Douglas Partners
Geotechnics / Environment / Groundwater

Integrated Practical Solutions

Report on
Geotechnical Investigation

Proposed Development
46 Prince Alfred Parade, Newport

Prepared for
Adams Consulting Engineers Pty Ltd

Project 212173.00
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Douglas Partners
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The undersigned, on behalf of Douglas Partners Pty Ltd, confirm that this document and all attached drawings, logs and test results have been checked and reviewed for errors, omissions and inaccuracies.

	Signature	Date
Author		14 July 2023
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Douglas Partners acknowledges Australia's First Peoples as the Traditional Owners of the Land and Sea on which we operate. We pay our respects to Elders past and present and to all Aboriginal and Torres Strait Islander peoples across the many communities in which we live, visit and work. We recognise and respect their ongoing cultural and spiritual connection to Country.

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Proposed Development
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1. Introduction

This revised report presents the results of a geotechnical investigation undertaken for proposed alterations and additions at 46 Prince Alfred Parade, Newport. The addition assessment was undertaken in accordance with Douglas Partners' email dated

We understand that the development will include the construction of a two-storey addition to the west of the existing yacht club. It is understood the new addition will be built on reclaimed (filled) foreshore land. In addition, proposed works will also include the construction of a new internal lift access, new walkway, and associated retaining wall adjacent to the existing slope, west of the carpark area.

This report includes a review of available information, a site inspection by a Senior Geotechnical Engineer, the drilling of five boreholes, and laboratory testing of selected samples. The details of the field work are presented in this report, together with comments on design and construction practice.

This report is a revision of DP's previous geotechnical report (DP November 2022) which addresses the geotechnical issues associated with the proposed walkway and associated retaining wall.

2. Site Description

The site is located to the west of Prince Alfred Parade, Newport. The site is an irregular shaped area of approximately 1.5 hectares with maximum site plan dimensions of about 145 m by 140 m. The site is accessible by Prince Alfred Parade to the east. The general layout of the site is shown on Drawing 1 in Appendix B.

It is bounded by the following:

- To the north by residential development;
- To the south and west by Pittwater foreshore; and,
- To the east by residential development and Prince Alfred Parade

The existing club building has been constructed partly over the lower slopes and toe of the hillside and partly on the reclaimed land. The lower Marina carpark which forms the existing building and pool area, is generally flat, grading from about RL 2.5 near the building to about RL 1.8 m along the foreshore.

Surface levels fall approximately 21 m (RL 23 m to 2 m) to the south and west. There are several changes in slope due to a series of terraces which form the carpark areas and the existing Royal Motor Yacht Club building.

The reclaimed land is retained by a series of retaining walls constructed of brick and sandstone. The Walls are estimated to be about 2 m in height.

3. Regional Geology and Mapping

3.1 Geology

Reference to Sydney 1:100,000 Geology Sheet indicates the site is underlain by the Newport Formation and Garie Formation (Rnn) typically comprising Interbedded laminite, shale and quartz, to lithic-quartz sandstone. Reference to the NSW seamless Geology, indicates the site comprises Anthropocene deposits varying from large man-made clasts (concrete blocks to building demolition rubble) to quarried natural boulders, with interstitial sand-sized to clay matrix.

The slope materials are typically colluvial at the surface and residual at depth, consisting of fill material and sandy load topsoil, over sandy clays with rock fragments and floaters through the profile. The sandy clays and clays merge into the weathered zone of the under lying rocks at depths expected to be in the range of 1 m to 1.5 m. This was observed during the recent walkover assessment.

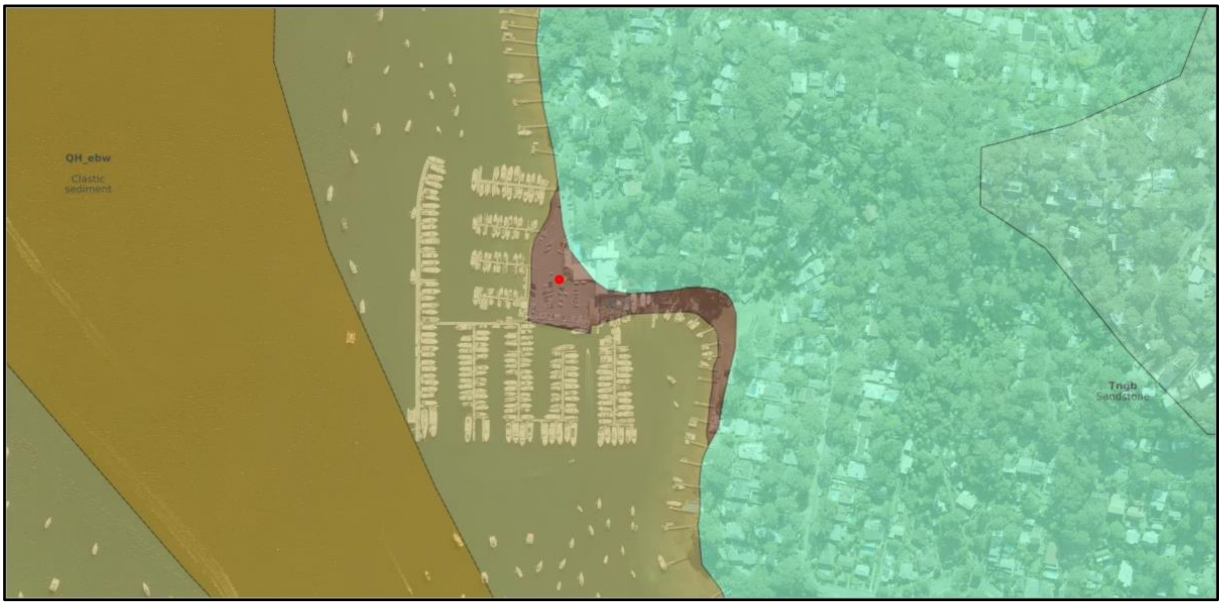


Figure 1: NSW Seamless Geology map with approximate site location

3.2 Acid Sulfate Soils

Reference to the 1:25 000 Acid Sulfate Soils (ASS) Risk map indicates the area no known occurrence of acid sulfate soils which is to be expected for this geology and topography. However, the sediments within Cahill Creek (Pittwater), adjacent to the site, are shown as having a high probability of ASS, therefore it is likely alluvial sediments beneath the fill may have a high probability of ASS. An extract from the published 1:25 000 Acid Sulfate Soil Risk Mapping, 1994-1998 (NSW Department of Environment and Climate Change) is presented in Figure 2.

Figure 2: Acid sulfate soil mapping with approximate site location: Acid sulfate soil mapping with approximate site location

3.3 Hydrogeology

Based on the site's proximity to Pittwater, it is expected that the groundwater table would be at approximately sea level or slightly higher due to normal phreatic rise on land adjacent to waterways. It should be noted however that groundwater levels are transient and that fluctuations may occur in response to climatic and seasonal conditions. It should also be noted that ephemeral seepage may occur along the soil/rock interface particularly following rainfall and may also occur along bedding planes and fractured zones in the rock.

4. Field Work

4.1 Field Work Methods

The investigation was carried out on 14 and 15 February 2022 in the presence of a Senior Geotechnical Engineer from DP.

Prior to drilling, on-site ground penetrating radar (GPR) and electronic scanning for buried services at the proposed borehole locations was carried out.

The investigation comprised five boreholes, drilled to depths in the range of 3.3 m to 6.94 m. BH1 to BH4 were drilled using a ute-mounted drill rig fitted with a 100 mm diameter solid flight auger. Augering was initially progressed within the fill, soil and weathered rock profile and terminated on bedrock at depths in the range of 3 m to 3.6 m. Boreholes were then continued into the underlying rock using NMLC diamond core drilling techniques to obtain continuous core samples of the rock, to termination depths in the range of 6.2 m to 6.94 m. The rock cores were transported to DP's workshop where the cores were photographed and tested for Point Load Strength Index ($Is_{(50)}$).

Due to access conditions, boreholes BH5 was drilled using a 100 mm diameter 'Diacore' barrel to penetrate the concrete (slab) near the surface. Following the initial coring, the underlying material was drilled was continued to 3.3 m depth using a 50 mm 'Diacore' barrel in an attempt to obtain continuous core samples of the laminite bedrock.

Standard Penetration Tests (SPTs) were carried out at regular depth intervals within the deeper boreholes (BH1 to BH4) to assess the in-situ strength of the soil material. Disturbed soil samples were recovered from the boreholes and SPT split tubes at regular depth intervals for logging purposes and subsequent laboratory testing.

All boreholes were backfilled with drilling spoil upon completion. BH1 to BH4 were capped with cold mix bitumen. BH5 was backfilled with drilling spoil and reinstated with the existing pavers. The locations of the boreholes are shown on Drawing 1 in Appendix B. The elevations and co-ordinates for test locations were interpreted from survey plans and topographic data from NSW Department of Lands April 2009.

5. Field Work Results

5.1 Walkover Inspection

- There was no apparent evidence of distress or cracking of the existing yacht club and associated structures, which could be attributed to significant previous slope or footing movements;
- There was no evidence of overland water flows entering the site from upslope. Surface water flows would be experienced to be diverted by the carpark drainage and guttering.
- A high cut batter was located between the existing swimming pool terrace and carpark area (north of the existing yacht club). The batter was heavily vegetated an average slope angle of about 30° - 35°. The slope has a scattered cover of mature and semi-mature trees, with a dense covering of vegetation (Refer to Photo 1).
- Some shallow timber retaining walls form the upper terrace area (Photo 2) showed some signs of bulging;
- Some weathered rock exposures were observed under the existing building, south of the vegetated slope within the proposed walkway area (Photo 5 and 6). The cut exposure showed light brown sandy clays, over yellow-orange colluvial sandy clays, over extremely weathered sandstone.

5.2 Subsurface Investigation

The subsurface conditions encountered in the boreholes are presented in the borehole logs in Appendix C, together with notes explaining classification methods and defining descriptive terms used.

The general subsurface profile encountered is summarised below:

FILL	-	Asphalt concrete over fine to medium angular gravel (possibly DGB20), within BH1 to BH4, to a depth of 0.15 m. BH5 encountered pavers then concrete to 0.46 m depth; underlain by,
	-	Typically, variably compacted sandy clay and clayey sand, to depths in the range of 0.77 m to 2.5 m. The fill contained possible sandstone cobbles and boulders in BH1, BH2 and BH4; overlying,
RESIDUAL SOILS	-	Sandy clay, generally stiff, with fine to medium grained sand to depths of 2.6 m; over,
BEDROK (LAMINITE)	-	Typically very low to medium strength shale/siltstone laminite with interbedded sandstone layers, extremely to highly weathered, fine to medium grained, fractured to highly fractured. The bores were terminated in sandstone at depths in the range of 3.3 m to 6.94 m.

Groundwater was observed within boreholes BH1, BH2 and BH4 at the completion of drilling, at depths of 1.4 m, 2 m and 2.2 m, respectively (RL 0.1-0.7 m). Groundwater levels are transient and will fluctuate with weather and tides and may be expected to rise by 1-2 m above the measured levels during periods of high tides and following heavy rainfall.

6. Laboratory Testing

6.1 Aggressivity

Laboratory testing was carried out on three soil samples to determine the soil aggressivity for exposure classification of buried concrete and steel elements. The results of the laboratory testing are presented in Table 1. The detailed laboratory test reports are given in Appendix D.

Table 1: Summary of Chemical Laboratory Test Results

Borehole ID	Depth (m)	Material Description	Conductivity (µS/cm)	pH (pH Units)	Cl (ppm)	SO_4 (ppm)
BH1	1-1.45	FILL/Sandy Clay	360	7.2	380	82
BH2	2.5-2.95	Sandy Clay	130	8.6	200	57
BH4	0.4-0.6	FILL/Clayey Sand	200	7.8	67	200

Notes: Cl = Chloride ion concentration, SO_4 = Sulphate ion concentration, ppm = parts per million

6.2 Acid Sulfate Soils

Three soil samples from the boreholes were screened for preliminary signs of actual sulfate soils (ASS) and potential acid sulphate soils (PASS). The screening involved measurement of the pH value of each soil sample after the addition of distilled water (pH_F). Hydrogen peroxide was then added to oxidise the sample and the pH value (pH_{Fox}) was measured again after at least 1 hour. The results for the pH screening are presented in Table 2.

The Acid Sulfate Soils Management Advisory Committee (ASSMAC) prepared an Acid Sulfate Soils Manual (August 1998) which includes guidelines for assessing AASS and PASS. The ASS screening and laboratory testing regime has been developed in general accordance with the ASSMAC Acid Sulfate Soil Manual.

Table 2: PASS and ASS Screening Test Results

Bore No.	Depth (m)	Material Description	Screening Tests			
			Natural pH_F	Oxidised pH_{FOX}	Change in pH	Reaction
BH1	1-1.45	FILL/Sandy Clay	7.5	4.0	3.5	High
BH3	1-1.45	FILL/Sandy Clay	7.5	5.0	2.0	Low
BH4	0.4-0.6	FILL/Clayey Sand	8.2	4.1	4.2	Medium

Note: Samples in **bold** selected for chromium reducible sulfur testing; 1.7 – red font exceeds action criteria

The screening test results were assessed for the possible presence of AASS or PASS on the basis of the following guidance indicators specified in the ASSMAC Guidelines:

- $pH_F \leq 4$ strongly indicates oxidation has occurred in the past and that AASS are likely to be present; and,
- $pH_{FOX} < 3.5$ plus preferably one or more of the following strongly indicates the presence of PASS
  - o a pH_{FOX} reading at least one pH unit below the corresponding pH_F,
  - o a strong reaction with peroxide, change in soil colour from grey tones to brown tones, or
  - o a release of sulphurous gases.

6.3 Point Load Strength Index

Selected samples of the rock core obtained from BH1 to BH5 were tested in the laboratory to determine the Point Load Strength Index (Is_{50}) values to assist with the rock strength classification. The results of the testing are shown on the borehole logs at the corresponding depth.

The Is_{50} values for the rock have been used to estimate the unconfined compressive strength (UCS) based on a UCS:Is_{50} ratio of 20:1. It is noted that point load tests are not readily carried out on extremely low to very low strength rock or highly fractured rock and hence strength classification of the weaker rock is based on visual/tactile assessments of the rock core.

The Is_{50} values of the rock cores from the investigation typically ranged from 0.15 MPa to 0.8 MPa, corresponding to a very low to medium strength classification (inferred UCS ranging from 3 MPa to 16 MPa).

7. Proposed Development

It is understood that the proposed development on the site involves the following:

- Alterations and additions to the existing building, including the provision for a lift well (located near BH5);
- Construction of a new two storey building around the existing inground pool. The new build will be linked to the existing building;
- Upgrade of the existing inground pool and existing terrace retaining wall;
- A new pathway will be constructed. Following demolition of the existing brushwood fence and timber retaining wall, where required. The pathway is understood to extend from the new building to the driveway to the north. Reference to prepared drawings indicates the location of the proposed walkway is indicative only.

It is expected that the building area will undergo minor regrading to account for any change in site levels. No structural loads were provided at the time of preparing this report.

8. Geotechnical Model

For design purposes, of the proposed extensions and lift well, the subsurface profile observed during the investigation has been grouped into three geotechnical units. The interpreted depth and reduced levels (RL) at the top of the various units at each test location is shown in Table 3. Reference should be made to the borehole logs for more detailed information and descriptions of the soil profile.

Geotechnical cross-sections (Sections A-A' to C-C') showing the interpreted subsurface profile, are shown on Drawings 2 to 4 in Appendix B. It should be noted that the interpreted boundaries shown on the section s are accurate at the borehole locations only.

