Geotechnical Consultants Australia

MHM Construction Group

Geotechnical Investigation Report

Proposed Development at:
262 Aberglasslyn Road
Aberglasslyn NSW 2320

G22572-1

5th December 2022

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Address: 262 Aberglasslyn Road Aberglasslyn NSW 2320

GCA Report No.:	G22572-1
Date:	5th December 2022

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Report Revision	Details	Report No.	Date	Amended By
0	Original Report	G22572-1	5th December 2022	-
Issued By:			Joe Nader	

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1. INTRODUCTION

1.1 Background

This geotechnical engineering report presents the results of a geotechnical investigation undertaken by Geotechnical Consultants Australia Pty Ltd (GCA) for a proposed development at No. 262 Aberglasslyn Road Aberglasslyn NSW 2320 (the site). The investigation was commissioned by Mr. Geoff Hart of MHM Construction Group (the client) and was carried out on the 22nd November 2022.

The purpose of the investigation was to assess the subsurface conditions over the site at the selected boreholes and testing locations (where accessible and feasible), and provide necessary recommendations from a geotechnical perspective for the proposed development.

The findings presented in this report are based on our subsurface investigation, laboratory testing results and our experience with subsurface conditions in the area. This report presents our assessment of the geotechnical conditions, and has been prepared to provide preliminary geotechnical 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 engineering reports.

1.2 Proposed Development

Information provided by the client indicates the proposed development comprises demolition of the existing infrastructures within the southern portion of the site, followed by construction of a new childcare centre facility, overlying a single basement level for carparking and storage.

The Finished Floor Levels (FFL)s for the proposed development are set to be at Reduced Levels (RL)s of:

- Basement level: RL34.700m Australian Height Datum (AHD).
- Ground floor level: RL37.300m AHD.

Based on this information and the existing site levels and topography, maximum inferred excavation depths varying from approximately 1.0m to 3.1m are expected to be required for construction of the proposed development, with cut and fill in certain areas of the site. Locally deeper excavations for the proposed lift shaft, building footings and service trenches are also anticipated as part of the planned development.

It should be noted that excavation depths are expected to vary across the site and are inferred off the FFLs shown on the architectural drawings, referenced in Section 1.3 below.

1.3 Provided Information

The following relevant information was provided to GCA prior to the geotechnical investigation and during preparation of this report:

- Architectural drawings prepared by Greenscape Design & Associates, titled "Proposed Childcare Center At 262 Aberglasslyn Rd, Aberglasslyn NSW", and referenced project No. 201002.

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1.4 Geotechnical Assessment Objectives

The objective of the geotechnical investigation was to assess the site surface and subsurface conditions at the selected boreholes and testing locations within the site (where accessible and feasible), and to provide professional geotechnical advice and recommendations on the following based on requirements provided to GCA by the client:

- 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 to restrict any ground vibrations.
- Recommendations on suitable shoring (retention) 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 ground conditions within the site.
- Groundwater levels which may be determined during the geotechnical investigation.
- Recommendations on groundwater maintenance and limiting.
- Preliminary site lot classification in accordance with Australian Standards (AS) 2870-2011.
- Preliminary subsoil class for earthquake design for the site in accordance with AS 1170.4-2007.
- Preliminary slope risk assessment in accordance with guidelines published by the Australian Geomechanics Society (AGS) "Practice Note Guidelines for Landslide Risk Management – AGS 2007c".
- Aggressivity and salinity assessment within the site based on laboratory testing results at the selected borehole locations.
- General geotechnical advice on site preparation, filling and subgrade preparation.

1.5 Scope of Works

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

- Submit and review Before You Dig Australia (BYDA) plans and any other plans provided by the client on existing buried services within the site.
- Service locating carried out using electromagnetic detection equipment to ensure the area is free of any underground services at the selected boreholes and testing locations.
- Review of site plans and drawings to determine appropriate testing locations (where accessible and feasible), and identify any relevant features of the site.
- Machine drilling of four (4) boreholes at selected locations within the site (where accessible and feasible) by a specialised trailer mounted drilling rig, using solid flight augers equipped with a 'Tungsten Carbide' (TC) bit, and identified as boreholes BH1 to BH4 inclusive. The drilling rig is owned and operated by a specialist subcontractor.
    - The boreholes were drilled to varying practical TC bit terminated/refusal depths of approximately 3.0m to 5.0m below the existing ground level within the site (bgl).
- Dynamic Cone Penetrometer (DCP) testing immediately adjacent to boreholes BH1 and BH3 using hand operated equipment to varying practical terminated/refusal depths of approximately 1.16m to 2.3m bgl. The DCP tests are identified as DCP1 and DCP2.
        - The approximate locations of the boreholes and DCP tests are shown on **Figure 2, Appendix B** of this report.
- Collection of soil and rock samples during drilling for the following laboratory testing required:
    - Laboratory testing by a National Association of Testing Authorities, Australia (NATA)

accredited laboratory (ALS Environmental) on four (4) selected samples collected during drilling of the boreholes, to determine the pH, chloride and sulphate content, and electrical conductivity of the selected samples.

- Reinstatement of the boreholes with available soil displaced during drilling.
- Preparation of this geotechnical engineering report.

1.6 Constraints

The discussions and recommendations provided in this report have been based on the results obtained at the selected testing locations within the site (where accessible and feasible). It is recommended that further geotechnical inspections be carried out during construction to confirm the subsurface conditions across the site and foundation bearing capacities have been achieved.

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 located within a residential area along Aberglasslyn Road carriageway, approximately 2.2km north-east of New England Highway thoroughfare.
Site Address	262 Aberglasslyn Road Aberglasslyn NSW 2320
Approximate Site Area[1]	8,250m^2
Local Government Authority	Maitland City Council
Site Description	At the time of the investigation, residential dwellings were present within the site, accompanied by associated concrete pavements and a detached shed. The remaining site area was mainly covered in grass, vegetation and mature trees scattered throughout.
Approximate Distances to Nearest Watercourses (i.e. rivers, lakes, creeks, etc.)	• Tributary of Hunter River – 80m north-east of the site. • Hunter River – 1.2km north-east of the site.
Site Surroundings	The site is located within an area of residential use and is bounded by: • Residential properties at No. 27, No. 29, No. 31, No. 33, No. 35 and No. 37 Finch Crescent to the north-east. • Residential properties at No. 4, No. 6, No. 8 and No. 10 Gilder Close to the south-east. • Aberglasslyn Road carriageway to the south-west. • Residential property at No. 266 Aberglasslyn Road to the north-west.

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

2.2 Topography

The local and site topography generally falls towards the north to north-east. We note that no site survey plan was provided to GCA during preparation of this report.

Preliminary assessment of the site area indicates an overall very gentle to moderate slope of approximately 2° to 5° (varying throughout).

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It should be noted that the site topography, levels and slopes are approximate only and based off measurements undertaken off Mecone Mosaic (https://meconemosaic.au/), observations made during the site investigation and reference to NSW Six Maps (https://maps.six.nsw.gov.au/). It should also be noted that some areas within the site were observed to have slightly gentler/steeper slopes than those discussed in this report.

The actual topography in areas inaccessible during the site investigation (i.e. dense vegetation, trees, existing infrastrcutures, etc.), along with the site and local topography and levels are expected to vary from those outlined in this report.

2.3 Regional Geology

Information obtained on the local regional subsurface conditions, referenced from the Department of Mineral Resources, Newcastle 1:100,000 Geological Series Sheet 9232, dated 1975, by the Geological Survey of New South Wales, indicates the site is located approximately at a geological boundary/region generally underlain by Permian Aged Conglomerate (Pmb) and Permian Aged Sandstone (Pd) of the Maitland Group.

The Conglomerate (Pmb) normally comprises "sandstone, siltstone, conglomerate and erratics", and the Sandstone (Pd) generally contains "sandstone, mudstone, siltstone, shale, tuff, basalt flows and erratics".

Furthermore, reference made to MinView by the State of New South Wales through Regional NSW 2021 specifies the site is positioned within a geological region underlain by Conglomerate (Pmtb) and in close proximity to Sandstone (Pmtu).

A review of the regional maps by the NSW Government Environment and Heritage shows the site is set within the Bolwarra Heights (bh) landscape group. The Bolwarra Heights (bh) landscape group is generally recognised by rolling low hills on Permian sediments in the centre-west of the sheet in the East Maitland Hills region. Slopes are 5% to 20%, elevation to 100m and local relief to 80m. Soils of the Bolwarra Heights group typically have moderate foundation hazard, water erosion hazard, high run-on (localised), seasonal waterlogging (localised) and localised steep slopes with mass movement hazard. Soils of the Bolwarra Heights group are generally slightly (pH 6.5) to strongly (pH 4.5) acidic.

The Bolwarra Heights (bh) landscape group report is attached in **Appendix H**.

3. SUBSURFACE CONDITIONS AND ASSESSMENT RESULTS

3.1 Stratigraphy

A summary of the surface and subsurface conditions from across the site during this geotechnical investigation are summarised in Table 2 below and are interpreted from the assessment results. It should be noted that 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.

It should be noted that estimated rock strengths and soil consistency/strength assessed by observation during auger drilling penetration resistance and DCP testing, respectively, are approximate and variances should be expected throughout the site. It is worth noting that auger penetration within various bedrock formations vary from each drilling rig and estimated rock strength variances across the site are expected.

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 borehole drilling 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 program, no matter how comprehensive, can reveal and identify all subsurface conditions underlying the site.

Based on the geotechnical investigation at the selected boreholes and DCP testing locations, along with our experience and observations made within the site and local region, it is inferred that bedrock of variable composition, strength and weathering is underlying majority of the proposed development area at varying depths of approximately 1.0m to 2.7m bgl (expected to vary throughout).

In addition, variable composition and consistency/strength natural soils are also likely to be present throughout the site, predominately at locations and depths not assessed during the geotechnical investigation.

It should be noted that DCP testing and higher blow counts encountered may be affected by factors such as gravels, ironstone bands, well consolidated soils and highly cemented sands, and other deleterious materials which may be present within the underlying soils, along with tree rootlets extending throughout the soils from trees and vegetation within the vicinity. These results should be read in conjunction with the boreholes and geotechnical confirmation made during construction by inspection, as site conditions may vary. DCP testing results are attached in **Appendix E**.

Table 2. Summary of Subsurface Conditions

			Borehole/DCP ID	**BH1 (DCP1)**	**BH2**	**BH3 (DCP2)**	**BH4**
Unit	**Unit Type**	**Description**	**Estimated Consistency/ Strength**	**Depth/Thickness of Unit (m bgl)**			
1	Fill	Silty CLAY, gravel inclusions.	N/A	0.0 – 0.5	0.0 – 1.0	0.0 – 0.8	0.0 – 0.6
2	Natural Soils	Silty CLAY, medium to high plasticity, gravel inclusions.	Soft to firm, becoming stiff, then very stiff to hard with depth	0.5 – 2.0	–	0.8 – 1.2	0.6 – 2.7
3	Bedrock[1]	CONGLOMERATE, clay and sand matrix, occasional sandstone clasts.	*Inferred* very low to low, grading low to medium estimated strength with depth.	2.0 – 5.0	1.0 – 4.5	1.2 – 4.0	2.7 – 3.0

[1]The composition, class, depth and estimated strength of the underlying bedrock material should be confirmed prior to construction by further borehole drilling and rock strength testing, or during construction by inspection.

Notes:

- N/A = Not Applicable.
- Clay seams, defects and fractured/extremely weathered zones are expected to be present throughout the underlying bedrock, predominately at depths and locations unobserved during the geotechnical investigation.
- Estimated rock strengths are based on observations made during auger penetration resistance at the time of drilling.
- Bedrock estimated strength is expected to vary across the site, due to the limited investigation carried out.
- Ground conditions are expected to vary across the site and should be confirmed by a geotechnical engineer, predominately in areas unobserved during the geotechnical investigation.

3.2 Groundwater

No groundwater was encountered or observed during or shortly after (<15 minutes) drilling of the boreholes to a maximum depth of about 5.0m bgl (BH1).

It is noted that the boreholes were backfilled following completion of fieldwork which precluded longer term monitoring of groundwater levels. Although no groundwater was encountered or observed during the investigation, its presence should not be precluded within the site and during construction.

