GCA

Geotechnical Consultants Australia

Designcorp Architects Pty Ltd

Geotechnical Investigation Report

Proposed Development at:
30 Mitchell Street
St Marys NSW 2760

G1887-1
11th August 2018

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1. INTRODUCTION

1.1 Background

This report presents the results of a geotechnical investigation undertaken by Geotechnical Consultants Australia Pty Ltd (GCA) for a proposed development at No. 30 Mitchell Street St Marys NSW 2760. The investigation was commissioned by Ms. Amber Huriwai of Designcorp Architects Pty Ltd, and was carried out on the 2nd August 2018. The commission was on a basis of a proposal provided by GCA and referenced P178-18.1.

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

The findings presented in this report are based on our subsurface investigation, laboratory test 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 advice and recommendations to assist in the preparation of preliminary designs and construction of the ground structures for the proposed development.

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

1.2 Proposed Development

Information provided by the client indicates the proposed development comprises the demolition of the existing dwelling and infrastructures on the site, and construction of a two storey townhouse building, overlying a single basement level. Access to the proposed basement will be via a ramp from Mitchell Street along the site northern boundary.

The following setbacks are proposed from the basement walls to the site boundaries:

- 8.523m from basement wall to the site northern boundary.
- 2.0m from the basement wall to the site eastern boundary
- 7.302m from the basement wall to the site southern boundary.
- 3.316m from the basement wall to the site western boundary.

It should be noted that the abovementioned setbacks are approximated from the architectural drawings and may vary.

The Finished Floor Levels (FFL)'s of the proposed basement level and ground floor level are set to be at Reduced Level (RL) of 31.500m Australian Height Datum (AHD) and RL34.500m AHD, respectively. Based on the existing site topography and levels, maximum excavation depths of approximately 2.0m within the rear portion of the site to approximately 3.2m within the front portion of the site are expected to be required for construction of the proposed development.

Locally deeper excavations will be required for the proposed lift shaft, footings and service trenches.

1.3 Provided Information

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

- Architectural Drawings prepared by Designcorp Architects Pty Ltd, titled "Proposed Townhouse Development @ 30 Mitchell St St Marys", dated August 2018, referenced project No. 2017-235 and included drawing nos. G000, and G1 to G3 inclusive.
- Site Survey Plan prepared by New South Surveys, titled "30 Mitchell Street, St Marys Topographical Survey", dated 28th November 2011 and referenced drawing No. 117490.

1.4 Geotechnical Assessment Objectives

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

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

1.5 Scope of Works

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

- Submit and review Dial Before You Dig (DBYD) plans, and any other plans provided by the client of existing buried services on the site.
- Service locating carried out using electromagnetic detection equipment to ensure the area is free of any underground services at the selected borehole locations.
- Review of site plans and drawings to determine testing locations, and identify any relevant features of the site.
- Machine drilling of three (3) boreholes at selected locations within the site by a specialised track mounted mini drilling rig, using solid flight augers equipped with a 'Tungsten Carbide' (TC) bit, and identified as boreholes BH1 to BH3 inclusive.
    - The boreholes were all drilled to TC bit terminated and refusal depths of approximately 6.0m to 9.0m below existing ground level (bgl).
    - Following the termination depth in borehole BH1, at a depth of approximately 6.0m bgl, drilling commenced using NMLC diamond coring technique to a final depth of approximately 7.11m bgl.
    - Installation of one (1) standpipe piezometer, identified as GW1 and installed in borehole BH3 to a depth of approximately 4.5m bgl (to RL29m AHD) for groundwater measurements and any future groundwater assessment which may be required.
    - Standard Penetration Tests (SPT's) were carried out as practicable in the boreholes during augering, to assess the soil strength (in-situ).
        - The approximate locations of the drilled boreholes and installed standpipe piezometer are shown on **Figure 1, Appendix B** of this report.
- Collection of soil and rock samples during drilling for laboratory tests.
    - Rock cores recovered from the borehole were boxed, logged and sent to a NATA

accredited laboratory (Geo-logic Solutions) for testing to estimate the point load strength index (Is_{50}) values. The rock core photographs and laboratory point load test results certificate are presented in **Appendix E** and **Appendix F**, respectively.
  - Laboratory testing by a NATA accredited laboratory (ALS Environmental) on the aggressivity and salinity of three (3) selected soil samples.
- Reinstatement of the boreholes with available soil displaced during drilling.
- Preparation of this geotechnical report.

1.6 Constraints

The discussions and recommendations provided in this report have been based on the results obtained during drilling and laboratory testing, at the locations of the boreholes. It is recommended that further geotechnical inspections should be carried out during construction to confirm the subsurface conditions across the site and foundation bearing capacities. Consideration should also be given to additional boreholes carried out to confirm the ground conditions, and to help assist in final designs of the proposed development. This recommendation should be confirmed by the project geotechnical engineer and structural engineer during/following design stages of the proposed development.

2. SITE DESCRIPTION

2.1 Overall Site Description

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

Table 1. Overall Site Description and Site Surroundings

Information	Details
Overall Site Location	The site is located along Mitchell Street road reserve, approximately 550m south of the Great Western Highway carriageway.
Site Address	30 Mitchell Street St Marys NSW 2760
Approximate Site Area[1]	1,363m² – based off the site survey plan.
Local Government Authority	Penrith City Council
Site Description	At the time of the investigation a single storey residential dwelling was present within the site, and positioned within the front portion of the site. The dwelling was accompanied by a detached garage and shed, which were both present within the middle to rear portion of the site. An in-ground swimming pool adjoined the garage within the middle portion of the site, and was accompanied by associated concrete pavements which extended throughout the site and towards the driveway. The remaining site area was covered in grass, vegetation and garden beds, with no observable trees at the time of investigation.
Approximate Distances to Nearest Watercourses (i.e. rivers, lakes, etc.)	• Byrnes Creek – 220 west of the site. • South Creek – 1.29km west of the site.
Site Surroundings	The site is located within an area of residential use, and is bounded by: • Mitchell Street road reserve to the north. • Residential dwelling at No. 28 Mitchell Street to the east. • Residential complex at unknown address to south. • Residential dwellings at No. 32 Mitchell Street, and No.48, No. 50 and No. 52 Mamre Road to

	the west.

[1]Site area is approximated and based off the site survey plan referenced in Section 1.3.

2.2 Topography

The local topography falls towards the west to south-west. The site topography also falls towards the south to south-west at a gentle slope. The site levels vary from approximately RL33.06m AHD to approximately RL34.99m AHD. It should be noted levels are approximated off the site survey plan and vary across the site.

2.3 Regional Geology

Information obtained on the local regional subsurface conditions, referenced from the Department of Mineral Resources, Penrith 1:100,000 Geological Series Sheet 9030 Edition 1, dated 1991, by the Geological Survey of New South Wales, indicates the site is underlain by Bringelly Shale (Rwb) of the Wianamatta Group. The Bringelly Shale typically comprises "shale, carbonaceous claystone, laminite, fine to medium grained lithic sandstone, rare coal".

The site is also situated approximately 240m east of a geological boundary underlain by Quaternary Aged Holocene Deposits (Qal). The Holocene Deposits typically comprise "Fine grained sand, silt and clay".

3. SUBSURFACE CONDITIONS AND ASSESSMENT RESULTS

3.1 Stratigraphy

A summary of the surface and subsurface conditions from across the site are summarised in Table 2 below, and are interpreted from the assessment results. It should be noted that 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 rock strengths assessed by observation during auger penetration resistance in the boreholes are approximate and strength variances should be expected throughout the site. Due to the variable ground conditions throughout the site, along with limited information gathered in areas not accessible during the site investigation, it is recommended that confirmation of the subsurface materials be carried out during construction, or by additional boreholes carried out following demolition. It should also be noted that ground conditions within the site are expected to differ from those encountered and inferred in this report, since no geotechnical or geological exploration programme, no matter how comprehensive, can reveal and identify all subsurface conditions underlying the site.