Table 3: Summary of Geotechnical Model

Unit	Material	Depth m to Top of Each Unit (Reduced Level, m AHD)				
		BH1	BH2	BH3	BH4	BH5
1	Filling	0.0 (2.1)	0.0 (2.1)	0.0 (2.2)	0.0 (2.3)	0.0 (2.4)
2	Residual Soils	2.1 (0.0)	2.0 (0.1)	1.0 (1.2)	2.0 (0.3)	NE (N.E.)
3	Class V - Laminite	2.6	2.5	1.5	2.5	0.15

Unit	Material	Depth m to Top of Each Unit (Reduced Level, m AHD)				
		BH1	BH2	BH3	BH4	BH5
		(-0.5)	(-0.4)	(0.7)	(-0.2)	(3.25)

The interpreted geotechnical profile for the existing slope is blanketed by silty sand (slope wash), over colluvial sandy clays to depths of up to 1 – 2 m, underlain by extremely weathered rock. The depth of the stratigraphy and to rock could not be confirmed in the current assessment and will need to be confirmed by further investigation.

Groundwater was observed within boreholes BH1, BH2 and BH4 at the completion of drilling, at depths of 1.4 m, 2 m and 2.2 m, respectively (RL 0.1-0.7 m). Groundwater levels are transient and will fluctuate with weather and tides and may be expected to rise by 1-2 m above the measured levels during periods of high tides and following heavy rainfall. Designs should allow for potential rises in groundwater of at least 2 m higher than measured levels.

It is considered that from a geotechnical perspective the proposed excavation and development is readily achievable, provided the recommendations outlined within this report are implemented, the project is appropriately engineer designed and sound engineering/construction practices are adopted.

9. Stability Assessment

Visual inspection of the site and surrounding areas, visible retaining walls, external of the Royal Motor Yacht Club walls did not identify any features or defects that could be attributable to previous overall slope instability.

At this stage, it is not known if the proposed development will require significant excavation into the existing slope, supporting the carpark above. Accordingly the works are considered to have a low risk of causing slope instability on the slope above the existing batter. The existing vegetated batter has likely remained in place for 20 years and may likely remain so, if left in place without disturbance. Managing the risk will require regular and ongoing monitoring to check for signs of tension cracking and slope movement. Such monitoring may not be practical over the long term and this should be considered when assessing the need for remediation prior finalising the design of the proposed works.

9.1 Slope Risk Analysis

The potential hazards above, below and beside the site have been assessed for risk to property and life using the general methodology outlined by the Australian Geomechanics Society (Landslide Risk Management AGS Subcommittee 2007).

For the purposes of this assessment, an acceptable level of geotechnical risk for residential buildings is "Low" while an accepted annual probability of loss of life is 1×10^{-6}. For the proposed walkway and new retaining wall, a tolerable level for property is "Moderate".

Identified hazards within and adjacent to the site are summarised in Table 4, together with qualitative assessments of likelihood, consequence and slope instability risk to the proposed structures after completion of construction which has had appropriate engineering design and construction methodologies.

Table 4: Property Risk Assessment

Hazard	Likelihood	Consequence	Risk
Failure of the existing hillside slope	Unlikely – no evidence of previous instability. The existing slope is well vegetated, if remains untouched	Medium – could impact the terrace below and carpark, or building above	Low
Significant failure of existing slope following proposed works	Rare – if advice provided in this report is follow	Medium – could impact the terrace below and carpark, or building above	Low
Gradual soil creep on the existing slope impacting proposed works	Unlikely – provided the recommendations given in this report are followed.	Medium – could impact the terrace below and carpark, or building above	Low
Collapse of the slope before the permanent walkway/retaining wall is constructed	Unlikely – provided the recommendations given in this report are followed.	Minor - redesign and additional construction works	Low
Collapse of the permanent retaining wall that supports the existing slope above	Unlikely – provided the recommendations given in this report are followed.	Medium – would require repairs to the retaining wall	Low

For loss of life, the individual risk can be calculated from:

$R_{(LoL)} = P_{(H)} \times P_{(S:H)} \times P_{(T:S)} \times V_{(D:T)}$

where:

$R_{(LoL)}$ *is the risk (annual probability of loss of life (death) of an individual)*

$P_{(H)}$ *is the annual probability of the hazardous event occurring (e.g. failure of the residence footings)*

$P_{(S:H)}$ *is the probability of spatial impact by the hazard (e.g. of the failure reaching the residence, taking into account the distance of a given event from the residence)*

$P_{(T:S)}$ *is the temporal probability (e.g. of the residence being occupied by the individual) at the time of the spatial impact*

$V_{(D:T)}$ *is the vulnerability of the individual (probability of loss of life of the individual given the impact).*

The assessed individual risk to life (person most at risk) resulting from slope instability is summarised in Table 5.

Table 5: Life Risk Assessment

Hazard	$P_{(H)}$	$P_{(S:H)}$	$P_{(T:S)}$	$V_{(D:T)}$	Risk $R_{(LoL)}$
Failure of the existing hillside slope	1×10^{-4}	0.3	0.5	0.5	7.5×10^{-6}
Gradual soil creep on the existing slope impacting proposed works	1×10^{-4}	0.5	0.5	0.01	2.5×10^{-7}
Collapse of the slope before the permanent walkway/retaining wall is constructed	1×10^{-4}	0.2	0.1	0.1	2×10^{-7}
Collapse of the permanent retaining wall / walkway that supports the existing slope above	1×10^{-5}	0.1	0.5	0.01	5×10^{-9}

For the purposes of this assessment, an accepted annual probability of loss of life is 1×10^{-6}.

When compared to the Landslide Risk Management Guidelines of the AGS, it is considered that the site meets ‘Acceptable Risk Management’ criteria with respect to both property and life for new developments under current and foreseeable conditions.

Provided construction is undertaken in accordance with the recommendations contained in this report, construction of the proposed works is not expected to affect the overall stability of the site or negatively influence the geotechnical hazards identified above.

10. Comments

10.1 Site Preparation

Any existing fill that is required to support structures or pavements will need to be reworked to reduce the potential for unacceptable settlements associated with poorly or variably compacted fill. Any new fill should also be placed in accordance with the following specification. To achieve this, the following scope of works will be necessary:

- Excavate and remove any vegetation, oversize pieces (>200mm), organic rich topsoil, existing fill and other building materials;
- Test roll the exposed soil surface in the presence of the geotechnical consultant and improve the condition of the stripped surface where so directed. This may include the removal and replacement of any soft or wet areas to a maximum depth of 0.5 m, where present. A 10 tonne smooth drum roller is suggested for test rolling;

- After approval of test rolling, place approved fill in uniform thickness layers not exceeding 300 mm loose thickness and compact each layer to a minimum dry density ratio of 98% Standard, increasing to 100% Standard within the upper 0.3 m of the subgrade. Ensure each fill layer is appropriately tested by the geotechnical consultant to verify the required compaction criterion has been achieved;
- Testing should be undertaken in accordance with AS3798-2007 Guidelines on Earthworks for Commercial and Residential Developments; and
- Fill and subgrade materials should be placed and maintained at a moisture content between -3% to +1% of Standard OMC.

10.2 Excavation Conditions

Excavations are expected to be limited to removal of fill soils and may encountered the top of residual soils to allow construction of the building, services trenches or lift pits. Based on the conditions encountered in the bores, it is estimated that excavation of the existing filling may be undertaken by general medium to large sized excavation plant, such as 15-20 tonne (or larger) excavators. Though some ripping assistance may be required for removal of concrete structures and boulders within the fill. Generally, hydraulic rock breakers in conjunction with heavy ripping would be employed for effective removal of medium strength rock (if encountered).

Where excavation for piled foundations is required into any rock stronger than low strength a large capacity hydraulic rotary rig, such as a Soilmec track mounted machine with telescopic kelly-bar, rock augers and rock core barrels would be required.

Although no free-standing groundwater was encountered within the proposed lift well (near BH5) at the time of the investigation, it is anticipated that seepage will generally occur from along the soil/rock interface and from bedding planes within bedrock.

During construction and in the long term, it is anticipated that seepage into the excavation could be controlled by perimeter and subfloor drainage connected to a sump-and-pump system. Generally, water collected from dewatering operations should be suitable for disposal by pumping to stormwater drains subject to confirmation testing of groundwater quality and approval from the Council.

It is anticipated that excavation for the proposed pool may be below the water table. In the event that dewatering is required, it should be carried out in accordance with regulatory requirements. Generally the groundwater level should be lowered to at least 1 m below the pool excavation level to allow man access and machinery to operate, and to prevent flooding during heavy rainfall. To reduce the risk of lowering the groundwater table outside the site and potentially damaging adjacent structures, etc, a system of recharge wells close to the pits may be required to reinject pumped groundwater back into the ground to maintain the groundwater level outside the pool excavation.

Monitoring of the groundwater levels outside the perimeter walls of the pits will, nevertheless, be required to ensure that adjacent structures are not adversely affected. Ongoing monitoring of groundwater levels should be carried out to obtain more detailed information on fluctuations in groundwater levels if this is likely to impact on the design or construction. This monitoring should include periods of extended wet weather.

Previous experience in Sydney is that seepage will likely contain relatively high levels of soluble iron that will form a precipitate in the form of a gelatinous ‘sludge’ when exposed to oxygen. This ‘sludge’ has the potential to block-up subsoil (gravel) drains and ‘seize-up’ pumps. Therefore, detailing of subfloor drains, sumps and pumps should incorporate provision for regular maintenance such as flushing and ‘rodding’ of drains and/or “baffle” pits.

10.3 Disposal of Excavated Material

All excavated materials to be removed from site will need to be disposed of in accordance with the provisions of the current legislation and guidelines including the Waste Classification Guidelines (NSW EPA, 2014) and the recommendations provided within the preliminary waste classification report.

10.4 Vibrations

During excavation, it will be necessary to use appropriate methods and equipment to keep ground vibrations at adjacent buildings and structures within acceptable limits. Most of the excavation is expected to be within sands which should result in relatively minor vibrations. Breaking up concrete is likely to generate the most vibration. Further advice on vibrations can be provided once details on the proposed development and equipment to be used are known.

10.5 Dilapidation Surveys

Dilapidation surveys (of building condition) should be undertaken on surrounding properties prior to commencing work on the site to document any existing defects so that any claims for damage due to construction related activities can be accurately assessed. As a minimum this should include the existing building and adjacent buildings.

10.6 Acid Sulfate Soils

The results indicate that potential acid sulphate soils (PASS) may be present in the natural soils below the fill although actual acid sulphate soils (AASS) do not appear to be present. The extent of PASS has not been clearly defined by the current investigation and may warrant further investigation.

If the natural soils below the water table will be excavated by the proposed works, it should be assumed that the soils potentially include PASS and they will need to be managed on site prior to disposal or reused in an appropriate manner. Management generally involves mixing the soils with lime in a controlled manner. If the proposed works are expected to disturb the natural soils below the fill and below the water table then an Acid Sulfate Soil Management Plan will be required to document the procedures and processes that should be adopted to handle PASS.

10.7 Aggressivity

The results of the aggressivity testing compared to the values presented in Australian Standards (AS) 2159-2009 "Piling – Design and installation" indicate that the soil conditions are 'mild' for buried concrete and 'non-aggressive' for buried steel. Recommendations are presented in AS 2159-2009 for appropriate minimum concrete strengths and minimum cover to reinforcing steel for various exposure classifications. Designers also need to consider the potential for PASS in the natural soils below the water table.

10.8 Seismicity

A Hazard Factor (*Z*) of 0.08 would be appropriate for the development site in accordance with Australian Standard AS 1170.4 – 2007 *Structural design actions – Part 4: Earthquake actions in Australia*. The site sub-soil class would be Class C_e.

The Earthquake Design Category could then be assessed based on a Probability Factor, k_p, (which is related to an Annual Probability of being Exceeded) as defined in Table 3.1 of AS 1170.4 – 2007.

10.9 Batter Slopes

Suggested temporary and permanent batter slopes for unsupported excavations up to a maximum height of 3 m, above the water table, are shown in Table 6. Batter slopes higher than 3 m will require analysis of each slope. If surcharge loads are applied near the crest of the slope then analysis will be required and probably flatter batters or support may be required.

Table 6: Batter Slopes

Unit	Material	Excavation Height (m)	Maximum Temporary Batter Slope (H : V)	Maximum Permanent Batter Slope (H : V)
1	Filling	<3 m	1.5 : 1	2.5 : 1
2	Residual	<3 m	1 : 1	2 : 1
3	Class V - Laminite	<3 m	0.75 : 1	1 :1

Deeper excavations and those that encounter groundwater may need to be battered considerably flatter, possibly less than 3H:1V and/or require dewatering for stability or require support measures such as sheet piling or shoring boxes.

For the existing batter, carful consideration should be given to the long term stability of the existing batter. Consideration should be given to permanent stabilisation of the existing embankment. Support measures could include construction of a retaining wall, or anchors and shotcrete, where the above batter slopes cannot be achieved. Sub-surface geotechnical conditions will need to be confirmed within the batter by a geotechnical investigation, prior to the final design. The factor of safety for the current and remediated slope (if required) should also be determined using a program such as SLOPE/W.

10.10 Excavation Support

Vertical excavation in soil and rock will require both temporary and permanent lateral support during and after excavation. Anchored soldier pile walls with shotcrete infill panels are often used to provide temporary retention support in the expected ground conditions, with the structure being designed to support the wall in the long term. Soldier piles are usually spaced at between 2 m to 2.5 m centres; however, closer spaced piles may be required to reduce wall movements, or prevent collapse of infill materials, particularly where pavements, structures or services are in close proximity to the excavation.

Careful attention will need to be given to the design of excavation support, particularly in the vicinity of the adjacent existing buildings. At these locations, the proposed retention system will need to provide adequate support to the existing footings to reduce lateral movement to tolerable levels. This could be achieved by closing the pile spacing and/or increasing anchor capacity.

Cantilevered pile walls should not be used where adjacent to existing structures within a distance equal to the height of the excavation from the shoring wall. Cantilevered pile walls have a greater propensity for outward rotation and the consequently higher risk of disturbing adjacent footings.

Design pressures for retaining walls should take into account the requirement to limit movement of the surrounding ground and adjacent structures and to ensure an adequate factor of safety is maintained against failure (for temporary and permanent retaining walls).

It is suggested that preliminary design of cantilevered shoring systems (or shoring with one row of anchors/props) be based on a triangular earth pressure distribution using the earth pressure coefficients provided in Table 7. 'Active' earth pressure coefficient (K_a) values may be used where some wall movement is acceptable, and 'at rest' earth pressure (K_o) values should be used where the wall movement needs to be reduced (i.e., adjacent to existing structures or utilities).

Table 7: Recommended Design Parameters for Shoring Systems

Unit	Material	Unit Weight (kN/m^3)	Earth Pressure Coefficient		Effective Cohesion c' (kPa)	Effective Friction Angle (Degrees)
			Active (K_a)	At Rest (K_o)		
1	Filling	20	0.4	0.6	2	25
2	Residual Soil	20	0.3	0.45	5	25
3	Class V Laminite	24	0.3	0.45	10	28

Passive resistance for piles founded in rock below bulk excavation (including allowance for services and/or footings) may be based on the ultimate passive restraint values provided in Table 8. This ultimate value represents the pressure mobilised at high displacements and therefore it will be necessary to incorporate a factor of safety of at least 3 to limit wall movement. The top 0.5 m of the socket should be taken into account due to possible disturbance and over-excavation. The minimum socket depth should be equal to the greater of one pile diameter or 1.0 m below the lowest level of any nearby excavation (including any detailed excavations), but subject to analysis.

Table 8: Recommended Passive Resistance Values

Unit	Foundation Stratum	Maximum Ultimate Passive Pressure (kPa)
3	Class V Laminite	400*

Note: * provided no adversely oriented discontinuities are present and subject to geotechnical inspection

10.11 Pile Foundations

It is recommended that all footings for the structures be founded on rock with similar strength in order to provide uniform support for the proposed structures and to reduce the potential for differential settlement.