Thus, based on the above observations and data available at the time of reporting, groundwater which may be present within the site is *expected* to be in the form of seepage through voids within the underlying fill material and pore spaces between particles of unconsolidated natural soils, or through networks of fractures and solution openings in consolidated bedrock underlying the site.

It should also 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.

Groundwater monitoring should be carried out during construction to assess any groundwater inflows within the site as no provision was made for longer term groundwater monitoring. Where groundwater conditions vary from those outlined in this report, GCA should be contacted for further advice.

4. PRELIMINARY LANDSLIDE RISK ASSESSMENT

4.1 General

The overall stability of the site including slope angles, depth of soils and overall strength, movements of groundwater and surface runoff, drainage and potential slides planes within the interfaces of rocks and soils were assessed by GCA as part of the geotechnical investigation. The overall assessment was carried out in accordance with guidelines published by the AGS "Practice Note Guidelines for Landslide Risk Management – AGS 2007c".

Due to the sloping nature of this site, a geotechnical investigation and assessment in accordance with guidelines published by the AGS was carried out in order to demonstrate that the proposed development is justified in terms of geotechnical stability. Therefore, the following sections are a preliminary assessment based on the AGS guidelines for the stability of the site prior and following construction.

It should be noted that this preliminary landslide risk assessment is limited to the proposed development area and areas accessible during the time of our site investigation, including information available at the time of reporting.

4.2 Site Assessment

The overall site area and topography generally slopes towards the north to north-east, as discussed in Section 2.2 of this report. Table 3 summarises results of the overall stability within the site.

Table 3. Summary of Overall Site Stability

Observations		Identification	Comments
Site Topography		N/A	The overall site area varies throughout and slopes generally towards the north to north-east, as discussed in Section 2.2 and shown in **Figure 1, Appendix B** of this report. Reference should be made to this section and site plan for a general description of the site area.
Overall Site Description		N/A	The site area was generally covered in healthy mature trees, vegetation and grass. Associated concrete pavements, retaining walls, and the existing dwellings covered the remaining site area.
Groundwater		No	No groundwater was encountered or observed during the geotechnical investigation at all testing locations, as discussed in Section 3.2. It is expected that groundwater which may be encountered within the site will be in the form of seepage through the voids within the underlying soils and defects in the underlying inferred bedrock. Based on the regional and site topography, we expect groundwater flows (including surface water) to flow towards the north to north-east (varying throughout).
Surface Water		No	No surface water or ponding, seepage or drainage pathways were observed within the site. Soils were predominately moist and comprise mainly fill material and natural soils, underlain by bedrock at varying depths throughout the proposed development area (as discussed in Section 3).
Outcrops		No	N/A
Loose Boulders or Rock Mass'		No	No loose boulders were observed across the site and adjoining properties.
Bedrock Deterioration		N/A	N/A
Structural Distress	Existing Dwelling and Infrastructures	No	No sign of structural distress or movement were observed to the existing dwellings and infrastructures within the site. Any cracks observed within the concrete pavements are inferred to be associated with concrete shrinkage and loading over time
	Retaining Walls		No sign of structural distress or movement were observed to any retaining walls throughout the site.
Adjoining Properties		N/A	Infrastructures adjoining the site were observed to be in a generally good condition and trees within the vicinity were observed to have no sign of deformation.
Ground Movement		No	No signs of cracks in the ground, slumping, or other signs of landslip observed within the site. No ground deformation was observed within the site (where accessible and feasible).

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Tilting or Bending Trees	No	No trees showed any signs of tilting or bending at the time of the investigation within the site and investigation area. Typically, tilting, bending or curved trees can indicate rotation due to soil creep or movement (where accessible and feasible).
Soil Creep or Shallow Failure	No	No sign of soil creep or shallow failure was observed within the fill material or any natural soils present within the site, and throughout the site and adjoining properties (where accessible and feasible).

It should be noted that trees, vegetation and grass present within the site are considered to contribute to the stability of the site. Retaining walls in their current state are also considered to contribute to the stability and retention of the soils behind the retaining walls.

Based on the subsurface conditions encountered within the site during the geotechnical investigation, it is anticipated that fill material and natural soils will underlie the majority of the proposed development area, overlying bedrock at varying depths throughout, as discussed in Section 3 above.

4.3 Pre-Development (Assessed Risk to Property)

Based on the geotechnical investigation, site topography and existing ground conditions within the site, assessment on the potential effects which may be associated with the hazards on the adjoining properties, along with the buildings, lands and occupiers within the adjoining properties, and existing dwelling/proposed development have been considered as part of the risk levels to the property pre-development, and is summarised in Table 4 below.

Table 4. Pre-Development – Assessed Risk To Property

Potential Hazard	Qualitative Measures of Likelihood (AGS)	Qualitative Measures of Consequences to Property (AGS)	Qualitative Risk Analysis – Level of Risk to Property (AGS)
Soil Creep[1]	C – Possible (10^{-3})	3 – Medium (20%)	Moderate
Shallow Failure[1]	C – Possible (10^{-3})	3 – Medium (20%)	Moderate

[1]Within the fill material and natural soils present within the site.

Based on the assessed conditions within the site, the overall slope instability assessed risk to the property under the existing conditions prior to construction of the currently proposed development is assessed to be “moderate”.

According to AGS 2007c, the “moderate risk level” may be tolerated in certain circumstances, however, requires investigation, planning and implementation of treatment options which will be required to reduce the risk to “low level risk”.

It should be noted that the AGS guidelines recommend tolerable loss of life for the person most at risk for exiting slope/existing development to be 1 x 10^{-4}/annum.

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4.4 Mitigation and Control Measures

To ensure the stability of the site and to reduce the risk (to "low" risk) of any instability for the proposed development within the site, the following recommendations should be considered along with recommendations presented in this report (not limited to):

- The design and construction of earthworks, foundations, retaining structures, excavation stabilisation and drainage measure for the proposed development should adhere to good engineering practice for hillside construction as set out in Appendix G of AGS 2007c Vol. 42 guidelines, attached as **Appendix I** in this report.
- Any cause of instability of the ground profile within the site and neighbouring properties should be addressed prior to any excavation or construction work, and proposed stabilisation actions should be implemented. In this case, GCA should be contacted on further geotechnical advice for any stabilisations actions which may be required.
- Excavation, pile installation and any rock ripping and hammering (or the like) are expected to cause vibrations within the underlying bedrock. Monitoring of any existing retaining walls, soils and bedrock underlying the site should be carried out and inspected by a geotechnical engineer during construction.
    - Any observable movement within the underlying soils and/or any retaining walls should cease work immediately, and GCA be contacted for further advice.
- Any excavation should be monitored by a suitably qualified geotechnical engineer, which should monitor ground movement and vibrations, as well as any retaining walls or infrastructures within and adjoining the site.
- Any vertical cut or fill exceeding 0.5m in depth within soils should be retained by an appropriately designed retaining wall. This should include construction of an appropriately designed retention system along all elevation walls of the proposed basement level, in order to reduce the potential for soil/bedrock movement following excavation. Section 6.9 provides further recommendations.
- All retaining walls should be designed using appropriate geotechnical design parameters for the subject site and ground conditions provided in Section 6.9.3 below.
- Any excavation should be commenced from higher levels and should be carried out in stages progressing towards the lower levels within the site. Excavations (including any batter slopes) are to be monitored and approved by a geotechnical engineer familiar with the site conditions.
- Backfilling should be placed and compacted to engineering standards in accordance with AS 3798-2007 and AS 1289, with reference to Section 6.11 and Section 6.12 of this report. This includes all batters, pavements, driveways, etc.
    - Reference should be made to these sections for preparation of pavements within the site. Further advice should also be sought from GCA prior and during construction.
- Backfilling behind any walls should also be carried out in accordance with AS 3798-2007 and AS 1289. This should include appropriate materials, compaction criteria and testing, site preparation and fill construction, methods of testing and inspections, and constant testing. Appropriate backfill drainage should also be provided.
- Appropriate drainage methods should be incorporated to ensure all surface and subsurface water flows are diverted away from the slopes, adjoining properties and proposed development, into a stormwater drainage system or appropriate discharge. This includes appropriate drainage behind the proposed garage (basement) walls, as discussed in Section 6.8.
    - All stormwater and drainage within the site should be in accordance with the approved stormwater engineering drawing.
- The foundation system for the proposed development should comprise combination of shallow footings and piles sufficiently founded/embedded into consistent and competent bedrock underlying the site, as discussed in Section 6.10 of this report. Piles are necessary in order to increase resistance against sliding.

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- Foundation systems for the proposed development, building structures, retaining walls and any water tanks, etc., should be sufficiently founded/embedded into the underlying bedrock, and where necessary designed for lateral earth pressures induced by soil movement along the interface between soils and the underlying bedrock.
- Foundations should be inspected and approved by a suitably qualified geotechnical engineer, with all structural elements also inspected and approved by the project structural engineer.
- All retaining walls and footings to be designed by a qualified structural engineer in accordance with recommendations in this report, and any future geotechnical investigation report which may be necessary for the site.
- Maintenance and inspection of permanent retaining walls should be carried out regularly.
- Inspection of surface and subsurface movement following any removal of trees or vegetation within the site.
- Plantation of trees and vegetation following construction of any proposed development in the future. Specific advice should be sought on plantation of trees near structures from AS 2870-2011.
- Construction activities should be carefully observed by a geotechnical engineer, where further assessment and necessary mitigation and control measures may be provided.
- Care should be taken for all construction activities within the site, with constant supervision by the project site manager, geotechnical engineer and structural engineer. Any observable movement within the underlying soils and/or retaining walls should cease work immediately, and GCA be contacted for further advice.
- Vibration levels during excavation and construction should be maintained to appropriate levels within the site, predominately where existing sensitive structures exist. Section 6.7 of this report provides general guidance on recommended vibration control measures.

Implementation of the measures recommended in this report (not limited to these measures) should constitute as “Hold Points”.

4.5 Quantitative Risk Assessment (Risk to Life)

The annual probability of loss of life (death) of an individual post-development has been calculated using the following formula:

$$R_{(LOL)} = P_{(H)} \times P_{(S:H)} \times P_{(T:S)} \times v_{(D:T)}$$

Where:

R (LOL)	is the risk (annual probability of loss of life (death) of an individual).
P (H)	is the annual probability of the landslide.
P (S:H)	is the probability of spatial impact of the landslide impacting a building (location) taking into account the travel distance and travel direction given the event.
P (T:S)	is the temporal spatial probability (e.g. of the building or location being occupied by the individual) given the spatial impact and allowing for the possibility of evacuation given there is warning of the landslide occurrence.
V (D:T)	is the vulnerability of the individual (probability of loss of life of the individual given the impact).

It should be noted that the AGS guidelines recommend tolerable loss of life for the person most at risk for a new development to be 1 x 10^{-5}/annum.

Annual Probability of Landslide

No evidence of movement was observed on the site during the time of the investigation.

P (H) = 0.0001/annum

Probability of Spatial Impact

The construction of the proposed childcare centre facility is anticipated to be located throughout the higher end of the site. Review of the proposed developments architectural drawings and from onsite investigations, we anticipate an area of approximately 2,080m^2 to be at risk of soil creep and shallow failure, which is roughly 25% of the total site area.

P (S:H) = 0.3

Possibility of the Location Being Occupied During Failure

The childcare is taken to be occupied by 110 people. It is estimated/assumed that 110 people are in the childcare for 18 hours a day, 5 days a week.

$$\left(\frac{110}{110} \times \frac{18}{24} \times \frac{5}{7}\right) = 0.54$$

P (T:S) = 0.54

Probability of Loss of Life on Impact of Failure

Based on the volume of land sliding and its likely velocity when it impacts the childcare, it is estimated that the vulnerability of a person to being killed in the structure when a landslide hits is 0.1.

V (D:T) = 0.1

Risk Estimation

R (LOL) = 0.0001 x 0.3 x 0.54 x 0.1

= 0.00000162

R (LOL) = 1.62 x 10^{-6}/annum.

Therefore, in accordance with AGS (2007c) this level of risk is considered to be "ACCEPTABLE".

