to carry out further investigation to assess the presence of groundwater and selection of suitable dewater system within the bulk excavation, before finalise the shoring system. It is recommended to carry out seepage recharge rate assessment during the further investigation.

A groundwater modelling using a commercial software is advisable to carried out to the proposed shoring system during the construction stage. Based on the modelling, groundwater impact to the neighbouring infrastructures should be assessed to ensure that the impact is within the acceptable limits. It is recommended to implement a Groundwater Management Plan (GMP) during the construction stage. Continues inspection and monitoring of the GMP should be required throughout the construction and should constitute as “Hold Points”.

Design and construction of the development should be adopted waterproofing of the basement floor and walls with installation of adequate drainage system. Appropriate site drainage should be designed and installed during the construction to collect and remove groundwater into the public stormwater system for the design life of the proposed development. Drainage system should be constructed in accordance with local council specification.

5.9 Foundations

The foundation level of the proposed development is anticipated to be within shale ground profiles. It is therefore considered footing system are likely to be reinforced concrete thickened raft slab with pad and strip footings placed at a minimum depth of 0.5m below the bulk excavation level.

Table 6 provides design parameters recommended for shallow and piled foundations.

Table 6: Foundation Design Parameters

Ground Profile	Allowable End Bearing Pressure (kPa)	Allowable Shaft Adhesion Compression (kPa)
Fill	N/A	N/A
Residual Soils	300	N/A
Class V Shale	700	30

Note:

- With a minimum embedment depth of 0.5m for deep foundations and 0.4m for shallow foundations.
- Clean rock socket of roughness of at least grooves of depth 1mm to 4mm and width greater than 5mm at spacing of 50mm to 200mm.
- Shaft Adhesion in Tension is 50% of Compression, applicable to piles only.

It is recommended to carried out detailed geotechnical modelling of raft foundation to determine the magnitude and distribution of settlement occurring under working load. Where additional bearing pressure is required and/or excessive settlement is occurred, consideration should be given to adoption of pile foundation socking into stronger stratum.

Piles will also be used to increase the resistance against the lateral seismic and wind loads. Shallow and pile foundation can be designed in accordance with Australian Standards AS2870-2011 and AS2159-2009, respectively.

It is recommended that all footings are to be founded on the same stratum to minimise and avoid potential future differential settlement. Detail design of the foundation system should be carried out by the project structural engineering and reviewed by project geotechnical engineer.

A qualified geotechnical engineer should inspect the footing excavations to confirm appropriate founding materials, and to ensure the serviceability bearing pressures could be met. Footing excavations should be cleaned and wet and debris should be removed prior to the concrete placement. Verification of the capacity of the shallow and pile foundations by inspections would be required and inspections should constitute as ‘Hold Points’.

5.10 Site Earthquake Classification

Based on the ground condition and in accordance with Australian Standard AS 1170.4-2007, the site may be classified as "Shallow soil site" (Class Ce) for design of foundations and retaining walls embedded in the underlying bedrock. The Hazard Factor (Z) is considered to be 0.08.

5.11 Earthwork and Subgrade Preparation

Earthwork and subgrade preparation should be carried out in accordance with local council specification and Australian Standard 3798-2007. However, a general procedure is provided below for the development areas:

- Fill materials or topsoils or unsuitable materials should be removed from the site.
- Excavated materials for reuse as engineering fill or remove to spoil should be stockpiled separately.
- Exposed surface after excavation should be treated to adjust its moisture condition to not to vary more than 2% from its Optimum Moisture Content (OMC) and then compacted using at least 12tonnes vibrating compacter to design density ratio.
- Exposed surface should be tested with appropriate density testing and proof rolling with a smooth drum roller.
- Soft or loose areas should be excavated, treated with moisture and then recompacted or replaced with appropriate imported fill material.

Final surface of the cut areas and every layer of the fill areas should be treated with moisture and compacted to design parameters in order to achieve the adequate strength for the proposed development. The general recommendation for the compaction of fill layers is listed below:

- Moisture content of fill materials should be treated to ±2% of OMC of the material.
- Minimum density ratio of 98% of the maximum dry density for the proposed development area.
- Placement of loose thickness of fill layers should not exceed 200mm during the compaction.

General recommendation for suitability of imported materials for the fill layers are provided below:

- The materials should be clean (i.e. free of contaminants, deleterious or organic material), free of inclusions of >120mm in size.
- Material with excessive moisture content should not be used.
- The materials should satisfy the Australian Standard AS 3798-2007.