A range of pile types can be considered. Continuous flight auger (CFA), concrete injected piles could be considered for this site. This type of pile is associated with relatively low levels of noise and vibration. Only experienced and reputable piling contractors should install the CFA piles with equipment capable of drilling through the boulder filling. Open bored piles may not be appropriate due to the potential for soil collapse and groundwater inflow Bored piles would likely need casing and/or drilling mud. The use of steel screw piles may not be appropriate due to the obstructions in the fill. Any piling method which returns soil to the ground surface will probably require treatment of the spoil for PASS.

Maximum allowable bearing pressures for preliminary design of foundations supported within the bedrock likely to be encountered at the site are provided in Table 9.

Table 9: Recommended Design Parameters for Foundation Design (After Pells et al[1])

Unit	Founding Stratum	Allowable Pressure (Serviceability)		Ultimate Pressure (Ultimate)		Field Young's Modulus, E (MPa)
		End Bearing (kPa)	Shaft Adhesion (Compression) (kPa)	End Bearing (kPa)	Shaft Adhesion (Compression) (kPa)	
2	Residual Soil Stiff to Very Stiff Clay or Stronger	150	-	450	-	25
3	Class V Laminite	700	70	3,000	100	70

Foundations proportioned on the basis of the allowable bearing pressure in Table 9 would be expected to experience total settlements of less than 1% of the footing width under the applied working load, with differential settlements between adjacent columns expected to be less than half of this value.

[1] Design Values for Foundations on Sandstone and Shale in the Sydney Region, Pells Mostyn and Walker. Australian Geomechanics December 1998.

Footings designed using ultimate values and Limit State Design will need to consider serviceability which usually governs the design in this case. For pile design, a basic geotechnical strength reduction factor, Φ_{gb}, of 0.4 is required if pile load testing is not carried out.

For uplift or tension loading, 70% of the above shaft adhesion parameters may be adopted for design of foundations or large anchors. In addition to traditional 'piston pull-out' or sidewall slip failure mechanisms, the uplift capacity should be checked for 'cone pull-out' failure modes. This should be based on an assumed cone angle of 90° from the mid-height of the bond length considering the submerged weight of the rock (where below groundwater table) and adopting a factor of safety of 1 against 'cone pull-out'.

All footings and bored piles should be inspected by an experienced geotechnical professional during construction to check the adequacy of the foundation material and, in the case of piles, to check the socket cleanliness and roughness. Seepage should be removed from excavations prior to pouring concrete.

10.12 Preliminary Site Classification

It is recommended that the site be designated 'Class P', which precludes the use of standard footing designs as presented in AS 2870-2011 and will therefore require design by engineering principles. The 'Class P' designation is given for the presence of 'uncontrolled' filling material across the site and the potential for differential settlement to occur (and consequential cracking) where standard footing arrangements are founded on materials of differing compressibility, such as natural soil, controlled filling and rock.

The laboratory testing from Coffey (2019) report indicates that the natural clays at the site are of medium to high plasticity and therefore likely to be susceptible to shrink-swell movements in response to seasonal variations in soil moisture content. Based on the soil depth, and the results of laboratory testing, it is considered that the natural soil profile would generally be consistent with a Class "M" site as per AS 2870. AS2870 indicates that characteristic surface movements (ys) of up 40 mm are expected for a Class "M" site.

10.13 Stormwater Disposal and Site Drainage

The current method of stormwater disposal does not appear to have resulted in any geotechnical issues on the site. Notwithstanding this, the proposed development may represent an increase in the stormwater generated. Accordingly, the builder should, as a minimum, expose and assess the functionality of the existing pits and pipe work.

Drainage measures will also be required immediately upslope of the proposed development. These could comprise a concrete lined dish above the crest discharging to the sides of the excavation or through pipes down the face. All drainage from the excavation face and down from above the crest should be connected to the site's stormwater disposal system, subject to appropriate treatment (if required).

Modification or replacement of the existing stormwater system may be required if the existing system is assessed as not adequate to carry the stormwater volumes from the new development.

10.14 Seismicity

A Hazard Factor (*Z*) of 0.08 would be appropriate for the development site in accordance with Australian Standard AS 1170.4 – 2007 *Structural design actions – Part 4: Earthquake actions in Australia*. The site sub-soil class would be Class C_e.

11. Design Life and Requirement for Future Geotechnical Assessments

DP interprets the reference to design life requirements specified within the GRMP to refer to structural elements designed to retain the subject slope and maintain the risk of instability within acceptable limits.

Specific structures that may affect the maintenance of site stability in relation to the proposed development on this site are considered to comprise:

- existing (and any proposed) stormwater surface drains and buried pipes leading to the stormwater disposal system; and
- existing (and proposed) retaining walls on the site.

In order to attain a structure life of 100 years as required by the Council Policy, it will be necessary for the structural engineer to incorporate appropriate construction detailing and for the property owner to adopt and implement a maintenance and inspection program. A typical program for developments on sloping sites is given in Table 10.

Table 10: Recommended Maintenance and Inspection Program

Structure	Maintenance/Inspection Task	Frequency
Drainage Lines	Inspect to ensure the line is flowing and not blocked.	Every year or following each significant rainfall event.
Drainage Pits	Inspect to ensure that pits are free of debris and sediment build-up. Clear surface grates of vegetation/litter build-up.	During normal ground maintenance and following each significant rainfall event.
Retaining Walls	Inspect walls for deviation from the as-constructed condition.	Every year or following each significant rainfall event.
General slopes	Inspect for possible erosion or tension cracks.	Every year or following each significant rainfall event.

Where changes to site conditions are identified during the maintenance and inspection program, reference should be made to a relevant professional (e.g. structural engineer or geotechnical engineer).

The site should be maintained in accordance with the *Australian Geoguide's LR7 (Landslide Risk) and LR8 (Construction Practice)*, copies of which is given in Appendix F. Whilst it must be accepted that minor cracking in most structures is inevitable, the guide describes suggested site maintenance practices aimed at minimising foundation movement to keep cracking within acceptable limits.

12. Further Assessment

The above investigation is to provide information for the purpose of preliminary design only. To better refine structural properties and other design elements, it is recommended that a 'detailed investigation' within the existing batter is carried out well in advance of construction.

13. References

AS 1170.4. (2007). *Structural Design Actions, Part 4: Earthquake Actions in Australia.* Reconfirmed 2018. Incorporating Amendments 1 & 2: Standards Australia.

AS 2159. (2009). *Piling - Design and Installation.* Standards Australia.

Bertuzzi, R., & Pells, J. (2002). Geotechnical Parameters of Sydney Sandstone and Shale. *Australian Geomechanics*, Vol 37 No 5 41-54.

NSW EPA. (2014). *Waste Classification Guidelines, Part 1: Classifying Waste.* NSW Environment Protection Authority.

Wilson, G. M. (1983). *Sydney 1:100,000 Geology Sheet.* NSW, Australia: NSW Department of Mines.

14. Limitations

Douglas Partners (DP) has prepared this report for this project at 46 Prince Alfred Parade, Newport in accordance with DP's proposal dated 12 January 2022 and acceptance received from Mark Wu of Adams Consulting Engineers Pty Ltd. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of Adams Consulting Engineers Pty Ltd for this project only and for the purposes as described in the report. It should not be used by or relied upon for other projects or purposes on the same or other site or by a third party. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of DP, does so entirely at its own risk and without recourse to DP for any loss or damage. In preparing this report DP has necessarily relied upon information provided by the client and/or their agents.

The results provided in the report are indicative of the sub-surface conditions on the site only at the specific sampling and/or testing locations, and then only to the depths investigated and at the time the work was carried out. Sub-surface conditions can change abruptly due to variable geological processes and also as a result of human influences. Such changes may occur after DP's field testing has been completed.

DP's advice is based upon the conditions encountered during this investigation. The accuracy of the advice provided by DP in this report may be affected by undetected variations in ground conditions across the site between and beyond the sampling and/or testing locations. The advice may also be limited by budget constraints imposed by others or by site accessibility.

The assessment of atypical safety hazards arising from this advice is restricted to the geotechnical components set out in this report and based on known project conditions and stated design advice and assumptions. While some recommendations for safe controls may be provided, detailed 'safety in design' assessment is outside the current scope of this report and requires additional project data and assessment.

This report must be read in conjunction with all of the attached and should be kept in its entirety without separation of individual pages or sections. DP cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by DP. This is because this report has been written as advice and opinion rather than instructions for construction.

The scope of work for this investigation/report did not include the assessment of surface or sub-surface materials or groundwater for contaminants, within or adjacent to the site. Should evidence of fill of unknown origin be noted in the report, and in particular the presence of building demolition materials, it should be recognised that there may be some risk that such fill may contain contaminants and hazardous building materials.

Douglas Partners Pty Ltd

Appendix A

About This Report

Introduction

These notes have been provided to amplify DP's report in regard to classification methods, field procedures and the comments section. Not all are necessarily relevant to all reports.

DP's reports are based on information gained from limited subsurface excavations and sampling, supplemented by knowledge of local geology and experience. For this reason, they must be regarded as interpretive rather than factual documents, limited to some extent by the scope of information on which they rely.

Copyright

This report is the property of Douglas Partners Pty Ltd. The report may only be used for the purpose for which it was commissioned and in accordance with the Conditions of Engagement for the commission supplied at the time of proposal. Unauthorised use of this report in any form whatsoever is prohibited.

Borehole and Test Pit Logs

The borehole and test pit logs presented in this report are an engineering and/or geological interpretation of the subsurface conditions, and their reliability will depend to some extent on frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or core drilling will provide the most reliable assessment, but this is not always practicable or possible to justify on economic grounds. In any case the boreholes and test pits represent only a very small sample of the total subsurface profile.

Interpretation of the information and its application to design and construction should therefore take into account the spacing of boreholes or pits, the frequency of sampling, and the possibility of other than 'straight line' variations between the test locations.

Groundwater

Where groundwater levels are measured in boreholes there are several potential problems, namely:

- In low permeability soils groundwater may enter the hole very slowly or perhaps not at all during the time the hole is left open;
- A localised, perched water table may lead to an erroneous indication of the true water table;
- Water table levels will vary from time to time with seasons or recent weather changes. They may not be the same at the time of construction as are indicated in the report; and
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must first be washed out of the hole if water measurements are to be made.

More reliable measurements can be made by installing standpipes which are read at intervals over several days, or perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from a perched water table.

Reports

The report has been prepared by qualified personnel, is based on the information obtained from field and laboratory testing, and has been undertaken to current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal, the information and interpretation may not be relevant if the design proposal is changed. If this happens, DP will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical and environmental aspects, and recommendations or suggestions for design and construction. However, DP cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions. The potential for this will depend partly on borehole or pit spacing and sampling frequency;
- Changes in policy or interpretations of policy by statutory authorities; or
- The actions of contractors responding to commercial pressures.

If these occur, DP will be pleased to assist with investigations or advice to resolve the matter.

About this Report

Site Anomalies

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, DP requests that it be immediately notified. Most problems are much more readily resolved when conditions are exposed rather than at some later stage, well after the event.

Information for Contractual Purposes

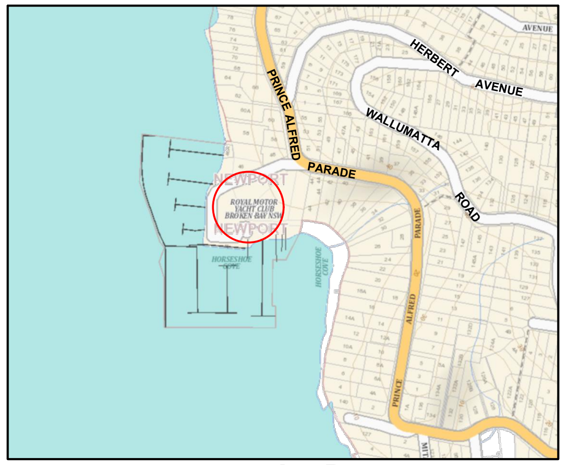
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. DP would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Site Inspection

The company will always be pleased to provide engineering inspection services for geotechnical and environmental aspects of work to which this report is related. This could range from a site visit to confirm that conditions exposed are as expected, to full time engineering presence on site.

Appendix B

Drawings



Locality Plan

NOTE:
1: Base image from MetroMap (Dated 16.08.2021)
2: Existing and Proposed Outlines from MCHP Architects, Drawing No. 21-079 DA006 & 009, Revision B & F (Dated 28.03.2023)

0 5 10 15 20 30 40 60 80m

1:800 @ A3

LEGEND

- Approximate Site Boundary
- Existing Buildings Outline
- Proposed Buildings Outline
- Proposed New Wall
- Borehole Locations
- Geological Cross Section

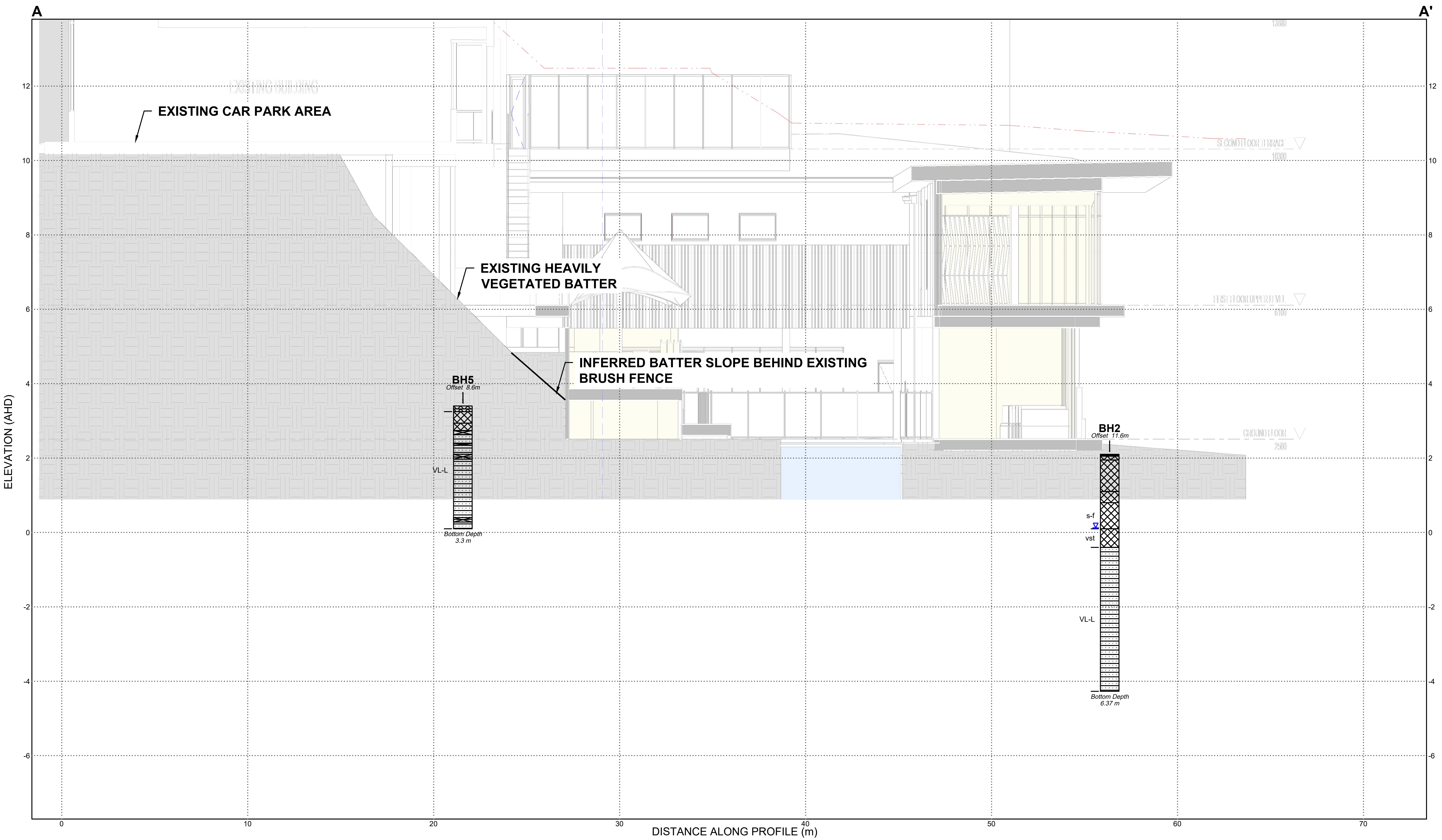
Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Adams Consulting Engineers Pty Ltd	
OFFICE: Sydney	DRAWN BY: MG/MN
SCALE: 1:800 @ A3	DATE: 14.07.2023

