Geotechnical Consultants Australia

Hills Awqaf Pty Ltd

Geotechnical Investigation Report

Proposed Development at:
1 Larapinta Place
Glenhaven NSW 2156

G18202-2

26th November 2018

Report Distribution

Geotechnical Investigation Report

Address: 1 Larapinta Place Glenhaven NSW 2156

GCA Report No.:	G18202-2
Date:	26th November 2018

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Report Revision	Details	Date	Amended By
0	Original Report	26th November 2018	-
Issued By:		**Joe Nader**	

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1. INTRODUCTION

1.1 Background

This report presents the results of a geotechnical investigation undertaken by Geotechnical Consultants Australia Pty Ltd (GCA) for a proposed development at No. 1 Larapinta Place Glenhaven NSW 2156 (the site). The investigation was commissioned by Mr. Sohail Shamsi of Iconfm, on behalf of Hills Awqaf Pty Ltd. The investigation was carried out on the 20th November 2018, on the basis of a proposal provided by GCA to the client, referenced P1168-18.1, and dated 15th November 2018.

The purpose of the investigation was to assess the subsurface conditions over the site, and provide necessary recommendations from a geotechnical perspective for the proposed development.

The findings presented in this report are based on our subsurface investigation and our experience with subsurface conditions in the area. This report presents our assessment of the geotechnical conditions, and has been prepared to provide advice and recommendations to assist in the preparation of designs and construction of the ground structures for the proposed development.

For your review, **Appendix A** contains a document prepared by GCA entitled "Important Information About Your Geotechnical Report", which summarises the general limitations, responsibilities, and use of geotechnical reports.

1.2 Proposed Development

Information provided by the client indicates the proposed development comprises demolition of the existing dwelling and infrastructures within the southern portion of the site, followed by the construction of a two (2) storey Mosque and associated car parking area to the rear, overlying a single basement level. Access to the proposed car parking area and basement will be via an entry from Larapinta Place along the site western boundary.

The Finished Floor Level's (FFL)'s of the proposed basement and ground floor level are set to be at a Reduced Level (RL) of 99.650m and RL102.650m Australian Height Datum (AHD), respectively. The car parking area is expected to be at similar levels to the existing site levels within the proposed development area.

Based on this information and existing site topography and levels, maximum excavation depths of approximately 0.8m within the rear portion of the proposed building, gradually increasing to approximately 3.7m within the front portion of the proposed building (varying throughout) are expected for construction of the proposed basement. Locally deeper excavation for the proposed lift shafts, footings and service trenches are also expected to be required as part of the proposed development.

It should be noted that excavation depths are expected to vary across the site, and have been inferred based off existing site levels shown on the site survey plan attached to the preliminary architectural drawings and proposed basement FFL.

1.3 Provided Information

The following relevant information was provided to GCA prior to the site investigation:

- Preliminary architectural drawings prepared by iDraft Architects, titled project "Proposed Mosque", referenced job No. 28569, included drawing nos. 1001, and 1003 to 1005 inclusive, and dated 15th November 2018.

1.4 Geotechnical Assessment Objectives

The objective of the geotechnical investigation was to assess the site surface and subsurface conditions at the testing locations, and to provide professional advice and recommendations on the following:

- General assessment of any potential geotechnical issues that may affect any surrounding infrastructures, buildings, council assets, etc., along with the proposed development.
- Excavation conditions and recommendations on excavation methods in soils and rocks, to restrict any ground vibrations.
- Recommendations on suitable shoring systems for the site.
- Design parameters based on the ground conditions within the site, for retaining walls, cantilever shoring walls and propped shoring.
- Recommendations on suitable foundation types and design for the site.
- End bearing capacities and shaft adhesion for shallow and deep foundations based on the ground conditions within the site (for ultimate limit state and serviceability loads).
- Groundwater levels which may be determined during the site investigation, along with the effects on the proposed development construction.
- Recommendations on groundwater maintenance and limiting (if required).
- Preliminary subsoil class for earthquake design for the site in accordance with Australian Standards (AS) 1170.4-2007.
- Preliminary site lot classification in accordance with AS 2870-2011.

1.5 Scope of Works

Fieldwork for the geotechnical investigation was undertaken by an experienced geotechnical engineer, following in general the guidelines outlined in AS 1726-2017. The scope of works included:

- Submit and review Dial Before You Dig (DBYD) plans, and any other plans provided by the client of existing buried services on the site.
- Service locating carried out using electromagnetic detection equipment to ensure the area is free of any underground services at the selected borehole and test locations.
- Review of site plans and drawings to determine testing locations, and identify any relevant features of the site.
- Machine drilling of six (6) borehole at selected locations within the site (where accessible) by a specialised trailer mounted drilling rig, using solid flight augers equipped with a Tungsten Carbide (TC) bit, and identified as boreholes BH1 to BH6 inclusive.
    - Boreholes BH1 to BH6 inclusive were all drilled to TC bit refusal depths of approximately 1.0m to 1.5m below existing ground level (bgl).
    - It should be noted that boreholes BH1 to BH5 inclusive were carried out within the proposed development area, whilst borehole BH6 was carried out to the rear of the proposed development.
        - The approximate locations of the boreholes are shown on **Figure 1, Appendix B** of this report.
- Collection of soil and rock samples during drilling for any laboratory testing which may be required.
- Reinstatement of the boreholes with available soil displaced during drilling.
- Preparation of this geotechnical report.

1.6 Constraints

The discussions and recommendations provided in this report have been based on the results obtained during testing at the locations of the boreholes locations (where accessible). It is recommended that geotechnical inspections are carried out on the proposed developments foundation system during construction, to confirm the subsurface conditions, and design bearing capacities are achieved.

Consideration should also be given to additional machine drilled boreholes and rock strength testing carried out to confirm the ground conditions, and to help assist in final designs of the proposed development. This recommendation should be confirmed by the project geotechnical engineer and structural engineer during/following design stages of the proposed development.

2. SITE DESCRIPTION

2.1 Overall Site Description

The overall site description and its surrounding are presented in Table 1 below.

Table 1. Overall Site Description and Site Surroundings

Information	Details
Overall Site Location	The site is an irregular shaped land and located within a residential and rural area at the intersection of Glenhaven Road and Larapinta Place, approximately 3.0km west of the Old Northern Road carriageway.
Site Address	1 Larapinta Place Glenhaven NSW 2156
Approximate Site Area[1]	2.04ha – based off NSW Six Maps.
Local Government Authority	The Hills Shire Council
Site and Investigation Area Description	At the time of the investigation, a residential dwelling and attached awning, accompanied by associated concrete pavements and walkways was present within the front portion of the site. A detached shed was also present within the middle portion of the site, along the site western boundary, with the remaining site area being covered in well-maintained grass, vegetation and a mature trees scattered throughout, predominately within the middle to rear portion of the site. It should be noted that access during the site investigation to the rear half portion of the site was not feasible due to the presence of dense vegetation and mature trees.
Approximate Distances to Nearest Watercourses (i.e. rivers, lakes, etc.)	• Dooral Dooral Creek – intersecting through portion of the rear of the site, and approximately 217m north of the site. • Cattai Creek – 170m south of the site.
Site Surroundings	The site is located within an area of rural and residential use, and is bounded by: • Residential/rural lot at No. 2 Larapinta Place to the north. • Residential/rural lot at No. 121 Glenhaven Road to the east. • Glenhaven Road carriageway to the south. • Larapinta Place road reserve to the west.

[1]Site area is approximated and based off NSW Six Maps - https://maps.six.nsw.gov.au/.

2.2 Topography

The local topography surrounding the site generally falls towards the north to north-east, and towards the north-west. The overall site topography generally also gently slopes towards the north to north-east. It should be noted that the site levels and topography are approximated off the site survey plan attached to the preliminary architectural drawings and are expected vary across the site. It should also be noted that the site survey plan is limited to approximate "tree line" within the site, extending to approximately half the site area.

2.3 Regional Geology

Information obtained on the local regional subsurface conditions, referenced from the Department of Mineral Resources, Penrith 1:100,000 Geological Series Sheet 9030 Edition 1, dated 1991, by the Geological Survey of New South Wales, indicates the site is situated within a geological region underlain by Triassic Aged Hawkesbury Sandstone (Rh). The Hawkesbury Sandstone typically comprises "medium to very coarse grained quartz sandstone, minor laminated mudstone and siltstone lenses".

3. SUBSURFACE CONDITIONS AND ASSESSMENT RESULTS

3.1 Stratigraphy

A summary of the surface and subsurface conditions from across the site are summarised in Table 2 below, and are interpreted from the assessment results. It should be noted that for Table 2 presents a summary of the overall site conditions, and reference should be made to the detailed engineering borehole logs presented in **Appendix D**, in conjunction with the geotechnical explanatory notes detailed in **Appendix C**. Rock description has been based on Pells P.J.N, Mostyn G. & Walker B.F. Foundations on Sandstone and Shale in the Sydney Region, Australian Geomechanics Journal, December 1998.

Due to the variable ground conditions throughout the site, it is recommended that confirmation of the subsurface materials be carried out during construction, or by additional boreholes and rock strength testing. It should also be noted that ground conditions within the site are expected to differ from those encountered and inferred in this report, since no geotechnical or geological exploration programme, no matter how comprehensive, can reveal and identify all subsurface conditions underlying the site.

Based on the site investigation, along with our experience and observations made within the local region, it is inferred that shale bedrock is underlying the majority of the proposed development area at relatively shallower depths varying from approximately 0.1m to 0.7m (varying and possibly greater), and is inferred to vary across the site. Residual soils are also expected to vary across the site with variable composition and consistency, predominately at locations and depths not observed during the site investigation.

Sandstone outcrops were also observed throughout the site, predominately within the middle to rear portion of the site. Sandstone outcrops were visually assessed to be generally extremely to highly weathered, low to medium estimated strength.

