Douglas Partners
Geotechnics / Environment / Groundwater

# Report on
# Geotechnical Desktop Assessment

Proposed Dual Occupancy
10 Jennifer Street, Ryde

Prepared for
Clermont Holdings Pty Limited

Project 200861.00
July 2022

Integrated Practical Solutions

**Douglas Partners**

Geotechnics / Environment / Groundwater

## Document History

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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 | pp | 1 July 2022 |
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## Table of Contents

# Report on Geotechnical Desktop Assessment
# Proposed Dual Occupancy
# 10 Jennifer Street, Ryde

## 1. Introduction

This report presents the results of a geotechnical desktop assessment undertaken for a proposed dual occupancy building at 10 Jennifer Street, Ryde. The assessment was commissioned in an email dated 13 August 2021 by Ray Younes of Clermont Holdings Pty Limited ('Client') and was undertaken in accordance with Douglas Partners Pty Ltd (DP) proposal 200861.00.P.002.Rev0 dated 19 May 2021.

It is understood that a parcel of land at 6 and 10 Clermont Avenue, Ryde will be subdivided into three smaller lots, namely:

- Lot 1: 10 Jennifer Street, Ryde – located on the northwestern portion of 10 Clermont Avenue, with frontage along Jennifer Street;
- Lot 2: 12 Clermont Avenue, Ryde – located on the eastern side of 6 and 10 Clermont Avenue, with frontage along Clermont Avenue; and
- Lot 3: 8 Clermont Avenue, Ryde – located on the western and southwestern portions of 6 and 10 Clermont Avenue, with frontage along Clermont Avenue.

This report focuses on Lot 1 (10 Jennifer Street, Ryde). Separate reports will be prepared for Lot 2 (DP Report 200861.00.R.002.Rev0) and Lot 3 (DP Report 200861.00.R.003.Rev0).

The aim of the desktop assessment is to assess the subsurface conditions at the site from existing sources in order to provide information on the expected soil and rock profile, and likely opportunities and constraints in relation to geotechnical issues on the site, including preliminary advice on design and construction. It is understood that the geotechnical report is required to accompany the Development Application (DA) to Council for the site.

The assessment comprised a review of available information in the public domain and a previous geotechnical investigation carried out by DP for a larger parcel of land including five existing lots at 6 and 10 Clermont Avenue, and 7, 8 and 9 Jennifer Street, Ryde. The previous investigation (DP Project 85044.00) included drilling of three boreholes on the properties at 6 and 10 Clermont Avenue. No additional site inspection or intrusive investigations have been undertaken for this assessment.

## 2. Proposed Development

It is understood that the proposed development of the site will include the demolition of existing structures on site followed by the construction of a dual occupancy building. The building will comprise two-storey dwellings, a pool and a carport located on the ground floor for each dwelling, and a basement for storage purposes located under the living areas of each dwelling.

The supplied preliminary architectural drawings prepared by Studio_BD Architecture & Interiors (Reference: Drawing No. 782DA_A_04 Rev E (S4 55 Issue) issued on 20 June 2022) indicate that the proposed basement levels for the two dwellings will be at RL 81.5 m AHD (western dwelling) and RL 82.0 m AHD (eastern dwelling), which will require excavations to about 3 – 4 m below existing ground levels.

## 3. Previous Investigation

DP completed a geotechnical investigation in September 2015 for five properties at 6 and 10 Clermont Avenue and 7 – 9 Jennifer Street, Ryde (Reference: DP Report 85044.00.R.001.Rev0).

The investigations located on the site of 6 and 10 Clermont Avenue comprised drilling of two cored boreholes (BH1 and BH2) and one hand-augered borehole (BH4), dynamic cone penetrometer (DCP) testing adjacent to BH4 to assess the strength of the underlying soil profile, and installation of groundwater monitoring well in BH1.

The previous borehole locations are shown on Drawing 1.1 in Appendix B, together with the outlines of the proposed basement footprints.

The detailed subsurface conditions encountered in the boreholes are presented in the borehole logs in Appendix C. Notes defining descriptive terms and classification methods are also included in Appendix C.

The subsurface conditions encountered in the boreholes can be summarised as:

- **PAVEMENT:** asphalt pavement and roadbase (200 mm thick) in BH1 only;
- **FILL:** silty clay fill (BH1 and BH2) and silty sand fill/topsoil (BH4) with various minor components (i.e. gravel, sand, roots, ripped sandstone, brick and terracotta fragments) to depths of 0.4 m; overlying
- **RESIDUAL SOIL:** stiff and very stiff clay to depths of 1.3 m and 1.4 m; overlying
- **WEATHERED BEDROCK:** encountered in BH1 and BH2 and comprised initially extremely low strength shale, becoming very low strength below about 2 m depth. Low to high strength ironstone bands were present within the weathered bedrock profile.

Free groundwater was not observed in any of the boreholes during augering to depths of 1.3 m (BH4) and 1.5 m (BH1 and BH2), and the use of drilling fluid in the cored boreholes (BH1 and BH2) below 1.5 m depth prevented groundwater observations during rotary wash-boring and coring. A water level was measured in the groundwater monitoring well installed in BH1 in September 2015 at 1.5 m depth (RL 82.2 m AHD).

## 4. Site Description and Regional Geology

The site is irregularly shaped with an area of 927 m$^2$ and is located on the northwestern portion of 10 Clermont Avenue, Ryde. The existing structure at 10 Clermont Avenue comprises a two-storey nursing home constructed of brick and tiles, with surrounding grass and paved areas. The ground surface within the area where the proposed development will be built is at about RL 84 m to RL 85 m AHD.

Reference to the Sydney 1:100,000 Geological Series Sheet indicates that the site is underlain by Ashfield Shale of the Wianamatta Group. Ashfield Shale typically comprises black to grey shale and laminite. This is consistent with the results of the investigation carried out by DP within the vicinity of the site.