4.6 Post-Development (Assessed Risk to Property)

Based on the proposed development and existing site conditions, maximum inferred excavation depths varying from approximately 1.0m to 3.1m are expected to be required for construction of the proposed development, with cut and fill in certain areas. Locally deeper excavations for the proposed lift shaft, building footings and service trenches are also anticipated to be required.

Therefore, appropriate measures against the potential for any instability should be incorporated into the design and construction of the proposed development, predominately into the design and construction of the building foundations and any retaining walls, as discussed and outlined in this report.

On the condition that the recommendations and design parameters presented in this report are taken into consideration during the design and construction of the proposed development, as well as post construction, the following assessed risks relating to the stability of the property upon completion of any infrastructures, building foundations and retaining walls are presented in Table 5 below.

Table 5. Post-Development – Assessed Risk To Property

Potential Hazard	Qualitative Measures of Likelihood (AGS)	Qualitative Measures of Consequences to Property (AGS)	Qualitative Risk Analysis – Level of Risk to Property (AGS)
Soil Creep[1]	D – Unlikely (10^{-4})	4 – Minor (5%)	Low
Shallow Failure[1]	D – Unlikely (10^{-4})	4 – Minor (5%)	Low

[1]Within the fill material and any natural soils present within the site.

Based on the assessed conditions within the site, the overall slope instability assessed risk to the property following construction of the currently proposed development is assessed to be "low".

Therefore, the above risk is considered acceptable for the proposed development within the site, providing the recommendations outlined in Section 4.4 and Section 6 are implemented for the design and construction of the proposed development.

Geotechnical inspections are to be undertaken during construction of the proposed development foundation system in order to confirm ground conditions and allowable bearing capacities have been achieved. Appropriate certifications should also be provided during staged inspections by the project structural engineer and geotechnical engineer.

5. LABORATORY TEST RESULTS

5.1 Aggressivity and Salinity

Four (4) selected samples were sent to a NATA accredited testing laboratory, ALS Environmental, to determine the pH, chloride and sulphate content, and electrical conductivity of the samples.

A summary of the laboratory tests results is provided in Table 6 below with laboratory certificates presented in **Appendix F** of this report.

Table 6. Summary of Laboratory Test Results (Aggressivity and Salinity)

Borehole ID		**BH1**	**BH2**	**BH3**	**BH4**
Approximate Depth (m bgl)		0.7 – 0.8	4.4 – 4.5	1.9 – 2.0	1.4 – 1.5
Strata Type		Natural Soils	Bedrock	Bedrock	Natural Soils
Aggressivity and Salinity	**pH**	5.4	5.8	5.8	5.4
	Moisture Content (%)	20.6	4.7	9.3	12.4
	Chloride (mg/kg)	150	180	90	160
	Sulphate SO_4 (mg/kg)	100	30	40	90
Electrical Conductivity (µS/cm)	**EC (µS/cm)**	95	140	86	160
	EC (dS/m)	0.095	0.140	0.086	0.160
	Multiplication Factor[1]	8	20	8	20
	Saturation Extract ECe (dS/m)	0.8	2.8	0.7	3.2

[1]Multipication factor obtained from NSW Government, Catchment Management Authority, "Calculating Electrical Conductivity and Salinity" and Department of Natural Resources (DNR) publication "Site Investigations for Urban Salinity" – 2002.

6. GEOTECHNICAL ASSESSMENT AND RECOMMENDATIONS

6.1 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 excavations and vicinity of the proposed development. A dilapidation survey will record the condition of existing defects prior to any works being carried out within the site. Preparation of a dilapidation report should constitute as a "Hold Point".

6.2 General Geotechnical Issues

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

- Aggressivity and salinity assessment.
- Preliminary site lot classification.
- Excavation conditions.
- Groundwater management.
- Stability of excavation and retention of adjoining properties and infrastructures.
- Preliminary earthquake site risk classification.
- Foundations.

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

6.3 Aggressivity and Salinity Assessment

In accordance to AS 2159-2009 "Piling – Design and Installation" (as outlined in Table 7 below), results of laboratory tests and introduction of a multiplication factor for electrical conductivity on the selected samples indicates the following classification:

Table 7. Aggressivity and Salinity Reference Table

Reference	Element Type	High Perm. Soils	Low Perm. Soils	pH	Chloride (mg/kg)	Sulphate SO_4 (mg/kg)
AS 2159-2009	Concrete Elements	Mild	Non	>5.5	N/A	<5,000
		Moderately	Mild	4.5 – 5.5		5,000 – 10,000
		Severely	Moderately	4.0 – 4.5		10,000 – 20,000
		Very Severely	Severely	<4.0		>20,000
	Steel Elements	Non	Non	>5.0	<5,000	N/A
		Mild	Non	4.0 – 5.0	5,000 – 20,000	
		Moderately	Mild	3.0 – 4.0	20,000 – 50,000	
		Severely	Moderately	<3.0	>50,000	
Dry Salinity 1993	Electrical Conductivity Saturation Extract ECe (dS/m) value range, based on an introduction of a multiplication factor from DNR publication.				Non-Saline <2 Slightly Saline 2 – 4 Moderately Saline 4 – 8 Very Saline 8 – 16 Highly Saline >16	

- Underlying natural soils (from boreholes BH1 and BH4):
  - Non aggressive for buried steel structural elements in low and high permeability soils.
  - Mildly aggressive for buried concrete structural elements in low permeability soils.
  - Moderately aggressive for buried concrete structural elements in high permeability soils.
  - Electrical conductivity of saturated extract (ECe) ranging from approximately 0.7ds/m to 0.8ds/m, indicating generally "non" saline natural soils underlying the site.
- Underlying bedrock (from boreholes BH2 and BH3):
  - Non aggressive for buried steel structural elements in low and high permeability soils.
  - Non aggressive for buried concrete structural elements in low permeability soils.
  - Mildly aggressive for buried concrete structural elements in high permeability soils.
  - Electrical conductivity of saturated extract (ECe) ranging from approximately 2.8ds/m to 3.2ds/m, indicating generally "slightly" saline bedrock underlying the site.

It should be note that soil aggressivity and salinity may vary throughout the site and is based on testing at the selected borehole locations to the maximum depths indicated, in conjunction with multiplication factors for electrical conductivity, as described above. Ground conditions and soil aggressivity and salinity are expected to vary across the site as discussed in this report since no geotechnical or geological exploration program, no matter how comprehensive, can reveal and identify all subsurface conditions underlying the site.

Consideration should be given to additional borehole drilling and laboratory testing following demolition of existing infrastructures onsite, in order to confirm the preliminary findings presented above.

6.4 Preliminary Site Lot Classification

Based on the geotechnical investigation and observations made at the selected testing locations within the site, fill and natural soils are expected to be underlain by bedrock at varying depths across the site 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 existing infrastructures and trees within and adjoining the site, causing abnormal and changing moisture conditions.
- The presence of deep fill material in certain areas of the site, considered as "uncontrolled fill".

Based on the boreholes and DCP tests carried out within the site, and proposed basement excavations which will result in removal of fill material and natural clayey soils, AS 2870-2011 indicates the site may be classified as a "Class M" site for design and construction of the proposed basement foundation system, founded below any soft/loose soils, topsoil, slopewash, fill or other deleterious material, being <u>entirely</u> on bedrock underlying the proposed development area.

A higher classification of "Class H1" should be adopted for structures built at ground surface level (i.e. portion of the proposed ground floor level, fences, etc.), or where fill and natural soils are present at depths of equal to or greater than 1.8m below the proposed developments FFLs. This should be confirmed/monitored during construction.

The above classification is solely based on assessment of the subsurface conditions at the selected boreholes and testing locations/depths within the site and current architectural drawings, and confirmation should be carried out as outlined in this report.

It should be noted that the classification given above is appropriate for the undeveloped lot at the time of this report and as such, AS 2870 recommends that the classification of a site should be reconsidered if the depth of subsequent cutting exceeds 0.5m or depth of subsequent filling exceeds 0.4m.

Foundation design and construction should be carried out as outlined in Section 6.10 below, with reference made to AS 2870-2011. Geotechnical inspections and confirmation of the actual depth of underlying fill material, natural soils and bedrock should be made during construction by inspection.

GCA should be contacted where ground conditions vary from those outlined in this report at the boreholes and testing locations. Where the building foundations are not proposed to be constructed on the bedrock underlying the site, GCA should also be contacted 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 preliminary 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 G**.

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6.5 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 excavations) of the proposed development. This should be carried out prior to any demolition, excavation or construction activities, 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 soils, which will determine the need for additional support, such as underpinning, prior to installation of shoring piles, or any demolition, excavation and construction activities.

6.6 Excavation

Maximum excavation depths varying from approximately 1.0m to 3.1m are expected to be required for construction of the proposed development, with cut and fill in certain areas of the site. Locally deeper excavations for the proposed lift shaft, building footings and service trenches are also anticipated as part of the planned development.

Based on this information and existing ground conditions as encountered during the geotechnical investigation, it is anticipated that excavations will extend through Unit 1 (fill) to Unit 3 (bedrock), throughout majority of the proposed development area, as discussed in Section 3 above.

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

Estimated bedrock strength variances and higher strength rock bands are expected across the site area. Therefore, consultation should be made with subcontractors to discuss the feasibility and capability of machinery for the proposed development for the existing site conditions.

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6.6.1 Excavation Assessment

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

Excavation of medium to higher estimated strength bedrock which is anticipated to be encountered during construction would necessitate 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, building footings and service trenches may require the use of heavy ripping and rock breaking equipment or vibratory rock breaking equipment, with the possibility of rock saw cutting.

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

Demolition, excavation and construction activities (or the like) will generate both vibration and noise, predominately whilst being carried out within the underlying bedrock. Therefore, vibration control measures should be considered as part of the construction process, mainly where excavations are expected to be conducted within the underlying bedrock of higher estimated strength and fall within the “zone of influence” of adjoining infrastructures.

All excavation works should be undertaken in accordance with the NSW WorkCover code of practice for excavation work.

6.7 Vibration Monitoring and Controls

Particular care will be required to ensure that adjacent buildings and infrastructures (i.e. road reserves, buildings, etc.), are not damaged during demolition, excavation and construction activities (or the like) due to excessive vibrations. Therefore, appropriate excavation and construction methods should be adopted which will limit ground vibrations to limits not exceeding the following maximum Peak Particle Velocity (PPV) for adjacent structures, as outlined in AS 2187.2-2006:

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

In order to reduce resonant frequencies, rock hammers should be used in short bursts and oriented away from the site boundaries and adjoining structures, and into the proposed excavation area.

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.

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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 8 below.

Table 8. 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.

Consideration should be given to a vibration monitoring plan to monitor construction activities and their effects on adjoining infrastructures, mainly where excavations are expected to be conducted within the underlying bedrock of higher estimated strength and fall within the “zone of influence” of 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. If adopted/considered, consultation should be made with appropriate subcontractors/consultants for the installation of vibration monitoring instruments.

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. Rock excavation methodology should also consider acceptable noise limits as per the “Interim Construction Noise Guideline” (NSW EPA). It is recommended a dilapidation report be carried out prior to any excavation or construction, as discussed in Section 6.1. This should be considered a “Hold Point”.

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6.8 Groundwater Management

Based on the geotechnical investigation at the selected borehole locations within the site (summarised in Section 3), *inferred* groundwater seepage which may be encountered during construction is expected to be at varying depths across the site and possibly above the proposed basement FFL.

It should be noted that no provision was made for longer term groundwater monitoring within the site, and the presence of groundwater should not be precluded during construction and in the long term design life of the proposed building. It should also be noted that these 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 any groundwater inflow into the excavation to be in the form of seepage through voids within the underlying soils and defects (such as bedding planes, joints, etc.) in the underlying weathered bedrock. Seepage may also occur within the excavation areas through 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 the voids in the natural soils and defects in the underlying inferred bedrock are drained, and 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):

- A conventional sump and pump system which may be used both during construction and for permanent groundwater control below the basement level floor slab.
- Drainage installed around the perimeter of the basement level behind all retaining walls, and below the 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 be given to waterproofing of the basement level walls and slabs, with allowance given for nominal hydrostatic uplift.

It is recommended that test pits are carried out by a suitable excavator within the site following demolition of the existing infrastructures and prior to construction, in order to confirm and monitor groundwater levels and inflow rates which may be intercepted during construction within the excavation areas.