5.12 Road and Martine Services (RMS) Requirement

Due to bulk excavation within the vicinity of the Parramatta Road, detailed geotechnical design should be complied with RMS “Technical Direction – Geotechnology GTD2012/001”.

For example, adoption of Life Traffic Load during the calculation of surcharge, Groundwater Management, Ground Deformation and Retaining Wall Deflection Analysis, Anchoring System and Ground Movement Monitoring of excavation wall within the vicinity of Parramatta Road, but not limited, should be complied with “RMS Technical Direction – Geotechnology GTD2012/001”.

It is therefore recommended to incorporate the “RMS Technical Direction – Geotechnology GTD2012/001” during the design and constitute stages and should constitute as ‘Hold Point’.

5.13 Additional Geotechnical Investigation

Prior to commencement of construction stage, drilling of additional appropriate number of boreholes is recommended to undertake in order to confirm the recommendations presented in this report. The geotechnical investigation should be undertaken in accordance with Australian Standard AS 1726-1993 by a Geotechnical Engineer familiar with the contents of this report

6.0 CONCLUSIONS

This report presents the findings of the geotechnical investigation and recommendations for the proposed development at 52-54 Powell Street and 125 Parramatta Road, Homebush NSW 2140. It considers that the proposed development is feasible in this site if the recommendations provided in this report are considered in design and construction of this development.

For and on behalf of

Benviron Group

Murali Muralitharan
Senior Geotechnical Engineer

Reviewed by

Ben Buckley

Ben Buckley
Director

LIMITATIONS

The assessment of the sub-surface profile within the proposed development area and the recommendations presented in this report are based on limited information available to date.

The recommendations and advice presented in this report on soil and rock condition is considered to be indicative only as only very limited areas were assessed on site to date. Site inspection by a consulting Geotechnical Engineer or Engineering Geologist are to be undertake when further investigation works are to be carried out to confirm the condition of founding materials in which this geotechnical assessment recommends.

Anecdotal evidence and Information provided by client is assumed to be relevant and to the best of knowledge be appropriate for its interpretation.

There is a possibility that the actual geotechnical and groundwater conditions across the site could differ from the inferred geotechnical assumptions and derivations on which our recommendations are presented in this report. In that case, Benviron Group should be contacted for further advise and review of the information provided in this report. Benviron Group does not accept any liabilities for the conditions not provided and/or accessible during the preparation of this report. Any ensuring liability resulting from use of this report by third parties cannot be transferred to Benviron Group.

REFERENCES

1. Australian Standard – AS1726-1993 'Geotechnical Site Investigation.
2. Australian Standard – AS 1170.4-2007 Structural Design Actions – Part 4: Earthquake actions in Australia
3. Australian Standard – AS 2870-2011 Residential slabs and footings.
4. Australian Standard – AS 2159-2009 Piling - Design and installation.
5. Pells, P.J.N, Mostyn, E and Walker , B F – Foundations on Sandstone and Shale in the Sydney Region, Australian Geomechanics Journal, Dec 1998.
6. Pells, P.J.N, Douglas D.J, Rodway, B, Thorne C, McManon B.K – Design Loadings for Foundations on Shale and Sandstone in the Sydney Region. Australian Geomechanics Journal, 1978.
7. Road and Martine Services (RMS) "Technical Direction – Geotechnology GTD2012/001"

Appendix A

Site Plan

Key

● Approximate Borehole Locations

Not for scale

Benviron group
simple sustainable solutions

DRAWN	MM
FIGURE	1
Job #	G386

Site Plan

Amna Holdings Pty Ltd

52-54 Powell Street and 125 Parramatta Road, Homebush NSW 2140

Appendix B

Engineering Borehole Logs

BOREHOLE : BH1

PAGE 1 OF 2

CLIENT NAME: Amna Holdings Pty Ltd

JOB NUMBER: G386

SITE ADDRESS: 52-54 Powell Street and 125 Parramatta Road, Homebush NSW 2140

PROJECT: Geotechnical Investigation

Date Started : 31/10/2019

Completed : 31/10/2019

Logged By : RL

Checked By : MM

Borehole Location : Refer to Site Plan

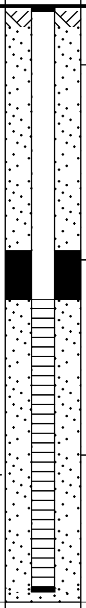
Surface RL : 8.6

Datum : m AHD

Equipment : Drilling Rig

Borehole Size : 100mm

Slope : -90°



Method	Water	Well Details	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Moisture	Consistence	Samples Tests Remarks	Additional Observations
ADT	▼ At 4.8m on 6/11/2019						CONCRETE, approximately 100mm thick				Pavement
			8.5	0.10			FILL, silty clay, low to medium plasticity, brown-dark brown, with some fine gravel	M	F		Fill
			7.6	1.00		CH	Silty CLAY, medium to high plasticity, grey-pale brown-red	M	St		Residual Soils
			5.3	3.30			SHALE, extremely weathered, extremely low strength, grey-pale brown, with some clay bands	M			Bedrock
			2.5	6.10			Borehole BH1 continued as cored hole from 6.10m				