Based on results during the site investigation within the boreholes, bedrock of varying strength and weathering is inferred to be generally dipping towards the rear of the site, towards the south to south-west. Higher strength bedrock is inferred to be encountered at shallower depths within the front portion of the site, dipping towards the south to south-west (rear portion of the site), where bedrock of inferred similar strength was encountered at greater depths as outlined in Table 2 below.

It should also be noted that higher blow counts encountered during SPT within the boreholes may be affected by factors such as gravels and ironstone bands which are present within the underlying soils, and any other deleterious material extending throughout the soils. These results should be read in conjunction with the boreholes, and geotechnical confirmation should be carried out during construction or by additional boreholes as site conditions are expected to vary.

Table 2. Summary of Subsurface Conditions

		Borehole ID	**BH1**	**BH2**	**BH3**
Unit	**Unit Type**	**Description**	**Depth/Thickness of Unit (m)**		
		Proposed Lower Basement FFL (m AHD)	RL31.500		
		Approximate Maximum Excavation at Borehole (m)	3.2	2.6	2
		Approximate RL Top of Borehole (m AHD)	34.7	34.1	33.5
1	Fill	Gravelly SAND, fine to medium grained, dark grey to blackish grey, fine to coarse grained gravel.	0.2 – 0.4	0.2 – 0.4	–
		Clayey SAND, fine grained, dark brown to blackish brown, low plasticity clay, with fine to medium grained gravel, grass rootlets.	Not Encountered		0.0 – 0.5
2	Alluvial Soils	Silty CLAY, high plasticity, brown, pale grey, pale brown and reddish brown laminations, fine to coarse grained ironstone gravel, some fine grained sand, estimated stiff to very stiff. (conglomerate pebbles in borehole BH3)	Not Encountered	0.4 – 1.4	0.5 – 1.0 3.2 – 3.5 4.6 – 5.5
		Sandy CLAY, medium to high plasticity, pale grey, pale brown and reddish brown laminations, fine grained sand, with fine to coarse grained ironstone and sandstone gravel, ironstone bands, estimated stiff to hard.		1.4 – 2.5	2.0 – 3.2 3.5 – 4.6 5.5 – 6.3
		Clayey SAND, low plasticity, pale grey, reddish brown to pale brown laminations, some fine to medium grained ironstone gravel, fine grained sand, estimated medium dense to dense.	Not Encountered		1.0 – 2.0
3	Residual Soils	Silty CLAY, low to medium plasticity, reddish brown and brown, pale grey, with fine to coarse grained ironstone gravel, fine grained sand, ironstone bands at depth, estimated firm to stiff, becoming estimated very stiff to hard.	0.4 – 3.3	2.5 – 4.7	6.3 – 7.8
		Shaly CLAY, low plasticity, grey to dark grey, some siltstone and coal laminations, estimated hard.	3.3 – 3.6	4.7 – 5.0	Not Encountered
4	Class V Shale	SHALE, dark brown to dark grey, with clay bands, some fine grained sand, extremely weathered, extremely low estimated strength. Class V Shale.	3.6 – 4.8	5.0 – 7.0	
5	Class V Siltstone	SILTSTONE, grey, dark grey laminations, some pale grey laminite, clay bands and fine grained sand, extremely weathered, extremely low estimated strength, becoming very low to low estimated strength at depth. Class V Siltstone.	4.8 – 6.25	7.0 – 9.0	7.8 – 9.0
6	Class IV Siltstone	SILTSTONE, dark grey to grey, pale grey sandstone patches, moderately weathered, low to medium estimated strength. Class IV Siltstone.	6.25 – 7.11	Inferred 9.0[1]	Unknown

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

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

Table 3. Approximate Reduced Level's Top of Units

	Borehole ID	**BH1**	**BH2**	**BH3**
Unit	**Unit Type**	**Approximate RL Top of Unit[1] (RL m AHD)**		
1	Fill	34.5	33.9	33.5
2	Alluvial Soils	Not Encountered	33.7	33
3	Residual Soils	34.3	31.6	27.2
4	Class V Shale[2]	31.1	29.1	Not Encountered
5	Class V Siltstone[2]	29.9	27.1	25.7
6	Class IV Siltstone[2]	28.45	Inferred 25.1	Unknown

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

[2] Confirmation of the actual depth and thickness of the Class V Shale, Class V Siltstone and Class IV Shale should be carried out by a geotechnical engineer by additional borehole drilling, or during construction.

3.2 Groundwater

No groundwater was observed or encountered during augering in boreholes BH1 to BH3 inclusive, to a maximum depth of approximately 9.0m (RL24.5m AHD). Water introduced during the NMLC coring process in borehole BH1 from below the termination depth at approximately 6.0m (RL28.7m AHD), further precluded any groundwater level indications.

Following completion of drilling in borehole BH3, a standpipe piezometer was installed to a depth of approximately 4.5m bgl (to RL29m AHD). It should be noted that the installed standpipe piezometer had no groundwater present within during the time of installation.

Groundwater measurements carried out on the 4th August 2018 indicates groundwater levels to be present at a depth of approximately 3.9m (RL29.6m AHD) within the site, at the time of the measurement. It is noted that observations made on the SPT sample collected from borehole BH3 at a depth of approximately 3.0m (RL30.5m AHD) indicated the presence of some groundwater.

Thus, groundwater is expected to be in the form of seepage through the pore spaces between particles of unconsolidated natural soils or through networks of fractures and solution openings in consolidated bedrock underlying the site. It should be noted that groundwater levels are subject to fluctuate during daily or seasonal factors. Groundwater monitoring should be carried out during construction, to assess groundwater inflow throughout the excavation areas.

4. LABORATORY TEST RESULTS

4.1 Rock Samples

Two (2) samples selected from the collected rock cores from borehole BH3 were tested by a NATA accredited laboratory, being Geo-logic Solutions, for diametral and axial point load strength index (Is_{50}). The results ranged between a point load index (Is_{50}) from 0.04MPa to 0.29MPa for diametral testing, and from 0.04MPa to 0.16MPa for axial testing. Test results correspond to predominately to very low to low strength rock, with possible medium strength rock layers. The point load test results laboratory certificate is presented in **Appendix F**.

4.2 Aggressivity and Salinity

Three (3) selected soil samples were sent to a NATA accredited testing laboratory, being ALS Environmental, to determine the pH, Chloride and Sulphate content, and electrical conductivity of the soils. A summary of the laboratory tests results are provided in Table 4 below, with laboratory certificates of the test results presented in **Appendix G** of this report.