Reference to the Sydney 1:100,000 Soils Landscape Sheet indicates that the site is underlain by Glenorie soils, which is an erosional soil landscape and is characterised by topography of undulating rolling hills on Wianamatta Group shales, with local relief of 50 m to 80 m and slope gradients of 5% to 20%. The soil landscape is typically represented by narrow ridges, hillcrests and valleys. Glenorie soils typically have a high soil erosion hazard, exhibit localised areas of impermeable highly plastic subsoil and are moderately reactive.

## 5. Geotechnical Model

Based on the regional geology and previous investigations within the vicinity of the site, it is expected that the development area may be underlain by a thin layer of fill over residual clays and then weathered shale bedrock.

The residual clays are expected to be stiff and very stiff and may extend to depths of about 1.5 m. The residual clays are derived from weathering of the underlying Ashfield Shale and are expected to be moderately reactive.

The underlying Ashfield Shale is expected to be initially extremely low strength, then becoming very low strength below about 2 m depth. Low to high strength ironstone bands should be expected within the weathered shale profile. The rock is expected to typically grade to stronger and less weathered rock with depth.

Groundwater was measured in the groundwater well installed within borehole BH1 at a depth of 1.5 m (RL 82.8 m). It is likely, however, that the measured water is local seepage flowing through the soils above the top of rock and given the surrounding topography, the regional groundwater table is expected to be much deeper.

## 6. Comments

### 6.1 Site Classification

The site is expected to be underlain by moderately reactive clay soils to depths of about 1.5 m. In accordance with the guidelines given in Australian Standard AS2870-2011 Residential Slabs and Footings, an M classification is suggested for the site.

Class M sites are underlain by moderately reactive clay or silt and may experience moderate ground movements from changes in the moisture of the soils. Shallow footings on Class M sites would be expected to experience differential movements of up to 30 mm. If there are large trees on the site, then higher movements may occur.

### 6.2 Site Preparation

Any existing fill that is required to support structures and 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 guidelines.

- Strip any organic-rich topsoil from areas in which new engineered fill, structures and/or pavements are proposed;
- Excavate existing fill from areas in which new engineered fill, structures and/or pavements are proposed;
- Compact the exposed surface and proof-roll using a roller of 10 tonne deadweight (or equivalent) in the presence of a geotechnical engineer. Any areas exhibiting unacceptable movements during the proof-roll may require further rectification;
- Place fill in maximum 250 mm thick loose layers and compact to achieve a dry density ratio of between 98% and 102% relative to Standard compaction. The upper 0.5 m of pavement subgrade areas should be compacted to achieve a dry density ratio of between 100% and 102% relative to Standard compaction, with moisture contents maintained within 2% of Standard optimum moisture content;
- Poor trafficability should be expected across any unpaved areas of the sites following rainfall. A layer of granular product (e.g. roadbase, recycled crushed concrete, etc.) should be considered as the top layer of fill to improve trafficability on site during construction; and
- Density testing should be undertaken on fill in accordance with the requirements of AS3798-2007 Guidelines on Earthworks for Commercial and Residential Developments.

From a geotechnical perspective, the existing fill is likely to be suitable for re-use as engineered fill, provided that it is free of oversize particles (>100 mm) and deleterious material. The underlying residual clays are also likely to be suitable for re-use, however, as they are likely to be moderately reactive, it will be very important to control the moisture content of these soils during compaction. For moderately (and highly) reactive soils, it is recommended that the soils be compacted at moisture contents between 100% and 102% of the Standard optimum moisture content to reduce the risk of swell.

The suitability of reusing site-won fill and natural soil should also be considered from a contamination perspective.

If fill is imported to the site, then the engineering properties (e.g. plasticity, reactivity, etc.) should ideally be equivalent, or superior, to the existing materials on site.

## 6.3 Excavation

Excavation for the basement level to depths of about 3 – 4 m is expected to be through fill, residual soil and weathered rock with some low to high strength ironstone bands. Excavation in fill, soil and extremely low to very low strength rock should be readily achievable using conventional earthmoving equipment such as hydraulic excavators with bucket attachments.

Excavation in the low strength and stronger bands may require the use of ripping equipment or hydraulic rock hammers. It is noted that the stronger rock within the anticipated excavation zone appears to be present in bands which may aid extraction.

The use of rock hammers will cause vibrations that could possibly result in damage to nearby structures. It is suggested that vibrations be limited to a peak component particle velocity (PPVi) of 8 mm/s at the foundation level of the adjacent buildings to protect the architectural features of the buildings and to reduce discomfort for the occupants. A site-specific vibration monitoring trial may be required to determine vibration attenuation once excavation plant and methods have been finalised.

It should be noted that any off-site disposal of spoil will generally require assessment for re-use or classification in accordance with current Waste Classification Guidelines (NSW EPA, 2014).

## 6.4 Excavation Support

### 6.4.1 General

Vertical excavations in fill, soil and weathered rock are not expected to be stable in either the short or long-term. Where space permits, temporary batters of 1(H):1(V) or flatter could be used for the sides of the excavation. If there is insufficient space, shoring support will be required from the ground surface down to the bulk excavation level.

Soldier piles with infill reinforced shotcrete panels are commonly used to support excavations in residual clays and shale. The soldier piles would generally be spaced at about 2 m to 3 m centres and should be founded at least two pile diameters below the lowest excavation level (both bulk and detailed) adjacent to the pile location. Shotcreting will be needed over the full excavation depth and should be undertaken in maximum 2.5 m ‘drops’ in order to reduce the risk of local slippages and collapse between soldier piles. Temporary ground anchors may also be required to prevent excessive lateral deformation of shoring or retaining walls. For the permanent situation, the basement structure should be designed to provide the required lateral support to the perimeter excavation once any temporary anchors are de-stressed.

### 6.4.2 Design

Excavation faces retained either temporarily or permanently will be subjected to earth pressures from the ground surface down to the bulk excavation level. Table 1 outlines material and strength parameters that may be used for the preliminary design of excavation support structures.