Table 2. Summary of Subsurface Conditions

		Borehole ID	**BH1**	**BH2**	**BH3**	**BH4**	**BH5**	**BH6**
Unit	**Unit Type**	**Description**	**Depth/Thickness of Unit (m)**					
		Proposed Basement FFL (m AHD)	RL99.650					
		Approximate Maximum Excavation at Borehole (m)	3.2	2.3	N/A	2.3	N/A	N/A
		Approximate RL Top of Borehole (m AHD)	RL102.8	RL101.9	RL97.7	RL101.9	RL97.2	N/A
1	Fill	Clayey SAND, fine grained, low plasticity clay, with gravel.	0.0 – 0.3	0.0 – 0.1	0.0 – 0.1	–	0.0 – 0.1	0.0 – 0.1
		Silty SAND, fine grained, with gravel.	–	–	–	0.0 – 0.1	–	–
2	Residual Soils[1]	Clayey SAND, fine grained, low plasticity clay, with gravel.	0.3 – 0.7	0.1 – 0.4	0.1 – 0.5	–	–	–
		Sandy CLAY, medium plasticity, fine grained sand, with gravel.	–	–	–	–	0.1 – 0.6	0.1 – 0.7
3	Sandstone Bedrock[2]	SANDSTONE, fine grained, some clay, EW, EL, grading to VL.	0.7 – 1.5	0.4 – 1.0	0.5 – 1.2	0.1 – 1.3	0.6 – 1.2	0.7 – 1.3

[1]Strength and consistency of underlying residual soils are expected to vary across the site. The potential for weak or softer layers throughout the unit should be considered. Consideration should be given to additional boreholes and testing carried out prior to construction.

[2]Higher strength or class bedrock (low estimated strength) may be present below the auger termination depths as indicated in Table 2 based on observations made during auger penetration resistance at the time of drilling. Confirmation of the actual depth and thickness of the underlying sandstone bedrock should be carried out by a geotechnical engineer by additional borehole drilling, or during construction. Ground conditions are expected to vary across the site, and should be confirmed by a geotechnical engineer, predominately in areas unobserved during the site investigation.

Notes:

- Rock strengths are based on observations made during auger penetration resistance at the time of drilling. Ground conditions are expected to vary across the site, and should be confirmed by a geotechnical engineer, predominately in areas unobserved during the site investigation.
- EW = Extremely Weathered, EL = Extremely Low Estimated Strength, VL = Very Low Estimated Strength.

Table 3 below represents approximate RL's to the top of each unit encountered during the site investigation at the borehole locations.

Table 3. Approximate Reduced Level's Top of Units

		Borehole ID	**BH1**	**BH2**	**BH3**	**BH4**	**BH5**	**BH6**
Unit	**Unit Type**		**Approximate RL Top of Unit[1] (RL m AHD)**					
1	Fill		102.8	101.9	97.7	101.9	97.2	N/A
2	Residual Soils		102.5	101.8	97.6	–	97.1	
3	Sandstone Bedrock[2]	EL – VL	102.1	101.5	97.2	101.8	96.6	
		L[3]	101.3	100.9	96.5	100.6	96	

[1]RL's are approximate and based off the site survey plan referenced in Section 1.3, and depths during drilling.

[2]Confirmation of the actual depth and thickness of the underlying sandstone bedrock should be carried out by a geotechnical engineer by additional borehole drilling, or during construction.

[3]Higher strength or class bedrock (low estimated strength) is inferred to be at depths indicated in Table 3 based on observations made during auger penetration resistance at the time of drilling. Ground conditions are expected to vary across the site, and should be confirmed by a geotechnical engineer, predominately in areas unobserved during the site investigation. Confirmation of

the underlying inferred sandstone bedrock strength and continuity should be made during construction, or by additional borehole drilling and testing.

Notes:

- Rock strengths are based on observations made during auger penetration resistance at the time of drilling. Ground conditions are expected to vary across the site, and should be confirmed by a geotechnical engineer, predominately in areas unobserved during the site investigation.
- EL = Extremely Low Estimated Strength, VL = Very Low Estimated Strength, L = Low Estimated Strength.

3.2 Groundwater

No groundwater was encountered or observed during drilling of boreholes BH1 to BH6 inclusive to maximum depths of approximately 1.5m in borehole BH1 or to approximately RL96m AHD in borehole BH5. It is noted that boreholes BH1 to BH6 inclusive were immediately backfilled following completion of augering which precluded longer term monitoring of groundwater levels.

Groundwater which may be present within the site is expected to be in the form of seepage through the voids within the underlying fill material, and through the pore spaces between particles of unconsolidated natural soils or through networks of fractures and solution openings in consolidated bedrock underlying the site. Although no groundwater was encountered or observed at the majority of the test locations during the site investigation, its presence should not be precluded.

It should be noted that groundwater levels have the potential to elevate during daily or seasonal influences such as tidal fluctuations, heavy rainfall, damaged services, flooding, etc., and moisture content within soils may be influenced by events within the site and adjoining properties.

4. GEOTECHNICAL ASSESSMENT AND RECOMMENDATIONS

4.1 Dilapidation Survey

It is recommended that prior to demolition, excavation and construction, a detailed dilapidation survey be carried out on all adjacent buildings, structures, council assets, road reserves and infrastructures that fall within the "zone of influence" of the proposed excavation and within the vicinity of the proposed development. A dilapidation survey will record the condition of existing defects prior to any works being carried out. Preparation of a dilapidation report should constitute as a "Hold Point".

4.2 General Geotechnical Issues

The following aspects have been considered main geotechnical issues for the proposed development:

- Preliminary site lot classification.
- Excavation conditions.
- Groundwater management.
- Stability of basement excavation and retention of adjoining properties and infrastructure.
- Foundations.
- Preliminary site earthquake classification.

Based on results of our assessment, a summary of the geotechnical aspects above and recommendations for construction and designs are presented below.

4.3 Preliminary Site Lot Classification.

Based on the geotechnical investigation and observations made during the site investigation, fill and residual soils are expected to be underlain by sandstone bedrock of variable strength and weathering at relatively shallower depths across the site varying from approximately 0.1m to 0.7m (varying and possibly greater), with sandstone outcrops exposed in certain areas across the site. Due to the site and subsurface conditions, no laboratory testing was carried out on any natural soils present underlying the proposed development area.

The governing site lot classification in accordance with AS 2870-2011 has been identified as **"Class P" (Problematic Site)** for the overall site, due to the presence of mature trees and existing infrastructures within the site causing abnormal and changing moisture conditions.

AS 2870-2011 indicates the site may be classified as a **"Class A"** site, for design and construction of the foundation system founded below any natural soils, topsoil, slopewash, fill or other deleterious material, being on the sandstone bedrock underlying the proposed development area (subject to confirmation). Foundation design and construction should be carried out as outlined in Section 4.9 below, with reference made to AS 2870-2011. Geotechnical inspections and confirmation of the actual depth of underlying soils and sandstone bedrock should be made prior to construction by additional borehole drilling and rock strength testing, or during construction by a geotechnical engineer.

Where ground conditions vary from those outlined at the test locations, and confirmation of the actual depth of underlying soils and sandstone bedrock has not been carried out by a geotechnical engineer as outlined in this report, and where the building foundations are not proposed to be constructed on the sandstone bedrock underlying the site, GCA should be contacted immediately, and the building foundations be designed and constructed as a "Class P" site.

Footing designs should take into consideration the effect of recent removal and planting of trees, along with any future tree removal within the vicinity of the proposed development on soil moisture conditions. Sufficient time should be given for soil moisture to re-equilibrate following any removal or planting of trees within the proposed development area, or specific engineering assessment and design will be required on the foundation design.

Although trees and vegetation are considered to contribute to the stability of the site, we recommend that planting of trees around the development area (i.e. in close proximity to the proposed building foundations) be limited as they can also affect moisture changes within the soil and cause significant displacement/damage within the building foundations by extensive tree root system movement.

Based on the site lot classification outlined above, it is recommended that reference is made to the recommendations provided by CSIRO "Guide to Home Owners on Foundation Maintenance and Footing Performance", attached as **Appendix E.**

4.4 Inspection Pits and Underpinning

Consideration should be given to inspection pits carried out for the existing adjacent buildings and infrastructures, particularly where they fall within the "zone of influence" (obtained by drawing a line 45º above horizontal from the base of the proposed basement walls) of the proposed development. This should be carried out prior to any demolition or excavation, and will provide an assessment of the existing foundations of the adjacent buildings.

The assessment of the adjacent building footings should include assessment of the underlying soil, which will determine the need for additional support, such as underpinning, prior to installation of shoring piles and excavation.

4.5 Excavation

Maximum excavation depths of approximately 0.8m to 3.7m (varying throughout) are expected for construction of the proposed basement, and locally deeper excavations to be required for the proposed lift shafts, footings and service trenches.

Based on this information and existing ground conditions as encountered during the site investigation, it is anticipated that excavation will extend through Unit 1 (fill) to Unit 3 (sandstone bedrock) of variable strength and weathering throughout the majority of the proposed development area, as outlined in Table 2 and Table 3 above. The possibility for encountering higher strength sandstone bedrock (i.e. medium estimated strength or better), should not be precluded due to the limited investigation carried out within the site.

4.5.1 Excavation Assessment

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.

Where required, excavation of medium to higher strength bedrock (if encountered) 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 lift shaft, 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 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. Excavation will generate both vibration and noise whilst being carried out within the bedrock. Vibration control measures should be implemented as part of the excavation process.

4.6 Vibration Monitoring and Controls

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

- Sensitive or historical structures – **2mm/sec**
- Residential and low rise structures – **5mm/sec**
- Unreinforced or brick structures – **10mm/sec**
- Reinforced or steel structures and/or commercial/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 4 below.

Table 4. Rock Breaking Equipment Recommendations

Distance From Adjoining Structures (m)	Maximum PPV 5mm/sec		Maximum PPV 10mm/sec[1]	
	Equipment	Operating Limit (Maximum Capacity %)	Equipment	Operating Limit (Maximum Capacity %)
1.5 to 2.5	Jack Hammer Only (hand operated)	100	300kg Rock Hammer	50
2.5 to 5.0	300kg Rock Hammer	50	300kg Rock Hammer	100
			600kg Rock Hammer	50
5.0 to 10.0	300kg Rock Hammer	100	600kg Rock Hammer	100
	600kg Rock Hammer	50	900kg Rock Hammer	50

[1]Vibration monitoring is recommended for the use of a maximum PPV of 10mm/sec.