This assessment should also be carried out to ensure a suitable drainage and retention system has been implemented for the proposed development, as discussed in Section 6.9 below, and to provide confirmation of the hydrogeological characteristics prior to construction.

Groundwater monitoring of seepage should also 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 stormwater engineer.

Should the proposed development change and excavation depths exceed those inferred in this report, GCA should be made aware.

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6.9 Excavation Stability

Maximum excavation depths are expected to vary within the site from approximately 1.0m to 3.1m for construction of the proposed development, with cut and fill in certain areas of the site. Locally deeper excavations for the proposed lift shaft, building footings and service trenches are also anticipated to be required.

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.

6.9.1 Batter Slopes

Temporary or permanent batters may be considered for certain areas of the proposed development where sufficient space exists between the proposed basement floor level walls and adjoining infrastructures. It should be noted that due to the nature of fill material, natural soils and weathered bedrock underlying the site, and the potential for elevated groundwater levels within the excavation area, unsupported vertical cuts of the soils carry the potential for slump failure.

Temporary or permanent batter slopes should only be considered where sufficient space exists between the proposed development and adjoining infrastructures, and where the adjacent infrastructures are located outside the "zone of influence" (obtained by drawing a line 45° above horizontal from the base of the proposed excavations).

Table 9 provides maximum recommended slopes for permanent and temporary batters.

Table 9. Recommended Maximum Batter Slopes

Unit		Maximum Batter Slope (H:V)[1]	
		Permanent	**Temporary**
Fill (Unit 1)		4:1	2:1
Natural (Clayey) Soils (Unit 2)		3:1	1.5:1 to 1:1
Conglomerate Bedrock (Unit 3)	**VL**	2:1	1:1 to 0.75:1
	L – M or better[2]	1:1	0.5:1

[1]Subject to inspection and confirmation by a geotechnical engineer or engineering geologist. Remedial options may be required (i.e. soil nailing, rock bolting, shotcreting, etc.).
[2]Preliminary only and inferred to be present within the site at depth. Assumes the presence of conglomerate bedrock underlying the entire site area.
Notes:
- VL = Very Low estimated strength, L = Low estimated strength, M = Medium 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 45° above horizontal from the base of the proposed excavations) plus an additional 1.0m. A geotechnical engineer or engineering geologist 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 should be provided by covering the batter slopes with plastic sheets extending at least 1.5m behind the crest of the cut face or up to the common site boundaries. The sheets should be positioned and fastened to prevent any water infiltration onto or into the batter slopes. Other applicable methods may be adopted for temporary surface protection, and all surface protection should be placed following inspection of the temporary batters by a geotechnical engineer.

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

6.9.2 Excavation Retention Support Systems

Where there is insufficient space between the proposed development and adjoining infrastructures, or where adjacent infrastructures are located within the "zone of influence" (as outlined in Section 6.9.1 above), consideration should be given to a suitable retention system such as a soldier pile wall solution, with piles sufficiently embedded into consistent and competent strength bedrock underlying the site, and concrete and reinforcement infill panels for the support of the excavation and soils.

Closer spaced piles are recommended and may be required to reduce lateral movements particularly where adjacent infrastructures, such as buildings or pavements and road reserves are located near the excavation, and to prevent the collapse of loose/soft 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.), and long term loadings.

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

The use of a more rigid retention system such as a cast in-situ contiguous pile wall solution should also be considered to reduce the lateral movements and risk of potential damage to adjacent infrastructures (i.e. buildings, infrastructures, adjacent road reserves, etc.). This option may also be adopted where excessive surcharges are adjacent to the proposed excavation and to meet acceptable deflection criteria, where loose/soft soils are required to be retained, or where there is a potential for undermining of any adjoining building/infrastructures (refer to Section 6.5).

All piles should be sufficiently embedded into consistent and competent strength bedrock underlying the site and should be inspected and approved by a suitably qualified geotechnical engineer. Piles should not be founded into any soft or weak bands/layers (i.e. clay seams and/or extremely weathered/fractured zones) underlying the site. Furthermore, the retention system should be carefully selected by the project structural engineer, with all structural elements also inspected and approved by a suitably qualified structural engineer.

It should be noted that groundwater inflow may pass through shoring pile gaps during excavation. This may be controlled by the installation of strip drains behind the retention system connected to the buildings stormwater system. Shotcreting or localised grouting may also be used in weak areas of the retention system, predominately where groundwater seepage and loose/soft soils are visible. Shoring design should take into consideration both short term (during construction) and permanent conditions, along with surcharge loading and footing loads from adjacent infrastructures.

The design of retaining walls will depend on the method of constructed being adopted. Common methods include (not limited to):

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

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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 walls.

If considered, the shoring wall can be designed using the recommended design parameters provided in Section 6.9.3. 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.

6.9.3 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 development. 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 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 excavations.

Support system designed using the earth pressure approach may be based on the parameters given in Table 10 below for soils and rock horizons underlying the site. Table 10 also provides preliminary coefficients of lateral earth pressure for the soils and rock horizons encountered in the site. These are based on fully drained conditions and that the ground behind the retention walls is horizontal.

Table 10. Preliminary Geotechnical Design Parameters

Material	Fill (Unit 1)	Natural Soils (Unit 2)	Conglomerate Bedrock[3, 5] (Unit 3)
Unit Weight (kN/m^3)[4]	16	18	20
Effective Cohesion c' (kPa)	0	4	15
Angle of Friction ϕ' (°)	24	26	28
Modulus of Elasticity E_{sh} (MPa)	3	8 (soft) 10 (firm) 15 (stiff, or better)	40
Earth Pressure Coefficient At Rest Ko[1]	0.59	0.56	0.53
Earth Pressure Coefficient Active Ka[2]	0.42	0.39	0.36
Earth Pressure Coefficient Passive Kp[2]	2.37	2.56	2.77
Poisson Ratio ν	0.4	0.35	0.3

[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 and conglomerate bedrock underlies the site. All excavation rock faces should be inspected on a regular basis by an experienced engineering geologist or geotechnical engineer.
[4]Above groundwater levels.
[5]Subject to confirmation by a geotechnical engineer by additional borehole drilling and rock strength testing, or during construction by inspection.

6.10 Foundations

Following excavation depths to the FFLs of the proposed development and based on the boreholes and DCP tests carried out within the site, we expect varying ground conditions comprising predominately Unit 1 (fill) to Unit 3 (bedrock) inclusive of variable estimated strength and weathering to be exposed at bulk excavation level of the proposed basement (depending on the actual amount of excavation required).

It should be noted that construction on fill or loose soils present throughout the site can lead to total and differential settlement under working loads, and not adequately support shallow foundations for the proposed development. Removal of any fill material within the proposed development area prior to foundation construction is recommended.

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 program, 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 during construction by inspection.

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6.10.1 Geotechnical Assessment

Based on the proposed development and assessment of the subsurface conditions, a suitable foundation system comprising combination of shallow foundations typically comprising pad and/or strip footings, and a piled foundation system are likely to be adopted for the proposed development, and should be constructed and sufficiently embedded into consistent and competent strength bedrock underlying the site.

All piles should be sufficiently embedded into consistent and competent strength bedrock in areas where bedrock is not exposed at bulk excavation level and should fully support the building/infrastructures. Shallow foundations should only be considered in areas where bedrock is expected to be exposed at or shortly below bulk excavation level and should include local slab thickening to support internal walls and columns for shallow foundations, with consideration given to settlement reducing piles. Conglomerate layers underlying the site may be in the form of boulders or incur weakened zones which may cause deformation of the building in the long term. Thus, adequate foundation analysis should be carried out by the project structural engineer when adopting a shallow foundation system.

Installation of piles and foundation construction should be complemented by inspections carried out by a geotechnical engineer during construction, to confirm ground conditions are consistent throughout and allowable bearing capacities have been achieved. The actual depth and embedment of the piles should be assessed by the project structural engineer, with all structural elements of the proposed development also inspected and approved by a suitably qualified structural engineer. Consultation should be made with specialist subcontractors to discuss the feasibility of piles for the existing ground conditions.

Given the potential for variable ground conditions and soil reactivity across the site, it is recommended that all foundations are constructed on consistent and competent 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 excavation level, and pile foundations elsewhere. Reference should be made to the estimated levels of the subsurface conditions outlined in this report, and compared to the final bulk excavation levels across the site.

Installation of piles may be required where the axial and working loads transmitted through the building walls and columns exceed the bearing pressure of the bedrock exposed at the proposed developments FFLs. These should be socketed into consistent and appropriate bedrock underlying the site. For cases where resistance against lateral loading induced by earthquakes or winds, and to achieve higher bearing capacities, piles may also be required.

Piles sufficiently socketed into higher strength bedrock may achieve greater allowable bearing capacities, subject to confirmation by a geotechnical engineer by additional borehole drilling and rock strength testing, or by inspection during construction.

Where higher estimated strength bedrock is present within the site, or where ground conditions vary from those encountered during the geotechnical investigation, GCA should be contacted for further advice.

Table 11 provides preliminary recommended geotechnical design parameters.

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Table 11. 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
Natural Soils (Unit 2)	N/A	N/A	N/A
Conglomerate Bedrock (Unit 3)[2]	400	20	10

[1]Minimum embedment of 0.4m for shallow foundations and 0.5m for piles. Assumes the presence of conglomerate bedrock underlying the entire site area.
[2]The composition, class, depth and estimated strength of the underlying bedrock material should be confirmed prior to construction by further borehole drilling and rock strength testing, or during construction by inspection.
Notes:
- Higher allowable bearing capacities may be attained for higher estimated strength rock assessed and confirmed by a geotechnical engineer.
- All shaft adhesion parameters are based on adequately clean and rough sockets of category "R2", or better.
- N/A = Not Applicable. Not recommended for the proposed development.
- 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.

Footings designed using ultimate values and limit state design will need to consider serviceability which usually governs designs in these cases. For pile designs, a basic geotechnical reduction factor (Φ_{gb}) should be calculated by the structural engineer from AS 2159-2009, taking into consideration the design, installation method and associated risk rating. Furthermore, the design structural engineer should check both 'piston' pull-out and 'cone' pull-out mechanics in accordance with AS 4678-2002.

6.10.2 Geotechnical Comments

Bearing capacity and settlement behaviour varies according to foundation depth, shape and dimensions, including method of installation for piles. Consultation should be made with specialist subcontractors to discuss the feasibility of piles for the existing site conditions. It should be noted that higher bearing capacities may be justified for the proposed foundations subject to confirmation by inspection during construction, and by additional borehole drilling and appropriate testing.

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.

It is recommended that suitable drainage and the use of impermeable surfaces be implemented as a precaution as part of the design and construction of the proposed development in order to divert surface water away from the building, and help eliminate or minimise surface water infiltration to minimise moisture within the soils. 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.

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

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or runoff is encountered dewatering should be carried out prior to pouring concrete in the foundations. Due to the possibility of groundwater being encountered and possible groundwater seepage during installation of bored piles within the site (encountered during borehole drilling), 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 (subject to confirmation) 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%). It is recommended that where piles penetrate expansive soils present within the site, which are susceptible to shrink and swell due to daily and seasonal moisture, shaft adhesion be ignored due to the potential of shrinkage cracking. Pile inspections should be complemented by downhole CCTV camera.

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”.

6.11 Filling

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

- Place horizontal loose layers not more than 150mm 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 and AS 1289. 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”.

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6.12 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.

7. ADDITIONAL GEOTECHNICAL RECOMMENDATIONS

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

- Implementation of mitigation and control measures for the proposed development, as outlined in this report.
- Dilapidation survey report on adjacent properties and infrastructures.
- Monitoring and supervision of excavations within the site.
- The composition, class, depth and estimated strength of the underlying bedrock material should be confirmed prior to construction by further borehole drilling and rock strength testing, or during construction by inspection, predominately in areas and at depths not assessed during the geotechnical investigation.
- Geotechnical inspections of exposed materials at bulk excavation level.
- Geotechnical inspections of shoring wall piles installations.
- Geotechnical inspections of foundations (shallow and pile foundations) to confirm the preliminary bearing capacities have been achieved.
- Monitoring of any groundwater inflows into the excavation areas within the site.
- Provision for longer term groundwater monitoring within the site.
- 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.