TITLE: **Test Location Plan**
Proposed Development
46 Prince Alfred Parade, Newport

N

PROJECT No:	212173.00
DRAWING No:	1
REVISION:	1



LEGEND

- Core Loss
- Asphaltic Concrete
- Filling
- Laminite

NOTES:

1. Subsurface conditions are accurate at the borehole locations only.
   Variations in subsurface conditions may occur between borehole locations.
   Interpreted strata boundaries are approximate and should be used as a guide only.
2. Summary logs only and should be read in conjunction with detailed logs.
3. Horizontal and vertical scales are not equal.
4. Base Section from MCHP Architects, Drawing No. 21-079 DA023, Revision D (Dated 22.03.2023)

ROCK STRENGTH

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

SOIL CONSISTENCY

- vs - Very Soft
- s - Soft
- f - Firm
- st - Stiff
- vst - Very Stiff
- h - Hard

TESTS / OTHER

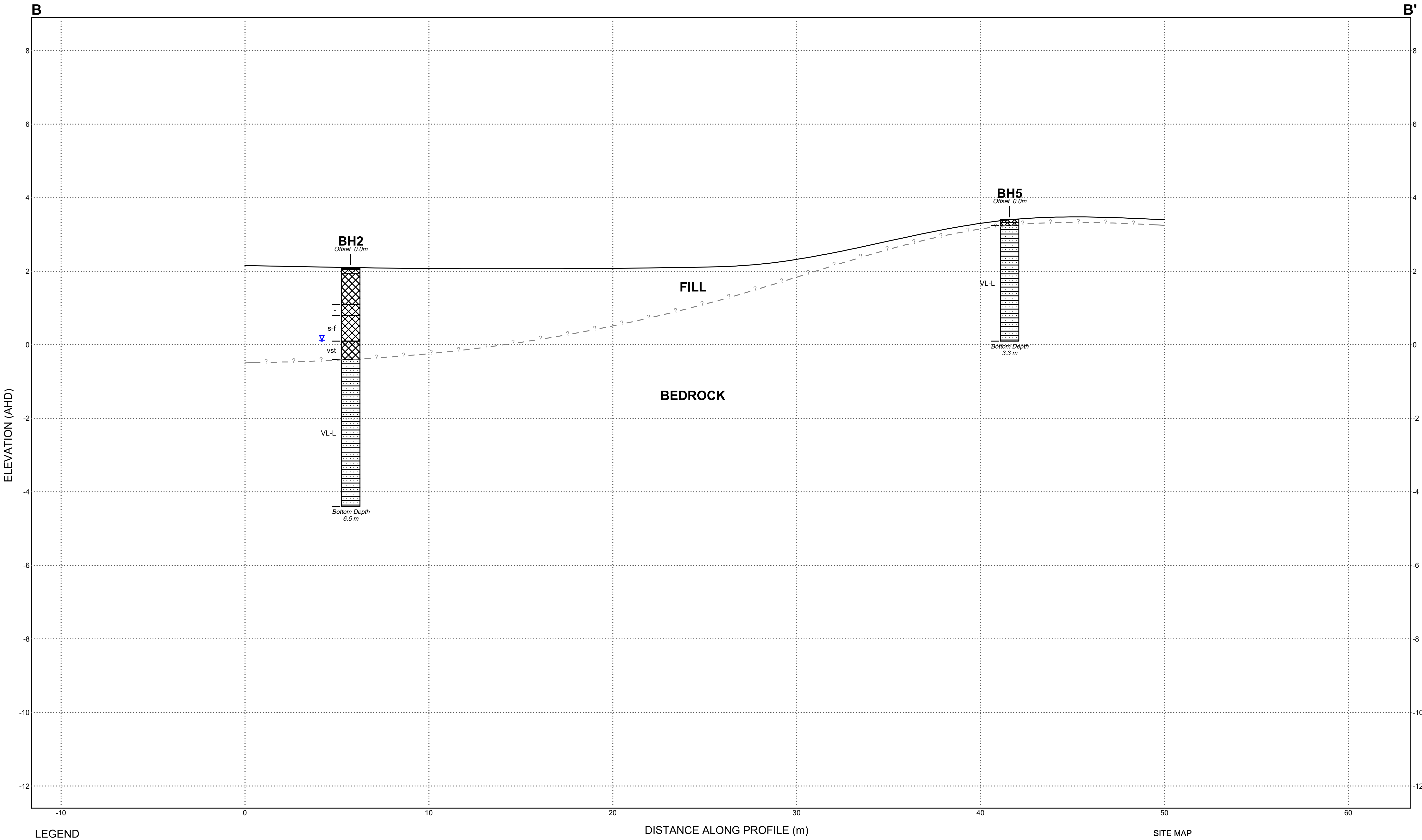
- — ? — — - Interpreted geotechnical boundary
- Water level

0 4

Horizontal Scale (metres)

Vertical Exaggeration = 2.0

Douglas Partners Geotechnics / Environment / Groundwater	CLIENT: Adams Consulting Engineers Pty Ltd	TITLE: **Interpreted Geotechnical Cross-Section A-A'**	PROJECT No: 212173.00
	OFFICE: Sydney / DRAWN BY: MN	**Proposed Addition**	DRAWING No: 2
	SCALE: 1:200 (H) 1:100 (V) @ A3 / DATE: 12.07.2023	**46 Prince Alfred Parade, Newport**	REVISION: 1



LEGEND

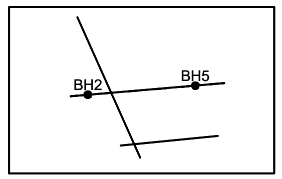
- Asphaltic Concrete
- Filling
- Laminite

NOTES:

1. Subsurface conditions are accurate at the borehole locations only. Variations in subsurface conditions may occur between borehole locations. Interpreted strata boundaries are approximate and should be used as a guide only.
2. Summary logs only and should be read in conjunction with detailed logs.
3. Horizontal and vertical scales are not equal.

ROCK STRENGTH		SOIL CONSISTENCY		TESTS / OTHER	
EL -	Extremely Low	vs -	Very Soft	N	- Standard penetration test value
VL -	Very Low	s -	Soft	— ? — —	- Interpreted geotechnical boundary
L -	Low	f -	Firm	⊻	- Water level
M -	Medium	st -	Stiff		
H -	High	vst -	Very Stiff		
VH-	Very High	h -	Hard		

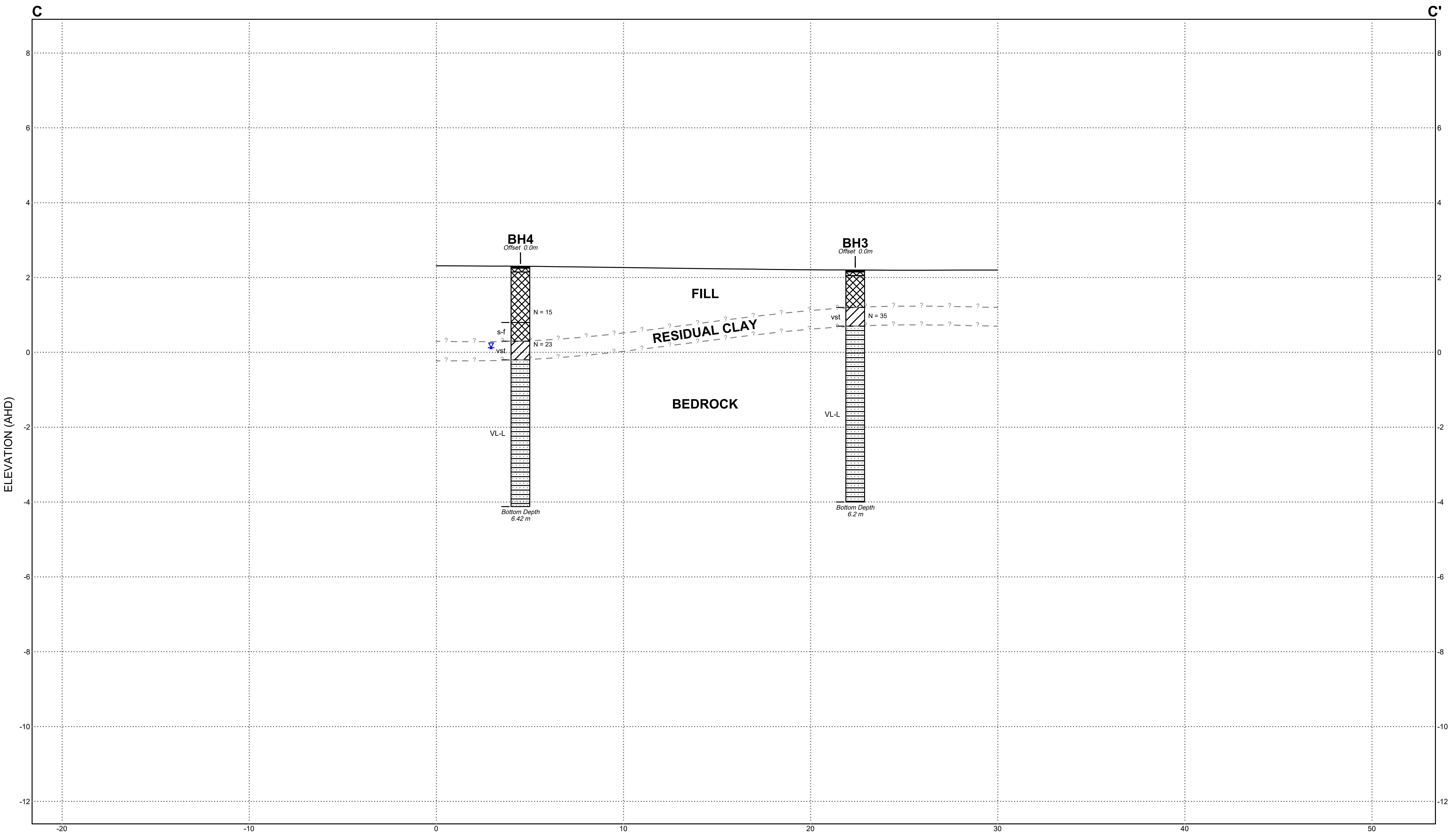
SITE MAP



0 4

Horizontal Scale (metres)
Vertical Exaggeration = 2.0

Douglas Partners Geotechnics / Environment / Groundwater	CLIENT: Royal Motor Yacht Club		TITLE: **Interpreted Geotechnical Cross-Section B-B'**	PROJECT No: 212173.00
	OFFICE: Sydney	DRAWN BY: MG	**Proposed Addition**	DRAWING No: 3
	SCALE: 1:200 (H) 1:100 (V) @ A3	DATE: 18.02.2022	**46 Prince Alfred Parade, Newport**	REVISION: 0



LEGEND

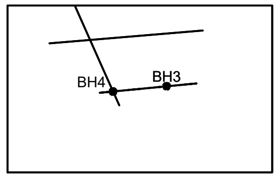
- Asphaltic Concrete
- Filling
- Sandy Clay
- Laminite

NOTES:

1. Subsurface conditions are accurate at the borehole locations only. Variations in subsurface conditions may occur between borehole locations. Interpreted strata boundaries are approximate and should be used as a guide only.
2. Summary logs only and should be read in conjunction with detailed logs.
3. Horizontal and vertical scales are not equal.

ROCK STRENGTH		SOIL CONSISTENCY		TESTS / OTHER	
EL -	Extremely Low	vs -	Very Soft	N	- Standard penetration test value
VL -	Very Low	s -	Soft	— ? — —	- Interpreted geotechnical boundary
L -	Low	f -	Firm	⊻	- Water level
M -	Medium	st -	Stiff		
H -	High	vst -	Very Stiff		
VH-	Very High	h -	Hard		

SITE MAP



0 4

Horizontal Scale (metres)
Vertical Exaggeration = 2.0

Douglas Partners
Geotechnics / Environment / Groundwater

CLIENT: Royal Motor Yacht Club		TITLE: **Interpreted Geotechnical Cross-Section C-C'**	PROJECT No: 212173.00
OFFICE: Sydney	DRAWN BY: MG	**Proposed Addition**	DRAWING No: 4
SCALE: 1:200 (H) 1:100 (V) @ A3	DATE: 18.02.2022	**46 Prince Alfred Parade, Newport**	REVISION: 0

Appendix C

Results of the Investigation

Sampling

Sampling is carried out during drilling or test pitting to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing it to obtain a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Test Pits

Test pits are usually excavated with a backhoe or an excavator, allowing close examination of the in-situ soil if it is safe to enter into the pit. The depth of excavation is limited to about 3 m for a backhoe and up to 6 m for a large excavator. A potential disadvantage of this investigation method is the larger area of disturbance to the site.

Large Diameter Augers

Boreholes can be drilled using a rotating plate or short spiral auger, generally 300 mm or larger in diameter commonly mounted on a standard piling rig. The cuttings are returned to the surface at intervals (generally not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube samples.

Continuous Spiral Flight Augers

The borehole is advanced using 90-115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are disturbed and may be mixed with soils from the sides of the hole. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively low reliability, due to the remoulding, possible mixing or softening of samples by groundwater.

Non-core Rotary Drilling

The borehole is advanced using a rotary bit, with water or drilling mud being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from the rate of penetration. Where drilling mud is used this can mask the cuttings and reliable identification is only possible from separate sampling such as SPTs.

Continuous Core Drilling

A continuous core sample can be obtained using a diamond tipped core barrel, usually with a 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in weak rocks and granular soils), this technique provides a very reliable method of investigation.

Standard Penetration Tests

Standard penetration tests (SPT) are used as a means of estimating the density or strength of soils and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, Methods of Testing Soils for Engineering Purposes - Test 6.3.1.

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150 mm of, say, 4, 6 and 7 as:

  4,6,7
  N=13

- In the case where the test is discontinued before the full penetration depth, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm as:

  15, 30/40 mm

The results of the SPT tests can be related empirically to the engineering properties of the soils.

Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests

Dynamic penetrometer tests (DCP or PSP) are carried out by driving a steel rod into the ground using a standard weight of hammer falling a specified distance. As the rod penetrates the soil the number of blows required to penetrate each successive 150 mm depth are recorded. Normally there is a depth limitation of 1.2 m, but this may be extended in certain conditions by the use of extension rods. Two types of penetrometer are commonly used.

- Perth sand penetrometer - a 16 mm diameter flat ended rod is driven using a 9 kg hammer dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands and is mainly used in granular soils and filling.
- Cone penetrometer - a 16 mm diameter rod with a 20 mm diameter cone end is driven using a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). This test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various road authorities.

Symbols & Abbreviations

Douglas Partners

Introduction

These notes summarise abbreviations commonly used on borehole logs and test pit reports.

Drilling or Excavation Methods

C	Core drilling
R	Rotary drilling
SFA	Spiral flight augers
NMLC	Diamond core - 52 mm dia
NQ	Diamond core - 47 mm dia
HQ	Diamond core - 63 mm dia
PQ	Diamond core - 81 mm dia

Water

▷	Water seep
$\underline{\nabla}$	Water level

Sampling and Testing

A	Auger sample
B	Bulk sample
D	Disturbed sample
E	Environmental sample
U_{50}	Undisturbed tube sample (50mm)
W	Water sample
pp	Pocket penetrometer (kPa)
PID	Photo ionisation detector
PL	Point load strength Is(50) MPa
S	Standard Penetration Test
V	Shear vane (kPa)

Description of Defects in Rock

The abbreviated descriptions of the defects should be in the following order: Depth, Type, Orientation, Coating, Shape, Roughness and Other. Drilling and handling breaks are not usually included on the logs.