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, as discussed in Section 5.1. This should be considered a "Hold Point".

4.7 Groundwater Management

Although no groundwater was encountered or observed during the site investigation to a maximum depth of approximately 1.5m in borehole BH1 or to approximately RL96m AHD in borehole BH5, its presence should not be precluded within the site and during construction. It should be noted that groundwater levels have the potential to elevate during daily or seasonal influences such as tidal fluctuations, heavy rainfall, damaged services, flooding, etc. Thus, we expect groundwater which may enter the site to be in the form of seepage throughout the voids within the underlying soils, and through defects in the underlying bedrock. Seepage may also occur within the fill material, and at the fill/natural soils and natural soils/bedrock interfaces, predominately following heavy rain.

The rate of flow which may enter the excavation may initially be rapid, but is expected to decrease over time as local water ingress decreases. As noted, groundwater levels are subject to fluctuations on a daily and seasonal basis, and the potential for groundwater to enter the excavation as moderate to rapid seepage should be considered as part of the long term design life of the building. The amount of seepage into the excavation will also depend on the shoring system being adopted.

Therefore, consideration should be given to precautionary drainage measures including (not limited to):

- Drainage installed around the perimeter of the basement behind all basement retaining walls, and below the basement slab. This drainage should be connected to a sump and pump out system and discharged into the stormwater system (which may require council approval).
- Collection trenches or pipes and stormwater pits may be installed in conjunction with the above method, and connected to the building stormwater system.

Where a suitable drainage system has not been implemented or provided for the proposed development to collect and remove any groundwater, consideration may also been given to waterproofing of the basement walls and slabs, with allowance given for nominal hydrostatic uplift.

It is recommended that monitoring of seepage (if encountered) be implemented during the excavation stage to confirm the capacity of the drainage system and groundwater entering the excavation area. This should be monitored by the project geotechnical engineer, in conjunction with the project hydraulic/stormwater engineer.

4.8 Excavation Stability

Maximum excavation depths are expected to vary within the site from approximately 0.8m to 3.7m (varying throughout) for construction of the proposed basement. Based on the ground conditions within the site, the total depth of excavation and the extent of the basement 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.

4.8.1 Batter Slopes

Temporary or permanent batters are considered to be suitable for construction of the proposed basement, providing sufficient space exists between the basement walls and adjoining infrastructures. It should be noted that due to the nature of natural soils and weathered bedrock, 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 basement walls and adjoining infrastructures, and where the adjacent infrastructures are located outside the "zone of influence" (obtained by drawing a line 45º above horizontal from the base of the proposed basement walls) for the use temporary batter slopes. Table 5 provides maximum recommended slopes for permanent and temporary batters.

Table 5. Recommended Maximum Batter Slopes

Unit		Maximum Batter Slope (H : V)	
		Permanent	Temporary
Fill (Unit 1)		4 : 1	2 : 1
Residual Soils (Unit 2)		2 : 1	1 : 1
Sandstone Bedrock (Unit 3)	**EL – VL**	1.5 : 1	0.75 : 1
	L	1 : 1	0.5 : 1
	M[2]	0.5 : 1[1]	0.25 : 1[1]
	H[2]	Vertical to Semi-Vertical[1]	

[1]Subject to inspection by a geotechnical engineer. Remedial options may be required (i.e. rock bolting, shotcreting, etc.).
[2]Preliminary only. Not encountered or observed during the site investigation. Subject to confirmation by a geotechnical engineer.
Notes:

- EL = Extremely Low Estimated Strength, VL = Very Low Estimated Strength, L = Low Estimated Strength, M = Medium Estimated Strength, H = High Estimated Strength.

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 45º above horizontal from the base of the proposed basement walls) plus an additional 1.0m. A geotechnical engineer should inspect the batter slopes within the site. Consideration should be given to shotcreting and soil nailing where steeper batter slopes are to be used.

Temporary surface protection against erosion may 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 basement perimeter walls following any temporary or permanent batter slopes within the site. All retaining walls should be sufficiently constructed on inferred sandstone 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 inferred bedrock.

4.8.2 Excavation Retention Support Systems

Where there is insufficient space between the basement walls and adjoining infrastructures, or where adjacent infrastructures are located within the "zone of influence" (as outlined in Section 4.8.1 above), consideration should be given to a suitable retention system such as a soldier pile wall sufficiently embedded into the underlying sandstone bedrock, with concrete infill panels for the support of the excavation. Closer spaced piles may be required to reduce lateral movements particularly where adjacent structures, such as buildings or pavements are located near the excavation, and to prevent collapse of loose fill in-situ materials and natural soils. 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.), or 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 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 basement 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 is 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.

The design of the basement retaining wall will depend on the method of constructed being adopted. The two common methods include:

- Top-down construction.
- Bottom-up construction.
- Staged excavation and installation of props and/or partial berms.

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 4.8.3. 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.

4.8.3 Design Parameters (Earth Pressures)

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 45º above horizontal from the base of the proposed basement wall.

Support system designed using the earth pressure approach may be based on the parameters given in Table 6 below for soils and rock horizons underlying the site. Table 6 also provides preliminary coefficients of lateral earth pressure for the soils and rock horizons encountered in the site, along with preliminary earthquake site risk classification. These are based on fully drained conditions and that the ground behind the retention walls is horizontal.

Table 6. Preliminary Geotechnical Design Parameters

Material	Fill (Unit 1)	Residual Soils (Unit 2)	Sandstone Bedrock (Unit 3)[3]		
			EL – VL	L[5]	M – H[6]
Unit Weight (kN/m^3)[4]	17	20	20	22	24
Effective Cohesion c' (kPa)	0	5	40	75	125
Angle of Friction ϕ' (°)	26	24	28	30	33
Modulus of Elasticity E_{sh} (MPa)	5	12	75	250	450
Earth Pressure Coefficient At Rest Ko[1]	0.56	0.59	0.53	0.5	0.46
Earth Pressure Coefficient Active Ka[2]	0.39	0.42	0.36	0.33	0.29
Earth Pressure Coefficient Passive Kp[2]	2.56	2.37	2.77	3.0	3.39
Poisson Ratio *v*	0.35	0.35	0.3	0.3	0.25
Preliminary Earthquake Site Risk Classification		• AS 1170.4-2011 indicates the site may be classified as a **"Shallow Soil Site" (Class C_e)**. • AS 1170-4-2011 indicates a Hazard Factor (Z) for Sydney is **0.08**.			

[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 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]Inferred low estimated strength sandstone bedrock based on observations made during auger penetration resistance at the time of drilling. Confirmation of the underlying inferred sandstone strength and continuity should be made by additional borehole drilling and testing, or during construction by a geotechnical engineer.
[6]Preliminary only. Not encountered or observed during the site investigation. Subject to confirmation by additional borehole drilling and testing, or during construction by a geotechnical engineer.
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.

4.9 Foundations

Following excavation to the proposed basement) FFL of the proposed development to RL99.650AHD, and based on the boreholes carried out, we expect varying ground conditions comprising predominately Unit 3 (sandstone bedrock) of variable strength and weathering to be exposed at bulk level excavation (varying throughout). The possibility for encountering higher strength bedrock should not be precluded, providing the ground conditions are confirmed by a geotechnical engineer by additional borehole drilling and rock strength testing, or during construction by inspection. Residual soils are also expected to vary across the site, predominately in areas not inspection during the site investigation. Thus, the possibility for encountering residual soils at bulk level excavation in areas where shallower excavation depths exist, should not be precluded.

It is noted that ground conditions within the site is expected to differ from those encountered and inferred in this report, since no geotechnical or geological exploration programme, no matter how comprehensive, can reveal and identify all subsurface conditions underlying the site. It is therefore recommended that confirmation of the underlying ground conditions be confirmed by a geotechnical engineer prior to construction by additional borehole drilling, or during construction by inspection.

4.9.1 Geotechnical Assessment

Based on the proposed development and assessment of the subsurface conditions, a suitable foundation system comprising shallow foundations typically comprising pad or strip footings constructed on the inferred low estimated strength (or better) inferred sandstone bedrock underlying the site is likely to be adopted for the proposed development. Shallow foundations should include local slab thickening to support internal walls and columns for shallow foundations, with consideration given to settlement reducing piles.

It should be noted that due to the potential variable bedrock conditions throughout the site, precaution should be taken for the design of the building foundation system, taking into consideration the preliminary geotechnical design parameters 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 and soil reactivity (as discussed in Section 4.3) within the site, it is recommended that all foundations are constructed on consistent bedrock throughout, in order 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 (e.g. in areas where shallower excavation depths exist). 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.

Table 7 provides preliminary recommended geotechnical design parameters.

Table 7. Preliminary Recommended Geotechnical Design Parameters

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
Residual Soils (Unit 2)		100	N/A	N/A
Sandstone Bedrock (Unit 3)[2]	**EL – VL**	700	50	25
	L	1,000	100	50
	M[2, 3]	1,500[4]	150	75
	H[2, 3]	2,000[4]	200	100

[1]Minimum embedment of 0.4m for shallow foundations and 0.5m for deep foundations.
[2]Confirmation of the underlying bedrock strength and continuity should be carried out by additional borehole drilling, or during construction by a geotechnical engineer.
[3]Preliminary only. Not encountered or observed during the site investigation. Subject to confirmation by a geotechnical engineer.
[4]Subject to confirmation by a geotechnical engineer during construction, or by additional borehole drilling and rock strength testing.

Notes:

- EL = Extremely Low Estimated Strength, VL = Very Low Estimated Strength, L = Low Estimated Strength, M = Medium Estimated Strength, H = High Estimated Strength.
- N/A = Not Applicable. Not recommended for the proposed development.
- The depth of the underlying bedrock material should be confirmed either prior to construction by further borehole 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.

4.9.2 Geotechnical Comments

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.

Foundations located within the “zone of influence” of any services or sensitive structures should be supported by a piled foundation. The depths of the piles should extend below the “zone of influence” and should ignore any shaft adhesion. Appropriate measures should be taken to ensure that any services or sensitive structures located within the “zone of influence” of the proposed development are not damaged during and following construction.