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8. 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 **MHM Construction Group**, 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 Pty Ltd (GCA)

GeorgeA

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9. REFERENCES

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Mecone Mosaic.

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APPENDIX A

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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.

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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.

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APPENDIX B

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GCA Geotechnical Consultants Australia	Figure 1 Site Plan	**Geotechnical Investigation**	Drawn: MS/GA
		MHM Construction Group	Date: 05/12/2022
	Job No.: G22572-1	262 Aberglasslyn Road Aberglasslyn NSW 2320	Scale: NTS

Image source: NSW Six Maps - https://maps.six.nsw.gov.au. Accessed 30th November 2022.

Legend:

Approximate Borehole/DCP Testing Location

GCA Geotechnical Consultants Australia	Figure 2 Site Plan	**Geotechnical Investigation**	Drawn: MS/GA	
		MHM Construction Group	Date: 05/12/2022	
	Job No.: G22572-1	262 Aberglasslyn Road Aberglasslyn NSW 2320	Scale: NTS	

Image source: NSW Six Maps - https://maps.six.nsw.gov.au. Accessed 30th November 2022.

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
CC	Concrete Coring

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 1726-2017)

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

SOIL ORIGINS

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

- **Residual soils**: derived from in-situ weathering of the underlying rock (see "rock material weathering" below).
- **Transported soils**: formed somewhere else and transported by nature to the site.
- **Filling**: moved/placed by man.

Transported soils may be further subdivided into:

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

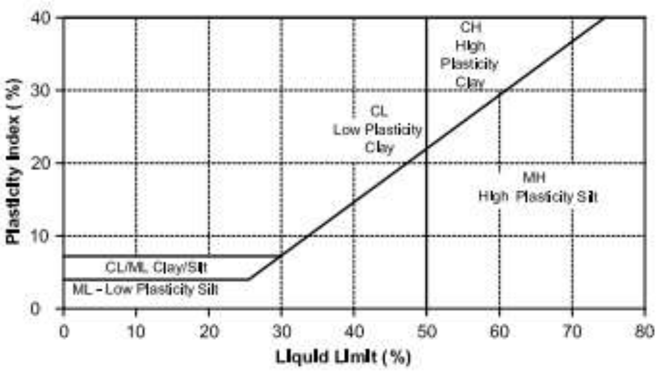
Method and Terms for Soil and Rock Descriptions Used on Borehole and Test Pit Reports

Soil and Rock is classified and described in reports of boreholes and test pits using the preferred method given in AS 1726-2017, 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		>200mm
Cobbles		63mm to 200mm
Gravel	coarse	20mm to 63mm
	medium	6mm to 20mm
	fine	2.36mm to 6mm
Sand	coarse	600µm to 2.36mm
	medium	200µm to 600µm
	fine	75µm to 200µm

PLASTICITY PROPERTIES



COHESIVE SOILS – CONSISTENCY (AS 1726-2017)

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
Friable	Fr	Easily crumbled or broken into small pieces by hand

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 1726-2017 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

ABREVIATIONS FOR DEFECT TYPES AND DECRIPTIONS

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

Type	Definition
B	Bedding
J	Joint
HJ	Horizontal to Sub-Horizontal Joint
VJ	Vertical to Sub-Vertical Joint
F	Fault
Cle	Cleavage
SZ	Shear Zone
SM	Shear Seam
FZ	Fractured Zone
CZ	Crushed Zone
CS	Crushed Seam
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

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BOREHOLE NUMBER BH1

CLIENT MHM Construction Group
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G22572-1
PROJECT LOCATION 262 Aberglasslyn Road Aberglasslyn NSW 2320

DATE STARTED 22/11/22 **COMPLETED** 22/11/22
R.L. SURFACE
DATUM
DRILLING CONTRACTOR NEO
SLOPE 90°
BEARING ---
EQUIPMENT Ute Mounted Drilling Rig
HOLE LOCATION Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter
LOGGED BY GA/GN
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 Drilling					Silty CLAY, medium to high plasticity, brown to dark brown, with fine gravel, grass rootlets, moist.		FILL
			0.5		CH	Silty CLAY, high plasticity, brown, grey, some fine gravel, moist, estimated soft.	DS	NATURAL SOILS
			1.0		CH	Silty CLAY, high plasticity, brown to pale brown, some fine gravel, moist, estimated soft. becoming estimated firm from 1.1m bgl. becoming estimated stiff from 1.2m bgl. becoming estimated very stiff from 1.3m bgl. becoming estimated hard from 1.4m bgl.		
			1.5					
			2.0			CONGLOMERATE, brown to brownish orange, sub angular gravel in a sand and clay matrix, occasional sandstone clasts, moist.		BEDROCK moderate drilling resistance from 2.0m bgl.
			2.5					
			3.0			CONGLOMERATE, brown to pale brown, sub angular gravel in a sand and clay matrix, occasional sandstone clasts, moist.		higher drilling resistance from 3.0m bgl.
			3.5					
			4.0					
			4.5					
			5.0					TC bit refusal at 5.0m bgl.

Borehole BH1 terminated at 5m

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BOREHOLE NUMBER BH2

CLIENT MHM Construction Group
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G22572-1
PROJECT LOCATION 262 Aberglasslyn Road Aberglasslyn NSW 2320

DATE STARTED 22/11/22 **COMPLETED** 22/11/22
R.L. SURFACE
DATUM
DRILLING CONTRACTOR NEO
SLOPE 90°
BEARING ---
EQUIPMENT Ute Mounted Drilling Rig
HOLE LOCATION Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter
LOGGED BY GA/GN
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 Drilling		0.0–1.0			Silty CLAY, medium to high plasticity, brown to dark brown, grey, with fine gravel, grass rootlets, moist.		FILL (REWORKED IN-SITU?)
			1.0–4.0			CONGLOMERATE, grey to pale grey, sub angular gravel in a sand and clay matrix, occasional sandstone clasts, moist. becoming brown from 1.5m bgl. becoming brownish orange from 2.8m bgl. becoming pale brown from 3.4m bgl.		BEDROCK moderate drilling resistance from 1.0m bgl.
			4.0–4.5			CONGLOMERATE, brown to pale brown, sub angular gravel in a sand and clay matrix, occasional sandstone clasts, moist.	DS	higher drilling resistance from 4.0m bgl. TC bit refusal at 4.5m bgl.
			4.5			Borehole BH2 terminated at 4.5m		

Depth scale (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0

BOREHOLE / TEST PIT BOREHOLE LOGS.GPJ GINT STD AUSTRALIA.GDT 5/12/22

BOREHOLE NUMBER BH3

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CLIENT MHM Construction Group
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G22572-1
PROJECT LOCATION 262 Aberglasslyn Road Aberglasslyn NSW 2320

DATE STARTED 22/11/22 **COMPLETED** 22/11/22
R.L. SURFACE
DATUM
DRILLING CONTRACTOR NEO
SLOPE 90°
BEARING ---
EQUIPMENT Ute Mounted Drilling Rig
HOLE LOCATION Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter
LOGGED BY GA/GN
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 Drilling		0.0–0.8			Silty CLAY, medium to high plasticity, brown to dark brown, grey, with fine gravel, grass rootlets, moist.		FILL
			0.8–1.2		CI-CH	Silty CLAY, medium to high plasticity, brown to pale brown, with fine gravel, moist, estimated stiff. becoming estimated hard from 0.9m bgl.		NATURAL SOILS
			1.2–4.0			CONGLOMERATE, brown to brownish orange, sub angular gravel in a sand and clay matrix, occasional sandstone clasts, moist.	DS (at approx. 2.0m)	BEDROCK moderate drilling resistance from 1.2m bgl. higer drilling resistance from 3.2m bgl. TC bit refusal at 4.0m bgl.
			4.0			Borehole BH3 terminated at 4m		

BOREHOLE NUMBER BH4

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CLIENT MHM Construction Group
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G22572-1
PROJECT LOCATION 262 Aberglasslyn Road Aberglasslyn NSW 2320

DATE STARTED 22/11/22 **COMPLETED** 22/11/22 **R.L. SURFACE** **DATUM**
DRILLING CONTRACTOR NEO **SLOPE** 90° **BEARING** ---
EQUIPMENT Ute Mounted Drilling Rig **HOLE LOCATION** Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter **LOGGED BY** GA/GN **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 Drilling		0.0–0.6			Silty CLAY, medium to high plasticity, brown to dark brown, with fine gravel, grass rootlets, moist.		FILL
			0.6–2.7		CI-CH	Silty CLAY, medium to high plasticity, grey to pale grey, some fine gravel, moist.		NATURAL SOILS
			1.3			becoming reddish brown, pale grey from 1.3m bgl.		
			1.4–1.5				DS	
			2.7–3.0			CONGLOMERATE, brown to brownish orange, sub angular gravels in a sand and clay matrix, moist.		BEDROCK moderate drilling resistance from 2.7m bgl.
			3.0			Borehole BH4 terminated at 3m		

Depth scale (m): 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0, 4.5, 5.0

APPENDIX E

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DYNAMIC CONE PENETOMETER RESULTS

Client:	MHM Construction Group	**Test Date:**	22/11/2022
Address:	262 Aberglasslyn Road Aberglasslyn NSW 2320	**Job No.:**	G22572-1

Depths (mm bgl)	DCP No.				Depths (mm bgl)	DCP No.			
	1	**2**							
0-100	2	2			0-100				
100-200	3	1			100-200				
200-300	3	5			200-300				
300-400	2	5			300-400				
400-500	4	2			400-500				
500-600	6	2			500-600				
600-700	3	2			600-700				
700-800	1	1			700-800				
800-900	2	6			800-900				
900-1000	1	15			900-1000				
1000-1100	1	23			1000-1100				
1100-1200	3	10/60mm			1100-1200				
1200-1300	4	**Bouncing**			1200-1300				
1300-1400	9				1300-1400				
1400-1500	13				1400-1500				
1500-1600	15				1500-1600				
1600-1700	19				1600-1700				
1700-1800	14				1700-1800				
1800-1900	14				1800-1900				
1900-2000	16				1900-2000				
2000-2100	26				2000-2100				
2100-2200	23				2100-2200				
2200-2300	30				2200-2300				
2300-2400	**Terminated**				2300-2400				
2400-2500					2400-2500				
2500-2600					2500-2600				
2600-2700					2600-2700				
2700-2800					2700-2800				
2800-2900					2800-2900				
2900-3000					2900-3000				
3000-3100					3000-3100				
3100-3200					3100-3200				
3200-3300					3200-3300				
3300-3400					3300-3400				
3400-3500					3400-3500				
3500-3600					3500-3600				
3600-3700					3600-3700				
3700-3800					3700-3800				
3800-3900					3800-3900				
3900-4000					3900-4000				

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Tested:	**GA/GN**	**©Geotechnical Consultants Australia Pty Ltd**	**Sheet:**	**1 of 1**

APPENDIX F

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ALS Environmental

CERTIFICATE OF ANALYSIS

Work Order	: **ES2242254**	Page	: 1 of 2
Client	: **GEOTECHNICAL CONSULTANTS AUSTRALIA PTY LTD**	Laboratory	: Environmental Division Sydney
Contact	: JOE NADER	Contact	: Customer Services ES
Address	: SUITE 5 5-7 VILLIERS STREET PARRAMATTA NSW 2151	Address	: 277-289 Woodpark Road Smithfield NSW Australia 2164
Telephone	: ----	Telephone	: +61-2-8784 8555
Project	: G22572-1 Geotechnical Investigation	Date Samples Received	: 22-Nov-2022 15:45
Order number	: ----	Date Analysis Commenced	: 25-Nov-2022
C-O-C number	: ----	Issue Date	: 05-Dec-2022 08:55
Sampler	: GEORGE A		
Site	: 262 Aberglassyln Road Aberglasslyn NSW 2320		
Quote number	: EN/333		
No. of samples received	: 4		
No. of samples analysed	: 4		

ilac-MRA NATA

Accreditation No. 825
Accredited for compliance with
ISO/IEC 17025 - Testing

This report supersedes any previous report(s) with this reference. Results apply to the sample(s) as submitted, unless the sampling was conducted by ALS. This document shall not be reproduced, except in full.