**Table 1: Material and Strength Parameters for Excavation Support Structures**

| Material | Bulk Density ($kN/m^3$) | Coefficient of Active Earth Pressure ($K_a$) | Coefficient of Earth Pressure at Rest ($K_o$) | Ultimate Passive Earth Pressure (kPa) |
|---|---|---|---|---|
| Fill | 18 | 0.4 | 0.6 | - |
| Residual Soil | 20 | 0.3 | 0.45 | - |
| Extremely Low to Very Low Strength Shale | 22 | 0.25[1] | 0.4[1] | 400[2] |

Notes: [1] Unless unfavourably jointed

[2] Only below bulk/detailed excavation level and where jointing is favourable

The ultimate passive pressure given in Table 1 should incorporate a suitable factor of safety to limit deflection. For rocks, jointing may be a controlling factor and should be considered.

Rock sockets below the bulk excavation level for the purpose of passive restraint should have a minimum length of two pile diameters below the lowest level of any nearby excavation (including any detailed excavations).

### 6.4.3 Ground Anchors

If necessary, the use of declined tie-back (ground) anchors may be used for the lateral restraint of perimeter piled walls. Such ground anchors should be declined below the horizontal to allow anchorage into the stronger bedrock at depth. The design of temporary ground anchors for the support of piled wall systems may be carried out using the allowable average bond stresses at the grout-rock interface given in Table 2.

**Table 2: Allowable Bond Stresses for Anchor Design**

| Material Description | Allowable Bond Stress (kPa) |
|---|---|
| Extremely Low to Very Low Strength Shale | 50 |

Ground anchors should be designed to have a free length equal to their height above the base of the excavation and have a minimum 3 m bond length. After installation they should be proof loaded to 125% of the design working load and locked-off at no higher than 80% of the working load. Periodic checks should be carried out during the construction phase to ensure that the lock-off load is maintained and not lost due to creep effects or other causes.

The parameter given in Table 2 assumes that the anchor holes are clean and adequately flushed, with grouting and other installation procedures carried out carefully and in accordance with good anchoring practice. Careful installation and close supervision by a geotechnical specialist may allow increased bond stresses to be adopted during construction, subject to testing.

In normal circumstances, the building will restrain the basement excavation over the long term and therefore ground anchors are expected to be temporary only. The use of permanent anchors would require careful attention to corrosion protection. Further advice on design and specification should be sought if permanent anchors are to be employed at this site.

It will be necessary to obtain permission from neighbouring landowners prior to installing anchors that will extend beyond the perimeter of the site. In addition, care should be taken to avoid damaging buried services, pipes and subsurface structures during anchor installation.

## 6.5 Groundwater

Groundwater was measured in the groundwater well installed within borehole BH1 at a depth of 1.5 m (RL 82.8 m). This, however, is likely to be seepage and the regional groundwater table is expected to be much deeper.

It is anticipated that there will be some seepage into the excavation through the soils and along strata boundaries. Based on experience, it is anticipated that the volume of seepage and flow rates will be very low and any seepage should be able to be controlled using a sub-floor drainage and collection system in the basement level. Seepage through Wianamatta Group shales sometimes results in iron precipitates which have the potential to block drainage material and additional precautions (e.g. ’wash-out points’ and ‘rodding points’, etc.) should be taken to avoid blocking of the drains over the medium to longer term.

Unless a gravity system can be designed, pumps will be required to periodically remove stored water from the sub-floor drainage system for the basement. Pumps may also be needed to remove seepage from bored pile excavations prior to the placement of concrete, if bored piles are used for shoring support.

## 6.6 Foundations

The proposed bulk excavation works are expected to expose extremely low and very low strength shale bedrock across the site. Spread footings (i.e. pad or strip footings) within the excavation should be suitable for supporting the proposed building loads and could be designed on the basis of an allowable bearing pressure provided in Table 3.

Bored piles used for shoring support could also be used to support structural loads providing they are founded below the bulk excavation level. Bored piles may be proportioned on the basis of the design parameters provided in Table 3.

**Table 3: Design Parameters for Bored Piles and Spread Footings**

| Material Description | Allowable End Bearing Pressure (kPa) | Allowable Shaft Adhesion (kPa) |
|---|---|---|
| Extremely Low to Very Low Strength Shale | 700 | 50 |

Settlement of a footing or pile is dependent on the loads applied to the footing and the foundation conditions. The total (long-term) settlement of a footing designed using the allowable parameters provided in Table 3 should be less than 1% of the footing width or pile diameter upon applicable of the design dead load.

In order to reduce the risks associated with differential settlements, it is strongly recommended that all foundations bear on material of similar strength.

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.

## 7. Limitations

Douglas Partners (DP) has prepared this report for this project at 10 Jennifer Street, Ryde in accordance with DP's proposal 200861.00.P.002.Rev0 dated 19 May 2021 and acceptance received from Ray Younes of Clermont Holdings Pty Limited dated 13 August 2021. The work was carried out under DP's Conditions of Engagement. This report is provided for the exclusive use of Clermont Holdings Pty Limited 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.

The results provided in the report are indicative of the sub-surface conditions on the site of 6 and 10 Clermont Avenue, Ryde 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 the previous investigation carried out at 6 and 10 Clermont Avenue, Ryde. 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.