The design and construction of the foundations should take into consideration the potential of flooding. All foundation excavations should be free of any loose debris and wet soils, and if groundwater seepage or runoff is encountered dewatering should be carried out prior to pouring concrete in the foundations. Due to the possibility of groundwater being encountered, or possible groundwater seepage during installation of bored piles within the site, it is recommended that consideration be given to other piling methods such as Continuous Flight Auger (CFA) piles.

Shaft adhesion may be applied to socketed piles adopted for foundations provided the socketed shaft lengths conform to appropriate classes of bedrock (i.e. sandstone) in accordance with Pells et. al, and shaft sidewall cleanliness and roughness are to acceptable levels. Shaft adhesion should be ignored or reduced within socket lengths that are smeared or fail to satisfy cleanliness requirements (i.e. at least 80%). The possibility of piles penetrating expansive soils which are susceptible to shrink and swell due to seasonal moisture should not be precluded, with shaft adhesion being ignored due to the potential of shrinkage cracking.

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".

4.10 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".

4.11 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 deleterious 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.

5. ADDITIONAL GEOTECHNICAL RECOMMENDATIONS

Following completion of the geotechnical investigation and report, GCA recommends the following additional work to be carried out:

- Dilapidation survey report on adjacent properties and infrastructures.
- Constant supervision and monitoring of excavation within the proposed development area.
- The depth and strength of the underlying inferred bedrock material should be confirmed either prior to construction by further borehole testing, or during construction by inspection.
- Geotechnical inspections of foundations (shallow and piles).
- Monitoring of any groundwater inflows into the excavation.
- Classification of all excavated material transported from the site.
- A meeting to be carried out to discuss any geotechnical issues and inspection requirements.
- Final architectural and structural design drawings are provided to GCA for further assessment.

6. LIMITATIONS

Geotechnical Consultants Australia Pty Ltd (GCA) 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 GCA'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 GCA's recommendations and conclusions, GCA 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. GCA's advice and accuracy may be limited by undetected variations in ground conditions between sampling locations.

GCA 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 **Hills Awqaf Pty Ltd**, 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 GCA.

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

For and behalf of

Geotechnical Consultants Australia (GCA)

Joe Nader
BE (Civil – Construction), Dip.Eng.Prac., MIEAust., AGS, ISSMGE
Cert. IV in Building and Construction
Geotechnical Engineer
Director

7. REFERENCES

Pells P.J.N, Mostyn, G. & Walker B.F., "Foundations on Sandstone and Shale in the Sydney Region", Australian Geomechanics Journal, 1998.

AS 3600-2009 Concrete Structures. Standards Australia.

AS 1726-2017 Geotechnical Site Investigation. Standards Australia.

AS 1170.4-2007 Structural Design Actions – Part 4: Earthquake actions in Australia. Standards Australia.

AS 3798-2007 Guidelines on Earthworks for Commercial and Residential Developments. Standards Australia.

AS 2870-2011 Residential slabs and footings. Standards Australia.

AS 2159-2009 Piling - Design and installation. Standards Australia.

NSW Department of Mineral Resources (1991) Penrith 1:100,000 Geological Series Sheet 9030 (Edition 1). Geological Survey of New South Wales. Department of Mineral Resources.

NSW Planning Portal.

NSW Six Maps.

APPENDIX A

Important Information About Your

Geotechnical Report

This geotechnical report has been prepared based on the scopes outlined in the project proposal. The works carried out by Geotechnical Consultants Australia Pty Ltd (GCA), have limitations during the site investigation, and may be affected by a number of factors. Please read the geotechnical invesitgation report in conjunction with this "Important Information About Your Geotechnical Report".

Geotechnical Services Are Performed for Specicif Projects, Clients and Purposes.
Due to the fact that each geotechnical investigation is unique and varies from sites, each geotechnical report is unique, and is prepared soley for the client. A geotechnical report may satisfy the needs of structural engineer, where is will not for a civil engineer or construction contractor. No one except the client should rely on the geotechnical report without first conferring with the specific geotechnical consultant who prepared the report. The report is prepared for the contemplated project or original purpose of the investigation. No one should apply this report to any other or similar project.

Reading The Full Report.
Do not read selected elements of the report or tables/figures only. Serious problems have occurred because those relying on the specially prepared geotechnical invesitgation report did not read it all in full context.

The Geotechnical Report is Based on a Unique Set of Project And Specific Factors.
When preparing a geotechnical report, the geotechnical engineering consultant considers a number of unique factors for the specific project. These typially include:

- Clients objectives, goals and risk management preferences;
- The general proposed development or nature of the structure involved (size, location, etc.); and
- Future planned or existing site improvements (parking lots, roads, underground services, etc.);

Care should be taken into identifying the reason of the geotechnical report, where you should not rely on a geotechnical engineering report that was:

- Not prepared for your project;
- Not prepared for the specific site;
- Not prepared for you;
- Does not take into consideration any important changes made to the project; or
- Was carried out prior to any new infrastructure on your subject site.

Typical changes that can affect the reliabiliy if an existing geotechical investigation report include those that affect:

- The function of the proposed structure, where it may change from one basement level to two basement levels, or from a light structure to a heavy loaded structure;
- Location, size, elevation or configuration of the proposed development;
- Changes in the structural design occur; or
- The owner of the proposed development/project has changed.

The geotecnical engineer of the project should always be notified of any changes – even minor – and be asked to evaluate if this has any impact. GCA does not accept responsibility or liability for problems that occur because its report did not consider developments which it was not informed of.

Subsurface Conditions Can Change
This report is based on conditions that existed at the time of the investigation, at the locations of the subsurface tests (i.e. boreholes) carried out during the site investigation. Subfurface conditions can be affected and modified by a number of factores including, but not limited to, the passage of time, man-made influences such as construction on or adjacent to the site, by natural forces such as floods, groundwater fluctuations or earthquakes. GCA should be contacted prior to submitting its report to determine if any further testing may be required. A minor amount of additional testing may prevent any major problems.

Geotechnical Findings Are Professional Opinions
Results of subsurface conditions are limited only to the points where the subsurface tests were carried out, or where samples were collected. The field and laboratory data is analysed and reviewed by a geotechnical engineer, who then applys their professional experience and recommendations about the site's subsurface conditions. Despite investigation, the actual subsurface conditions may differ – in some cases significantly – from the results presented in the geotechnical investigation report, since no subsurface exploration program, no matter how comprehensive, can reveal all subsurface anomalies and details.

Therefore, the recommendations in this report can only be used as preliminary. Retaining GCA as your geotechnical consultants on your project to provide construction observations is the most effective method of managing the risks associated with unanticipated subsurface conditions.

Geotechnical Report's Recommendations Are Not Final
Because geotechnical engineers provide recommendations based on experience and judgement, you should not overrely on the recommendations provided – they are not final. Only by observing the actual subsurface conditions revealed during construction may a geotechnical engineer finalise their recommendations. GCA does not assume responsibility or liability for the report's recommendations if no additional observations or testing is carried out.

Geotechnical Report's Are Subject to Misinterpretations
The project geotechnical engineer should consult with appropriate members of the design team following submission of the report. You should review your design teams plans and drawings, in conjunction with the geotechnical report to ensure they have all be incorporated. Due to many issues arising from misinterpretation of geotechnical reports between design teams and building contractors, GCA should participate in pre-construction meetings, and provide adequate construction observations.

Engineering Borehole Logs And Data Should Not be Redrawn
Geotechnical engineers prepare final borehole and testing logs, figure, etc. based on results and interpretation of field logs and laboratory data following the site investigation. The logs, figure, etc. provided in the geotechnical report should never be redrawn or altered for inclusion in any other documents from this report, includined architectural or other design drawings.

Providing The Full Geotechnical Report For Guidance
The project design teams, subcontactors and building contractors should have a copy of the full geotechnical investigation report to help prevent any costly issues. This should be prefaced with a clearly written letter of transmittal. The letter should clearly advise the aforementioned that the report was prepared for proposed development/project requirements, and the report accuracy is limited. The letter should also encourage them to confer with GCA, and/or carry out further testing as may be required. Providing the report to your project team will help share the financial responsibilities stemming from any unanticipated issues or conditions in the site.