This Certificate of Analysis contains the following information:

- General Comments
- Analytical Results

Additional information pertinent to this report will be found in the following separate attachments: Quality Control Report, QA/QC Compliance Assessment to assist with Quality Review and Sample Receipt Notification.

Signatories

This document has been electronically signed by the authorized signatories below. Electronic signing is carried out in compliance with procedures specified in 21 CFR Part 11.

Signatories	*Position*	*Accreditation Category*
Ankit Joshi	Senior Chemist - Inorganics	Sydney Inorganics, Smithfield, NSW
Edwandy Fadjar	Organic Coordinator	Sydney Inorganics, Smithfield, NSW

General Comments

The analytical procedures used by ALS have been developed from established internationally recognised procedures such as those published by the USEPA, APHA, AS and NEPM. In house developed procedures are fully validated and are often at the client request.

Where moisture determination has been performed, results are reported on a dry weight basis.

Where a reported less than (<) result is higher than the LOR, this may be due to primary sample extract/digestate dilution and/or insufficient sample for analysis.

Where the LOR of a reported result differs from standard LOR, this may be due to high moisture content, insufficient sample (reduced weight employed) or matrix interference.

When sampling time information is not provided by the client, sampling dates are shown without a time component. In these instances, the time component has been assumed by the laboratory for processing purposes.

Where a result is required to meet compliance limits the associated uncertainty must be considered. Refer to the ALS Contract for details.

Key : CAS Number = CAS registry number from database maintained by Chemical Abstracts Services. The Chemical Abstracts Service is a division of the American Chemical Society.
LOR = Limit of reporting
^ = This result is computed from individual analyte detections at or above the level of reporting
ø = ALS is not NATA accredited for these tests.
~ = Indicates an estimated value.

Analytical Results

Sub-Matrix: SOIL (Matrix: SOIL)			Sample ID	BH1 0.7m-0.8m	BH2 4.4m-4.5m	BH3 1.9m-2.0m	BH4 1.4m-1.5m	----
			Sampling date / time	22-Nov-2022 00:00	22-Nov-2022 00:00	22-Nov-2022 00:00	22-Nov-2022 00:00	----
Compound	CAS Number	LOR	Unit	ES2242254-001	ES2242254-002	ES2242254-003	ES2242254-004	--------
				Result	Result	Result	Result	----
EA002: pH 1:5 (Soils)								
pH Value	----	0.1	pH Unit	5.4	5.8	5.8	5.4	----
EA010: Conductivity (1:5)								
Electrical Conductivity @ 25°C	----	1	µS/cm	95	140	86	160	----
EA055: Moisture Content (Dried @ 105-110°C)								
Moisture Content	----	0.1	%	20.6	4.7	9.3	12.4	----
ED040S: Soluble Major Anions								
Sulfate as SO4 2-	14808-79-8	10	mg/kg	100	30	40	90	----
ED045G: Chloride by Discrete Analyser								
Chloride	16887-00-6	10	mg/kg	150	180	90	160	----

QUALITY CONTROL REPORT

Work Order	: **ES2242254**	Page	
Client	: **GEOTECHNICAL CONSULTANTS AUSTRALIA PTY LTD**	Laboratory	: Environmental Division Sydney
Contact	: JOE NADER	Contact	: Customer Services ES
Address	: SUITE 5 5-7 VILLIERS STREET PARRAMATTA NSW 2151	Address	: 277-289 Woodpark Road Smithfield NSW Australia 2164
Telephone	: ----	Telephone	: +61-2-8784 8555
Project	: G22572-1 Geotechnical Investigation	Date Samples Received	: 22-Nov-2022
Order number	: ----	Date Analysis Commenced	: 25-Nov-2022
C-O-C number	: ----	Issue Date	: 05-Dec-2022
Sampler	: GEORGE A		
Site	: 262 Aberglassyln Road Aberglasslyn NSW 2320		
Quote number	: EN/333		
No. of samples received	: 4		
No. of samples analysed	: 4		

Accreditation No. 825
Accredited for compliance with
ISO/IEC 17025 - Testing

This report supersedes any previous report(s) with this reference. Results apply to the sample(s) as submitted, unless the sampling was conducted by ALS. This document shall not be reproduced, except in full.

This Quality Control Report contains the following information:

- Laboratory Duplicate (DUP) Report; Relative Percentage Difference (RPD) and Acceptance Limits
- Method Blank (MB) and Laboratory Control Spike (LCS) Report; Recovery and Acceptance Limits
- Matrix Spike (MS) Report; Recovery and Acceptance Limits

Signatories

This document has been electronically signed by the authorized signatories below. Electronic signing is carried out in compliance with procedures specified in 21 CFR Part 11.

Signatories	*Position*	*Accreditation Category*
Ankit Joshi	Senior Chemist - Inorganics	Sydney Inorganics, Smithfield, NSW
Edwandy Fadjar	Organic Coordinator	Sydney Inorganics, Smithfield, NSW

General Comments

The analytical procedures used by ALS have been developed from established internationally recognised procedures such as those published by the USEPA, APHA, AS and NEPM. In house developed procedures are fully validated and are often at the client request.

Where moisture determination has been performed, results are reported on a dry weight basis.

Where a reported less than (<) result is higher than the LOR, this may be due to primary sample extract/digestate dilution and/or insufficient sample for analysis. Where the LOR of a reported result differs from standard LOR, this may be due to high moisture content, insufficient sample (reduced weight employed) or matrix interference.

Key : Anonymous = Refers to samples which are not specifically part of this work order but formed part of the QC process lot
CAS Number = CAS registry number from database maintained by Chemical Abstracts Services. The Chemical Abstracts Service is a division of the American Chemical Society.
LOR = Limit of reporting
RPD = Relative Percentage Difference
# = Indicates failed QC

Laboratory Duplicate (DUP) Report

The quality control term Laboratory Duplicate refers to a randomly selected intralaboratory split. Laboratory duplicates provide information regarding method precision and sample heterogeneity. The permitted ranges for the Relative Percent Deviation (RPD) of Laboratory Duplicates are specified in ALS Method QWI-EN/38 and are dependent on the magnitude of results in comparison to the level of reporting: Result < 10 times LOR: No Limit; Result between 10 and 20 times LOR: 0% - 50%; Result > 20 times LOR: 0% - 20%.

Sub-Matrix: **SOIL**

				Laboratory Duplicate (DUP) Report					
Laboratory sample ID	*Sample ID*	*Method: Compound*	*CAS Number*	*LOR*	*Unit*	*Original Result*	*Duplicate Result*	*RPD (%)*	*Acceptable RPD (%)*
EA002: pH 1:5 (Soils) (QC Lot: 4726719)									
ES2242254-001	BH1 0.7m-0.8m	EA002: pH Value	----	0.1	pH Unit	5.4	5.5	0.0	0% - 20%
ES2242177-004	Anonymous	EA002: pH Value	----	0.1	pH Unit	5.4	5.3	2.1	0% - 20%
EA010: Conductivity (1:5) (QC Lot: 4726718)									
ES2242254-001	BH1 0.7m-0.8m	EA010: Electrical Conductivity @ 25°C	----	1	µS/cm	95	92	3.0	0% - 20%
ES2242177-004	Anonymous	EA010: Electrical Conductivity @ 25°C	----	1	µS/cm	26	24	7.7	0% - 20%
EA055: Moisture Content (Dried @ 105-110°C) (QC Lot: 4726727)									
ES2242177-003	Anonymous	EA055: Moisture Content	----	0.1	%	11.7	12.2	3.6	0% - 50%
ES2242254-004	BH4 1.4m-1.5m	EA055: Moisture Content	----	0.1	%	12.4	12.6	2.0	0% - 20%
ED040S: Soluble Major Anions (QC Lot: 4726717)									
ES2242177-004	Anonymous	ED040S: Sulfate as SO4 2-	14808-79-8	10	mg/kg	30	30	0.0	No Limit
ED045G: Chloride by Discrete Analyser (QC Lot: 4726720)									
ES2242254-001	BH1 0.7m-0.8m	ED045G: Chloride	16887-00-6	10	mg/kg	150	160	0.0	No Limit
ES2242177-004	Anonymous	ED045G: Chloride	16887-00-6	10	mg/kg	<10	10	0.0	No Limit

Method Blank (MB) and Laboratory Control Sample (LCS) Report

The quality control term Method / Laboratory Blank refers to an analyte free matrix to which all reagents are added in the same volumes or proportions as used in standard sample preparation. The purpose of this QC parameter is to monitor potential laboratory contamination. The quality control term Laboratory Control Sample (LCS) refers to a certified reference material, or a known interference free matrix spiked with target analytes. The purpose of this QC parameter is to monitor method precision and accuracy independent of sample matrix. Dynamic Recovery Limits are based on statistical evaluation of processed LCS.

Sub-Matrix: **SOIL**

				Method Blank (MB) Report	Laboratory Control Spike (LCS) Report			
					Spike	Spike Recovery (%)	Acceptable Limits (%)	
Method: Compound	CAS Number	LOR	Unit	Result	Concentration	LCS	Low	High
EA002: pH 1:5 (Soils) (QCLot: 4726719)								
EA002: pH Value	----	----	pH Unit	----	4 pH Unit	101	98.8	101
				----	7 pH Unit	100	98.8	101
EA010: Conductivity (1:5) (QCLot: 4726718)								
EA010: Electrical Conductivity @ 25°C	----	1	µS/cm	<1	1412 µS/cm	102	92.0	108
ED040S: Soluble Major Anions (QCLot: 4726717)								
ED040S: Sulfate as SO4 2-	14808-79-8	10	mg/kg	<10	750 mg/kg	98.6	80.0	120
ED045G: Chloride by Discrete Analyser (QCLot: 4726720)								
ED045G: Chloride	16887-00-6	10	mg/kg	<10	250 mg/kg	96.6	75.0	125
				<10	5000 mg/kg	101	79.0	117

Matrix Spike (MS) Report

The quality control term Matrix Spike (MS) refers to an intralaboratory split sample spiked with a representative set of target analytes. The purpose of this QC parameter is to monitor potential matrix effects on analyte recoveries. Static Recovery Limits as per laboratory Data Quality Objectives (DQOs). Ideal recovery ranges stated may be waived in the event of sample matrix interference.

Sub-Matrix: **SOIL**

				Matrix Spike (MS) Report			
				Spike	SpikeRecovery(%)	Acceptable Limits (%)	
Laboratory sample ID	Sample ID	Method: Compound	CAS Number	Concentration	MS	Low	High
ED045G: Chloride by Discrete Analyser (QCLot: 4726720)							
ES2242177-001	Anonymous	ED045G: Chloride	16887-00-6	250 mg/kg	95.6	70.0	130

APPENDIX G

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Foundation Maintenance and Footing Performance: A Homeowner's Guide

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).

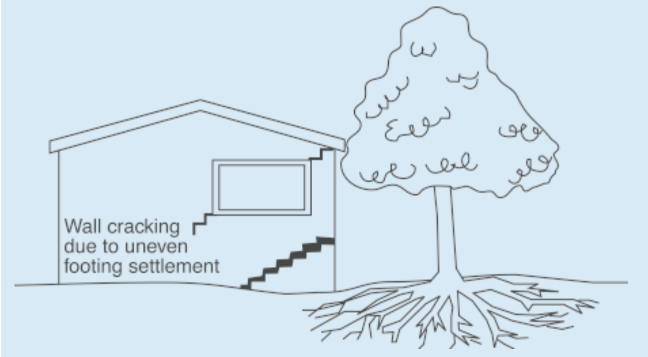
Isolated piers affected by erosion or saturation of foundations will eventually lose contact with the bearers they support and may tilt or fall over. The floors that have lost this support will become bouncy, sometimes rattling ornaments etc.

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.