**Douglas Partners Pty Ltd**

# Appendix A

## About This Report

# 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.

## 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

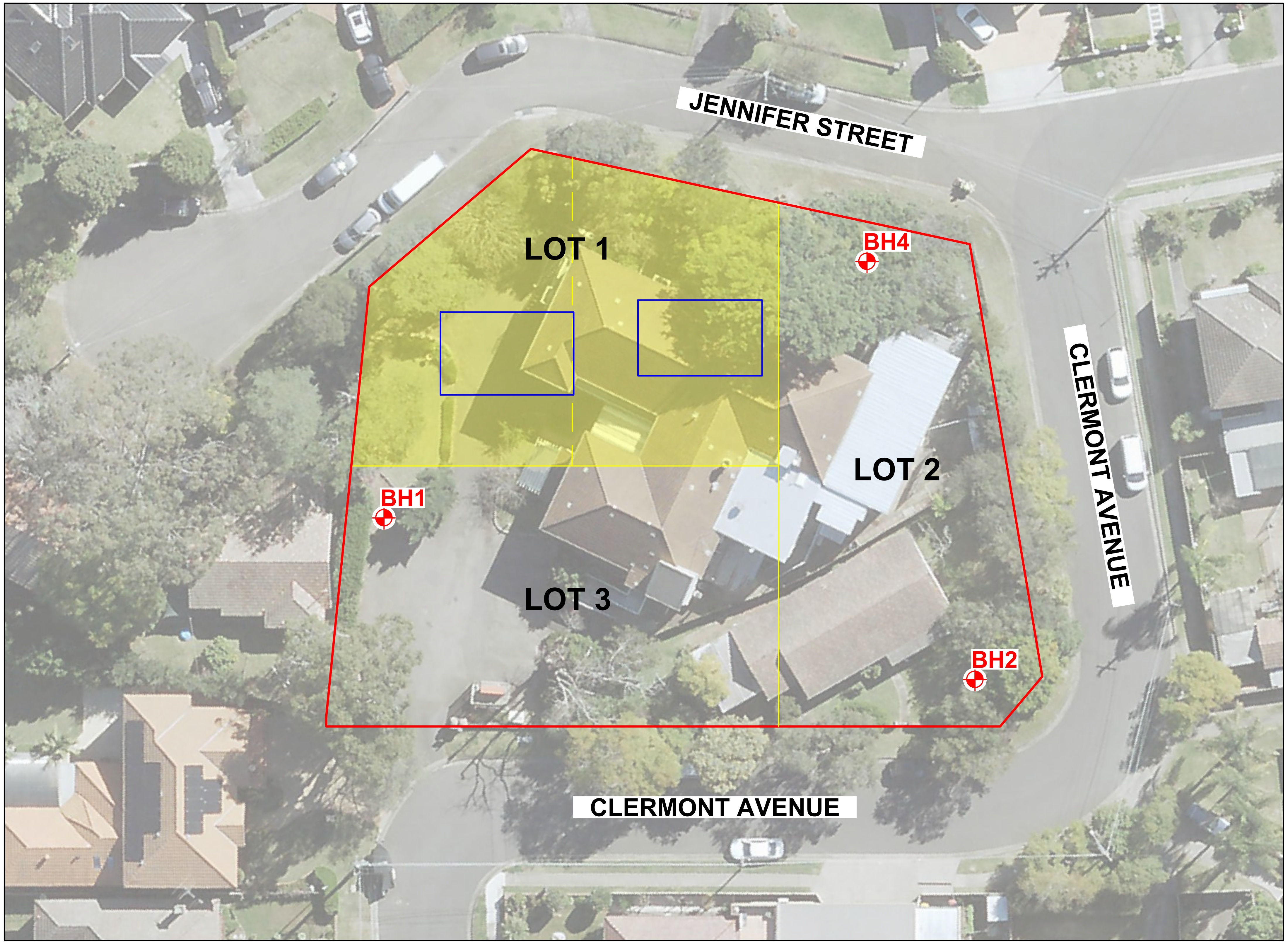
Where information obtained from this report is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate 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.

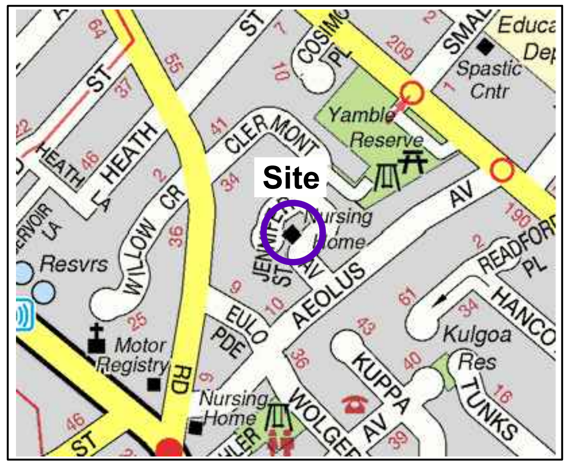
## Site Inspection

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

## Drawing





**Locality Plan**

NOTE:
1. Base image from Metromap.com dated 30 July 2021

0 2 4 6 8 10 15 20 30 40m
1:400 @ A3

## LEGEND

| | |
|---|---|
| 🟥 | Boundary of Full Subdivision<br>(6 and 10 Clermont Avenue, Ryde) |
| 🟦 | Basement Footprint (Reference: Studio_BD Architecture & Interiors Drawing No. 782DA_A_04 Rev B issued 31 March 2022) |
| ⊕ | Borehole Location (Reference: DP Report 85044.00.R.001.Rev0 dated September 2015) |

**Douglas Partners**
*Geotechnics / Environment / Groundwater*

CLIENT: Clermont Holdings Pty Limited
OFFICE: Sydney
DRAWN BY: IT
SCALE: 1:400 @ A3
DATE: 07.04.2022

TITLE: **Site and Test Location Plan**
**Proposed Dual Occupancy**
**10 Jennifer Street, Ryde**

PROJECT No: 200861.00
DRAWING No: 1.1
REVISION: 1

# Appendix C

## Field Work Results from Previous Investigation

# Sampling Methods

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## 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.

# Soil Descriptions

## Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard AS 1726, Geotechnical Site Investigations Code. In general, the descriptions include strength or density, colour, structure, soil or rock type and inclusions.

## 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 | 20 - 63 |
| Medium gravel | 6 - 20 |
| Fine gravel | 2.36 - 6 |
| Coarse sand | 0.6 - 2.36 |
| Medium sand | 0.2 - 0.6 |
| Fine sand | 0.075 - 0.2 |

The proportions of secondary constituents of soils are described as:

| Term | Proportion | Example |
|---|---|---|
| And | Specify | Clay (60%) and Sand (40%) |
| Adjective | 20 - 35% | Sandy Clay |
| Slightly | 12 - 20% | Slightly Sandy Clay |
| With some | 5 - 12% | Clay with some sand |
| With a trace of | 0 - 5% | Clay with a trace of sand |

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

## 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 |

## 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 | SPT N value | CPT qc value (MPa) |
|---|---|---|---|
| Very loose | vl | <4 | <2 |
| Loose | l | 4 - 10 | 2 -5 |
| Medium dense | md | 10 - 30 | 5 - 15 |
| Dense | d | 30 - 50 | 15 - 25 |
| Very dense | vd | >50 | >25 |

# *Soil Descriptions*

## 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;
- Transported soils - formed somewhere else and transported by nature to the site; or
- Filling - moved by man.