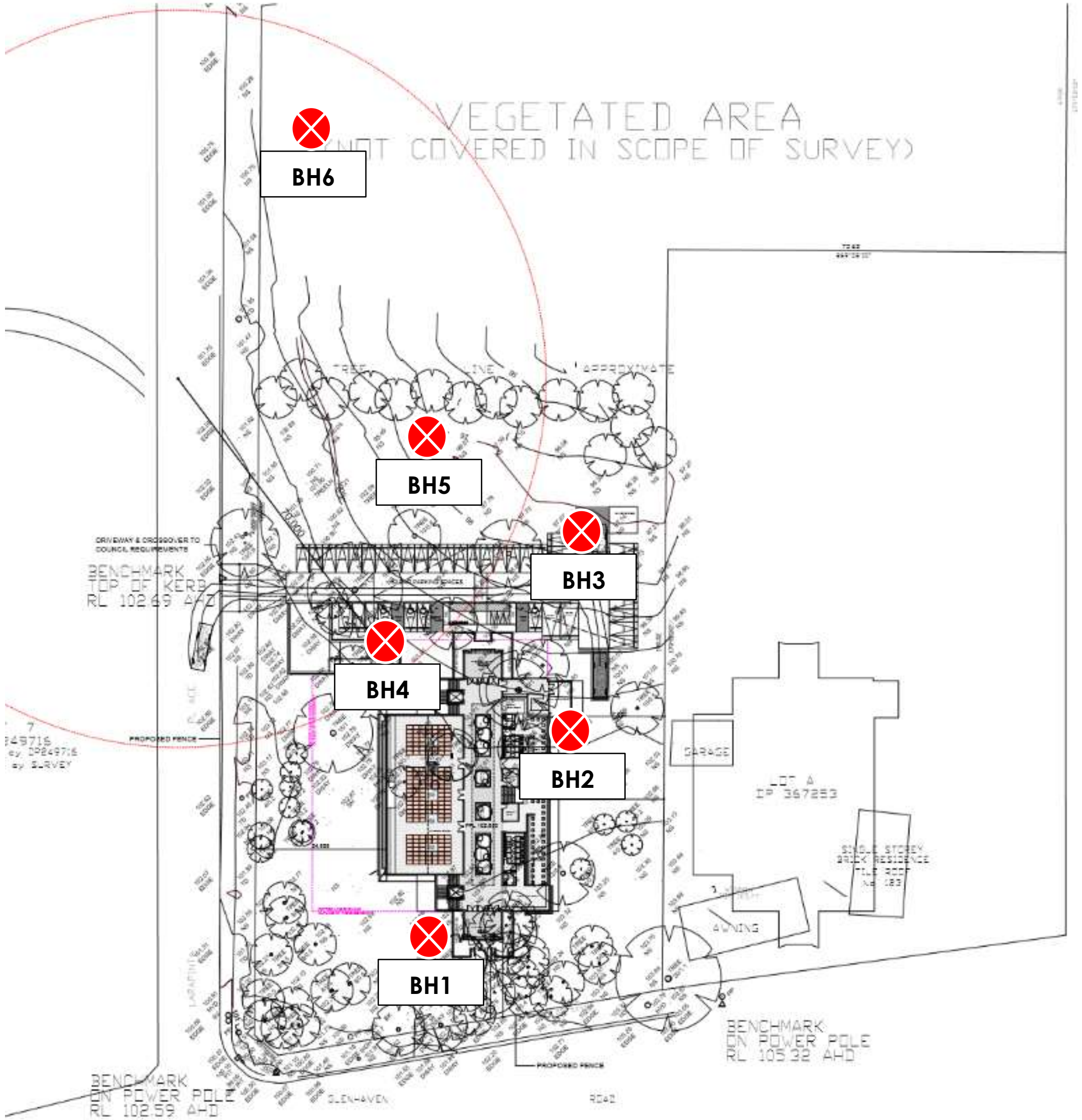
Understanding Limitation Provisions
As some clients, contractors and design professionals do not recognise geotechnical engineering is much broader and less exact than other engineering disciplines, this creates unrealistic expectations that lead to claims, disputs and other disappointments. As part of the geotechnical report, (in most cases) a 'limitations' explanatory provision is included, outlining the geotechnical engineers' limitations for your project – with the geotechnical engineers responsibilites to help other reduce their own. This should be read closely as part of your report.

Other Limitations
GCA will not be liable to revise or update the report to take into account any events or circumstances (seen or unforeseen), or any fact occurring or becoming apparent after the date of the report. This report is the subject of copyright and shall not be reproduced either totally or in part without the express permission of GCA. The report should not be used if there have been changes to the project, without first consulting with GCA to assess if the report's recommendations are still valid. GCA does not accept any responsibility for problems that occur due to project changes which have not been consulted.

APPENDIX B

Legend:

Approximate Borehole Location



GCA Geotechnical Consultants Australia	Figure 1 Site Plan	**Geotechnical Investigation**	Drawn: GN
		Hills Awqaf Pty Ltd	Date: 26/11/2018
	Job No.: G18202-2	1 Larapinta Place Glenhaven NSW 2156	Scale: NTS

Image Source: Preliminary architectural drawing prepared by iDraft Architects, titled project "Proposed Mosque", referenced job No. 28569, drawing No. 1001, and dated 15th November 2018.

APPENDIX C

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Explanation of Notes, Abbreviations and Terms Used on Borehole and Test Pit Reports

DRILLING/EXCAVATION METHOD

Method	Description
AS	Auger Screwing
BH	Backhoe
CT	Cable Tool Rig
EE	Existing Excavation/Cutting
EX	Excavator
HA	Hand Auger
HQ	Diamond Core-63mm
JET	Jetting
NMLC	Diamond Core –52mm
NQ	Diamond Core –47mm
PT	Push Tube
RAB	Rotary Air Blast
RB	Rotary Blade
RT	Rotary Tricone Bit
TC	Auger TC Bit
V	Auger V Bit
WB	Washbore
DT	Diatube

PENETRATIION/EXCAVATION RESISTANCE

These assessments are subjective and dependant on many factors including the equipment weight, power, condition of the drilling tools or excavation, and the experience of the operator..

L **Low Resistance.** Rapid penetration possible with little effort from the equipment used.
M **Medium Resistance.** Excavation possible at an acceptable rate with moderate effort required from the equipment used.
H **High Resistance.** Further penetration is possible at a slow rate and required significant effort from the equipment.
R **Refusal or Practical Refusal.** No further progress possible within the risk of damage or excessive wear to the equipment used.

WATER

▽ Water level at date shown ◁ Partial water loss

▷ Water inflow ◀ Complete water loss

Groundwater not observed: The observation of groundwater, whether present or not, was not possible due to drilling water, surface seepage or cave in of the borehole/test pit.

Groundwater not encountered: No free-flowing (springs or seepage) was intercepted, although the soil may be moist due to capillary water. Water may be observed in low permeable soils if the test pits/boreholes had been left open for at least 12-24 hours.

MOISTURE CONDITION (AS 1726-1993)

Dry - Cohesive soils are friable or powdery
Cohesionless soil grains are free-running

Moist - Soil feels cool, darkened in colour
Cohesive soils can be moulded
Cohesionless soil grains tend to adhere

Wet - Cohesive soils usually weakened
Free water forms on hands when handling

For cohesive soils the following codes may also be used:

MC>PL Moisture Content greater than the Plastic Limit.
MC~PL Moisture Content near the Plastic Limit.
MC<PL Moisture Content less than the Plastic Limit.

SAMPLING AND TESTING

Sample	Description
B	Bulk Disturbed Sample
DS	Disturbed Sample
Jar	Jar Sample
SPT*	Standard Penetration Test
U50	Undisturbed Sample –50mm
U75	Undisturbed Sample –75mm

*SPT (4, 7, 11 N=18). 4, 7, 11 = Blows per 150mm. N= Blows per 300mm penetration following 150mm sealing.
SPT (30/80mm). Where practical refusal occurs, the blows and penetration for that interval is recorded.

ROCK QUALITY

The fracture spacing is shown where applicable and the Rock Quality Designation (RQD) or Total Core Recovery (TCR) is given where:

$$TCR\ (\%) = \frac{\text{length of core recovered}}{\text{length of core run}}$$

$$RQD\ (\%) = \frac{\text{Sum of Axial lengths of core > 100mm long}}{\text{length of core run}}$$

ROCK STRENGTH TEST RESULTS

● Diametral Point Load Index test

■ Axial Point Load Index test

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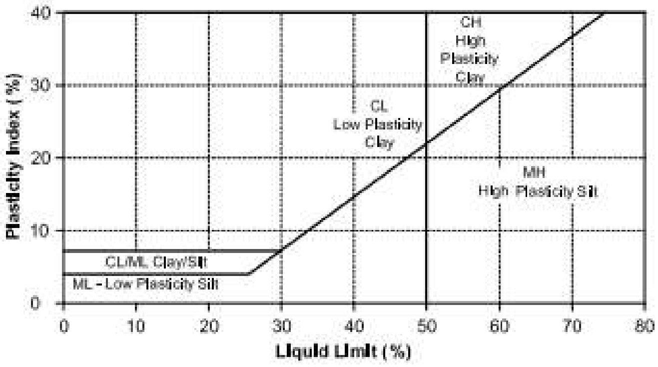
Method and Terms for Soil and Rock Descriptions Used on Borehole and Test Pit Reports

Soil and Rock is classified and described in reports of boreholes and test pits using the preferred method given in AS 1726-1993, Appendix A. The material properties are assessed in the field by visual/tactile methods. The appropriate symbols in the Unified Soil Classification are selected on the result of visual examination, field tests and available laboratory tests, such as, sieve analysis, liquid limit and plasticity index.

COHESIONLESS SOILS PARTICLE SIZE DESCRIPTIVE TERMS

Name	Subdivision	Size
Boulders		>200 mm
Cobbles		63 mm to 200 mm
Gravel	coarse	20 mm to 63 mm
	medium	6 mm to 20 mm
	fine	2.36 mm to 6 mm
Sand	coarse	600 µm to 2.36 mm
	medium	200 µm to 600 µm
	fine	75 µm to 200 µm

PLASTICITY PROPERTIES



COHESIVE SOILS – CONSISTENCY (AS 1726-1993)

Strength	Symbol	Undrained Shear Strength, C_u (kPa)
Very Soft	VS	< 12
Soft	S	12 to 25
Firm	F	25 to 50
Stiff	St	50 to 100
Very Stiff	VSt	100 to 200
Hard	H	> 200

PLASTICITY

Description of Plasticity	LL (%)
Low	<35
Medium	35 to 50
High	>50

COHESIONLESS SOILS - RELATIVE DENSITY

Term	Symbol	Density Index	N Value (blows/0.3 m)
Very Loose	VL	0 to 15	0 to 4
Loose	L	15 to 35	4 to 10
Medium Dense	MD	35 to 65	10 to 30
Dense	D	65 to 85	30 to 50
Very Dense	VD	>85	>50

UNIFIED SOIL CLASSIFICATION

USC Symbol	Description
GW	Well graded gravel
GP	Poorly graded gravel
GM	Silty gravel
GC	Clayey gravel
SW	Well graded sand
SP	Poorly graded sand
SM	Silty sand
SC	Clayey sand
ML	Silt of low plasticity
CL	Clay of low plasticity
OL	Organic soil of low plasticity
MH	Silt of high plasticity
CH	Clay of high plasticity
OH	Organic soil of high plasticity
Pt	Peaty Soil

ROCK MATERIAL WEATHERING

Symbol	Term	Definition
RS	Residual Soil	Soil definition on extremely weathered rock; the mass structure and substance are no longer evident; there is a large change in volume but the soil has not been significantly transported
EW	Extremely Weathered	Rock is weathered to such an extent that it has 'soil' properties, i.e. It either disintegrates or can be remoulded in water
HW *DW*	Highly Weathered *Distinctly Weathered (as per AS 1726)*	The rock substance is affected by weathering to the extent that limonite staining or bleaching affects the whole rock substance and other signs of chemical or physical decomposition are evident. Porosity and strength is usually decreased compared to the fresh rock. The colour and strength of the fresh rock is no longer recognisable.
MW	Moderately Weathered	The whole of the rock substance is discoloured, usually by iron staining or bleaching, to the extent that the colour of the fresh rock is no longer recognisable
SW	Slightly Weathered	Rock is slightly discoloured but shows little or no change of strength from fresh rock
FR	Fresh	Rock shows no sign of decomposition or staining