Table 4. Summary of Laboratory Test Results

Sample/Test ID		**BH1 5.0m – 5.2m**	**BH2 3.0m – 3.2m**	**BH3 1.5m – 1.7m**
Soil Type		Bedrock	Residual Soils	Alluvial Soils
Aggressivity and Salinity	**pH**	7.6	5.4	7.4
	Moisture Content (%)	9.2	13.7	8.8
	Chloride (mg/kg)	860	1,140	360
	Sulphate SO_4 (mg/kg)	170	280	140
Electrical Conductivity (µS/cm)	**EC (µS/cm)**	766	984	274
	EC (dS/m)	0.766	0.987	0.274
	Multiplication Factor	10	8	14
	Saturation Extract ECe (dS/m)	7.66	7.896	3.836

Table 5. Aggressivity and Salinity Reference Table

Reference	**Pile Type**	**High Perm. Soils**	**Low Perm. Soils**	**pH**	**Chloride (mg/kg)**	**Sulphate SO_4 (mg/kg)**
AS 2159-2009	**Concrete Piles**	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 Piles	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	

In accordance to AS 2159-2009 "Piling – Design and Installation", the results of the laboratory tests and introduction of a multiplication factor for electrical conductivity (from Department of Natural Resources (DNR) publication "Site Investigations for Urban Salinity" – 2002), as outlined in Table 5 above, on the sample soils pH, Chloride and Sulphate content, and electrical conductivity indicates the following classification:

- Bedrock (BH1):
  - Non aggressive to steel piles in low and high permeability soils.
  - Non aggressive to concrete piles in low permeability soils.
  - Mildly aggressive to concrete piles in high permeability soils.
  - Electrical conductivity of saturated extract (ECe) of approximately 7.66dS/m, indicating "**moderately**" saline soils at borehole BH1 location

- Residual Soils (BH2):
  - Non aggressive to steel piles in low and high permeability soils.
  - Mildly aggressive to concrete piles in low permeability soils.
  - Moderately aggressive to concrete piles in high permeability soils.
  - Electrical conductivity of saturated extract (ECe) of approximately 7.896dS/m, indicating "**moderately**" saline soils at borehole BH2 location, with the potential for "**very**" saline soils being encountered.

- Alluvial Soils (BH3):
    - Non aggressive to steel piles in low and high permeability soils.
    - Non aggressive to concrete piles in low permeability soils.
    - Mildly aggressive to concrete piles in high permeability soils.
    - Electrical conductivity of saturated extract (ECe) of approximately 3.836dS/m, indicating "**slightly**" saline soils at borehole BH3 location, with the potential for "**moderately**" saline soils being encountered.

It should be note that soil salinity may vary throughout the site, and is based on testing at borehole locations, in conjunction with multiplication factors for electrical conductivity, as described above.

5. GEOTECHNICAL ASSESSMENT AND RECOMMENDATIONS

5.1 Dilapidation Survey

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

5.2 General Geotechnical Issues

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

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

Based on results of our assessment, a summary of the geotechnical aspects above and recommendations for construction and designs are presented below. These have considered inferred groundwater seepage levels based on readings during the site investigation and in the installed piezometer to be at depths of approximately 3.0m (RL30.5m AHD) to approximately 3.9m (RL29.6m AHD).

It is noted that groundwater levels are subject to fluctuations and are expected to be in the form of seepage through the pore spaces between particles of unconsolidated natural soils or through networks of fractures and solution openings in consolidated bedrock.

5.3 Inspection Pits and Underpinning

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

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

5.4 Excavation

Maximum excavation depths of approximately 2.0m to 3.2m (varying throughout the site) are expected to be required for construction of the proposed development, with locally deeper excavations to be required for the proposed lift shaft, footings and service trenches. Based on this information and existing ground conditions, it is anticipated that excavation will extend through Unit 1 (fill) to Unit 3 (residual soils), inclusive, throughout the majority of the proposed development area, with the possibility of extending through Unit 4 (Class V Shale) in some areas of the site, as outlined in Table 2 and Table 3 above.

5.4.1 Excavation Assessment

Excavation through Unit 1 to Unit 3 inclusive (softer soils) and Unit 4 (extremely low to low 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.

5.5 Groundwater Management

Based on the site investigation and piezometer readings, groundwater levels are expected to be at depths below the proposed basement FFL of RL30.500m AHD. Although groundwater levels observed during the site investigation and measured within the piezometer indicate levels to be below the proposed basement FFL, it should be noted that these levels have the potential to elevate during daily or seasonal influences such as heavy rainfall, damaged services, flooding, etc. Thus, we expect any groundwater inflow into the excavation to be in the form of seepage through the voids in the natural soils, and through the defects (such as bedding planes, joints, etc.) in the underlying weathered bedrock. Seepage may also occur within the fill material, 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 defects in the rock (including bedding seams and joints) 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:

- A conventional sump and pump system which may be used both during construction and for permanent groundwater control below the basement floor slab.
- Drainage installed around the perimeter of the basement behind all basement retaining walls, and below the basement slab (adjoining the existing three storey building). This drainage should be connected to a sump and pump out system and discharged into the stormwater system (which may require council approval).
- Collection trenches or pipes and stormwater pits may be installed in conjunction with the above method, and connected to the building stormwater system.

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

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

5.6 Excavation Stability

Maximum excavation depths are expected to vary within the site from approximately 2.0m to 3.2m (varying throughout the site) for construction of the proposed development. 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.

5.6.1 Batter Slopes

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

Temporary or permanent batter slopes may be considered where sufficient space exists between the basement walls and adjoining infrastructures, and where the adjacent infrastructures are located outside the "zone of influence" (obtained by drawing a line 45º above horizontal from the base of the proposed basement walls) for the use temporary batter slopes. Table 6 provides maximum recommended slopes for permanent and temporary batters.

Table 6. Recommended Maximum Batter Slopes

Unit	Maximum Batter Slope (H : V)	
	Permanent	Temporary
Fill (Unit 1)	4 : 1	2 : 1
Alluvial Soils (Unit 2)	3 : 1	1.5 : 1
Residual Soils (Unit 3)	2 : 1	1 : 1
Class V Shale (Unit 4) – if encountered	1.5 : 1	0.75 : 1

[1]Subject to inspection by a geotechnical engineer. Remedial options may be required (i.e. rock bolting, shotcreting, etc.)

All batter slopes within the site should remain stable providing all surcharge and construction loads are kept out of the "zone of influence" (obtained by drawing a line 45º above horizontal from the base of the proposed basement walls) plus an additional 1.0m. A geotechnical engineer should inspect the batter slopes within the site. Consideration should be given to shotcreting and soil nailing where steeper batter slopes are to be used.

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

5.6.2 Excavation Retention Support Systems

Where there is insufficient space between the basement walls and adjoining infrastructures (predominately along the site eastern and western boundaries), or where adjacent infrastructures are located within the "zone of influence" (as outlined in Section 5.6.1 above), consideration should be given to a suitable retention system such as a soldier pile wall sufficiently embedded into the underlying bedrock, with concrete infill panels for the support of the excavation. Closer spaced piles may be required to reduce lateral movements particularly where adjacent structures, such as buildings or pavements are located near the excavation, and to prevent collapse of loose fill situ materials, natural soils and weathered bedrock. Pile spacing should be analysed and designed by the project structural engineer and should consider horizontal pressures due to surcharge loads from adjacent infrastructures (i.e. buildings, road reserves, etc.), or long term loadings.

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

The use of a more rigid retention system such as a cast in-situ contiguous pile wall should also be considered to reduce the lateral movements and risk of potential damage to adjacent infrastructures (i.e. adjacent road reserves and infrastructures). This option may also be adopted where excessive surcharges are adjacent to the basement excavation, and to meet acceptable deflection criteria.

It should be noted that groundwater inflow may pass through shoring pile gaps during excavation. This may be controlled by the installation of strip drains behind the retention system, connected to the buildings stormwater system. Shotcreting or localised grouting may also be used in weak areas of the retention system, predominately where groundwater seepage is visible. Shoring design should take into consideration both short term (during construction) and permanent conditions, along with surcharge loading and footing loads from adjacent infrastructures. Where groundwater is deemed to be relatively high, and permeability rates are excessive, it is recommended that consideration be given to a contiguous pile wall with strip drains installed behind the piles and shotcreting in weak areas susceptible to groundwater inflow.

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

- Top-down construction.
- Bottom-up construction.

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

If considered, the shoring wall can be designed using the recommended design parameters provided in Section 5.6.3. 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.