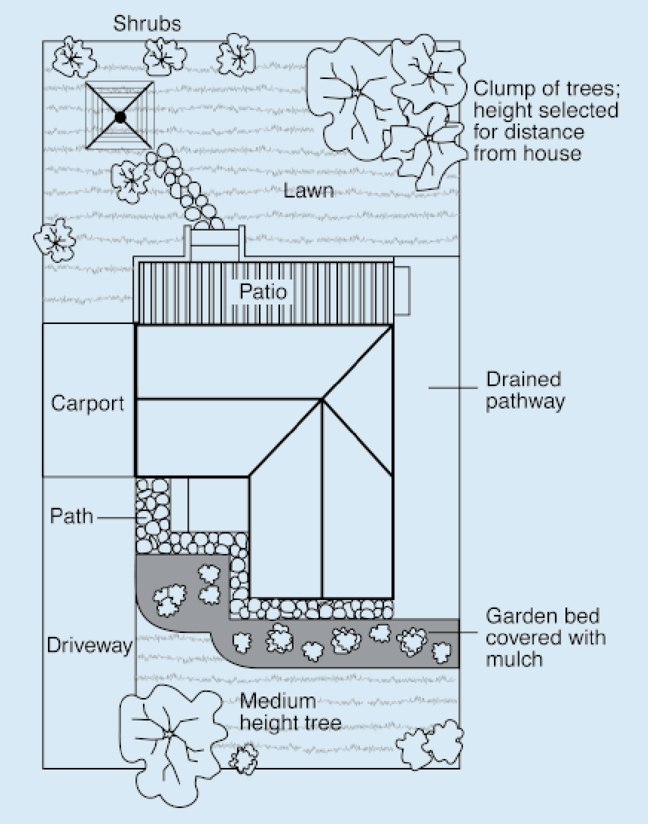
Protection of the building perimeter
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

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.

This BTF was prepared by John Lewer FAIB, MIAMA, Partner, Construction Diagnosis.

The information in this and other issues in the series was derived from various sources and was believed to be correct when published.

The information is advisory. It is provided in good faith and not claimed to be an exhaustive treatment of the relevant subject.

Further professional advice needs to be obtained before taking any action based on the information provided.

Distributed by

CSIRO PUBLISHING PO Box 1139, Collingwood 3066, Australia

Freecall 1800 645 051 Tel (03) 9662 7666 Fax (03) 9662 7555 www.publish.csiro.au

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APPENDIX H

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bh BOLWARRA HEIGHTS

Landscape—rolling low hills on Permian sediments in the centre-west of the sheet in the East Maitland Hills region. Slopes are 5–20%, elevation to 100 m, local relief to 80 m. Cleared tall open-forest.

Landscape Variant—bha—shallow (<55 cm) soils.

Soils—moderately deep (<150 cm), well-drained Yellow Podzolic Soils (Dy2.21, Dy2.31), Red Podzolic Soils (Dr2.31, Dr3.21) and Brown Podzolic Soils (Db1.21, Db1.11) with some moderately deep (<100 cm), well-drained Lithosols (Um1.41, Um1.42) on crests, moderately deep (<140 cm), imperfectly drained yellow Soloths (Dy2.41, Dy3.41) on lower slopes.

Qualities and Limitations—moderate foundation hazard, water erosion hazard, high run-on (localised), seasonal waterlogging (localised), localised steep slopes with mass movement hazard.

LOCATION

Rolling low hills on Permian sediments, predominantly in the East Maitland Hills region, in the centre-west of the area. Examples include Bolwarra Heights, Rutherford, Heddon Greta and Gillieston Heights. Type location is Bolwarra Heights (Area reference 3 **67*****E, 63 **81*****N).

LANDSCAPE

Geology and Regolith

Predominantly Branxton Formation of the Maitland Group—sandstone, siltstone, conglomerate, erratics. Also including small areas of Muree Sandstone—sandstone, conglomerate and siltstone. Greta Coal Measures—lenticular conglomerates, sandstone, shale, splitting coal seams; and Farley Formation—sandstone, mudstone, siltstone, shale, erratics.

Topography

Rolling low hills. Slopes range from 5–20%. Local relief is generally 50 m, but ranging to 80 m. Elevation is 40–100 m. Crests are broad (200–500 m) with short (300–500 m), convex sideslopes and narrow, incised drainage lines. Rock outcrop is localised, often occurring where Muree Sandstone is present (<2%).

Vegetation

Predominantly cleared tall open-forest. *Eucalyptus maculata* (spotted gum) is the most dominant species, with *E. fibrosa* (broad-leaved ironbark). *E. tereticornis* (forest red gum) occurs on some lower slopes. *Angophora floribunda* (rough-barked apple) and *Allocasuarina torulosa* (forest oak) may also occur, with *Casuarina glauca* (swamp oak) along drainage lines.

Land Use

The majority of this landscape is cleared. Urban development occurs at Rutherford and Bolwarra Heights, which is expanding onto traditional beef cattle grazing and hobby farm areas.

Existing Land Degradation

Minor to moderate sheet and rill erosion may occur where ground cover has been removed. Minor gully erosion may occur.

Landscape Variants

The areas marked as **bha** on the map have shallow (<55 cm) soils on massive conglomerate. The vegetation is dominated by *Angophora bakeri* (narrow-leaved apple). Otherwise, this variant has similar landscape features to the Bolwarra Heights **(bh)** soil landscape.

Included Soil Landscapes

Small areas of Rivermead **(ri)** soil landscape have been included at the boundary between Bolwarra Heights and Hunter soil landscapes.

SOILS

Dominant Soil Materials

bh1 — Brownish black gravelly loam (topsoil — A_1 horizon)

Colour	brownish black (10YR 2/2, 10YR 2/3, 7.5YR 3/2)
Texture	loam fine sandy to sandy clay loam, occasionally fine sandy loam
Structure	weak, 20–50 mm sub-angular blocky peds or massive
Fabric	rough ped or earthy
Field pH	moderately to slightly acid (pH 5.5–6.0)
Exposed condition	massive, hardsetting, rarely friable
Permeability	moderate to high
Coarse fragments	common to many subrounded conglomerate pebbles. Charcoal fragments rarely occur
Roots	common, branched, in-ped
Type location	Gillieston Heights, 1 km north-east along the Cessnock Rd (Grid Ref. 3 **626****E, 63 **747****N). Soil Data System card 36, 0–30 cm

bh2 — Earthy gravelly sandy clay loam (topsoil — A_2 horizon)

Colour	dark brown (10YR 3/3) or brownish black (10YR 2/3)
Texture	loam fine sandy to fine sandy clay loam
Structure	massive
Fabric	earthy
Field pH	moderately to slightly acid (pH 5.0–6.5)
Exposed condition	massive, hardsetting
Permeability	moderate
Coarse fragments	common conglomerate pebbles occur, often forming a stoneline at the base. Few to common charcoal fragments may occur
Roots	common
Type location	2 km along track leading from Tocal Rd, 5 km north of Bolwarra Heights, near Quarry Creek (Grid Ref. 3 **854****E, 63 **76**8**N). Soil Data System card 55, 0–20 cm

bh3 — Yellowish brown pedal clay (subsoil — B_2 horizon)

Colour	yellowish brown (10YR 5/6) to bright yellowish brown (10YR 6/6). Very few to common red/orange mottles may occur, often increasing with depth
Texture	light to medium clay
Structure	moderate to strong, 10–20 mm angular blocky peds which break down from 50–100 mm or 100–200 mm columnar, prismatic or angular blocky peds
Fabric	smooth ped
Field pH	strongly to moderately acid (pH 4.5–5.5)
Exposed condition	frets upon exposure to form fine (<5 mm) surface aggregates
Permeability	moderate to slow
Coarse fragments	conglomerate pebbles are absent to common. Few ironstones may occur
Roots	few to common, in- and ex-ped
Type location	2 km along track leading from Tocal Rd, 5 km north of Bolwarra Heights, near Quarry Creek (Grid Ref. 3 **654****E, 63 **877****N). Soil Data System card 55, >20 cm

bh4 — Reddish brown pedal mottled clay

Colour	reddish brown (5YR 4/6), brown (7.5YR 4/6, 10YR 4/4) or dark brown (10YR 3/4); few to common orange/grey mottles occur
Texture	light to medium clay
Structure	moderate to strong, 20–50 mm angular blocky peds which often break down from 50–100 mm or 100–200 mm columnar peds
Fabric	smooth ped
Field pH	moderately acid (pH 5.5–6.0)
Exposed condition	fine (<5 mm) fragments form at the surface
Permeability	moderate to slow
Coarse fragments	few conglomerate pebbles commonly occur
Roots	few, in- and ex-ped
Type location	Gillieston Heights, 1 km north-east along the Cessnock Road (Grid Ref. 3 **626****E, 63 **47****N). Soil Data System card 36, >30 cm

Associated Soil Materials

Gravelly bleached sandy clay loam. This material occurs as a topsoil (A_2 horizon) on poorly drained lower slopes.

Mottled brownish grey clay. This material is a structured light to medium clay which occurs as a deep subsoil (B_3/C horizon) on siltstone parent material, and which is prone to slip failure (Hannam, Elliott and Veness 1982).

Occurrence and Relationships

Generally. Up to 25 cm brownish black gravelly loam **(bh1)** overlies 15–20 cm earthy, gravelly fine sandy clay loam **(bh2)**, which in turn overlies 75–103 cm yellowish brown pedal clay **(bh3).** Often, **bh1** has been lost to erosion, exposing **bh2** at the surface. Rarely, >185 cm mottled brownish grey clay underlies **bh3**. Soil boundaries are sharp to clear. Total soil depth is <150 cm [well-drained Yellow Podzolic Soils (Dy2.21, Dy2.31)].

On some well-drained upper slopes and crests. Up to 25 cm **bh1** overlies 15–30 cm **bh2** which in turn overlies 30–45 cm reddish brown pedal mottled clay **(bh4).** Soil boundaries are abrupt to clear. Total soil depth is <100 cm [well-drained Red Podzolic Soils (Dr2.31, Dr3.21) and Brown Podzolic Soils (Db1.21)]. Occasionally, up to 35 cm **bh1** directly overlies 30–40 cm **bh4.** Soil boundaries are sharp to clear. Total soil depth is <100 cm [well-drained Brown Podzolic Soils (Db1.11)].

On some resistant sandstone and conglomerate crests. 5 cm **bh1** overlies up to 100 cm **bh2** which directly overlies bedrock. Boundaries are clear. Total soil depth is 50–120 cm [rapidly drained Lithosols (Um1.41, Um1.42)].

On poorly drained slopes. Up to 25 cm **bh1** overlies up to 20 cm gravelly bleached sandy clay loam, which in turn overlies up to 30 cm **bh3**. Soil boundaries are clear to abrupt. Total soil depth is <100 cm [imperfectly drained yellow Soloths (Dy2.41, Dy3.41)].

Where siltstone lenses outcrop. 25–40 cm **bh1** overlies 15–30 cm gravelly bleached sandy clay loam which overlies 35–>185 cm mottled brownish grey clay. Boundaries are clear to abrupt. Total soil depth is 80–>200 cm [well to imperfectly drained yellow Soloths (Dy2.41, Dy3.41)].

In drainage lines. >100 cm **bh1** occurs [poorly drained Structured Loams (Um6.32), Earthy Loams (Um5.52)].