ROCK STRENGTH (AS 1726-1993 and ISRM)

Term	Symbol	Point Load Index $I_{s(50)}$ (MPa)
Extremely Low	EL	<0.03
Very Low	VL	0.03 to 0.1
Low	L	0.1 to 0.3
Medium	M	0.3 to 1
High	H	1 to 3
Very High	VH	3 to 10
Extremely High	EH	>10

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ABREVIATIONS FOR DEFECT TYPES AND DECRIPTIONS

Term	Defect Spacing	Bedding
Extremely closely spaced	<6 mm	Thinly Laminated
	6 to 20 mm	Laminated
Very closely spaced	20 to 60 mm	Very Thin
Closely spaced	0.06 to 0.2 m	Thin
Moderately widely spaced	0.2 to 0.6 m	Medium
Widely spaced	0.6 to 2 m	Thick
Very widely spaced	>2 m	Very Thick

Type	Definition
B	Bedding
J	Joint
HJ	Horizontal to Sub-Horizontal Joint
F	Fault
Cle	Cleavage
SZ	Shear Zone
FZ	Fractured Zone
CZ	Crushed Zone
MB	Mechanical Break
HB	Handling Break

Planarity	Roughness
P – Planar	C – Clean
Ir – Irregular	Cl – Clay
St – Stepped	VR – Very Rough
U - Undulating	R – Rough
	S – Smooth
	Sl – Slickensides
	Po – Polished
	Fe – Iron

Coating or Infill	Description
Clean (C)	No visible coating or infilling
Stain	No visible coating or infilling but surfaces are discoloured by mineral staining
Veneer	A visible coating or infilling of soil or mineral substance but usually unable to be measured (<1mm). If discontinuous over the plane, patchy veneer
Coating	A visible coating or infilling of soil or mineral substance, >1mm thick. Describe composition and thickness
Iron (Fe)	Iron Staining or Infill.

APPENDIX D

BOREHOLE NUMBER BH1

Geotechnical Consultants Australia Pty Ltd
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CLIENT Hills Awqaf Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G18202-2
PROJECT LOCATION 1 Larapinta Place Glenhaven NSW 2156

DATE STARTED 20/11/18 **COMPLETED** 20/11/18 **R.L. SURFACE** 102.8 **DATUM** m AHD
DRILLING CONTRACTOR Australian Geotechnical Pty Ltd **SLOPE** 90° **BEARING** ---
EQUIPMENT Trailer Mounted Drilling Rig **HOLE LOCATION** Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter **LOGGED BY** JN **CHECKED BY** JN
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT	Not Encountered During Augering					Clayey SAND, fine grained, dark brown to brown, low plasticity clay, with fine to medium grained gravel, grass rootlets, moist.		FILL
		102.5			SC	Clayey SAND, fine grained, pale brown, yellowish brown, low plasticity clay, with fine grained gravel, moist.		RESIDUAL SOILS
			0.5					
		102.0				SANDSTONE, fine grained, pale grey, grey, clay seams, extremely weathered, extremely low estimated strength, moist.		BEDROCK
			1.0					
		101.5				becoming pale brown, grey laminations from 1.3m bgl.		
						becoming very low estimated strength from 1.4m bgl.		
			1.5			Borehole BH1 terminated at 1.5m		'TC' Bit refusal at 1.5m bgl.
		101.0						
			2.0					

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BOREHOLE NUMBER BH2

CLIENT Hills Awqaf Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G18202-2
PROJECT LOCATION 1 Larapinta Place Glenhaven NSW 2156

DATE STARTED 20/11/18 **COMPLETED** 20/11/18
R.L. SURFACE 101.9 **DATUM** m AHD
DRILLING CONTRACTOR Australian Geotechnical Pty Ltd
SLOPE 90° **BEARING** ---
EQUIPMENT Trailer Mounted Drilling Rig
HOLE LOCATION Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter
LOGGED BY JN **CHECKED BY** JN
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT	Not Encountered During Augering					Clayey SAND, fine grained, dark brown to brown, low plasticity clay, with fine to medium grained gravel, grass rootlets, moist.		FILL
		101.5			SC	Clayey SAND, fine grained, pale brown, yellowish brown, low plasticity clay, with fine to coarse grained gravel, some grass rootlets, moist.		RESIDUAL SOILS
		101.0	0.5			SANDSTONE, fine grained, reddish brown to dark reddish brown, extremely weathered, extremely low estimated strength, moist. becoming very low estimated strength from 0.7m bgl.		BEDROCK
		100.5 100.0	1.0 1.5 2.0			Borehole BH2 terminated at 1m		'TC' Bit refusal at 1.0m bgl.

BOREHOLE NUMBER BH3

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CLIENT Hills Awqaf Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G18202-2
PROJECT LOCATION 1 Larapinta Place Glenhaven NSW 2156

DATE STARTED 20/11/18 **COMPLETED** 20/11/18 **R.L. SURFACE** 97.7 **DATUM** m AHD
DRILLING CONTRACTOR Australian Geotechnical Pty Ltd **SLOPE** 90° **BEARING** ---
EQUIPMENT Trailer Mounted Drilling Rig **HOLE LOCATION** Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter **LOGGED BY** JN **CHECKED BY** JN
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT	Not Encountered During Augering					Clayey SAND, fine grained, dark brown to brown, low plasticity clay, with fine grained gravel, grass rootlets, moist.		FILL
		97.5			SC	Clayey SAND, fine grained, pale brown, yellowish brown, low plasticity clay, with fine to coarse grained gravel, some grass rootlets, moist.		RESIDUAL SOILS
		97.0	0.5			SANDSTONE, fine grained, pale brown, reddish brown laminations, some clay, extremely weathered, extremely low estimated strength, moist.		BEDROCK
			1.0			becoming very low estimated strength from 1.0m bgl.		
		96.5				Borehole BH3 terminated at 1.2m		'TC' Bit refusal at 1.2m bgl.
		96.0	1.5					
			2.0					

BOREHOLE NUMBER BH4

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CLIENT Hills Awqaf Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G18202-2
PROJECT LOCATION 1 Larapinta Place Glenhaven NSW 2156

DATE STARTED 20/11/18 **COMPLETED** 20/11/18
R.L. SURFACE 101.9 **DATUM** m AHD
DRILLING CONTRACTOR Geotechnical Consultants Australia Pty Ltd
SLOPE 90° **BEARING** ---
EQUIPMENT Hand Operated Equipment
HOLE LOCATION Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter
LOGGED BY JN **CHECKED BY** JN
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT	Not Encountered During Augering					Silty SAND, fine grained, brown to dark brown, with fine to coarse grained gravel, grass rootlets, moist.		FILL
		101.5	0.5			SANDSTONE, fine to medium grained, pale grey, grey laminations, extremely weathered, extremely low estimated strength, moist.		BEDROCK
						pale brown laminations from 0.8m bgl.		
		101.0	1.0			becoming very low estimated strength from 1.0m bgl.		
		100.5	1.5			Borehole BH4 terminated at 1.3m		'TC' Bit refusal at 1.3m bgl.
		100.0	2.0					

BOREHOLE NUMBER BH5

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CLIENT Hills Awqaf Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G18202-2
PROJECT LOCATION 1 Larapinta Place Glenhaven NSW 2156

DATE STARTED 20/11/18 **COMPLETED** 20/11/18 **R.L. SURFACE** 97.2 **DATUM** m AHD
DRILLING CONTRACTOR **SLOPE** 90° **BEARING** ---
EQUIPMENT **HOLE LOCATION** Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter **LOGGED BY** JN **CHECKED BY** JN
NOTES

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT	Not Encountered During Augering					Clayey SAND, fine grained, dark brown to brown, low plasticity clay, with fine to medium grained gravel, grass and tree rootlets, moist.		FILL
		97.0	0.5		CIS	Sandy CLAY, medium plasticity, brown, yellowish brown, fine grained sand, with fine grained gravel, moist.	DS	RESIDUAL SOILS
		96.5	1.0			SANDSTONE, fine grained, pale grey, pale brown laminations, some clay, extremely weathered, extremely low estimated strength, moist. becoming very low estimated strength from 1.0m bgl.		BEDROCK
		96.0 95.5	1.5 2.0			Borehole BH5 terminated at 1.2m		'TC' Bit refusal at 1.2m bgl.

BOREHOLE NUMBER BH6

GCA Geotechnical Consultants Australia Pty Ltd
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CLIENT Hills Awqaf Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G18202-2
PROJECT LOCATION 1 Larapinta Place Glenhaven NSW 2156

DATE STARTED 20/11/18 **COMPLETED** 20/11/18 **R.L. SURFACE** **DATUM**
DRILLING CONTRACTOR **SLOPE** 90° **BEARING** ---
EQUIPMENT **HOLE LOCATION** Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter **LOGGED BY** JN **CHECKED BY** JN
NOTES

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT	Not Encountered During Augering					Clayey SAND, fine grained, dark brown to brown, low plasticity clay, with fine to medium grained gravel, grass and tree rootlets, moist.		FILL
			0.5		CIS	Sandy CLAY, medium plasticity, pale brown, yellowish brown, fine grained sand, with fine grained gravel, moist.		RESIDUAL SOILS
			1.0			SANDSTONE, fine grained, reddish brown to pale reddish brown, extremely weathered, extremely low estimated strength, moist. becoming very low estimated strength from 1.1m bgl.	DS	BEDROCK
			1.5 2.0			Borehole BH6 terminated at 1.3m		'TC' Bit refusal at 1.3m bgl.

APPENDIX E

Foundation Maintenance and Footing Performance: A Homeowner's Guide

CSIRO

BTF 18 replaces Information Sheet 10/91

Buildings can and often do move. This movement can be up, down, lateral or rotational. The fundamental cause of movement in buildings can usually be related to one or more problems in the foundation soil. It is important for the homeowner to identify the soil type in order to ascertain the measures that should be put in place in order to ensure that problems in the foundation soil can be prevented, thus protecting against building movement.