5.6.3 Design Parameters (Earth Pressures)

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

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

Lateral active or "at rest" earth pressure:

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

Passive earth pressure:

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

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

Active earth pressure:

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

Where:

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

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

Support system designed using the earth pressure approach may be based on the parameters given in Table 7 below for soils and rock horizons underlying the site. Table 7 also provides preliminary coefficients of lateral earth pressure for the soils and rock horizons encountered in the site, along with preliminary earthquake site risk classification. These are based on fully drained conditions and that the ground behind the retention walls is horizontal.

Table 7. Preliminary Geotechnical Design Parameters

Material	Fill (Unit 1)	Alluvial Soils (Unit 2)	Residual Soils (Unit 3)	Class V Shale/Siltstone[3] (Unit 4/Unit 5)	Class IV Siltstone [3] (Unit 6)
Unit Weight (kN/m^3)[4]	17	19	20	22	22
Effective Cohesion c' (kPa)	0	3*	5*	25	50
Angle of Friction ϕ' (°)	26	24	24	27	28
Modulus of Elasticity E_{sh} (MPa)	5	8	12	75	250
Earth Pressure Coefficient At Rest Ko[1]	0.56	0.59	0.59	0.5	0.5
Earth Pressure Coefficient Active Ka[2]	0.39	0.42	0.42	0.3	0.3
Earth Pressure Coefficient Passive Kp[2]	2.56	2.37	2.37	3.0	3.0
Preliminary Earthquake Site Risk Classification	• AS 1170.4-2011 indicates the site may be classified as a "Shallow Soil Site" (Class C_e). • AS 1170-4-2011 indicates a Hazard Factor (Z) for Sydney is 0.08				

[1] Earth pressure coefficient at rest (Ko) can be calculated using Jacky's equation.
[2] Earth pressure coefficient of active (Ka) and passive (Kp) can be calculated using Rankine's or Coulomb's equation.
[3] The values for rock assume no defects of adverse dipping is present in the bedrock. All excavation rock faces should be inspected on a regular basis by an experienced engineering geologist or geotechnical engineer.
[4] Above groundwater levels.

Notes:

- For undrained (temporary) clay soils, higher earth pressures (K=1) will apply.
- *An effective cohesion c' of **0kPa** shall apply to the clayey sand material within the site.

5.7 Foundations

Following excavation to the proposed basement FFL of RL31.500m AHD, and based on the boreholes carried out, we expect varying ground conditions comprising predominately Unit 2 (alluvial soils) and Unit 3 (residual soils) to be exposed at bulk level excavation, with the potential for Unit 4 (Class V Shale) to be exposed in some areas of the site following bulk level excavation. Variable strength alluvial and residual soils are likely to result in total and differential settlement under working load, and not adequately support shallow foundations for the proposed development.

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

5.7.1 Geotechnical Assessment

Based on the proposed development and assessment of the subsurface conditions, a piled foundation system is likely to be adopted, with the building fully supported by piles sufficiently founded into appropriate bedrock underlying the site (i.e. Class IV Siltstone or better).

It should be noted that due to the potential variable bedrock conditions throughout the site (i.e. Class V Shale, Class V Siltstone and Class IV Siltstone), precaution should be taken for the design of the building foundation system, taking into consideration the preliminary geotechnical design parameters in Table 8 below. Higher bearing capacities may be justified subject to confirmation by inspection during construction, or by additional borehole drilling and rock strength testing. Bearing capacity and settlement behaviour varies according to foundation depth, shape and dimensions.

Given the potential for variable ground conditions within the site, it is recommended that all foundations are constructed on consistent bedrock, with piles sufficiently embedded into consistent bedrock underlying the site, in order to provide uniform support and reduce the potential for differential settlements. 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.

Piles with increased socket depths into higher strength bedrock may be considered, in order to increase the resistance against lateral loading induced by earthquakes or winds, and to achieve higher bearing capacities.

Table 8 provides recommended geotechnical design parameters.

Table 8. 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
Alluvial Soils (Unit 2)	100	N/A	N/A
Residual Soils (Unit 3)	150	N/A	N/A
Class V Shale (Unit 4)[2]	600	30	15
Class V Siltstone (Unit 5)[2]	700	50	25
Class IV Siltstone (Unit 6)[2]	1,000	100	50

[1] Minimum embedment of 0.4m for shallow foundations and 0.5m for deep foundations.

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

Notes:

- N/A = Not Applicable. Not recommended for the proposed development.
- The depth of the underlying bedrock material should be confirmed either prior to construction by further borehole testing, or during construction by inspection.
- It is recommended that geotechnical inspections on the foundations are completed by a geotechnical engineer to determine the material and confirm the required bearing capacity has been achieved.

5.7.2 Geotechnical Comments

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

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

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

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

5.8 Filling

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

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

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

5.9 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.

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

- Place horizontal loose layers not more than 300mm thickness over the prepared subgrade.
- Compact to a minimum dry density ratio not less than 98% of the maximum dry density for the building platforms.
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6. ADDITIONAL GEOTECHNICAL RECOMMENDATIONS

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

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

7. 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 **Designcorp Architects Pty Ltd**, and any misinterpretations or reliances by third parties of this report shall be at their own risk. Any legal or other liabilities resulting from the use of this report by other parties can not be religated to GCA.

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

For and behalf of

Geotechnical Consultants Australia (GCA)

Jnader

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

8. REFERENCES

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NSW Six Maps.

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APPENDIX A

Important Information About Your
Geotechnical Report

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

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

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

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

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

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

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

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

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

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

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

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

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

Geotechnical Report's Recommendations Are Not Final

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

Geotechnical Report's Are Subject to Misinterpretations

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

Engineering Borehole Logs And Data Should Not be Redrawn

Geotechnical engineers prepare final borehole and testing logs, figure, etc. based on results and interpretation of field logs and laboratory data following the site investigation. The logs, figure, etc. provided in the geotechnical report should never be redrawn or altered for inclusion in any other documents from this report, includined architectural or other design drawings.