Defect Type

B	Bedding plane
Cs	Clay seam
Cv	Cleavage
Cz	Crushed zone
Ds	Decomposed seam
F	Fault
J	Joint
Lam	Lamination
Pt	Parting
Sz	Sheared Zone
V	Vein

Orientation

The inclination of defects is always measured from the perpendicular to the core axis.

h	horizontal
v	vertical
sh	sub-horizontal
sv	sub-vertical

Coating or Infilling Term

cln	clean
co	coating
he	healed
inf	infilled
stn	stained
ti	tight
vn	veneer

Coating Descriptor

ca	calcite
cbs	carbonaceous
cly	clay
fe	iron oxide
mn	manganese
slt	silty

Shape

cu	curved
ir	irregular
pl	planar
st	stepped
un	undulating

Roughness

po	polished
ro	rough
sl	slickensided
sm	smooth
vr	very rough

Other

fg	fragmented
bnd	band
qtz	quartz

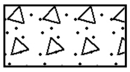
Symbols & Abbreviations

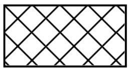
Graphic Symbols for Soil and Rock

General

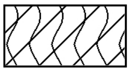
 Asphalt

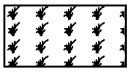
 Road base

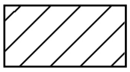
 Concrete

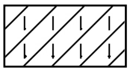
 Filling

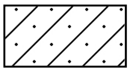
Soils

 Topsoil

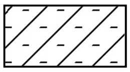
 Peat

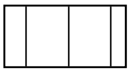
 Clay

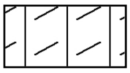
 Silty clay

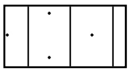
 Sandy clay

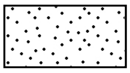
 Gravelly clay

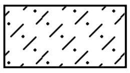
 Shaly clay

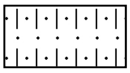
 Silt

 Clayey silt

 Sandy silt

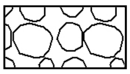
 Sand

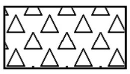
 Clayey sand

 Silty sand

 Gravel

 Sandy gravel

 Cobbles, boulders

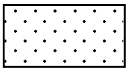
 Talus

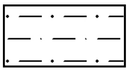
Sedimentary Rocks

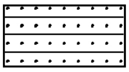
 Boulder conglomerate

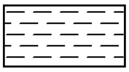
 Conglomerate

 Conglomeratic sandstone

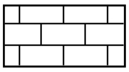
 Sandstone

 Siltstone

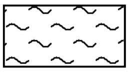
 Laminite

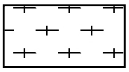
 Mudstone, claystone, shale

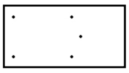
 Coal

 Limestone

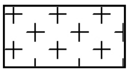
Metamorphic Rocks

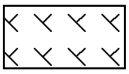
 Slate, phyllite, schist

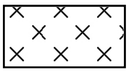
 Gneiss

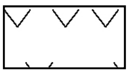
 Quartzite

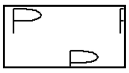
Igneous Rocks

 Granite

 Dolerite, basalt, andesite

 Dacite, epidote

 Tuff, breccia

 Porphyry

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are generally based on Australian Standard AS1726:2017, Geotechnical Site Investigations. In general, the descriptions include strength or density, colour, structure, soil or rock type and inclusions.

The soil group symbol classifications are given as follows based on two major soil divisions:

- Coarse-grained soils
- Fine-grained soils

Major Divisions				Description	
				Group Symbol*	Typical Name
COARSE-GRAINED SOILS	More than 65% by dry mass, (excluding that larger than 63 mm) is greater than 0.075 mm	GRAVEL	More than 50% of coarse grains are greater than 2.36 mm	GW	Well graded gravels and gravel-sand mixtures, little or no fines.
				GP	Poorly graded gravels and gravel-sand mixtures, little or no fines.
		GRAVELLY SOILS		GM	Silty gravels, gravel-sand-silt mixtures.
				GC	Clay gravels, gravel-sand-clay mixtures.
		SAND	More than 50% of coarse grains are less than 2.36 mm	SW	Well graded sands and gravelly sands, little or no fines.
				SP	Poorly graded sands and gravelly sands, little or no fines.
		SANDY SOILS		SM	Silty sand, sand-silt mixtures.
				SC	Clayey sands, sand-clay mixtures.

* For coarse grained soils where the fines content is between 5% and 12%, the soil shall be given a dual classification eg GP-GM.

FINE-GRAINED SOILS	More than 35% by dry mass, (excluding that larger than 63 mm) is less than 0.075 mm	Liquid Limit less than 35%	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity
		35% <LL< 50%	CI	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
		Liquid Limit greater than 50%	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity.
			Pt	Peat muck and other highly organic soils.

Soil Types

Soil types are described according to the predominant particle size, qualified by the grading of other particles present:

Type	Particle size (mm)
Boulder	>200
Cobble	63 - 200
Gravel	2.36 - 63
Sand	0.075 - 2.36
Silt	0.002 - 0.075
Clay	<0.002

The sand and gravel sizes can be further subdivided as follows:

Type	Particle size (mm)
Coarse gravel	19 - 63
Medium gravel	6.7 - 19
Fine gravel	2.36 – 6.7
Coarse sand	0.6 - 2.36
Medium sand	0.21 - 0.6
Fine sand	0.075 - 0.21

Definitions of grading terms used are:

- Well graded - a good representation of all particle sizes
- Poorly graded - an excess or deficiency of particular sizes within the specified range
- Uniformly graded - an excess of a particular particle size
- Gap graded - a deficiency of a particular particle size with the range

The proportions of secondary constituents of soils are described as follows:

In fine grained soils (>35% fines)

Term	Proportion of sand or gravel	Example
And	Specify	Clay (60%) and Sand (40%)
Adjective	>30%	Sandy Clay
With	15 – 30%	Clay with sand
Trace	0 - 15%	Clay with trace sand

In coarse grained soils (>65% coarse)
- with clays or silts

Term	Proportion of fines	Example
And	Specify	Sand (70%) and Clay (30%)
Adjective	>12%	Clayey Sand
With	5 - 12%	Sand with clay
Trace	0 - 5%	Sand with trace clay

In coarse grained soils (>65% coarse)
- with coarser fraction

Term	Proportion of coarser fraction	Example
And	Specify	Sand (60%) and Gravel (40%)
Adjective	>30%	Gravelly Sand
With	15 - 30%	Sand with gravel
Trace	0 - 15%	Sand with trace gravel

The presence of cobbles and boulders shall be specifically noted by beginning the description with 'Mix of Soil and Cobbles/Boulders' with the word order indicating the dominant first and the proportion of cobbles and boulders described together.

Soil Descriptions

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Cohesive Soils

Cohesive soils, such as clays, are classified on the basis of undrained shear strength. The strength may be measured by laboratory testing, or estimated by field tests or engineering examination. The strength terms are defined as follows:

Description	Abbreviation	Undrained shear strength (kPa)
Very soft	VS	<12
Soft	S	12 - 25
Firm	F	25 - 50
Stiff	St	50 - 100
Very stiff	VSt	100 - 200
Hard	H	>200
Friable	Fr	-

Cohesionless Soils

Cohesionless soils, such as clean sands, are classified on the basis of relative density, generally from the results of standard penetration tests (SPT), cone penetration tests (CPT) or dynamic penetrometers (PSP). The relative density terms are given below:

Relative Density	Abbreviation	Density Index (%)
Very loose	VL	<15
Loose	L	15-35
Medium dense	MD	35-65
Dense	D	65-85
Very dense	VD	>85

Soil Origin

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

- Residual soil - derived from in-situ weathering of the underlying rock;
- Extremely weathered material – formed from in-situ weathering of geological formations. Has soil strength but retains the structure or fabric of the parent rock;
- Alluvial soil – deposited by streams and rivers;
- Estuarine soil – deposited in coastal estuaries;
- Marine soil – deposited in a marine environment;
- Lacustrine soil – deposited in freshwater lakes;
- Aeolian soil – carried and deposited by wind;
- Colluvial soil – soil and rock debris transported down slopes by gravity;
- Topsoil – mantle of surface soil, often with high levels of organic material.
- Fill – any material which has been moved by man.

Moisture Condition – Coarse Grained Soils

For coarse grained soils the moisture condition should be described by appearance and feel using the following terms:

- Dry (D) Non-cohesive and free-running.
- Moist (M) Soil feels cool, darkened in colour.

  Soil tends to stick together.

  Sand forms weak ball but breaks easily.
- Wet (W) Soil feels cool, darkened in colour.

  Soil tends to stick together, free water forms when handling.

Moisture Condition – Fine Grained Soils

For fine grained soils the assessment of moisture content is relative to their plastic limit or liquid limit, as follows:

- 'Moist, dry of plastic limit' or 'w <PL' (i.e. hard and friable or powdery).
- 'Moist, near plastic limit' or 'w ≈ PL (i.e. soil can be moulded at moisture content approximately equal to the plastic limit).
- 'Moist, wet of plastic limit' or 'w >PL' (i.e. soils usually weakened and free water forms on the hands when handling).
- 'Wet' or 'w ≈LL' (i.e. near the liquid limit).
- 'Wet' or 'w >LL' (i.e. wet of the liquid limit).

Rock Strength

Rock strength is defined by the Unconfined Compressive Strength and it refers to the strength of the rock substance and not the strength of the overall rock mass, which may be considerably weaker due to defects.

The Point Load Strength Index $Is_{(50)}$ is commonly used to provide an estimate of the rock strength and site specific correlations should be developed to allow UCS values to be determined. The point load strength test procedure is described by Australian Standard AS4133.4.1-2007. The terms used to describe rock strength are as follows:

Strength Term	Abbreviation	Unconfined Compressive Strength MPa	Point Load Index * $Is_{(50)}$ MPa
Very low	VL	0.6 - 2	0.03 - 0.1
Low	L	2 - 6	0.1 - 0.3
Medium	M	6 - 20	0.3 - 1.0
High	H	20 - 60	1 - 3
Very high	VH	60 - 200	3 - 10
Extremely high	EH	>200	>10

* Assumes a ratio of 20:1 for UCS to $Is_{(50)}$. It should be noted that the UCS to $Is_{(50)}$ ratio varies significantly for different rock types and specific ratios should be determined for each site.

Degree of Weathering

The degree of weathering of rock is classified as follows:

Term	Abbreviation	Description
Residual Soil	RS	Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are no longer visible, but the soil has not been significantly transported.
Extremely weathered	XW	Material is weathered to such an extent that it has soil properties. Mass structure and material texture and fabric of original rock are still visible
Highly weathered	HW	The whole of the rock material is discoloured, usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable. Rock strength is significantly changed by weathering. Some primary minerals have weathered to clay minerals. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Moderately weathered	MW	The whole of the rock material is discoloured , usually by iron staining or bleaching to the extent that the colour of the original rock is not recognisable, but shows little or no change of strength from fresh rock.
Slightly weathered	SW	Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.
Fresh	FR	No signs of decomposition or staining.
Note: If HW and MW cannot be differentiated use DW (see below)		
Distinctly weathered	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by iron staining. Porosity may be increased by leaching or may be decreased due to deposition of weathered products in pores.

Rock Descriptions

Degree of Fracturing

The following classification applies to the spacing of natural fractures in diamond drill cores. It includes bedding plane partings, joints and other defects, but excludes drilling breaks.

Term	Description
Fragmented	Fragments of <20 mm
Highly Fractured	Core lengths of 20-40 mm with occasional fragments
Fractured	Core lengths of 30-100 mm with occasional shorter and longer sections
Slightly Fractured	Core lengths of 300 mm or longer with occasional sections of 100-300 mm
Unbroken	Core contains very few fractures

Rock Quality Designation

The quality of the cored rock can be measured using the Rock Quality Designation (RQD) index, defined as:

$$\text{RQD \%} = \frac{\text{cumulative length of 'sound' core sections > 100 mm long}}{\text{total drilled length of section being assessed}}$$

where 'sound' rock is assessed to be rock of low strength or stronger. The RQD applies only to natural fractures. If the core is broken by drilling or handling (i.e. drilling breaks) then the broken pieces are fitted back together and are not included in the calculation of RQD.

Stratification Spacing

For sedimentary rocks the following terms may be used to describe the spacing of bedding partings:

Term	Separation of Stratification Planes
Thinly laminated	< 6 mm
Laminated	6 mm to 20 mm
Very thinly bedded	20 mm to 60 mm
Thinly bedded	60 mm to 0.2 m
Medium bedded	0.2 m to 0.6 m
Thickly bedded	0.6 m to 2 m
Very thickly bedded	> 2 m

BOREHOLE LOG

CLIENT: Adams Consulting Engineers Pty Ltd
PROJECT: Proposed Alterations and Additions
LOCATION: 46 Prince Alfred Parade, Newport

SURFACE LEVEL: 2.1 AHD
EASTING: 342575.9
NORTHING: 6275215.1
DIP/AZIMUTH: 90°/--

BORE No: BH1
PROJECT No: 212173.00
DATE: 14/2/2022
SHEET 1 OF 1

Depth (m)	Description of Strata	Water	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
0.05	ASPHALTIC CONCRETE: possible AC10 wearing course						
0.15	PAVEMENT/Sandy GRAVEL: fine to medium, angular igneous gravel, grey, dry, fine to medium sand, dry, apparently well compacted, nominal 20mm roadbase (possible DGB20, base layer)						
1.0	FILL/Sandy CLAY, medium to high plasticity, red, fine to coarse sand, trace fine gravel, w>PL, generally in a firm condition	14-02-22		S			2,2,2 N = 4
1.7	Sandy Silty CLAY CI, medium plasticity, grey brown, fine to coarse sand, w>PL, generally in soft condition to firm condition	14-02-22					
2.1	Sandstone Boulder			S			15,25/100 refusal
2.6	Sandy CLAY CI: medium plasticity, pale brown, fine to medium sand, w>PL, in a very stiff condition						
	LAMINITE: Shale/Siltstone with interbedded sandstone layers, fine to medium grained, grey-brown, very low to low strength, highly weathered with clay seams, fractured, Newport and Garie Formation		Unless otherwise stated, fractures are J40-60, pl, ro, cly vn, fe stn and B0-5, pl, ro, fe stn 3m: Cs 130mm 3.19m: Cs 40mm 3.35m: Cs 60mm 3.42m: B10, pl, ro, cly co 3.47m: J50-60 (x2), pl, ro, cly co	C	100	10	
5.1			5m: CORE LOSS: 100mm	C	95	8	
6.94	Bore discontinued at 6.94m Target depth reached						

RIG: Ute Mounted Drill Rig **DRILLER:** Stratacore **LOGGED:** DS **CASING:** HW to 3m
TYPE OF BORING: Spiral Flight Auger (TC-bit) to 3 m, NMLC Coring to 6.94 m
WATER OBSERVATIONS: Free standing groundwater measured at 1.4m depth
REMARKS: Location coordinates are in MGA94 Zone 56. No visible signs of deformation within the pavement surface

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

Douglas Partners
Geotechnics / Environment / Groundwater

BORE: BH1 PROJECT: NEWPORT RMYC FEBRUARY 2022

3.00 – 6.94m

BOREHOLE LOG

CLIENT: Adams Consulting Engineers Pty Ltd
PROJECT: Proposed Alterations and Additions
LOCATION: 46 Prince Alfred Parade, Newport