This Building Technology File is designed to identify causes of soil-related building movement, and to suggest methods of prevention of resultant cracking in buildings.

Soil Types

The types of soils usually present under the topsoil in land zoned for residential buildings can be split into two approximate groups – granular and clay. Quite often, foundation soil is a mixture of both types. The general problems associated with soils having granular content are usually caused by erosion. Clay soils are subject to saturation and swell/shrink problems.

Classifications for a given area can generally be obtained by application to the local authority, but these are sometimes unreliable and if there is doubt, a geotechnical report should be commissioned. As most buildings suffering movement problems are founded on clay soils, there is an emphasis on classification of soils according to the amount of swell and shrinkage they experience with variations of water content. The table below is Table 2.1 from AS 2870, the Residential Slab and Footing Code.

Causes of Movement

Settlement due to construction
There are two types of settlement that occur as a result of construction:

- Immediate settlement occurs when a building is first placed on its foundation soil, as a result of compaction of the soil under the weight of the structure. The cohesive quality of clay soil mitigates against this, but granular (particularly sandy) soil is susceptible.
- Consolidation settlement is a feature of clay soil and may take place because of the expulsion of moisture from the soil or because of the soil's lack of resistance to local compressive or shear stresses. This will usually take place during the first few months after construction, but has been known to take many years in exceptional cases.

These problems are the province of the builder and should be taken into consideration as part of the preparation of the site for construction. Building Technology File 19 (BTF 19) deals with these problems.

Erosion
All soils are prone to erosion, but sandy soil is particularly susceptible to being washed away. Even clay with a sand component of say 10% or more can suffer from erosion.

Saturation
This is particularly a problem in clay soils. Saturation creates a bog-like suspension of the soil that causes it to lose virtually all of its bearing capacity. To a lesser degree, sand is affected by saturation because saturated sand may undergo a reduction in volume – particularly imported sand fill for bedding and blinding layers. However, this usually occurs as immediate settlement and should normally be the province of the builder.

Seasonal swelling and shrinkage of soil
All clays react to the presence of water by slowly absorbing it, making the soil increase in volume (see table below). The degree of increase varies considerably between different clays, as does the degree of decrease during the subsequent drying out caused by fair weather periods. Because of the low absorption and expulsion rate, this phenomenon will not usually be noticeable unless there are prolonged rainy or dry periods, usually of weeks or months, depending on the land and soil characteristics.

The swelling of soil creates an upward force on the footings of the building, and shrinkage creates subsidence that takes away the support needed by the footing to retain equilibrium.

Shear failure
This phenomenon occurs when the foundation soil does not have sufficient strength to support the weight of the footing. There are two major post-construction causes:

- Significant load increase.
- Reduction of lateral support of the soil under the footing due to erosion or excavation.
- In clay soil, shear failure can be caused by saturation of the soil adjacent to or under the footing.

GENERAL DEFINITIONS OF SITE CLASSES

Class	Foundation
A	Most sand and rock sites with little or no ground movement from moisture changes
S	Slightly reactive clay sites with only slight ground movement from moisture changes
M	Moderately reactive clay or silt sites, which can experience moderate ground movement from moisture changes
H	Highly reactive clay sites, which can experience high ground movement from moisture changes
E	Extremely reactive sites, which can experience extreme ground movement from moisture changes
A to P	Filled sites
P	Sites which include soft soils, such as soft clay or silt or loose sands; landslip; mine subsidence; collapsing soils; soils subject to erosion; reactive sites subject to abnormal moisture conditions or sites which cannot be classified otherwise

Tree root growth
Trees and shrubs that are allowed to grow in the vicinity of footings can cause foundation soil movement in two ways:

- Roots that grow under footings may increase in cross-sectional size, exerting upward pressure on footings.
- Roots in the vicinity of footings will absorb much of the moisture in the foundation soil, causing shrinkage or subsidence.

Unevenness of Movement

The types of ground movement described above usually occur unevenly throughout the building's foundation soil. Settlement due to construction tends to be uneven because of:

- Differing compaction of foundation soil prior to construction.
- Differing moisture content of foundation soil prior to construction.

Movement due to non-construction causes is usually more uneven still. Erosion can undermine a footing that traverses the flow or can create the conditions for shear failure by eroding soil adjacent to a footing that runs in the same direction as the flow.

Saturation of clay foundation soil may occur where subfloor walls create a dam that makes water pond. It can also occur wherever there is a source of water near footings in clay soil. This leads to a severe reduction in the strength of the soil which may create local shear failure.

Seasonal swelling and shrinkage of clay soil affects the perimeter of the building first, then gradually spreads to the interior. The swelling process will usually begin at the uphill extreme of the building, or on the weather side where the land is flat. Swelling gradually reaches the interior soil as absorption continues. Shrinkage usually begins where the sun's heat is greatest.

Effects of Uneven Soil Movement on Structures

Erosion and saturation
Erosion removes the support from under footings, tending to create subsidence of the part of the structure under which it occurs. Brickwork walls will resist the stress created by this removal of support by bridging the gap or cantilevering until the bricks or the mortar bedding fail. Older masonry has little resistance. Evidence of failure varies according to circumstances and symptoms may include:

- Step cracking in the mortar beds in the body of the wall or above/below openings such as doors or windows.
- Vertical cracking in the bricks (usually but not necessarily in line with the vertical beds or perpends).

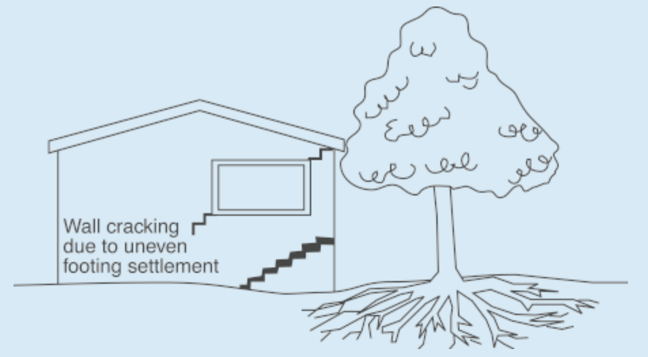
Isolated piers affected by erosion or saturation of foundations will eventually lose contact with the bearers they support and may tilt or fall over. The floors that have lost this support will become bouncy, sometimes rattling ornaments etc.

Seasonal swelling/shrinkage in clay
Swelling foundation soil due to rainy periods first lifts the most exposed extremities of the footing system, then the remainder of the perimeter footings while gradually permeating inside the building footprint to lift internal footings. This swelling first tends to create a dish effect, because the external footings are pushed higher than the internal ones.

The first noticeable symptom may be that the floor appears slightly dished. This is often accompanied by some doors binding on the floor or the door head, together with some cracking of cornice mitres. In buildings with timber flooring supported by bearers and joists, the floor can be bouncy. Externally there may be visible dishing of the hip or ridge lines.

As the moisture absorption process completes its journey to the innermost areas of the building, the internal footings will rise. If the spread of moisture is roughly even, it may be that the symptoms will temporarily disappear, but it is more likely that swelling will be uneven, creating a difference rather than a disappearance in symptoms. In buildings with timber flooring supported by bearers and joists, the isolated piers will rise more easily than the strip footings or piers under walls, creating noticeable doming of flooring.

Trees can cause shrinkage and damage



As the weather pattern changes and the soil begins to dry out, the external footings will be first affected, beginning with the locations where the sun's effect is strongest. This has the effect of lowering the external footings. The doming is accentuated and cracking reduces or disappears where it occurred because of dishing, but other cracks open up. The roof lines may become convex.

Doming and dishing are also affected by weather in other ways. In areas where warm, wet summers and cooler dry winters prevail, water migration tends to be toward the interior and doming will be accentuated, whereas where summers are dry and winters are cold and wet, migration tends to be toward the exterior and the underlying propensity is toward dishing.

Movement caused by tree roots
In general, growing roots will exert an upward pressure on footings, whereas soil subject to drying because of tree or shrub roots will tend to remove support from under footings by inducing shrinkage.

Complications caused by the structure itself
Most forces that the soil causes to be exerted on structures are vertical – i.e. either up or down. However, because these forces are seldom spread evenly around the footings, and because the building resists uneven movement because of its rigidity, forces are exerted from one part of the building to another. The net result of all these forces is usually rotational. This resultant force often complicates the diagnosis because the visible symptoms do not simply reflect the original cause. A common symptom is binding of doors on the vertical member of the frame.

Effects on full masonry structures
Brickwork will resist cracking where it can. It will attempt to span areas that lose support because of subsided foundations or raised points. It is therefore usual to see cracking at weak points, such as openings for windows or doors.

In the event of construction settlement, cracking will usually remain unchanged after the process of settlement has ceased.

With local shear or erosion, cracking will usually continue to develop until the original cause has been remedied, or until the subsidence has completely neutralised the affected portion of footing and the structure has stabilised on other footings that remain effective.

In the case of swell/shrink effects, the brickwork will in some cases return to its original position after completion of a cycle, however it is more likely that the rotational effect will not be exactly reversed, and it is also usual that brickwork will settle in its new position and will resist the forces trying to return it to its original position. This means that in a case where swelling takes place after construction and cracking occurs, the cracking is likely to at least partly remain after the shrink segment of the cycle is complete. Thus, each time the cycle is repeated, the likelihood is that the cracking will become wider until the sections of brickwork become virtually independent.

With repeated cycles, once the cracking is established, if there is no other complication, it is normal for the incidence of cracking to stabilise, as the building has the articulation it needs to cope with the problem. This is by no means always the case, however, and monitoring of cracks in walls and floors should always be treated seriously.

Upheaval caused by growth of tree roots under footings is not a simple vertical shear stress. There is a tendency for the root to also exert lateral forces that attempt to separate sections of brickwork after initial cracking has occurred.

The normal structural arrangement is that the inner leaf of brickwork in the external walls and at least some of the internal walls (depending on the roof type) comprise the load-bearing structure on which any upper floors, ceilings and the roof are supported. In these cases, it is internally visible cracking that should be the main focus of attention, however there are a few examples of dwellings whose external leaf of masonry plays some supporting role, so this should be checked if there is any doubt. In any case, externally visible cracking is important as a guide to stresses on the structure generally, and it should also be remembered that the external walls must be capable of supporting themselves.