Providing The Full Geotechnical Report For Guidance

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

Understanding Limitation Provisions

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

Other Limitations

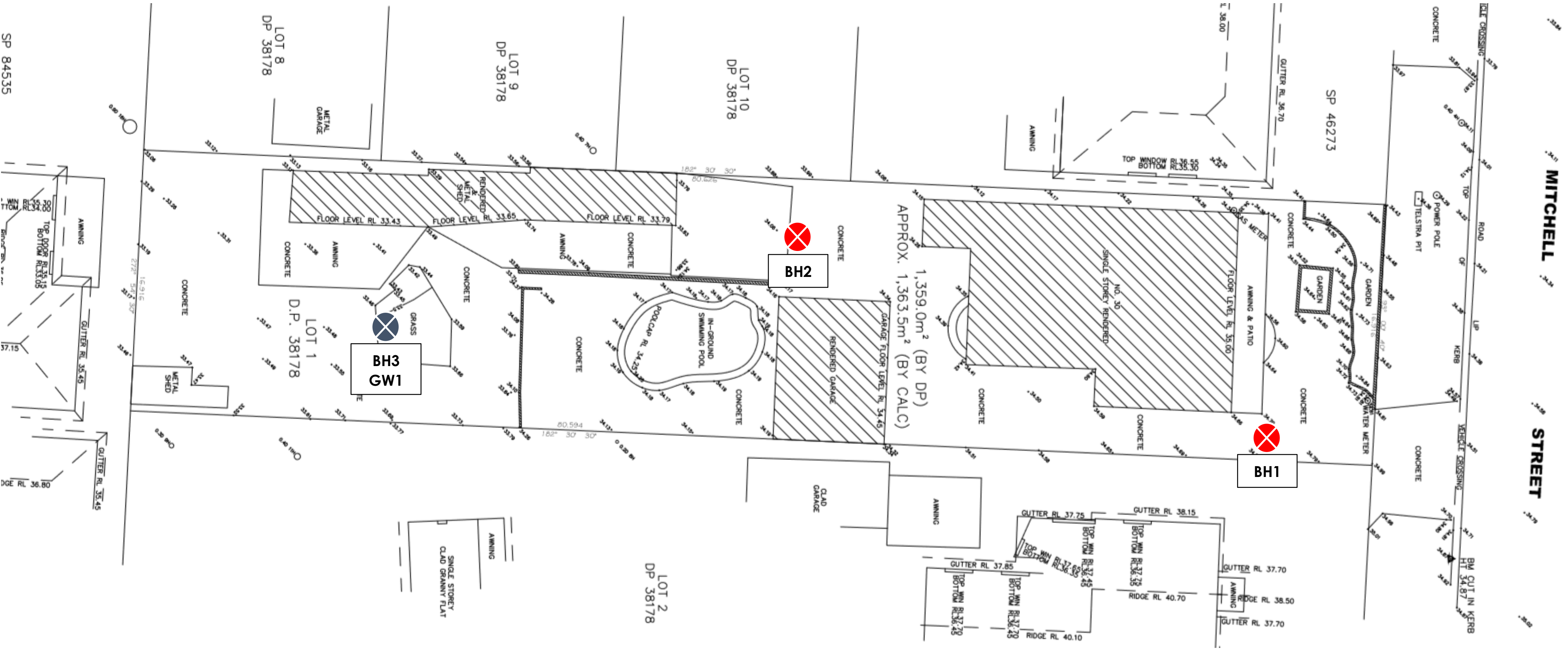
GCA will not be liable to revise or update the report to take into account any events or circumstances (seen or unforeseen), or any fact occurring or becoming apparent after the date of the report. This report is the subject of copyright and shall not be reproduced either totally or in part without the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

APPENDIX B

Legend:

- 🟥 Approximate Borehole Drilled
- ● Approximate Borehole Drilled/Piezometer

Borehole ID	Total Depth (m bgl)	Approx. RL (m AHD)
BH1	7.11	34.7
BH2	9.0	34.1
BH3/GW1	9.0	33.5



GCA
Geotechnical Consultants Australia

Figure 1
Site Plan

Job No: G1887-1

Geotechnical Investigation

Designcorp Architects Pty Ltd

30 Mitchell Street St Marys NSW 2760

Drawn: JN

Date: 11/08/2018

Scale: NTS

Image Source: Site Survey Plan prepared by New South Surveys, titled “30 Mitchell Street, St Marys Topographical Survey”, dated 28th November 2011 and referenced drawing No. 117490.

APPENDIX C

GCA
Geotechnical Consultants Australia

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

DRILLING/EXCAVATION METHOD

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

PENETRATIION/EXCAVATION RESISTANCE

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

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

WATER

▽	Water level at date shown	◁	Partial water loss
▷	Water inflow	◀	Complete water loss

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

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

MOISTURE CONDITION (AS 1726-1993)

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

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

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

For cohesive soils the following codes may also be used:

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

SAMPLING AND TESTING

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

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

ROCK QUALITY

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

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

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

ROCK STRENGTH TEST RESULTS

●	Diametral Point Load Index test
■	Axial Point Load Index test

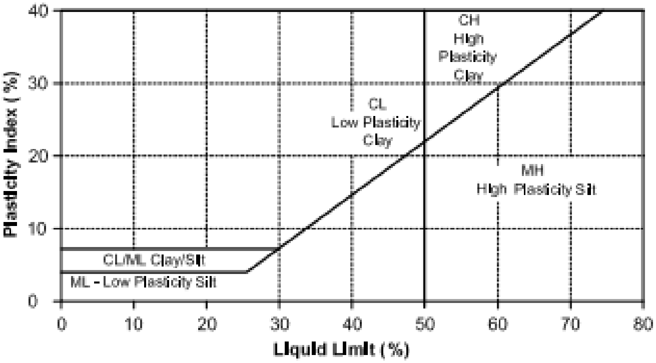
Method and Terms for Soil and Rock Descriptions Used on Borehole and Test Pit Reports

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

COHESIONLESS SOILS PARTICLE SIZE DESCRIPTIVE TERMS

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

PLASTICITY PROPERTIES



COHESIVE SOILS – CONSISTENCY (AS 1726-1993)

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

PLASTICITY

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

COHESIONLESS SOILS - RELATIVE DENSITY

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

UNIFIED SOIL CLASSIFICATION

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

ROCK MATERIAL WEATHERING

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

ROCK STRENGTH (AS 1726-1993 and ISRM)

Term	Symbol	Point Load Index $Is_{(50)}$ (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	<6 mm	Thinly Laminated
	6 to 20 mm	Laminated
Very closely spaced	20 to 60 mm	Very Thin
Closely spaced	0.06 to 0.2 m	Thin
Moderately widely spaced	0.2 to 0.6 m	Medium
Widely spaced	0.6 to 2 m	Thick
Very widely spaced	>2 m	Very Thick

Type	Definition
B	Bedding
J	Joint
F	Fault
C	Cleavage
SZ	Shear Zone
CZ	Crushed Zone
MB	Mechanical Break

Planarity	Roughness
	VR – Very Rough
P – Planar	R – Rough
Ir – Irregular	S – Smooth
St – Stepped	Sl – Slickensides
U – Undulating	Po – Polished

Coating or Infill	Description
Clean	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

APPENDIX D

Geotechnical Consultants Australia Pty Ltd
info@geoconsultants.com.au
www.geoconsultants.com.au

BOREHOLE NUMBER BH1

CLIENT Designcorp Architects Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G1887-1
PROJECT LOCATION 30 Mitchell Street St Marys NSW 2760

DATE STARTED 2/8/18
COMPLETED 2/8/18
R.L. SURFACE 34.7
DATUM m AHD
DRILLING CONTRACTOR BG Drilling Pty Ltd
SLOPE 90°
BEARING ---
EQUIPMENT Track Mounted Drilling Rig
HOLE LOCATION Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter
LOGGED BY JN
CHECKED BY JN
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT	Not Encountered During Augering	34.5		⁘		200mm Concrete Pavement.		PAVEMENT
				▩		Gravelly SAND, fine to medium grained, dark grey to blackish grey, fine to coarse grained gravel, moist.		FILL
		34.0 33.5	0.5 1.0	▨	CI	Silty CLAY, medium plasticity, reddish brown, pale grey laminations, with fine to medium grained ironstone gravel, moist, estimated firm to stiff.		RESIDUAL SOILS
		33.0 32.5	1.5 2.0	▨	CI	Silty CLAY, medium plasticity, pale grey to pale brown, fine to coarse grained ironstone gravel and fine grained sand, some ironstone bands, tree rootlets, moist, estimated stiff.	SPT 4, 5, 8 N=13	
		32.0	2.5 3.0	▨	CI	Silty CLAY, medium plasticity, pale grey to reddish brown, fine to coarse grained ironstone gravel, moist, estimated stiff to very stiff.	SPT 7, 11, 29 N=40	
		31.5		▨	CI	Silty CLAY, medium plasticity, pale grey, fine to coarse grained ironstone gravels and ironstone bands, moist, estimated very stiff.		
			3.5	▧		Shaly CLAY, low plasticity, grey to dark grey, some coal and siltstone laminations, moist, estimated hard.		
		31.0 30.5 30.0	4.0 4.5	☰		SHALE, dark brown to dark grey, with clay bands, some fine grained sand, extremely weathered, extremely low estimated strength, moist.		BEDROCK
			5.0	× ×		SILTSTONE, pale brown, dark grey laminations, pale grey laminite, some clay bands and fine grained sand, extremely weathered, extremely low estimated strength, moist.		