SURFACE LEVEL: 2.1 AHD
EASTING: 342576.8
NORTHING: 6275194.9
DIP/AZIMUTH: 90°/--

BORE No: BH2
PROJECT No: 212173.00
DATE: 14/2/2022
SHEET 1 OF 1

Depth (m)	Description of Strata	Water	Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)	Type	Core Rec. %	RQD %	Test Results & Comments
0.05	ASPHALTIC CONCRETE: possible AC10 wearing course						
0.15	PAVEMENT/Sandy GRAVEL: fine to medium, angular igneous gravel, grey, dry, fine to medium sand, dry, apparently well compacted, nominal 20mm roadbase (possible DGB20, base layer)						
1.0	FILL/Sandy CLAY, medium to high plasticity, red, fine to coarse sand, trace fine gravel, w>PL, generally in a firm condition						
1.3	Sandstone Boulder						
2.0	Sandy Silty CLAY CI, medium plasticity, grey brown, fine to coarse sand, w>PL, generally in soft condition to firm condition	14-02-22					
2.5	Sandy CLAY CI: medium plasticity, pale brown, fine to medium sand, w>PL, in a very stiff condition		Unless otherwise stated, fractures are J40-60, pl, ro, cly vn, fe stn				
	LAMINITE: Shale/Siltstone with interbedded sandstone layers, fine to medium grained, grey-brown, very low to low strength, highly weathered with clay seams, fractured, Newport and Garie Formation		3.35m: Cs 20mm 3.4m: J60, pl, ro, fe stn 3.8m: Cs 20mm 4.93m: Cs 70mm 5.25m: J60-90, cu, ro, fe stn 6.05-6.37m: J80, pl, ro	C	100	0	
6.37	Bore discontinued at 6.37m Target depth reached						

RIG: Ute Mounted Drill Rig **DRILLER:** Stratacore **LOGGED:** DS **CASING:** HW to 3.3m
TYPE OF BORING: Spiral Flight Auger (TC-bit) to 3.3 m, NMLC Coring to 6.5 m
WATER OBSERVATIONS: Free standing groundwater measured at 2m depth
REMARKS: Location coordinates are in MGA94 Zone 56. No visible signs of deformation within the pavement surface

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

Douglas Partners
Geotechnics / Environment / Groundwater

BORE: BH2   PROJECT: NEWPORT RMYC   FEBRUARY 2022

3.30 – 6.37m

BOREHOLE LOG

CLIENT: Adams Consulting Engineers Pty Ltd
PROJECT: Proposed Alterations and Additions
LOCATION: 46 Prince Alfred Parade, Newport

SURFACE LEVEL: 2.2 AHD
EASTING: 342610
NORTHING: 6275180.2
DIP/AZIMUTH: 90°/--

BORE No: BH3
PROJECT No: 212173.00
DATE: 15/2/2022
SHEET 1 OF 1

Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
0.05	ASPHALTIC CONCRETE: possible AC10 wearing course					
0.15	PAVEMENT/Sandy GRAVEL: fine to medium, angular igneous gravel, grey, dry, fine to medium sand, dry, apparently well compacted, nominal 20mm roadbase (possible DGB20, base layer)					
1.0	FILL/Sandy CLAY, medium to high plasticity, red, fine to coarse sand, trace fine gravel, w>PL, generally in a firm condition		S			9,12,23 N = 35
1.5	Sandy CLAY CI: medium plasticity, pale brown, fine to medium sand, w>PL, in a very stiff condition					
	LAMINITE: Shale/Siltstone with interbedded sandstone layers, fine to medium grained, grey-brown, very low to low strength, highly weathered with clay seams, fractured, Newport and Garie Formation	Unless otherwise stated, fractures are B0-5, pl, ro, fe stn 3.13m: J90, pl, ro, cln 3.37m: Cs 130mm 3.88m: J70, pl, ro, fe st 4.3m: J60, pl, fe he 4.37m: J60, pl, ro, fe st 4.48m: Cs 20mm 4.6m: Cs 40mm 4.66m: J90, pl, ro, fe st	C	100	42	
6.2	Bore discontinued at 6.2m Target depth reached					

Column headings: RL; Depth (m); Description of Strata; Degree of Weathering (EW, HW, MW, SW, FS, FR); Graphic Log; Rock Strength (Ex Low, Very Low, Low, Medium, High, Very High, Ex High); Water; Fracture Spacing (m) (0.01, 0.05, 0.10, 0.50, 1.00); Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault); Sampling & In Situ Testing (Type, Core Rec. %, RQD %, Test Results & Comments)

RIG: Ute Mounted Drill Rig **DRILLER:** Stratacore **LOGGED:** DS **CASING:** HW to 3.13m
TYPE OF BORING: Spiral Flight Auger (TC-bit) to 3.13 m, NMLC Coring to 6.4 m
WATER OBSERVATIONS: No free groundwater observed
REMARKS: Location coordinates are in MGA94 Zone 56. No visible signs of deformation within the pavement surface

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

Douglas Partners
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BORE: BH3 PROJECT: NEWPORT RMYC FEBRUARY 2022

3.13 – 6.20m

BOREHOLE LOG

CLIENT: Adams Consulting Engineers Pty Ltd
PROJECT: Proposed Alterations and Additions
LOCATION: 46 Prince Alfred Parade, Newport

SURFACE LEVEL: 2.3 AHD
EASTING: 342592.2
NORTHING: 6275178.4
DIP/AZIMUTH: 90°/--

BORE No: BH4
PROJECT No: 212173.00
DATE: 15/2/2022
SHEET 1 OF 1

Depth (m)	Description of Strata
0.05	ASPHALTIC CONCRETE: possible AC10 wearing course
0.15	PAVEMENT/Sandy GRAVEL: fine to medium, angular igneous gravel, grey, dry, fine to medium sand, dry, apparently well compacted, nominal 20mm roadbase (possible DGB20, base layer)
1.5	FILL/Clayey SAND, fine to coarse, red, medium plasticity, trace fine to medium gravel, w>PL, generally in a firm condition Below 1.4 m: Possible Sandstone Cobble, 100mm thick
2.0	Sandy Silty CLAY CI, medium plasticity, grey brown, fine to coarse sand, w>PL, generally in soft condition to firm condition
2.5	Sandy CLAY CI: medium plasticity, pale brown, fine to medium sand, w>PL, in a very stiff condition
	LAMINITE: Shale/Siltstone with interbedded sandstone layers, fine to medium grained, grey-brown, very low to low strength, highly weathered with clay seams, fractured, Newport and Garie Formation
6.42	Bore discontinued at 6.42m Target depth reached

Water: 15-02-22 (water level at approx. 2.2 m)

Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault)

Unless otherweise stated, fractures are B0-5, pl, ro, fe stn

- 3.6m: Cs 100mm
- 3.92m: J30, pl, ro, fe stn
- 4.21m: B0, pl, ro, fe, cly vn
- 4.26m: Cs 40mm
- 4.54m: B0, pl, sm, cly 5mm
- 4.58m: B0, pl, sm, cly 5mm
- 4.65-4.82m: B0 (x7), pl, ro, cly vn
- 5.47m: J70, pl, ro, fe stn
- 5.61m: Cs 40mm
- 5.76m: J70-90, ir, ca he
- 5.92m: B0, pl, ro, cly co 2mm

Sampling & In Situ Testing

Depth (m)	Type	Core Rec. %	RQD %	Test Results & Comments
1.0–1.45	S			1,7,8 N = 15
2.5–2.95	S			13,12,11 N = 23
3.6–6.42	C	100	26	

RIG: Ute Mounted Drill Rig **DRILLER:** Stratacore **LOGGED:** DS **CASING:** HW to 3.6m
TYPE OF BORING: Spiral Flight Auger (TC-bit) to 3.6 m, NMLC Coring to 6.42 m
WATER OBSERVATIONS: Free standing groundwater measured at 2.2m depth
REMARKS: Location coordinates are in MGA94 Zone 56. No visible signs of deformation within the pavement surface

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

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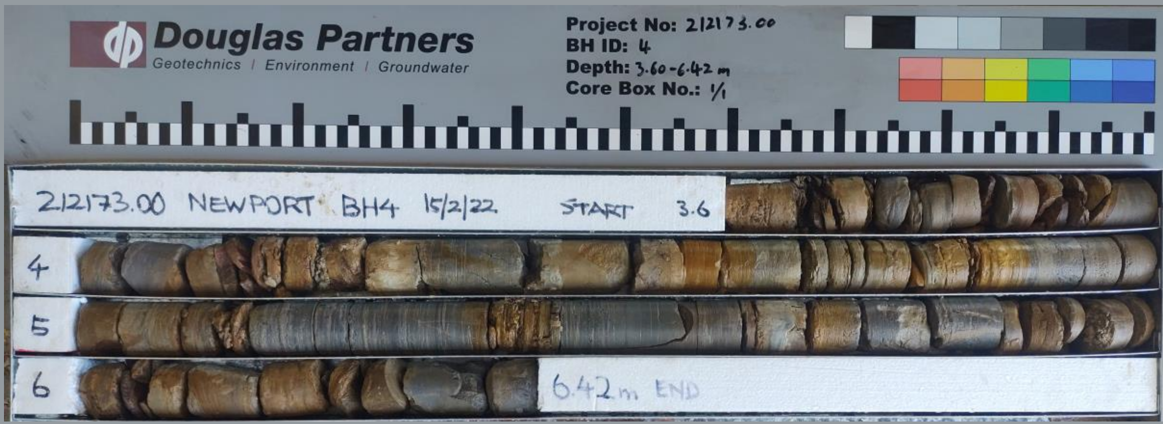
Geotechnics / Environment / Groundwater

BORE: BH4 PROJECT: NEWPORT RMYC FEBRUARY 2022



3.60 – 6.42m

BOREHOLE LOG

CLIENT: Adams Consulting Engineers Pty Ltd
PROJECT: Proposed Alterations and Additions
LOCATION: 46 Prince Alfred Parade, Newport

SURFACE LEVEL: 3.4 AHD
EASTING: 342612.5
NORTHING: 6275197.9
DIP/AZIMUTH: 90°/--

BORE No: BH5
PROJECT No: 212173.00
DATE: 14/2/2022
SHEET 1 OF 1

Depth (m)	Description of Strata	Discontinuities	Type	Core Rec. %	RQD %	Test Results & Comments
0.08	Paving	Unless otherweise stated, fractures are B0-5, pl, ro, fe stn				
0.15	PAVEMENT/Sandy GRAVEL: fine to medium, angular igneous gravel, grey, dry, fine to medium sand, dry, apparently in a medium dense condition	0.66m: CORE LOSS: 110mm				
0.46	Concrete	1m: CORE LOSS: 60mm				
0.67	FILL/Gravelly CLAY: medium to high plasticity, red, fine to coarse sand, fine to medium gravel, w>PL, generally in a firm condition	1.06m: J80, ir, ro, fe stn 1.3m: CORE LOSS: 130mm				
0.77	LAMINITE: Shale/Siltstone with interbedded sandstone layers, fine to medium grained, grey-brown, very low to low strength, highly weathered with clay seams, fractured, Newport and Garie Formation	1.6m: J90, ir, ro, fe stn 1.76m: J70, ir, ro, fe he 1.9m: J70, ir, ro, fe he 2.4m: J70, ir, ro, fe he 2.55m: Fg 40mm 2.76m: J60, pl, ro, fe stn 3m: CORE LOSS: 120mm 3.22m: J70, ir, fe he	C	90	0	
3.12						
3.3	Bore discontinued at 3.3m Target depth reached					

Column headings: RL; Depth (m); Description of Strata; Degree of Weathering (EW, HW, MW, SW, FS, FR); Graphic Log; Rock Strength (Ex Low, Very Low, Low, Medium, High, Very High, Ex High); Water; Fracture Spacing (m) (0.01, 0.05, 0.10, 0.50, 1.00); Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault); Sampling & In Situ Testing (Type, Core Rec. %, RQD %, Test Results & Comments).

RIG: Portable Drilling Rig **DRILLER:** A1 **LOGGED:** DS **CASING:** HW to 0.15m
TYPE OF BORING: Hand Auger to 0.15 m, Dia-core to 3.3 m
WATER OBSERVATIONS: No free groundwater observed
REMARKS: Location coordinates are in MGA94 Zone 56. No visible signs of deformation within the pavement surface

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	G	Gas sample	PID	Photo ionisation detector (ppm)
B	Bulk sample	P	Piston sample	PL(A)	Point load axial test Is(50) (MPa)
BLK	Block sample	U_x	Tube sample (x mm dia.)	PL(D)	Point load diametral test Is(50) (MPa)
C	Core drilling	W	Water sample	pp	Pocket penetrometer (kPa)
D	Disturbed sample	▷	Water seep	S	Standard penetration test
E	Environmental sample	▼	Water level	V	Shear vane (kPa)

Douglas Partners
Geotechnics / Environment / Groundwater

BORE: BH5 PROJECT: NEWPORT RMYC FEBRUARY 2022

0.15 – 3.30m

Appendix D

Laboratory Test Results

Envirolab Services Pty Ltd
ABN 37 112 535 645
12 Ashley St Chatswood NSW 2067
ph 02 9910 6200 fax 02 9910 6201
customerservice@envirolab.com.au
www.envirolab.com.au

CERTIFICATE OF ANALYSIS 289698

Client Details

Client	Douglas Partners Pty Ltd
Attention	David Smith
Address	96 Hermitage Rd, West Ryde, NSW, 2114

Sample Details

Your Reference	**212173.00**
Number of Samples	5 Soil
Date samples received	24/02/2022
Date completed instructions received	24/02/2022

Analysis Details

Please refer to the following pages for results, methodology summary and quality control data.

Samples were analysed as received from the client. Results relate specifically to the samples as received.

Results are reported on a dry weight basis for solids and on an as received basis for other matrices.

Please refer to the last page of this report for any comments relating to the results.

Report Details

Date results requested by	01/03/2022
Date of Issue	01/03/2022

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Accredited for compliance with ISO/IEC 17025 - Testing. **Tests not covered by NATA are denoted with ***

Results Approved By
Priya Samarawickrama, Senior Chemist

Authorised By

Nancy Zhang, Laboratory Manager

Misc Inorg - Soil				
Our Reference		289698-1	289698-3	289698-5
Your Reference	UNITS	BH1 1.0-1.45	BH2 2.5-2.95	BH4 0.4-0.6
Date Sampled		14/02/2022	14/02/2022	15/02/2022
Type of sample		Soil	Soil	Soil
Date prepared	-	28/02/2022	28/02/2022	28/02/2022
Date analysed	-	28/02/2022	28/02/2022	28/02/2022
pH 1:5 soil:water	pH Units	7.2	8.6	7.8
Electrical Conductivity 1:5 soil:water	µS/cm	360	130	200
Chloride, Cl 1:5 soil:water	mg/kg	380	100	67
Sulphate, SO4 1:5 soil:water	mg/kg	82	57	200

sPOCAS field test				
Our Reference		289698-1	289698-4	289698-5
Your Reference	UNITS	BH1 1.0-1.45	BH3 1-1.45	BH4 0.4-0.6
Date Sampled		14/02/2022	15/02/2022	15/02/2022
Type of sample		Soil	Soil	Soil
Date prepared	-	28/02/2022	28/02/2022	28/02/2022
Date analysed	-	28/02/2022	28/02/2022	28/02/2022
pH_F (field pH test)*	pH Units	7.5	7.5	8.2
pH_{FOX} (field peroxide test)*	pH Units	4.0	5.0	4.1
Reaction Rate*	-	High reaction	Low reaction	Medium reaction

Method ID	Methodology Summary
Inorg-001	pH - Measured using pH meter and electrode in accordance with APHA latest edition, 4500-H+. Please note that the results for water analyses are indicative only, as analysis outside of the APHA storage times.
Inorg-002	Conductivity and Salinity - measured using a conductivity cell at 25°C in accordance with APHA latest edition 2510 and Rayment & Lyons.
Inorg-063	pH- measured using pH meter and electrode. Soil is oxidised with Hydrogen Peroxide or extracted with water. Based on section H, Acid Sulfate Soils Laboratory Methods Guidelines, Version 2.1 - June 2004. To ensure accurate results these tests are recommended to be done in the field as pH may change with time thus these results may not be representative of true field conditions.
Inorg-081	Anions - a range of Anions are determined by Ion Chromatography, in accordance with APHA latest edition, 4110-B. Waters samples are filtered on receipt prior to analysis. Alternatively determined by colourimetry/turbidity using Discrete Analyser.