Comments:

D - Dry
M - Moist
W - Wet

VS - Very Soft
S - Soft
F - Firm
St - Stiff
VSt - Very Stiff
H - Hard

VL - Very Loose
L - Loose
MD - Medium Dense
D - Dense
VD - Very Dense

BOREHOLE : BH1

CLIENT NAME: Amna Holdings Pty Ltd
JOB NUMBER: G386
SITE ADDRESS: 52-54 Powell Street and 125 Parramatta Road, Homebush NSW 2140
PROJECT: Geotechnical Investigation

Date Started : 31/10/2019
Completed : 31/10/2019
Logged By : RL
Checked By : MM
Borehole Location : Refer to Site Plan
Surface RL : 8.6
Datum : m AHD
Equipment : Drilling Rig
Borehole Size : 100mm
Slope : -90°

Method	Water	Well Details	RL (m)	Depth (m)	Graphic Log	Material Description	Weathering	Estimated Strength (EL VL L M H VH EH)	$Is_{(50)}$ (MPa)	RQD %	Defect Spacing (mm) (20 60 180 540 1620)	Defect Description	Depth (m)
				2									2
				4									4
				6		Continued from non-cored borehole							6
NMLC			2.5	6.10		SHALE, grey-pale brown, with some clay bands	EW		D A 0 0.07	0			
				8						0			8
									D A 0 0.02	0			
			-0.9	9.50		becoming grey-light grey laminations	EW/HW		D A 0.17 0.23			9.50m, FZ, 40mm 9.58m, CS, 10mm 9.64m, J, P, S, 15-20 deg 9.69m, EW, 10mm 9.73m, EW, 20mm 9.76m, J, P, S, 85-90 deg 9.79m, EW, 10mm	
			-1.2	9.80		BH1 terminated at 9.80m							
				10									10
				12									12
				14									14

EL VL L M H VH EH
20 60 180 540 1620 4860

Comments:

Weathering
EW - Extremely
HW - Highly
MW - Moderately
SW - Slightly
Fr - Fresh

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

D - Diametral
A Axial

J - Joint
B - Bedding Plan
CS - Clay Seams
FZ - Fractured Zone
IS - Infill Seam
SS - Sheared Seam
CZ - Crushed Zone

MB - Mechanical Break
HB - Handling Break
Pl - Planar
Ir - Irregular
Cu - Curved
St - Stepped

S - Smooth
R - Rough
P - Polished
Qz - Quartz
Fe - Iron Stain

BOREHOLE : BH2

PAGE 1 OF 1

CLIENT NAME: Amna Holdings Pty Ltd

JOB NUMBER: G386

SITE ADDRESS: 52-54 Powell Street and 125 Parramatta Road, Homebush NSW 2140

PROJECT: Geotechnical Investigation

Date Started : 1/11/2019

Completed : 1/11/2019

Logged By : RL

Checked By : MM

Borehole Location : Refer to Site Plan

Surface RL : 6.1

Datum : m AHD

Equipment : Drilling Rig

Borehole Size : 100mm

Slope : -90°

Method	Water	RL (m)	Depth (m)	Classification Symbol	Material Description	Moisture	Consistence	Samples Tests Remarks	Additional Observations
ADT	Well was dry on 6/11/2019				CONCRETE, approximately 100mm thick				Pavement
		6.0	0.10		FILL, silty clay, low to medium plasticity, dark brown, with some fine gravel	M	F		Fill
		5.7	0.40	CH	Silty CLAY, medium to high plasticity, grey-pale brown-red	M	St	SPT 6, 7, 12 N=19	Residual Soils
		-0.1	6.00		SHALE, extremely weathered, extremely low strength, dark grey, with some clay bands	M		SPT Bouncing N>30	Bedrock
		-2.9	9.00		Borehole BH2 terminated at 9.00m				

Comments:

D - Dry
M - Moist
W - Wet

VS - Very Soft
S - Soft
F - Firm
St - Stiff
VSt - Very Stiff
H - Hard

VL - Very Loose
L - Loose
MD - Medium Dense
D - Dense
VD - Very Dense

BOREHOLE : BH3

PAGE 1 OF 1

CLIENT NAME: Amna Holdings Pty Ltd

JOB NUMBER: G386

SITE ADDRESS: 52-54 Powell Street and 125 Parramatta Road, Homebush NSW 2140

PROJECT: Geotechnical Investigation

Date Started : 1/11/2019

Completed : 1/11/2019

Logged By : RL

Checked By : MM

Borehole Location : Refer to Site Plan

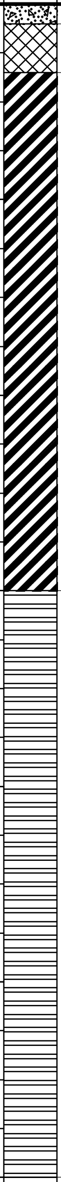
Surface RL : 7.7

Datum : m AHD

Equipment : Drilling Rig

Borehole Size : 100mm

Slope : -90°

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Moisture	Consistence	Samples Tests Remarks	Additional Observations	Depth (m)
ADT	Not observed during the augering					CONCRETE, approximately 200mm thick				Pavement	
		7.5	*0.20*			FILL, gravelly silt, low plasticity, medium to course gravel, dark brown	M	F		Fill	
		7.0	*0.70*		CH	Silty CLAY, medium to high plasticity, grey-pale brown-red	M	St		Residual Soils	
			2								2
									SPT 6, 10, 10 N=20		
			4								4
			6								6
		1.7	*6.00*			SHALE, extremely weathered, extremely low strength, dark grey, with some clay bands	M		SPT Bouncing N>30	Bedrock	
			8								8
			10								10
			12								12
		-4.3	*12.00*			Borehole BH3 terminated at 12.00m					
			14								14

Comments:

D - Dry
M - Moist
W - Wet

VS - Very Soft
S - Soft
F - Firm
St - Stiff
VSt - Very Stiff
H - Hard

VL - Very Loose
L - Loose
MD - Medium Dense
D - Dense
VD - Very Dense

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BOREHOLE : BH4

PAGE 1 OF 1

CLIENT NAME: Amna Holdings Pty Ltd
JOB NUMBER: G386
SITE ADDRESS: 52-54 Powell Street and 125 Parramatta Road, Homebush NSW 2140
PROJECT: Geotechnical Investigation

Date Started : 1/11/2019
Completed : 1/11/2019
Logged By : RL
Checked By : MM
Borehole Location : Refer to Site Plan
Surface RL : 7.2
Datum : m AHD
Equipment : Drilling Rig
Borehole Size : 100mm
Slope : -90°

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Moisture	Consistence	Samples Tests Remarks	Additional Observations	Depth (m)
ADT	Not observed during the augering					CONCRETE, approximately 100mm thick				Pavement	
		7.1	0.10			FILL, silty clay, low to medium plasticity, dark brown, with some fine gravel	M	F		Fill	
		6.7	0.50		CH	Silty CLAY, medium to high plasticity, grey-pale brown-red	M	St		Residual Soils	
									SPT 5, 9, 11 N=20		
			2								2
									SPT 3, 11, 18 N=29		
			4								4
									SPT 5, 15, 28 N=43		
		2.2	5.00			SHALE, extremely weathered, extremely low strength, grey-pale brown, with some clay bands	M			Bedrock	
			6								6
									SPT Bouncing N>30		
			8								8
			10								10
			12								12
		-6.3	13.50			Borehole BH4 terminated at 13.50m					
			14								14

Comments:

D - Dry
M - Moist
W - Wet

VS - Very Soft
S - Soft
F - Firm
St - Stiff
VSt - Very Stiff
H - Hard

VL - Very Loose
L - Loose
MD - Medium Dense
D - Dense
VD - Very Dense

Appendix C

Photograph of Core Samples

G386 Borehole BH1
Depth: 6.1m – 9.8m
Date: 31/10/2019

Appendix D

Results of Laboratory Testing

Results of Point Load Index Tests

Client:	Amna Holdings Pty Ltd	**Ref No:**	G386
Project:	Geotechnical Investigation	**Test Method:**	AS 4133.4.1
Location:	52-54 Powell St & 125 Parramatta Rd, Homebush NSW 2140	**Test Date:**	4/11/2019

Borehole	Depth (m)	Materials Description	Moisture Content	I_{s50} (MPa)	
				Diametral	Axial
BH1	6.44	Shale, grey-pale brown	Moist	0.00	0.07
BH1	8.25	Shale, grey-pale brown	Moist	0.00	0.02
BH1	9.63	Shale, grey-light grey	Moist	0.17	0.23