BOREHOLE NUMBER BH1

PAGE 2 OF 3

CLIENT Designcorp Architects Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G1887-1
PROJECT LOCATION 30 Mitchell Street St Marys NSW 2760

DATE STARTED 2/8/18
COMPLETED 2/8/18
R.L. SURFACE 34.7
DATUM m AHD
DRILLING CONTRACTOR BG Drilling Pty Ltd
SLOPE 90°
BEARING ---
EQUIPMENT Track Mounted Drilling Rig
HOLE LOCATION Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter
LOGGED BY JN
CHECKED BY JN
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT		29.5		× ×		SILTSTONE, pale brown, dark grey laminations, pale grey laminite, some clay bands and fine grained sand, extremely weathered, extremely low estimated strength, moist. *(continued)*	DS	
			5.5	× ×				
		29.0		× ×		becoming very low estimated strength from 5.8m bgl.		
			6.0	× ×				
		28.5				Borehole BH1 continued as cored hole		
			6.5					
		28.0						
			7.0					
		27.5						
			7.5					
		27.0						
			8.0					
		26.5						
			8.5					
		26.0						
			9.0					
		25.5						
			9.5					
		25.0						
			10.0					

BOREHOLE NUMBER BH1

CLIENT Designcorp Architects Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G1887-1
PROJECT LOCATION 30 Mitchell Street St Marys NSW 2760

DATE STARTED 2/8/18 **COMPLETED** 2/8/18
R.L. SURFACE 34.7
DATUM m AHD
DRILLING CONTRACTOR BG Drilling Pty Ltd
SLOPE 90°
BEARING ---
EQUIPMENT Track Mounted Drilling Rig
HOLE LOCATION Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter
LOGGED BY JN
CHECKED BY JN
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Material Description	Weathering	Estimated Strength EL VL L M H VH EH	Is(50) MPa D- diametral A- axial	RQD %	Defect Spacing mm 30 100 300 1000 3000	Defect Description
		29.5 29.0	5.5 6.0		Continued from non-cored borehole						
NMLC		28.5			Core Loss 250mm.						
		28.0	6.5 7.0	× ×	SILTSTONE, dark grey to grey, pale grey sandstone laminations and occasional patches.	MW		D A 0.29 0.16 D A 0.04 0.04	24		6.28m, Joint (J), Smooth (S), Clean (C), Undulating (U), 5 degrees (deg) 6.42m, J, S, C, U, 5 deg 6.55m, Bedding (B), Rough (R), Sandstone (S.S), U, 0-5 deg, 30mm 6.61m, J, S, C, U, 0-5 deg 6.76m, J, S, C, U, 0-5 deg 6.83m, J, S, C, U, 0-5 deg 6.89m, B, C, Clay (Cl), U, 10 deg 6.96m, Extremely Weathered (EW), Cl, 30mm 7.00m, EW, Cl, 110mm
		27.5 27.0 26.5 26.0 25.5 25.0	7.5 8.0 8.5 9.0 9.5 10.0		BH1 terminated at 7.11m						

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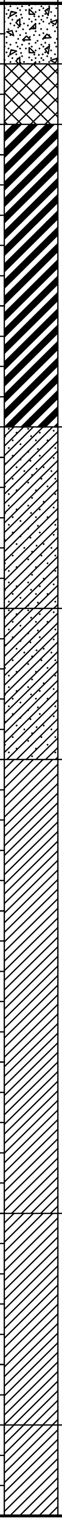
www.geoconsultants.com.au

BOREHOLE NUMBER BH2

PAGE 1 OF 2

CLIENT Designcorp Architects Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G1887-1
PROJECT LOCATION 30 Mitchell Street St Marys NSW 2760

DATE STARTED 2/8/18
COMPLETED 2/8/18
R.L. SURFACE 34.1
DATUM m AHD
DRILLING CONTRACTOR BG Drilling Pty Ltd
SLOPE 90°
BEARING ---
EQUIPMENT Track Mounted Drilling Rig
HOLE LOCATION Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter
LOGGED BY JN
CHECKED BY JN
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate



Method	Water	RL (m)	Depth (m)	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT	Not Encountered During Augering	34.0			200mm Concrete Pavement.		PAVEMENT
					Gravelly SAND, fine to medium grained, dark grey to blackish grey, fine to coarse grained gravel, moist.		FILL
		33.5 33.0	0.5 1.0	CH	Silty CLAY, high plasticity, brown, pale grey to reddish brown laminations, fine to medium grained gravel, some fine grained sand, moist, estimated stiff.		ALLUVIAL SOILS
		32.5	1.5	CIS	Sandy CLAY, medium plasticity, pale grey, pale brown and reddish brown laminations, fine grained sand, with fine to medium grained ironstone gravel, some tree rootlets, moist estimated stiff. becoming estimated very stiff from 1.8m bgl.	SPT 5, 9, 14 N=23	RESIDUAL SOILS
		32.0	2.0	CHS	Sandy CLAY, high plasticity, pale brown, pale grey laminations, fine grained sand, with fine to coarse grained sandstone and ironstone gravel, moist, estimated very stiff.		
		31.5 31.0 30.5	2.5 3.0 3.5	CL-CI	Silty CLAY, low to medium plasticity, pale grey, reddish brown and brown laminations, some ironstone bands and fine to medium grained gravel, moist, estimated very stiff. becoming estimated hard from 3.3m bgl.	SPT 8, 17, 22 N=39 DS	
		30.0 29.5	4.0 4.5	CL-CI	Silty CLAY, low to medium plasticity, pale grey, pale brown laminations, some fine to medium grained gravel and fine grained sand, moist, estimated hard.	SPT 11, 30 Terminated	
			5.0		Shaly CLAY, low plasticity, grey to dark grey, some coal and siltstone laminations, moist, estimated hard.		

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BOREHOLE NUMBER BH2

CLIENT Designcorp Architects Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G1887-1
PROJECT LOCATION 30 Mitchell Street St Marys NSW 2760

DATE STARTED 2/8/18
COMPLETED 2/8/18
R.L. SURFACE 34.1
DATUM m AHD
DRILLING CONTRACTOR BG Drilling Pty Ltd
SLOPE 90°
BEARING ---
EQUIPMENT Track Mounted Drilling Rig
HOLE LOCATION Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter
LOGGED BY JN
CHECKED BY JN
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	Water	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT		29.0				SHALE, dark brown to dark grey, with clay bands, some fine grained sand, extremely weathered, extremely low estimated strength, moist.		BEDROCK
			5.5					
		28.5						
			6.0					
		28.0						
			6.5					
		27.5						
			7.0					
		27.0				SILTSTONE, pale brown, dark grey laminations, pale grey laminite, some clay bands and fine grained sand, extremely weathered, extremely low estimated strength, moist.		
			7.5					
		26.5						
			8.0					
		26.0				becoming very low estimated strength from 8.0m bgl.		
			8.5					
		25.5				becoming very low to low estimated strength from 8.6m bgl.		
			9.0					TC Bit Refual at 9.0m bgl.
		25.0				Borehole BH2 terminated at 9m		
			9.5					
		24.5						
			10.0					