Transported soils may be further subdivided into:

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

# Rock Descriptions

## Rock Strength

Rock strength is defined by the Point Load Strength Index ($Is_{(50)}$) and 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 test procedure is described by Australian Standard 4133.4.1 - 1993. The terms used to describe rock strength are as follows:

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

* Assumes a ratio of 20:1 for UCS to $Is_{(50)}$

## Degree of Weathering

The degree of weathering of rock is classified as follows:

| Term | Abbreviation | Description |
|---|---|---|
| Extremely weathered | EW | Rock substance has soil properties, i.e. it can be remoulded and classified as a soil but the texture of the original rock is still evident. |
| Highly weathered | HW | Limonite staining or bleaching affects whole of rock substance and other signs of decomposition are evident. Porosity and strength may be altered as a result of iron leaching or deposition. Colour and strength of original fresh rock is not recognisable |
| Moderately weathered | MW | Staining and discolouration of rock substance has taken place |
| Slightly weathered | SW | Rock substance is slightly discoloured but shows little or no change of strength from fresh rock |
| Fresh stained | Fs | Rock substance unaffected by weathering but staining visible along defects |
| Fresh | Fr | No signs of decomposition or staining |

## 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 some fragments |
| Fractured | Core lengths of 40-200 mm with some shorter and longer sections |
| Slightly Fractured | Core lengths of 200-1000 mm with some shorter and loner sections |
| Unbroken | Core lengths mostly > 1000 mm |

# Rock Descriptions

## Rock Quality Designation

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

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

where 'sound' rock is assessed to be rock of low strength or better. 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 |

# Symbols & Abbreviations

## 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 |

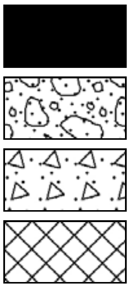
### Other

| | |
|---|---|
| fg | fragmented |
| bnd | band |
| qtz | quartz |

# *Symbols & Abbreviations*

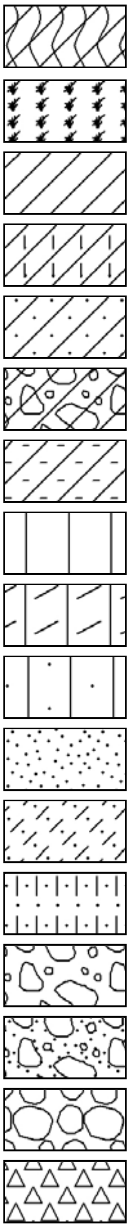
## Graphic Symbols for Soil and Rock

### General



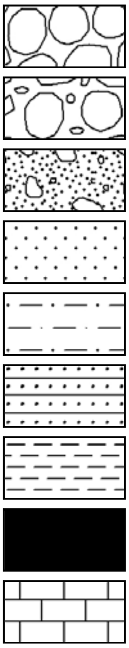
- Asphalt
- Road base
- Concrete
- Filling

### Soils



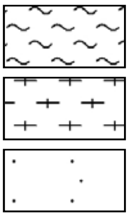
- Topsoil
- Peat
- Clay
- Silty clay
- Sandy clay
- Gravelly clay
- Shaly clay
- Silt
- Clayey silt
- Sandy silt
- Sand
- Clayey sand
- Silty sand
- Gravel
- Sandy gravel
- Cobbles, boulders
- Talus

### Sedimentary Rocks



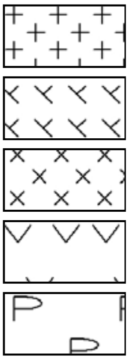
- Boulder conglomerate
- Conglomerate
- Conglomeratic sandstone
- Sandstone
- Siltstone
- Laminite
- Mudstone, claystone, shale
- Coal
- Limestone

### Metamorphic Rocks



- Slate, phyllite, schist
- Gneiss
- Quartzite

### Igneous Rocks



- Granite
- Dolerite, basalt, andesite
- Dacite, epidote
- Tuff, breccia
- Porphyry