QUALITIES AND LIMITATIONS

Landscape Limitations

Steep slopes (localised)
Mass movement hazard (localised, steeper (>20%) slopes or where mottled brownish grey clay occurs)
Seasonal waterlogging (localised, lower slopes)
High run-on (localised, lower slopes)
Water erosion hazard
Shallow soils (localised)
Foundation hazard
Rock outcrop (localised)

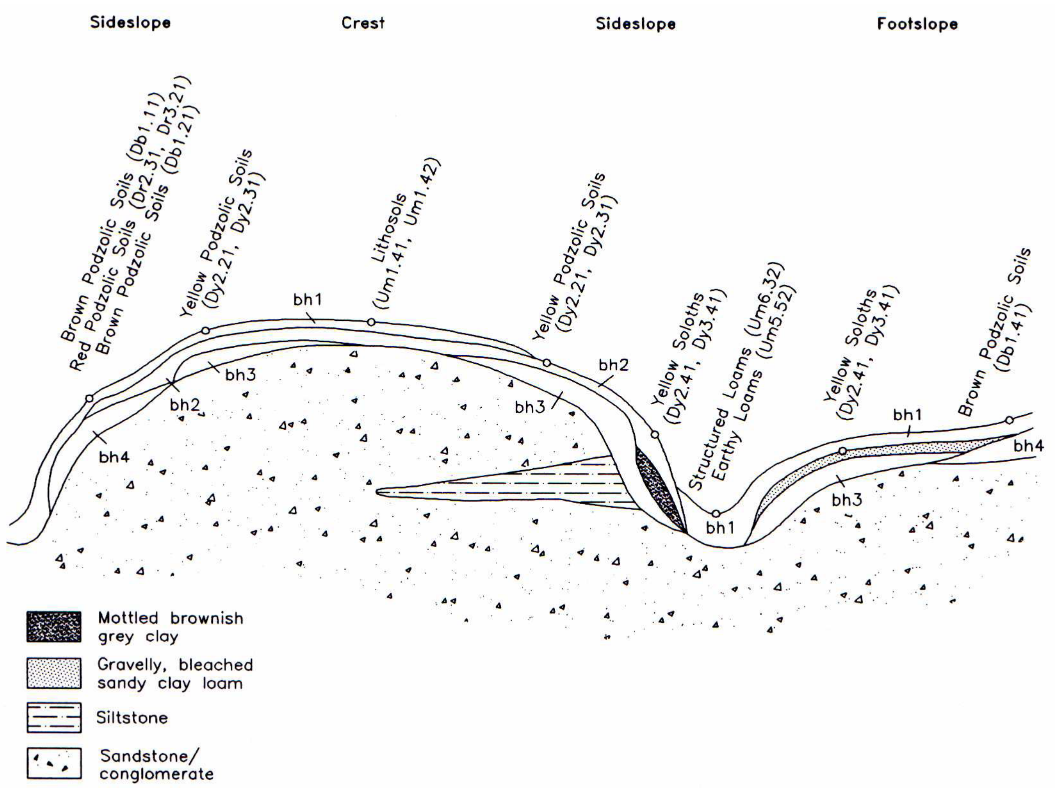
Soil Limitations

bh1	Low wet bearing strength
	Stoniness (localised)
	Strong acidity
bh2	Stoniness (localised)
	Hardsetting surface
	Very strong acidity
	Potential aluminium toxicity
	Very low fertility
bh3	Sodicity/dispersion (localised)
	Very strong acidity
	High potential aluminium toxicity
bh4	High plasticity
	Moderate shrink-swell potential

Fertility

Soil Materials as Plant Growth Media. Soil material suitability as growth media is moderate **(bh1, bh4)** to low **(bh2, bh3)**. Topsoil **bh1** has very high organic matter content, but is strongly acid with low CEC . Subsoil **bh4** has moderate CEC and low to very high exchangeable cations, very high water retention capacity, but very high phosphorus sorption. Soil materials **bh2** and **bh3** are very strongly acid and have high potential aluminium toxicity, while **bh2** has very low pH buffer capacity.

Soil Profile Fertility. Soil profile suitability as a plant growth medium is low to moderate for moderately deep, well-drained Yellow Podzolic Soils, moderate for moderately deep, well-drained Red Podzolic Soils and Brown Podzolic Soils and low for moderately deep, rapidly drained Lithosols and moderately deep to deep imperfectly drained yellow Soloths. Soil volumes available for root penetration are generally moderate.



■ *Schematic cross-section of Bolwarra Heights soil landscape illustrating the occurrence and relationship of the dominant soil materials.*

Erodibility

	K factor	Non-concentrated flows	Concentrated flows	Wind
bh1	0.033	moderate	high	very low
bh2	0.045	high	high	very low
bh3	0.028	moderate	moderate	very low
bh4	0.022	moderate	moderate	very low

Erosion Hazard

	Non-concentrated flows	Concentrated flows	Wind
grazing	low	moderate	slight
cultivation	high	high	slight
urban	moderate	moderate	slight

Foundation Hazard

Moderate foundation hazard, due to moderate shrink-swell (reactive) subsoils **(bh4)**. Localised high foundation hazard on steeper (>20%) slopes. Localised high foundation hazard where associated mottled brownish grey clay occurs, as this material is predisposed to slip failure (Hannan *et al.* 1982). Topsoil depth is 20–>50 cm. Total soil depth is 50–>200 cm.

Urban Capability

Generally moderate limitations for urban development due to moderate foundation hazard. High limitations where slopes are >20%.

Rural Capability

Generally moderate limitations for cultivation and low limitations for grazing.

Sustainable Land Management Recommendations

Topsoils have low potential fertility, but may benefit from lime applications. Organic matter incorporation may help to improve structural stability and moisture holding capacity. A permanent pasture cover should be maintained at 75% or greater. Areas under improved pasture may have the potential to acidify. Topsoils are prone to structural degradation; therefore, cultivation of soils should be minimised and soils should not be worked while too dry or too wet. Soil conservation practices, including ploughing on the contour, apply.

Soil Conservation Earthworks

Moderate limitations for earthworks, including high shrink-swell subsoils **(bh4)**. Soils tested have earthworks categories I for **bh1**, J for **bh2**, A for **bh3** and G for **bh4**.

APPENDIX I

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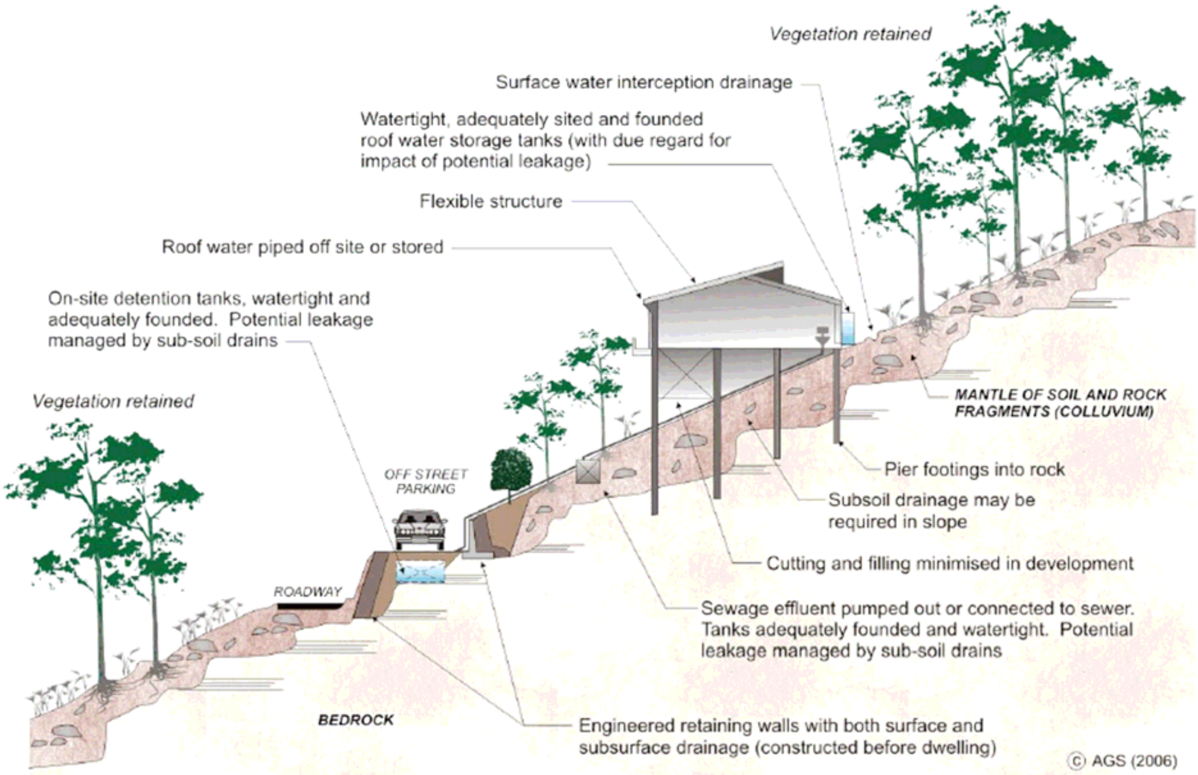
APPENDIX G - SOME GUIDELINES FOR HILLSIDE CONSTRUCTION

	GOOD ENGINEERING PRACTICE	*POOR ENGINEERING PRACTICE*
ADVICE		
GEOTECHNICAL ASSESSMENT	Obtain advice from a qualified, experienced geotechnical practitioner at early stage of planning and before site works.	Prepare detailed plan and start site works before geotechnical advice.
PLANNING		
SITE PLANNING	Having obtained geotechnical advice, plan the development with the risk arising from the identified hazards and consequences in mind.	Plan development without regard for the Risk.
DESIGN AND CONSTRUCTION		
HOUSE DESIGN	Use flexible structures which incorporate properly designed brickwork, timber or steel frames, timber or panel cladding. Consider use of split levels. Use decks for recreational areas where appropriate.	Floor plans which require extensive cutting and filling. Movement intolerant structures.
SITE CLEARING	Retain natural vegetation wherever practicable.	Indiscriminately clear the site.
ACCESS & DRIVEWAYS	Satisfy requirements below for cuts, fills, retaining walls and drainage. Council specifications for grades may need to be modified. Driveways and parking areas may need to be fully supported on piers.	Excavate and fill for site access before geotechnical advice.
EARTHWORKS	Retain natural contours wherever possible.	Indiscriminatory bulk earthworks.
CUTS	Minimise depth. Support with engineered retaining walls or batter to appropriate slope. Provide drainage measures and erosion control.	Large scale cuts and benching. Unsupported cuts. Ignore drainage requirements
FILLS	Minimise height. Strip vegetation and topsoil and key into natural slopes prior to filling. Use clean fill materials and compact to engineering standards. Batter to appropriate slope or support with engineered retaining wall. Provide surface drainage and appropriate subsurface drainage.	Loose or poorly compacted fill, which if it fails, may flow a considerable distance including onto property below. Block natural drainage lines. Fill over existing vegetation and topsoil. Include stumps, trees, vegetation, topsoil, boulders, building rubble etc in fill.
ROCK OUTCROPS & BOULDERS	Remove or stabilise boulders which may have unacceptable risk. Support rock faces where necessary.	Disturb or undercut detached blocks or boulders.
RETAINING WALLS	Engineer design to resist applied soil and water forces. Found on rock where practicable. Provide subsurface drainage within wall backfill and surface drainage on slope above. Construct wall as soon as possible after cut/fill operation.	Construct a structurally inadequate wall such as sandstone flagging, brick or unreinforced blockwork. Lack of subsurface drains and weepholes.
FOOTINGS	Found within rock where practicable. Use rows of piers or strip footings oriented up and down slope. Design for lateral creep pressures if necessary. Backfill footing excavations to exclude ingress of surface water.	Found on topsoil, loose fill, detached boulders or undercut cliffs.
SWIMMING POOLS	Engineer designed. Support on piers to rock where practicable. Provide with under-drainage and gravity drain outlet where practicable. Design for high soil pressures which may develop on uphill side whilst there may be little or no lateral support on downhill side.	
DRAINAGE		
SURFACE	Provide at tops of cut and fill slopes. Discharge to street drainage or natural water courses. Provide general falls to prevent blockage by siltation and incorporate silt traps. Line to minimise infiltration and make flexible where possible. Special structures to dissipate energy at changes of slope and/or direction.	Discharge at top of fills and cuts. Allow water to pond on bench areas.
SUBSURFACE	Provide filter around subsurface drain. Provide drain behind retaining walls. Use flexible pipelines with access for maintenance. Prevent inflow of surface water.	Discharge roof runoff into absorption trenches.
SEPTIC & SULLAGE	Usually requires pump-out or mains sewer systems; absorption trenches may be possible in some areas if risk is acceptable. Storage tanks should be water-tight and adequately founded.	Discharge sullage directly onto and into slopes. Use absorption trenches without consideration of landslide risk.
EROSION CONTROL & LANDSCAPING	Control erosion as this may lead to instability. Revegetate cleared area.	Failure to observe earthworks and drainage recommendations when landscaping.
DRAWINGS AND SITE VISITS DURING CONSTRUCTION		
DRAWINGS	Building Application drawings should be viewed by geotechnical consultant	
SITE VISITS	Site Visits by consultant may be appropriate during construction/	
INSPECTION AND MAINTENANCE BY OWNER		
OWNER'S RESPONSIBILITY	Clean drainage systems; repair broken joints in drains and leaks in supply pipes. Where structural distress is evident see advice. If seepage observed, determine causes or seek advice on consequences.	

EXAMPLES OF GOOD HILLSIDE PRACTICE



EXAMPLES OF POOR HILLSIDE PRACTICE