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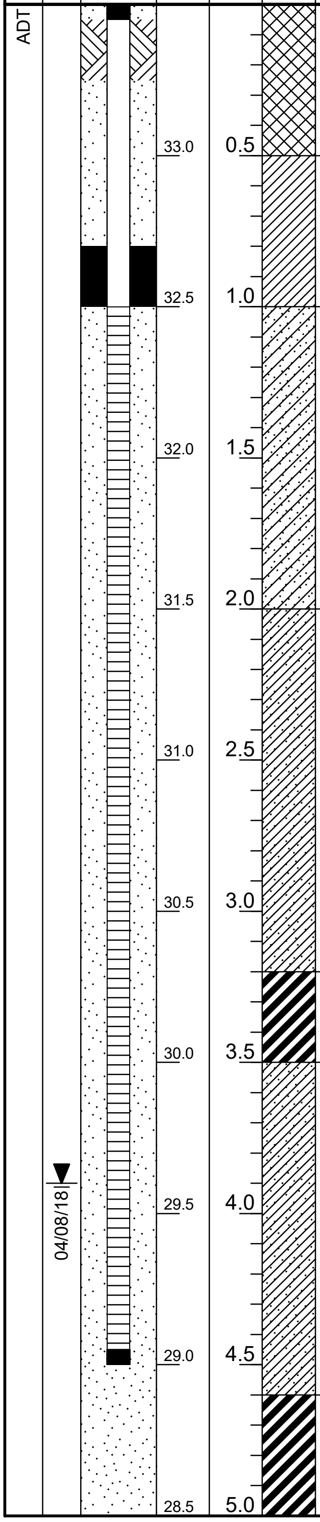
BOREHOLE NUMBER BH3

PAGE 1 OF 2

CLIENT Designcorp Architects Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G1887-1
PROJECT LOCATION 30 Mitchell Street St Marys NSW 2760

DATE STARTED 2/8/18 **COMPLETED** 2/8/18
R.L. SURFACE 33.5 **DATUM** m AHD
DRILLING CONTRACTOR BG Drilling Pty Ltd
SLOPE 90° **BEARING** ---
EQUIPMENT Track Mounted Drilling Rig
HOLE LOCATION Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter
LOGGED BY JN **CHECKED BY** JN
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate



Method	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT		Clayey SAND, fine grained, dark brown to blackish brown, with fine to medium grained gravel, low plasticity clay, grass rootlets, moist.		FILL
	CL-CI	Silty CLAY, low to medium plasticity, brown, pale brown to pale grey laminations, with fine to coarse grained ironstone gravel, moist, estimated stiff.		ALLUVIAL SOILS
	SC	Clayey SAND, fine grained, pale grey, reddish brown to pale brown laminations, low plasticity clay, some fine to medium grained ironstone gravel, moist, estimated medium dense. becoming medium dense to dense from 1.8m bgl.	SPT 16, 17, 16 N=33 DS	
	CIS	Sandy CLAY, medium plasticity, reddish brown to dark reddish brown, pale grey laminations, some fine to coarse grained sandstone and ironstone gravel, moist, estimated very stiff to hard.	SPT 10, 19, 20 N=39	
	CI-CH	Silty CLAY, medium to high plasticity, pale grey to grey, reddish brown and brown laminations, with fine grained sand, ironstone bands and fine to coarse grained ironstone gravel, moist, estimated hard. conglomerate pebbles from 3.3m to 3.5m bgl.		
	CIS	Sandy CLAY, medium plasticity, dark brown, with fine to medium grained gravel, fine grained sand, moist, estimated hard.		
	CI-CH	Silty CLAY, medium to high plasticity, pale grey to grey, reddish brown and brown laminations, fine grained sand, ironstone bands and fine to coarse grained ironstone gravel, moist, hard.	SPT 15, 22, 28 N=50	

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BOREHOLE NUMBER BH3

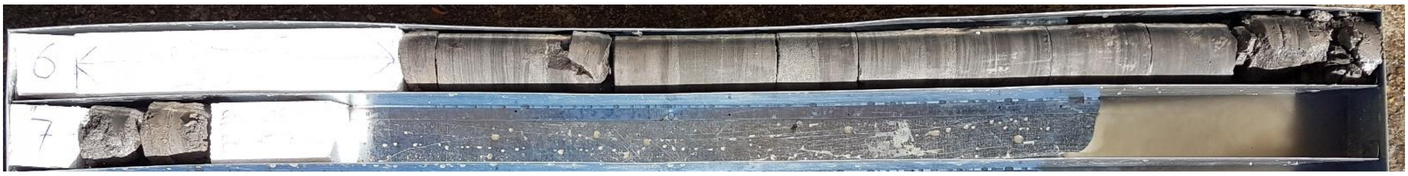
CLIENT Designcorp Architects Pty Ltd
PROJECT NAME Geotechnical Investigation
PROJECT NUMBER G1887-1
PROJECT LOCATION 30 Mitchell Street St Marys NSW 2760

DATE STARTED 2/8/18
COMPLETED 2/8/18
R.L. SURFACE 33.5
DATUM m AHD
DRILLING CONTRACTOR BG Drilling Pty Ltd
SLOPE 90°
BEARING ---
EQUIPMENT Track Mounted Drilling Rig
HOLE LOCATION Refer To Site Plan (Figure 1) For Test Locations
HOLE SIZE 100mm Diameter
LOGGED BY JN
CHECKED BY JN
NOTES RL To The Top Of The Borehole & Depths Of The Subsurface Conditions Are Approximate

Method	Water	Well Details	RL (m)	Depth (m)	Graphic Log	Classification Symbol	Material Description	Samples Tests Remarks	Additional Observations
ADT		░	28.0	5.5	▨	CI-CH	Silty CLAY, medium to high plasticity, pale grey to grey, reddish brown and brown laminations, fine grained sand, ironstone bands and fine to coarse grained ironstone gravel, moist, hard. *(continued)*		
		░	27.5	6.0	▨	CIS-CHS	Sandy CLAY, medium to high plasticity, pale brown, pale grey laminations, fine grained sand, some fine to medium grained gravel, moist, estimated hard.	SPT 12, 23, 28 N=51	
		░	27.0 26.5 26.0	6.5 7.0 7.5	▨	CI-CH	Silty CLAY, medium to high plasticity, grey to dark grey, blackish grey and pale brown laminations, moist, estimated hard.		RESIDUAL SOILS
		░	25.5 25.0 24.5	8.0 8.5 9.0	× ×		SILTSTONE, pale brown, dark grey laminations, pale grey laminite, some clay bands and fine grained sand, extremely weathered, extremely low estimated strength, moist.		BEDROCK
			24.0 23.5	9.5 10.0			Borehole BH3 terminated at 9m		

APPENDIX E

BOREHOLE BH1 CORING STARTS FROM 6.0m to 7.11m



GCA Geotechnical Consultants Australia	**Geotechnical Investigation**	Core Box Photographs
	Designcorp Architects Pty Ltd	Job No: G1887-1
	30 Mitchell Street St Marys NSW 2760	Date: 11/08/2018

APPENDIX F

Geo-Logic Solutions
Geotechnical & Environmental Testing

Unit 3 /112 Fairfield Street, Fairfield East NSW 2165
www.geo-logic.com.au
ABN: 57 621 548 294
PH: 0402 597 452
Email: samer@geo-logic.com.au

Point Load Strength Index

TEST METHOD: AS4133.4.1

Client:	**Geotechnical Consultants Australia Pty Ltd**
Project :	**Materials Testing**
Location:	**G1883-1 St Marys**
Project No.	**G136**
Date Reported:	**10/08/2018**
Report No.	**G136-Rev1**
Sample Procedure	**Sampled By Client: Core Samples Supplied**

Sample Number	**BH1**	**BH1**			
Sample Depth (m)	**6.85**	**7.07**			
Date Sampled	**2/08/2018**	**2/08/2018**			
Sample Description/ Rock Type	Shale	Shale			
Sample size when received	Approx 50mm X 100mm Cylinder	Approx 50mm X 100mm Cylinder			
Test Type	Diametral	Diametral			
Is - (Mpa)	0.29	0.04			
Is(50) - (Mpa)	0.29	0.04			
Moisture Condition	Moist	Moist			
Test Type	Axial	Axial			
Is - (Mpa)	0.17	0.04			
Is(50) - (Mpa)	0.16	0.04			
Moisture Condition	Moist	Moist			
Sample Weakness Description	N/A	N/A			
Sample Storage History	Sealed Bag	Sealed Bag			
Date Tested	8/08/2018	8/08/2018			

NOTES:

All equipment used in testing process has been calibrated by a NATA accredited laboratory.