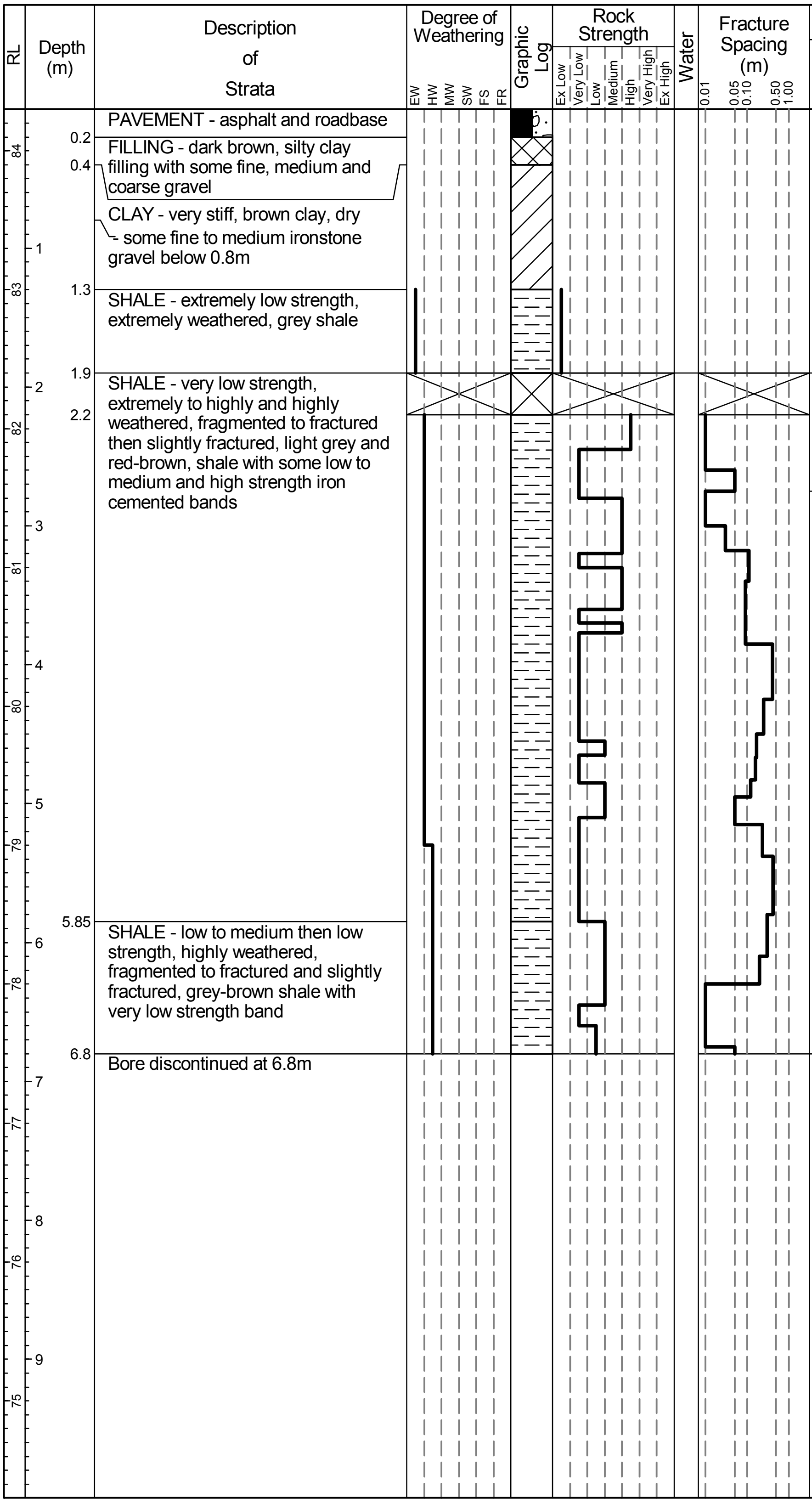
# BOREHOLE LOG

**CLIENT:** Clermont Aged Care Pty Ltd
**PROJECT:** Proposed Aged Care Facility
**LOCATION:** 6 & 14 Clermont Avenue and
7 - 9 Jennifer Street, Ryde

**SURFACE LEVEL:** 84.3 AHD
**EASTING:**
**NORTHING:**
**DIP/AZIMUTH:** 90°/--

**BORE No:** 1
**PROJECT No:** 85044
**DATE:** 31/8/2015
**SHEET** 1 OF 1

| Depth (m) | Description of Strata | Discontinuities (B - Bedding, J - Joint, S - Shear, F - Fault) | Type | Core Rec. % | RQD % | Test Results & Comments |
|---|---|---|---|---|---|---|
| | PAVEMENT - asphalt and roadbase | | A | | | |
| 0.2 | FILLING - dark brown, silty clay filling with some fine, medium and coarse gravel | | A | | | |
| 0.4 | CLAY - very stiff, brown clay, dry - some fine to medium ironstone gravel below 0.8m | Note: Unless otherwise stated, rock is fractured along smooth planar bedding dipping 0°- 10° | A<br>S | | | 4,6,24<br>N = 30 |
| 1.3 | SHALE - extremely low strength, extremely weathered, grey shale | | | | | |
| 1.9 | SHALE - very low strength, extremely to highly and highly weathered, fragmented to fractured then slightly fractured, light grey and red-brown, shale with some low to medium and high strength iron cemented bands | 1.9m: CORE LOSS: 300mm | C | 75 | 0 | |
| 2.2 | | 2.2-2.6m: fg, fe<br>2.7m: B0°, fe<br>2.75-3.0m: fg, fe<br>3.0-3.18m: B (x3) 0°, fe | | | | PL(A) = 1.4<br>PL(A) = 1 |
| | | 3.4m: J60°, pl, ro, fe<br>3.85m: B0°, fe<br>4.25m: J30°, pl, ro, fe<br>4.5m: J45°, pl, sm, cly<br>4.83-4.9m: J75°, pl, ro, cln | C | 100 | 20 | PL(A) = 0.3 |
| | | 5.38m: B20°, pl, ro, fe | C | 100 | 40 | |
| 5.85 | SHALE - low to medium then low strength, highly weathered, fragmented to fractured and slightly fractured, grey-brown shale with very low strength band | 5.8-5.9m: J, sv, pl, ro, cln<br>6.1 & 6.3m: B0°, fe | | | | PL(A) = 0.3<br>PL(A) = 0.2 |
| 6.8 | Bore discontinued at 6.8m | | | | | |