QUALITY CONTROL: Misc Inorg - Soil					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			28/02/2022	1	28/02/2022	28/02/2022		28/02/2022	[NT]
Date analysed	-			28/02/2022	1	28/02/2022	28/02/2022		28/02/2022	[NT]
pH 1:5 soil:water	pH Units		Inorg-001	[NT]	1	7.2	7.2	0	101	[NT]
Electrical Conductivity 1:5 soil:water	µS/cm	1	Inorg-002	<1	1	360	410	13	102	[NT]
Chloride, Cl 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	380	440	15	102	[NT]
Sulphate, SO4 1:5 soil:water	mg/kg	10	Inorg-081	<10	1	82	86	5	89	[NT]

QUALITY CONTROL: sPOCAS field test					Duplicate				Spike Recovery %	
Test Description	Units	PQL	Method	Blank	#	Base	Dup.	RPD	LCS-1	[NT]
Date prepared	-			28/02/2022	[NT]	[NT]	[NT]	[NT]	28/02/2022	[NT]
Date analysed	-			28/02/2022	[NT]	[NT]	[NT]	[NT]	28/02/2022	[NT]
pH_F (field pH test)*	pH Units		Inorg-063	[NT]	[NT]	[NT]	[NT]	[NT]	99	[NT]

Result Definitions

NT	Not tested
NA	Test not required
INS	Insufficient sample for this test
PQL	Practical Quantitation Limit
<	Less than
>	Greater than
RPD	Relative Percent Difference
LCS	Laboratory Control Sample
NS	Not specified
NEPM	National Environmental Protection Measure
NR	Not Reported

Quality Control Definitions

Blank	This is the component of the analytical signal which is not derived from the sample but from reagents, glassware etc, can be determined by processing solvents and reagents in exactly the same manner as for samples.
Duplicate	This is the complete duplicate analysis of a sample from the process batch. If possible, the sample selected should be one where the analyte concentration is easily measurable.
Matrix Spike	A portion of the sample is spiked with a known concentration of target analyte. The purpose of the matrix spike is to monitor the performance of the analytical method used and to determine whether matrix interferences exist.
LCS (Laboratory Control Sample)	This comprises either a standard reference material or a control matrix (such as a blank sand or water) fortified with analytes representative of the analyte class. It is simply a check sample.
Surrogate Spike	Surrogates are known additions to each sample, blank, matrix spike and LCS in a batch, of compounds which are similar to the analyte of interest, however are not expected to be found in real samples.

Australian Drinking Water Guidelines recommend that Thermotolerant Coliform, Faecal Enterococci, & E.Coli levels are less than 1cfu/100mL. The recommended maximums are taken from "Australian Drinking Water Guidelines", published by NHMRC & ARMC 2011.

The recommended maximums for analytes in urine are taken from “2018 TLVs and BEIs”, as published by ACGIH (where available). Limit provided for Nickel is a precautionary guideline as per Position Paper prepared by AIOH Exposure Standards Committee, 2016.

Guideline limits for Rinse Water Quality reported as per analytical requirements and specifications of AS 4187, Amdt 2 2019, Table 7.2

Laboratory Acceptance Criteria

Duplicate sample and matrix spike recoveries may not be reported on smaller jobs, however, were analysed at a frequency to meet or exceed NEPM requirements. All samples are tested in batches of 20. The duplicate sample RPD and matrix spike recoveries for the batch were within the laboratory acceptance criteria.

Filters, swabs, wipes, tubes and badges will not have duplicate data as the whole sample is generally extracted during sample extraction.

Spikes for Physical and Aggregate Tests are not applicable.

For VOCs in water samples, three vials are required for duplicate or spike analysis.

Duplicates: >10xPQL - RPD acceptance criteria will vary depending on the analytes and the analytical techniques but is typically in the range 20%-50% – see ELN-P05 QA/QC tables for details; <10xPQL - RPD are higher as the results approach PQL and the estimated measurement uncertainty will statistically increase.

Matrix Spikes, LCS and Surrogate recoveries: Generally 70-130% for inorganics/metals (not SPOCAS); 60-140% for organics/SPOCAS (+/-50% surrogates) and 10-140% for labile SVOCs (including labile surrogates), ultra trace organics and speciated phenols is acceptable.

In circumstances where no duplicate and/or sample spike has been reported at 1 in 10 and/or 1 in 20 samples respectively, the sample volume submitted was insufficient in order to satisfy laboratory QA/QC protocols.

When samples are received where certain analytes are outside of recommended technical holding times (THTs), the analysis has proceeded. Where analytes are on the verge of breaching THTs, every effort will be made to analyse within the THT or as soon as practicable.

Where sampling dates are not provided, Envirolab are not in a position to comment on the validity of the analysis where recommended technical holding times may have been breached.

Measurement Uncertainty estimates are available for most tests upon request.

Analysis of aqueous samples typically involves the extraction/digestion and/or analysis of the liquid phase only (i.e. NOT any settled sediment phase but inclusive of suspended particles if present), unless stipulated on the Envirolab COC and/or by correspondence. Notable exceptions include certain Physical Tests (pH/EC/BOD/COD/Apparent Colour etc.), Solids testing, total recoverable metals and PFAS where solids are included by default.

Samples for Microbiological analysis (not Amoeba forms) received outside of the 2-8°C temperature range do not meet the ideal cooling conditions as stated in AS2031-2012.

Report Comments

pH/ec Samples were out of the recommended holding time for this analysis.

Appendix E

Photographs

Photo 1: Existing Batter, looking south-west. Heavily vegetated. No leaning trees observed.

Photo 2: Terrace area, looking north-east.

Timber retaining wall, showing some evidence of bulging

Photo 3: View of the proposed lift area, looking north-east.

Showing the existing retaining wall and vegetated batter.

Photo 4: View of terraced area, looking south-east toward the vegetated batter,

behind the brush fence.

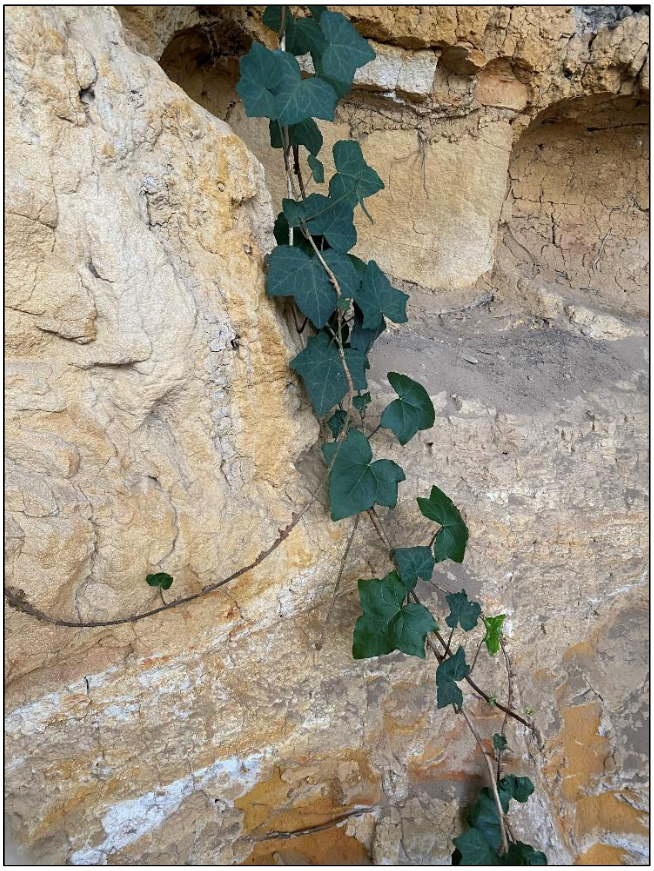


Photo 5: View of weathered rock beneath the existing building

Photo 6: View of exposed cut beneath the existing building, looking north-east.

Douglas Partners Geotechnics / Environment / Groundwater	Project: 86043.12	Site Photographs
	Date: 13 July 2023	Proposed Alterations and Additions 46 Prince Alfred Parade, Newport

Appendix F

AGS Guidelines and Forms 1 and 1(a)

LANDSLIDE RISK

Concept of Risk

Risk is a familiar term, but what does it really mean? It can be defined as *"a measure of the probability and severity of an adverse effect to health, property, or the environment."* This definition may seem a bit complicated. In relation to landslides, geotechnical practitioners (GeoGuide LR1) are required to assess risk in terms of the likelihood that a particular landslide will occur and the possible consequences. This is called landslide risk assessment. The consequences of a landslide are many and varied, but our concerns normally focus on loss of, or damage to, property and loss of life.

Landslide Risk Assessment

Some local councils in Australia are aware of the potential for landslides within their jurisdiction and have responded by designating specific "landslide hazard zones". Development in these areas is often covered by special regulations. If you are contemplating building, or buying an existing house, particularly in a hilly area, or near cliffs, go first for information to your local council.

Landslide risk assessment must be undertaken by a geotechnical practitioner. It may involve visual inspection, geological mapping, geotechnical investigation and monitoring to identify:

- potential landslides (there may be more than one that could impact on your site)
- the likelihood that they will occur
- the damage that could result
- the cost of disruption and repairs and
- the extent to which lives could be lost.

Risk assessment is a predictive exercise, but since the ground and the processes involved are complex, prediction tends to lack precision. If you commission a landslide risk assessment for a particular site you should expect to receive a report prepared in accordance with current professional guidelines and in a form that is acceptable to your local council, or planning authority.

Risk to Property

Table 1 indicates the terms used to describe risk to property. Each risk level depends on an assessment of how likely a landslide is to occur and its consequences in dollar terms. "Likelihood" is the chance of it happening in any one year, as indicated in Table 2. "Consequences" are related to the cost of repairs and temporary loss of use if a landslide occurs. These two factors are combined by the geotechnical practitioner to determine the Qualitative Risk.

TABLE 2: LIKELIHOOD

Likelihood	Annual Probability
Almost Certain	1:10
Likely	1:100
Possible	1:1,000
Unlikely	1:10,000
Rare	1:100,000
Barely credible	1:1,000,000

The terms "unacceptable", "may be tolerated", etc. in Table 1 indicate how most people react to an assessed risk level. However, some people will always be more prepared, or better able, to tolerate a higher risk level than others.

Some local councils and planning authorities stipulate a maximum tolerable level of risk to property for developments within their jurisdictions. In these situations the risk must be assessed by a geotechnical practitioner. If stabilisation works are needed to meet the stipulated requirements these will normally have to be carried out as part of the development, or consent will be withheld.

TABLE 1: RISK TO PROPERTY

Qualitative Risk		Significance - Geotechnical engineering requirements
Very high	VH	**Unacceptable** without treatment. Extensive detailed investigation and research, planning and implementation of treatment options essential to reduce risk to Low. May be too expensive and not practical. Work likely to cost more than the value of the property.
High	H	**Unacceptable** without treatment. Detailed investigation, planning and implementation of treatment options required to reduce risk to acceptable level. Work would cost a substantial sum in relation to the value of the property.
Moderate	M	**May be tolerated** in certain circumstances (subject to regulator's approval) but requires investigation, planning and implementation of treatment options to reduce the risk to Low. Treatment options to reduce to Low risk should be implemented as soon as possible.
Low	L	**Usually acceptable** to regulators. Where treatment has been needed to reduce the risk to this level, ongoing maintenance is required.
Very Low	VL	**Acceptable**. Manage by normal slope maintenance procedures.

Risk to Life

Most of us have some difficulty grappling with the concept of risk and deciding whether, or not, we are prepared to accept it. However, without doing any sort of analysis, or commissioning a report from an "expert", we all take risks every day. One of them is the risk of being killed in an accident. This is worth thinking about, because it tells us a lot about ourselves and can help to put an assessed risk into a meaningful context. By identifying activities that we either are, or are not, prepared to engage in we can get some indication of the maximum level of risk that we are prepared to take. This knowledge can help us to decide whether we really are able to accept a particular risk, or to tolerate a particular likelihood of loss, or damage, to our property (Table 2).

In Table 3, data from NSW for the years 1998 to 2002, and other sources, is presented. A risk of 1 in 100,000 means that, in any one year, 1 person is killed for every 100,000 people undertaking that particular activity. The NSW data assumes that the whole population undertakes the activity. That is, we are all at risk of being killed in a fire, or of choking on our food, but it is reasonable to assume that only people who go deep sea fishing run a risk of being killed while doing it.

It can be seen that the risks of dying as a result of falling, using a motor vehicle, or engaging in water-related activities (including bathing) are all greater than 1:100,000 and yet few people actively avoid situations where these risks are present. Some people are averse to flying and yet it represents a lower risk than choking to death on food. Importantly, the data also indicate that, even when the risk of dying as a consequence of a particular event is very small, it could still happen to any one of us any day. If this were not so, no one would ever be struck by lightning.

Most local councils and planning authorities that stipulate a tolerable risk to property also stipulate a tolerable risk to life. The AGS Practice Note Guideline recommends that 1:100,000 is tolerable in newly developed areas, where works can be carried out as part of the development to limit risk. The tolerable level is raised to 1:10,000 in established areas, where specific landslide hazards may have existed for many years. The distinction is deliberate and intended to prevent the concept of landslide risk management, for its own sake, becoming an unreasonable financial burden on existing communities. Acceptable risk is usually taken to be one tenth of the tolerable risk (1:1,000,000 for new developments and 1:100,000 for established areas) and efforts should be made to attain these where it is practicable and financially realistic to do so.