Laboratory Approved Signatory:

Samer Ghanem

Date:

10/08/2018

Sign:

APPENDIX G

ALS Environmental

CERTIFICATE OF ANALYSIS

Work Order	: **ES1822659**	Page	: 1 of 2
Client	: **GEOTECHNICAL CONSULTANTS AUSTRALIA**	Laboratory	: Environmental Division Sydney
Contact	: JOE NADER	Contact	: Customer Services ES
Address	: 90 Sorrell St North Parramatta NSW 2151	Address	: 277-289 Woodpark Road Smithfield NSW Australia 2164
Telephone	: ----	Telephone	: +61-2-8784 8555
Project	: G1887-1	Date Samples Received	: 02-Aug-2018 14:10
Order number	: G1887-1	Date Analysis Commenced	: 02-Aug-2018
C-O-C number	: ----	Issue Date	: 07-Aug-2018 09:16
Sampler	: JOE NADER		
Site	: ----		
Quote number	: EN/333		
No. of samples received	: 3		
No. of samples analysed	: 3		

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. 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	Inorganic Chemist	Sydney Inorganics, Smithfield, NSW
Celine Conceicao	Senior Spectroscopist	Sydney Inorganics, Smithfield, NSW
Edwandy Fadjar	Organic Coordinator	Sydney Inorganics, Smithfield, NSW

General Comments

The analytical procedures used by the Environmental Division have been developed from established internationally recognized procedures such as those published by the USEPA, APHA, AS and NEPM. In house developed procedures are employed in the absence of documented standards or by 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 Contact 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)**			*Client sample ID*	**BH1** **5.0-5.2m**	**BH2** **3.0-3.2m**	**BH3** **1.5-1.7m**	----	----
			Client sampling date / time	02-Aug-2018 00:00	02-Aug-2018 00:00	02-Aug-2018 00:00	----	----
Compound	*CAS Number*	*LOR*	*Unit*	**ES1822659-001**	**ES1822659-002**	**ES1822659-003**	--------	--------
				Result	Result	Result	----	----
EA002: pH 1:5 (Soils)								
pH Value	----	0.1	pH Unit	**7.6**	**5.4**	**7.4**	----	----
EA010: Conductivity (1:5)								
Electrical Conductivity @ 25°C	----	1	µS/cm	**766**	**984**	**274**	----	----
EA055: Moisture Content (Dried @ 105-110°C)								
Moisture Content	----	1.0	%	**9.2**	**13.7**	**8.8**	----	----
ED040S : Soluble Sulfate by ICPAES								
Sulfate as SO4 2-	14808-79-8	10	mg/kg	**170**	**280**	**140**	----	----
ED045G: Chloride by Discrete Analyser								
Chloride	16887-00-6	10	mg/kg	**860**	**1140**	**360**	----	----

QUALITY CONTROL REPORT

Work Order	: **ES1822659**	Page	: 1 of 3
Client	: **GEOTECHNICAL CONSULTANTS AUSTRALIA**	Laboratory	: Environmental Division Sydney
Contact	: JOE NADER	Contact	: Customer Services ES
Address	: 90 Sorrell St North Parramatta NSW 2151	Address	: 277-289 Woodpark Road Smithfield NSW Australia 2164
Telephone	: ----	Telephone	: +61-2-8784 8555
Project	: G1887-1	Date Samples Received	: 02-Aug-2018
Order number	: G1887-1	Date Analysis Commenced	: 02-Aug-2018
C-O-C number	: ----	Issue Date	: 07-Aug-2018
Sampler	: JOE NADER		
Site	: ----		
Quote number	: EN/333		
No. of samples received	: 3		
No. of samples analysed	: 3		

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. 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	Inorganic Chemist	Sydney Inorganics, Smithfield, NSW
Celine Conceicao	Senior Spectroscopist	Sydney Inorganics, Smithfield, NSW
Edwandy Fadjar	Organic Coordinator	Sydney Inorganics, Smithfield, NSW

General Comments

The analytical procedures used by the Environmental Division have been developed from established internationally recognized procedures such as those published by the USEPA, APHA, AS and NEPM. In house developed procedures are employed in the absence of documented standards or by 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 hig

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	*Client sample ID*	*Method: Compound*	*CAS Number*	*LOR*	*Unit*	*Original Result*	*Duplicate Result*	*RPD (%)*	*Recovery Limits (%)*
EA002: pH 1:5 (Soils) (QC Lot: 1849589)									
ES1822655-001	Anonymous	EA002: pH Value	----	0.1	pH Unit	9.1	9.1	0.00	0% - 20%
ES1822659-001	BH1 5.0-5.2m	EA002: pH Value	----	0.1	pH Unit	7.6	7.8	1.82	0% - 20%
EA010: Conductivity (1:5) (QC Lot: 1849588)									
ES1822655-001	Anonymous	EA010: Electrical Conductivity @ 25°C	----	1	µS/cm	314	322	2.52	0% - 20%
ES1822659-001	BH1 5.0-5.2m	EA010: Electrical Conductivity @ 25°C	----	1	µS/cm	766	732	4.54	0% - 20%
EA055: Moisture Content (Dried @ 105-110°C) (QC Lot: 1847952)									
ES1822655-004	Anonymous	EA055: Moisture Content	----	0.1	%	10.8	10.6	1.88	0% - 50%
ES1822660-003	Anonymous	EA055: Moisture Content	----	0.1	%	8.6	8.7	1.40	0% - 20%
ED040S: Soluble Major Anions (QC Lot: 1849587)									
ES1822655-001	Anonymous	ED040S: Sulfate as SO4 2-	14808-79-8	10	mg/kg	70	70	0.00	No Limit
ES1822659-001	BH1 5.0-5.2m	ED040S: Sulfate as SO4 2-	14808-79-8	10	mg/kg	170	160	0.00	0% - 50%
ED045G: Chloride by Discrete Analyser (QC Lot: 1849590)									
ES1822655-001	Anonymous	ED045G: Chloride	16887-00-6	10	mg/kg	120	100	11.8	No Limit
ES1822659-001	BH1 5.0-5.2m	ED045G: Chloride	16887-00-6	10	mg/kg	860	820	4.18	0% - 20%

Method Blank (MB) and Laboratory Control Spike (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 Spike (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 (%)	Recovery Limits (%)	
Method: Compound	*CAS Number*	*LOR*	*Unit*	*Result*	*Concentration*	*LCS*	*Low*	*High*
EA010: Conductivity (1:5) (QCLot: 1849588)								
EA010: Electrical Conductivity @ 25°C	----	1	µS/cm	<1	1412 µS/cm	# 110	92	108
ED040S: Soluble Major Anions (QCLot: 1849587)								
ED040S: Sulfate as SO4 2-	14808-79-8	10	mg/kg	<10	150 mg/kg	101	80	120
ED045G: Chloride by Discrete Analyser (QCLot: 1849590)								
ED045G: Chloride	16887-00-6	10	mg/kg	<10	50 mg/kg	109	75	125
				<10	5000 mg/kg	104	79	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(%)	Recovery Limits (%)	
Laboratory sample ID	*Client sample ID*	*Method: Compound*	*CAS Number*	*Concentration*	*MS*	*Low*	*High*
ED045G: Chloride by Discrete Analyser (QCLot: 1849590)							
ES1822655-001	Anonymous	ED045G: Chloride	16887-00-6	6250 mg/kg	108	70	130