Effects on framed structures
Timber or steel framed buildings are less likely to exhibit cracking due to swell/shrink than masonry buildings because of their flexibility. Also, the doming/dishing effects tend to be lower because of the lighter weight of walls. The main risks to framed buildings are encountered because of the isolated pier footings used under walls. Where erosion or saturation cause a footing to fall away, this can double the span which a wall must bridge. This additional stress can create cracking in wall linings, particularly where there is a weak point in the structure caused by a door or window opening. It is, however, unlikely that framed structures will be so stressed as to suffer serious damage without first exhibiting some or all of the above symptoms for a considerable period. The same warning period should apply in the case of upheaval. It should be noted, however, that where framed buildings are supported by strip footings there is only one leaf of brickwork and therefore the externally visible walls are the supporting structure for the building. In this case, the subfloor masonry walls can be expected to behave as full brickwork walls.

Effects on brick veneer structures
Because the load-bearing structure of a brick veneer building is the frame that makes up the interior leaf of the external walls plus perhaps the internal walls, depending on the type of roof, the building can be expected to behave as a framed structure, except that the external masonry will behave in a similar way to the external leaf of a full masonry structure.

Water Service and Drainage

Where a water service pipe, a sewer or stormwater drainage pipe is in the vicinity of a building, a water leak can cause erosion, swelling or saturation of susceptible soil. Even a minuscule leak can be enough to saturate a clay foundation. A leaking tap near a building can have the same effect. In addition, trenches containing pipes can become watercourses even though backfilled, particularly where broken rubble is used as fill. Water that runs along these trenches can be responsible for serious erosion, interstrata seepage into subfloor areas and saturation.

Pipe leakage and trench water flows also encourage tree and shrub roots to the source of water, complicating and exacerbating the problem.
Poor roof plumbing can result in large volumes of rainwater being concentrated in a small area of soil:

- Incorrect falls in roof guttering may result in overflows, as may gutters blocked with leaves etc.
- Corroded guttering or downpipes can spill water to ground.
- Downpipes not positively connected to a proper stormwater collection system will direct a concentration of water to soil that is directly adjacent to footings, sometimes causing large-scale problems such as erosion, saturation and migration of water under the building.

Seriousness of Cracking

In general, most cracking found in masonry walls is a cosmetic nuisance only and can be kept in repair or even ignored. The table below is a reproduction of Table C1 of AS 2870.

AS 2870 also publishes figures relating to cracking in concrete floors, however because wall cracking will usually reach the critical point significantly earlier than cracking in slabs, this table is not reproduced here.

Prevention/Cure

Plumbing
Where building movement is caused by water service, roof plumbing, sewer or stormwater failure, the remedy is to repair the problem. It is prudent, however, to consider also rerouting pipes away from the building where possible, and relocating taps to positions where any leakage will not direct water to the building vicinity. Even where gully traps are present, there is sometimes sufficient spill to create erosion or saturation, particularly in modern installations using smaller diameter PVC fixtures. Indeed, some gully traps are not situated directly under the taps that are installed to charge them, with the result that water from the tap may enter the backfilled trench that houses the sewer piping. If the trench has been poorly backfilled, the water will either pond or flow along the bottom of the trench. As these trenches usually run alongside the footings and can be at a similar depth, it is not hard to see how any water that is thus directed into a trench can easily affect the foundation's ability to support footings or even gain entry to the subfloor area.

Ground drainage
In all soils there is the capacity for water to travel on the surface and below it. Surface water flows can be established by inspection during and after heavy or prolonged rain. If necessary, a grated drain system connected to the stormwater collection system is usually an easy solution.

It is, however, sometimes necessary when attempting to prevent water migration that testing be carried out to establish watertable height and subsoil water flows. This subject is referred to in BTF 19 and may properly be regarded as an area for an expert consultant.

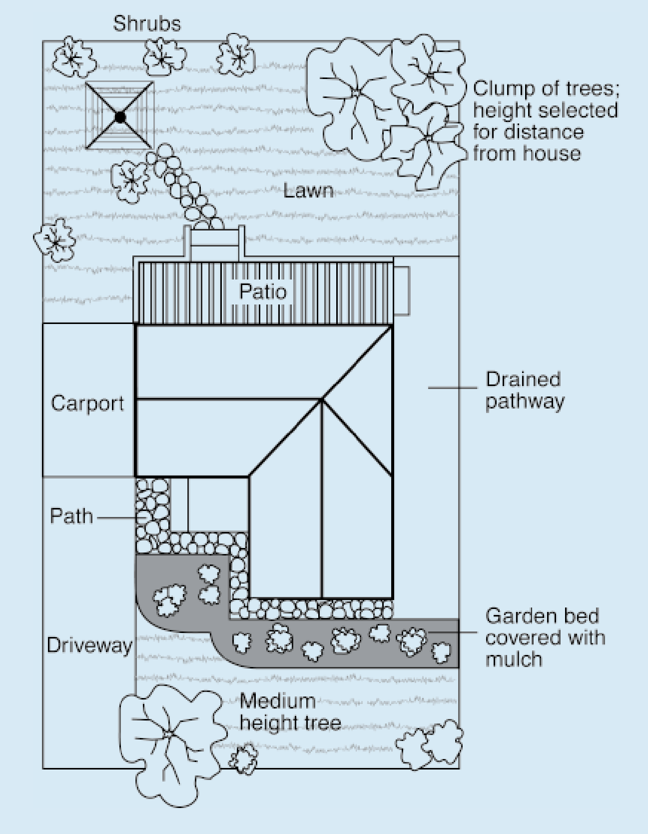
Protection of the building perimeter
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

For this reason, particularly where problems exist or are likely to occur, it is recommended that an apron of paving be installed around as much of the building perimeter as necessary. This paving

CLASSIFICATION OF DAMAGE WITH REFERENCE TO WALLS

Description of typical damage and required repair	Approximate crack width limit (see Note 3)	Damage category
Hairline cracks	<0.1 mm	0
Fine cracks which do not need repair	<1 mm	1
Cracks noticeable but easily filled. Doors and windows stick slightly	<5 mm	2
Cracks can be repaired and possibly a small amount of wall will need to be replaced. Doors and windows stick. Service pipes can fracture. Weathertightness often impaired	5–15 mm (or a number of cracks 3 mm or more in one group)	3
Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Window and door frames distort. Walls lean or bulge noticeably, some loss of bearing in beams. Service pipes disrupted	15–25 mm but also depend on number of cracks	4

Gardens for a reactive site



should extend outwards a minimum of 900 mm (more in highly reactive soil) and should have a minimum fall away from the building of 1:60. The finished paving should be no less than 100 mm below brick vent bases.

It is prudent to relocate drainage pipes away from this paving, if possible, to avoid complications from future leakage. If this is not practical, earthenware pipes should be replaced by PVC and backfilling should be of the same soil type as the surrounding soil and compacted to the same density.

Except in areas where freezing of water is an issue, it is wise to remove taps in the building area and relocate them well away from the building – preferably not uphill from it (see BTF 19).

It may be desirable to install a grated drain at the outside edge of the paving on the uphill side of the building. If subsoil drainage is needed this can be installed under the surface drain.

Condensation

In buildings with a subfloor void such as where bearers and joists support flooring, insufficient ventilation creates ideal conditions for condensation, particularly where there is little clearance between the floor and the ground. Condensation adds to the moisture already present in the subfloor and significantly slows the process of drying out. Installation of an adequate subfloor ventilation system, either natural or mechanical, is desirable.

Warning: Although this Building Technology File deals with cracking in buildings, it should be said that subfloor moisture can result in the development of other problems, notably:

- Water that is transmitted into masonry, metal or timber building elements causes damage and/or decay to those elements.
- High subfloor humidity and moisture content create an ideal environment for various pests, including termites and spiders.
- Where high moisture levels are transmitted to the flooring and walls, an increase in the dust mite count can ensue within the living areas. Dust mites, as well as dampness in general, can be a health hazard to inhabitants, particularly those who are abnormally susceptible to respiratory ailments.

The garden

The ideal vegetation layout is to have lawn or plants that require only light watering immediately adjacent to the drainage or paving edge, then more demanding plants, shrubs and trees spread out in that order.

Overwatering due to misuse of automatic watering systems is a common cause of saturation and water migration under footings. If it is necessary to use these systems, it is important to remove garden beds to a completely safe distance from buildings.

Existing trees

Where a tree is causing a problem of soil drying or there is the existence or threat of upheaval of footings, if the offending roots are subsidiary and their removal will not significantly damage the tree, they should be severed and a concrete or metal barrier placed vertically in the soil to prevent future root growth in the direction of the building. If it is not possible to remove the relevant roots without damage to the tree, an application to remove the tree should be made to the local authority. A prudent plan is to transplant likely offenders before they become a problem.

Information on trees, plants and shrubs

State departments overseeing agriculture can give information regarding root patterns, volume of water needed and safe distance from buildings of most species. Botanic gardens are also sources of information. For information on plant roots and drains, see Building Technology File 17.

Excavation

Excavation around footings must be properly engineered. Soil supporting footings can only be safely excavated at an angle that allows the soil under the footing to remain stable. This angle is called the angle of repose (or friction) and varies significantly between soil types and conditions. Removal of soil within the angle of repose will cause subsidence.

Remediation

Where erosion has occurred that has washed away soil adjacent to footings, soil of the same classification should be introduced and compacted to the same density. Where footings have been undermined, augmentation or other specialist work may be required. Remediation of footings and foundations is generally the realm of a specialist consultant.

Where isolated footings rise and fall because of swell/shrink effect, the homeowner may be tempted to alleviate floor bounce by filling the gap that has appeared between the bearer and the pier with blocking. The danger here is that when the next swell segment of the cycle occurs, the extra blocking will push the floor up into an accentuated dome and may also cause local shear failure in the soil. If it is necessary to use blocking, it should be by a pair of fine wedges and monitoring should be carried out fortnightly.

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Distributed by

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