TABLE 3: RISK TO LIFE

Risk (deaths per participant per year)	**Activity/Event Leading to Death** (NSW data unless noted)
1:1,000	Deep sea fishing (UK)
1:1,000 to 1:10,000	Motor cycling, horse riding , ultra-light flying (Canada)
1:23,000	Motor vehicle use
1:30,000	Fall
1:70,000	Drowning
1:180,000	Fire/burn
1:660,000	Choking on food
1:1,000,000	Scheduled airlines (Canada)
1:2,300,000	Train travel
1:32,000,000	Lightning strike

More information relevant to your particular situation may be found in other AUSTRALIAN GEOGUIDES:

- GeoGuide LR1 - Introduction
- GeoGuide LR2 - Landslides
- GeoGuide LR3 - Landslides in Soil
- GeoGuide LR4 - Landslides in Rock
- GeoGuide LR5 - Water & Drainage
- GeoGuide LR6 - Retaining Walls
- GeoGuide LR8 - Hillside Construction
- GeoGuide LR9 - Effluent & Surface Water Disposal
  GeoGuide LR10 - Coastal Landslides
- GeoGuide LR11 - Record Keeping

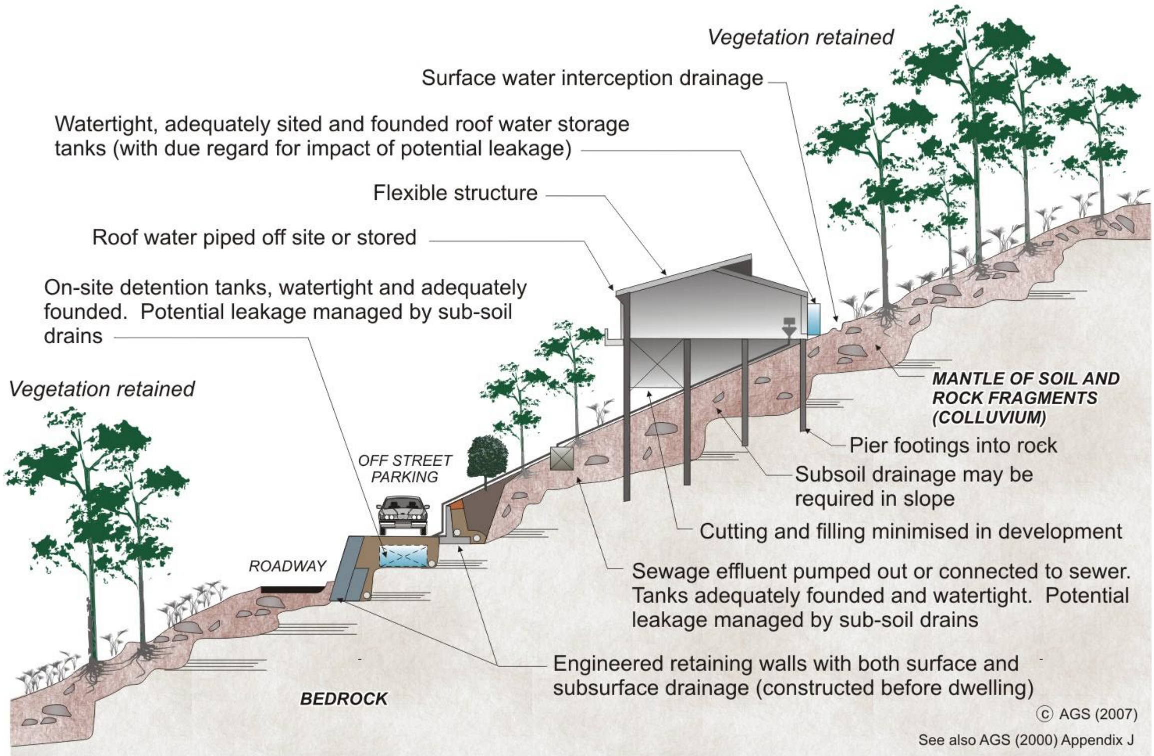
The Australian GeoGuides (LR series) are a set of publications intended for property owners; local councils; planning authorities; developers; insurers; lawyers and, in fact, anyone who lives with, or has an interest in, a natural or engineered slope, a cutting, or an excavation. They are intended to help you understand why slopes and retaining structures can be a hazard and what can be done with appropriate professional advice and local council approval (if required) to remove, reduce, or minimise the risk they represent. The GeoGuides have been prepared by the Australian Geomechanics Society, a specialist technical society within Engineers Australia, the national peak body for all engineering disciplines in Australia, whose members are professional geotechnical engineers and engineering geologists with a particular interest in ground engineering. The GeoGuides have been funded under the Australian governments' National Disaster Mitigation Program.

AUSTRALIAN GEOGUIDE LR8 (CONSTRUCTION PRACTICE)

HILLSIDE CONSTRUCTION PRACTICE

Sensible development practices are required when building on hillsides, particularly if the hillside has more than a low risk of instability (GeoGuide LR7). Only building techniques intended to maintain, or reduce, the overall level of landslide risk should be considered. Examples of good hillside construction practice are illustrated below.

EXAMPLES OF **GOOD** HILLSIDE CONSTRUCTION PRACTICE



WHY ARE THESE PRACTICES GOOD?

Roadways and parking areas - are paved and incorporate kerbs which prevent water discharging straight into the hillside (GeoGuide LR5).

Cuttings - are supported by retaining walls (GeoGuide LR6).

Retaining walls - are engineer designed to withstand the lateral earth pressures and surcharges expected, and include drains to prevent water pressures developing in the backfill. Where the ground slopes steeply down towards the high side of a retaining wall, the disturbing force (see GeoGuide LR6) can be two or more times that in level ground. Retaining walls must be designed taking these forces into account.

Sewage - whether treated or not is either taken away in pipes or contained in properly founded tanks so it cannot soak into the ground.

Surface water - from roofs and other hard surfaces is piped away to a suitable discharge point rather than being allowed to infiltrate into the ground. Preferably, the discharge point will be in a natural creek where ground water exits, rather than enters, the ground. Shallow, lined, drains on the surface can fulfil the same purpose (GeoGuide LR5).

Surface loads - are minimised. No fill embankments have been built. The house is a lightweight structure. Foundation loads have been taken down below the level at which a landslide is likely to occur and, preferably, to rock. This sort of construction is probably not applicable to soil slopes (GeoGuide LR3). If you are uncertain whether your site has rock near the surface, or is essentially a soil slope, you should engage a geotechnical practitioner to find out.

Flexible structures - have been used because they can tolerate a certain amount of movement with minimal signs of distress and maintain their functionality.

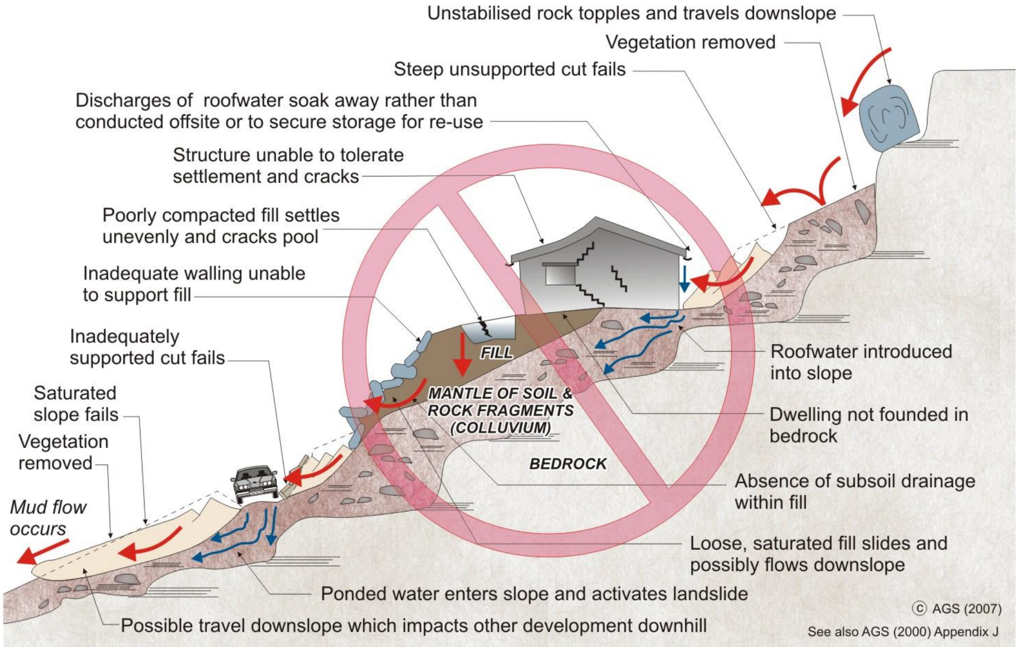
Vegetation clearance - on soil slopes has been kept to a reasonable minimum. Trees, and to a lesser extent smaller vegetation, take large quantities of water out of the ground every day. This lowers the ground water table, which in turn helps to maintain the stability of the slope. Large scale clearing can result in a rise in water table with a consequent increase in the likelihood of a landslide (GeoGuide LR5). An exception may have to be made to this rule on steep rock slopes where trees have little effect on the water table, but their roots pose a landslide hazard by dislodging boulders.

Possible effects of ignoring good construction practices are illustrated on page 2. Unfortunately, these poor construction practices are not as unusual as you might think and are often chosen because, on the face of it, they will save the developer, or owner, money. You should not lose sight of the fact that the cost and anguish associated with any one of the disasters illustrated, is likely to more than wipe out any apparent savings at the outset.

ADOPT GOOD PRACTICE ON HILLSIDE SITES

AUSTRALIAN GEOGUIDE LR8 (CONSTRUCTION PRACTICE)

EXAMPLES OF **POOR** HILLSIDE CONSTRUCTION PRACTICE



WHY ARE THESE PRACTICES POOR?

Roadways and parking areas - are unsurfaced and lack proper table drains (gutters) causing surface water to pond and soak into the ground.

Cut and fill - has been used to balance earthworks quantities and level the site leaving unstable cut faces and added large surface loads to the ground. Failure to compact the fill properly has led to settlement, which will probably continue for several years after completion. The house and pool have been built on the fill and have settled with it and cracked. Leakage from the cracked pool and the applied surface loads from the fill have combined to cause landslides.

Retaining walls - have been avoided, to minimise cost, and hand placed rock walls used instead. Without applying engineering design principles, the walls have failed to provide the required support to the ground and have failed, creating a very dangerous situation.

A heavy, rigid, house - has been built on shallow, conventional, footings. Not only has the brickwork cracked because of the resulting ground movements, but it has also become involved in a man-made landslide.

Soak-away drainage - has been used for sewage and surface water run-off from roofs and pavements. This water soaks into the ground and raises the water table (GeoGuide LR5). Subsoil drains that run along the contours should be avoided for the same reason. If felt necessary, subsoil drains should run steeply downhill in a chevron, or herring bone, pattern. This may conflict with the requirements for effluent and surface water disposal (GeoGuide LR9) and if so, you will need to seek professional advice.

Rock debris - from landslides higher up on the slope seems likely to pass through the site. Such locations are often referred to by geotechnical practitioners as "debris flow paths". Rock is normally even denser than ordinary fill, so even quite modest boulders are likely to weigh many tonnes and do a lot of damage once they start to roll. Boulders have been known to travel hundreds of metres downhill leaving behind a trail of destruction.

Vegetation - has been completely cleared, leading to a possible rise in the water table and increased landslide risk (GeoGuide LR5).

DON'T CUT CORNERS ON HILLSIDE SITES - OBTAIN ADVICE FROM A GEOTECHNICAL PRACTITIONER

More information relevant to your particular situation may be found in other Australian GeoGuides:

- GeoGuide LR1 - Introduction
- GeoGuide LR2 - Landslides
- GeoGuide LR3 - Landslides in Soil
- GeoGuide LR4 - Landslides in Rock
- GeoGuide LR5 - Water & Drainage
- GeoGuide LR6 - Retaining Walls
- GeoGuide LR7 - Landslide Risk
- GeoGuide LR9 - Effluent & Surface Water Disposal
- GeoGuide LR10 - Coastal Landslides
- GeoGuide LR11 - Record Keeping

The Australian GeoGuides (LR series) are a set of publications intended for property owners; local councils; planning authorities; developers; insurers; lawyers and, in fact, anyone who lives with, or has an interest in, a natural or engineered slope, a cutting, or an excavation. They are intended to help you understand why slopes and retaining structures can be a hazard and what can be done with appropriate professional advice and local council approval (if required) to remove, reduce, or minimise the risk they represent. The GeoGuides have been prepared by the Australian Geomechanics Society, a specialist technical society within Engineers Australia, the national peak body for all engineering disciplines in Australia, whose members are professional geotechnical engineers and engineering geologists with a particular interest in ground engineering. The GeoGuides have been funded under the Australian governments' National Disaster Mitigation Program.

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER

FORM NO. 1 – To be submitted with Development Application

Development Application for Royal Motor Yacht Club Broken Bay NSW
Name of Applicant
Address of site 46 Prince Alfred Parade, Newport

Declaration made by geotechnical engineer or engineering geologist or coastal engineer (where applicable) as part of a geotechnical report

I, Geoff McIntosh *on behalf of* Douglas Partners
(Insert Name) (Trading or Company Name)

on this the 14 July 2023 certify that I am a geotechnical engineer or engineering geologist ~~or coastal engineer~~ as defined by the Geotechnical Risk Management Policy for Pittwater - 2009 and I am authorised by the above organisation/company to issue this document and to certify that the organisation/company has a current professional indemnity policy of at least $10million.

~~I:~~ Douglas Partners

Please mark appropriate box

- ☑ have prepared the detailed Geotechnical Report referenced below in accordance with the Australia Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009
- ☐ am willing to technically verify that the detailed Geotechnical Report referenced below has been prepared in accordance with the Australian Geomechanics Society's Landslide Risk Management Guidelines (AGS 2007) and the Geotechnical Risk Management Policy for Pittwater - 2009
- ☐ have examined the site and the proposed development in detail and have carried out a risk assessment in accordance with Section 6.0 of the Geotechnical Risk Management Policy for Pittwater - 2009. I confirm that the results of the risk assessment for the proposed development are in compliance with the Geotechnical Risk Management Policy for Pittwater - 2009 and further detailed geotechnical reporting is not required for the subject site.
- ☐ have examined the site and the proposed development/alteration in detail and I am of the opinion that the Development Application only involves Minor Development/Alteration that does not require a Geotechnical Report or Risk Assessment and hence my Report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements.
- ☐ have examined the site and the proposed development/alteration is separate from and is not affected by a Geotechnical Hazard and does not require a Geotechnical Report or Risk Assessment and hence my Report is in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009 requirements.
- ☐ have provided the coastal process and coastal forces analysis for inclusion in the Geotechnical Report

Geotechnical Report Details:

Report Title: Proposed Development

Report Date: 14 July 2023

Author: David Smith

Author's Company/Organisation: Douglas Partners Pty Ltd

Documentation which relate to or are relied upon in report preparation:

Preliminary Architectural Drawing Set 21-079 dated March 2023 prepared by MCHP Architects

I am aware that the above Geotechnical Report, prepared for the abovementioned site is to be submitted in support of a Development Application for this site and will be relied on by Pittwater Council as the basis for ensuring that the Geotechnical Risk Management aspects of the proposed development have been adequately addressed to achieve an "Acceptable Risk Management" level for the life of the structure, taken as at least 100 years unless otherwise stated and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk.

Signature ……[signature]……

Name Geoff McIntosh

Chartered Professional Status FIEAust, CPEng

Membership No. 52855

Company Douglas Partners Pty Ltd

GEOTECHNICAL RISK MANAGEMENT POLICY FOR PITTWATER
FORM NO. 1(a) - Checklist of Requirements For Geotechnical Risk Management Report for Development Application

Development Application for Rpyal Motor Yacht Club Broken Bay NSW
Name of Applicant
Address of site 46 Prince Alfred Parade, Newport

The following checklist covers the minimum requirements to be addressed in a Geotechnical Risk Management Geotechnical Report. This checklist is to accompany the Geotechnical Report and its certification (Form No. 1).

Geotechnical Report Details:

Report Title: Proposed Development
Report Date: 14 July 2023
Author: David Smith
Author's Company/Organisation: Douglas Partners

Please mark appropriate box

э Comprehensive site mapping conducted 14 February 2022
(date)
э Mapping details presented on contoured site plan with geomorphic mapping to a minimum scale of 1:200 (as appropriate)
э Subsurface investigation required
- э No Justification ……………………………………………...
- э Yes Date conducted 14 February 2022

э Geotechnical model developed and reported as an inferred subsurface type-section
э Geotechnical hazards identified
- э Above the site
- э On the site
- э Below the site
- э Beside the site

э Geotechnical hazards described and reported
э Risk assessment conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
- э Consequence analysis
- э Frequency analysis

э Risk calculation
э Risk assessment for property conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
э Risk assessment for loss of life conducted in accordance with the Geotechnical Risk Management Policy for Pittwater - 2009
э Assessed risks have been compared to "Acceptable Risk Management" criteria as defined in the Geotechnical Risk Management Policy for Pittwater - 2009
э Opinion has been provided that the design can achieve the "Acceptable Risk Management" criteria provided that the specified conditions are achieved.
э Design Life Adopted:
- э 100 years
- э Other ……………………………………………
specify

э Geotechnical Conditions to be applied to all four phases as described in the Geotechnical Risk Management Policy for Pittwater - 2009 have been specified
э Additional action to remove risk where reasonable and practical have been identified and included in the report.
э ~~Risk assessment within Bushfire Asset Protection Zone.~~

I am aware that Pittwater Council will rely on the Geotechnical Report, to which this checklist applies, as the basis for ensuring that the geotechnical risk management aspects of the proposal have been adequately addressed to achieve an "Acceptable Risk Management" level for the life of the structure, taken as at least 100 years unless otherwise stated, and justified in the Report and that reasonable and practical measures have been identified to remove foreseeable risk.

Signature ……………………………………………………………

Name Geoff McIntosh

Chartered Professional Status FIEAust, CPEng

Membership No. 52855

Company Douglas Partners