**RIG:** Bobcat **DRILLER:** SY **LOGGED:** MB/SI **CASING:** HW to 1.7m

**TYPE OF BORING:** Solid flight auger to 1.5m; Rotary to 1.9m; NMLC-Coring to 6.8m

**WATER OBSERVATIONS:** No free groundwater observed whilst augering

**REMARKS:** Standpipe installed to 6.8m (screen 1.0-6.8m; gravel 0.8-6.8m; bentonite 0.3-0.8m; backfill to GL)

**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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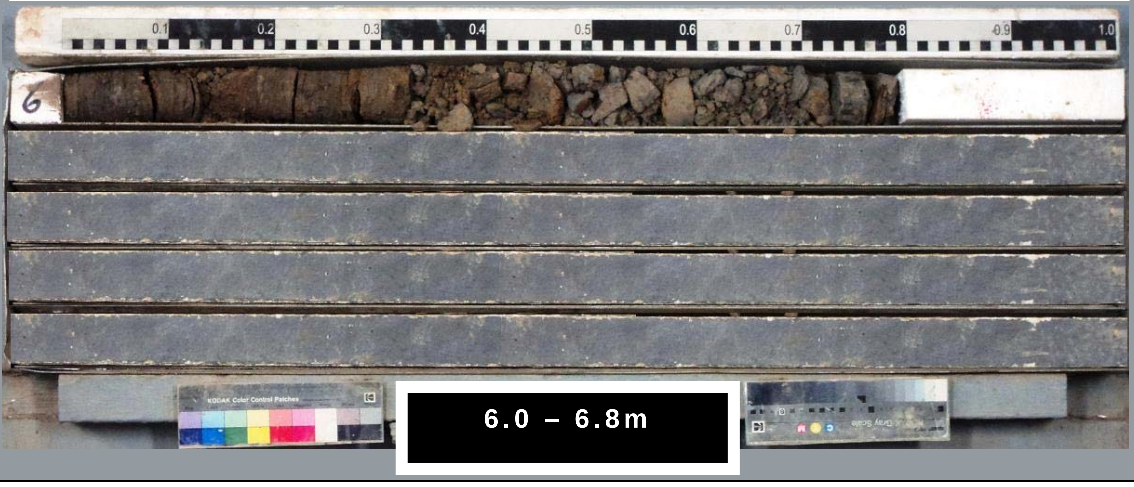
**BORE 1 PROJECT 85044 SEP 2015**

**1.9 – 6.0m**

**DOUGLAS PARTNERS PTY LTD**

**PROPOSED AGED CARE FACILITY - RYDE**

**BORE 1 PROJECT 85044 SEP 2015**



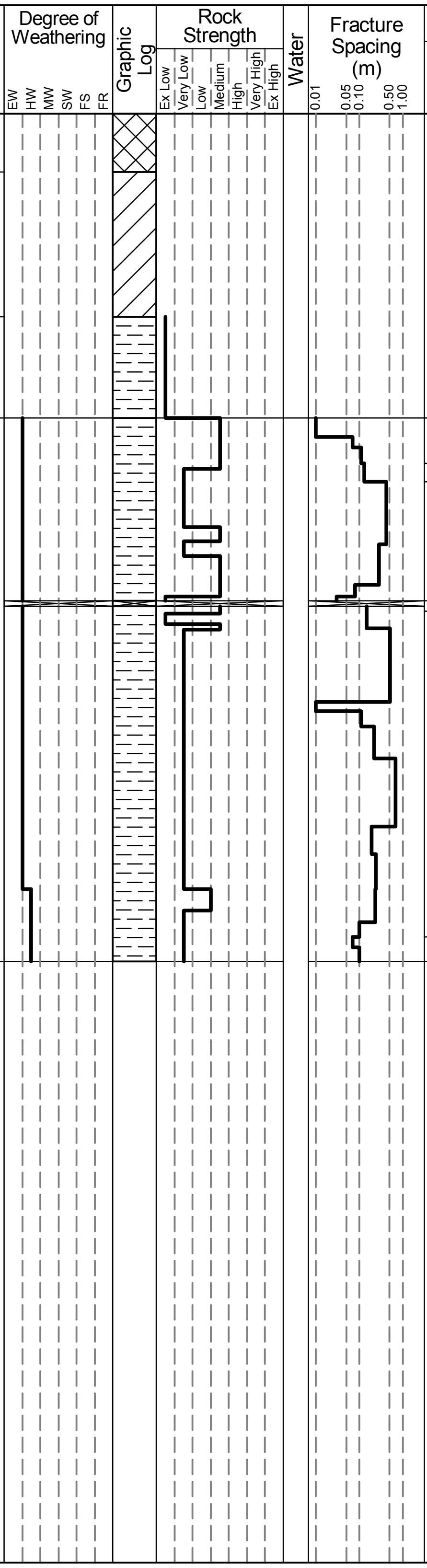
**6.0 – 6.8m**

# BOREHOLE LOG

**CLIENT:** Clermont Aged Care Pty Ltd
**PROJECT:** Proposed Aged Care Facility
**LOCATION:** 6 & 14 Clermont Avenue and
7 - 9 Jennifer Street, Ryde

**SURFACE LEVEL:** 85.0 AHD
**EASTING:**
**NORTHING:**
**DIP/AZIMUTH:** 90°/--

**BORE No:** 2
**PROJECT No:** 85044
**DATE:** 1/9/2015
**SHEET** 1 OF 1



| RL | Depth (m) | Description of Strata | Discontinuities<br>B - Bedding J - Joint<br>S - Shear F - Fault | Sampling & In Situ Testing: Type | Core Rec. % | RQD % | Test Results & Comments |
|---|---|---|---|---|---|---|---|
| 85 | | FILLING - dark grey, silty clay filling with some fine sand and gravel, moist | | A | | | |
| | 0.4 | CLAY - stiff, red-brown clay with ironstone gravel, moist | | A | | | |
| 84 | 1 | 1.0m: becoming very stiff | Note: Unless otherwise stated, rock is fractured along smooth planar bedding dipping 0°- 10° | A<br>S | | | 2,17,25/100mm refusal |
| | 1.4 | SHALE - extremely low strength, light grey-brown shale with ironstone bands | | | | | |
| 83 | 2 | | | | | | |
| | 2.1 | SHALE - very low strength, extremely to highly then highly weathered, fragmented to fractured, light grey-brown to red-brown, shale with low to medium and medium strength ironcemented bands | 2.1-2.23m: fg<br>2.3m: B0°, cly<br>2.41m: J30° & 75°, st, ti<br>2.54m: J60° & 85°, st, ro, fe<br>2.97m: B10°, cly<br>3.25m: J45°, pl, ro, cly | C | 100 | 30 | PL(A) = 0.6 |
| 82 | 3 | | | | | | |
| | 3.4 | | 3.36m: CORE LOSS: 40mm<br>3.43-3.5m: cly<br>4.06-4.12m: fg, fe<br>4.23m: J35°, pl, ro, fe<br>4.45 & 4.92m: J70°, he/cly<br>5.11m: J70°, pl, ro, cln<br>5.55 & 5.65m: B10°, fe<br>5.68m: J70°, pl, sm, fe | C | 98 | 0 | PL(A) = 0.1<br>PL(A) = 0.1<br>PL(A) = 0.3 |
| 81 | 4 | | | | | | |
| 80 | 5 | | | | | | |
| | 5.85 | Bore discontinued at 5.85m | | | | | |
| 79 | 6 | | | | | | |
| 78 | 7 | | | | | | |
| 77 | 8 | | | | | | |
| 76 | 9 | | | | | | |

**RIG:** Bobcat **DRILLER:** SY **LOGGED:** SI **CASING:** HW to 1.5m
**TYPE OF BORING:** Solid flight auger to 1.5m; Rotary to 2.1m; NMLC-Coring to 5.85m
**WATER OBSERVATIONS:** No free groundwater observed whilst augering
**REMARKS:**

**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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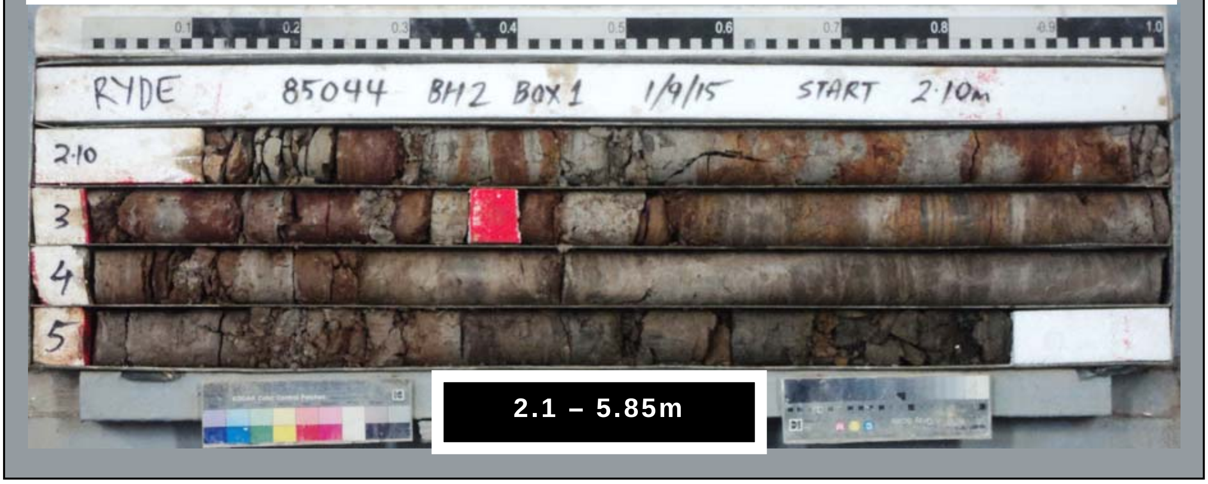
**PROPOSED AGED CARE FACILITY - RYDE**

**BORE 2 PROJECT 85044 SEP 2015**



**2.1 – 5.85m**

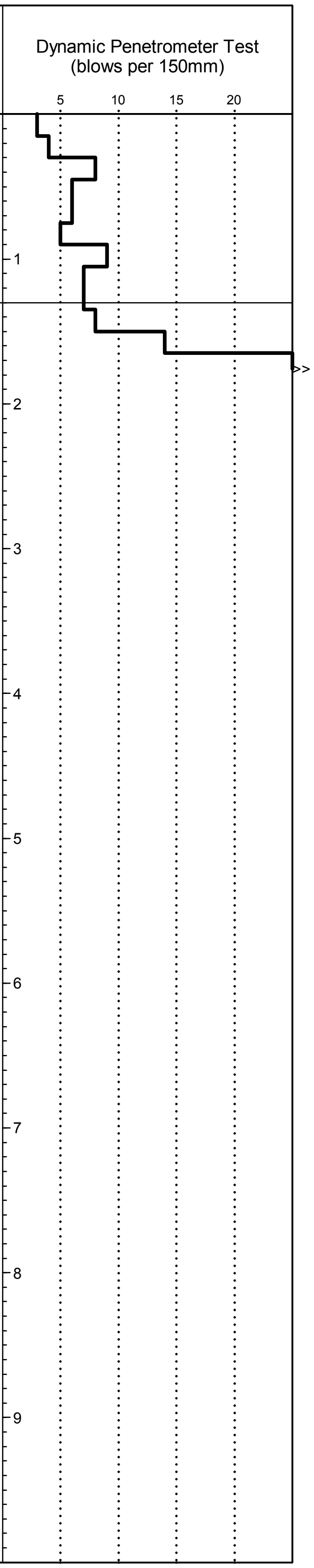
# BOREHOLE LOG

**CLIENT:** Clermont Aged Care Pty Ltd
**PROJECT:** Proposed Aged Care Facility
**LOCATION:** 6 & 14 Clermont Avenue and
7 - 9 Jennifer Street, Ryde

**SURFACE LEVEL:** 85.3 AHD
**EASTING:**
**NORTHING:**
**DIP/AZIMUTH:** 90°/--

**BORE No:** 4
**PROJECT No:** 85044
**DATE:** 31/8/2015
**SHEET** 1 OF 1

| RL | Depth (m) | Description of Strata | Graphic Log | Type | Depth | Sample | Results & Comments | Water |
|---|---|---|---|---|---|---|---|---|
| | 0.4 | FILLING - poorly compacted, dark grey, silty, medium sand (topsoil) filling with some roots, ripped sandstone, gravel, brick and terracotta fragments, damp<br>- becoming brown with some clay below 0.25m | ☒ | A | 0.1 | | | |
| | 1.3 | CLAY - stiff to very stiff, brown, clay with some fine to medium ironstone gravel, dry | ▨ | A<br>A | 0.5<br>0.9 | | | |
| | | Bore discontinued at 1.3m<br>- auger refusal on ironstone band | | | | | | |



**RIG:** Hand auger **DRILLER:** MB **LOGGED:** MB **CASING:** Uncased

**TYPE OF BORING:** Hand auger to 1.3m

**WATER OBSERVATIONS:** No free groundwater observed

**REMARKS:**

☐ Sand Penetrometer AS1289.6.3.3
☒ Cone Penetrometer AS1289.6.3.2

**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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