Apperly Village - Nelson Bay Road, Fern Bay

Preliminary Dewatering Management Plan

Principle Living Pty Ltd

Reference: SYDGE369539-AA

11 November 2024

APPERLY VILLAGE - NELSON BAY ROAD, FERN BAY

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11 November 2024

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EXECUTIVE SUMMARY[1]

Project Overview
Apperly Village, a senior living development by Principle Living Pty Ltd, requires a Preliminary Dewatering Management Plan (DMP) as part of its construction along Nelson Bay Road, Fern Bay, NSW. Due to groundwater presence within the proposed basement excavation areas, dewatering activities are necessary to create a suitable construction environment. This DMP is developed by Tetra Tech Coffey and provides initial strategies for groundwater management, potential impacts, and required further investigations to refine the dewatering process.

Site Conditions and Geology
The site, spanning approximately 6.5 hectares, sits within the Tomago-Tomaree-Stockton Sandbeds aquifer. Soils in this area primarily consist of loose to dense sands with underlying denser sand layers. Preliminary testing indicated groundwater levels may be encountered within the basement excavation depth for Apartments 1 and 2, warranting site-specific dewatering measures.

Hydrogeological Assessment and Groundwater Take
An assessment using Seep/W modelling estimated inflow rates of 28 L/s for Apartment 1 and 7.7 L/s for Apartment 2. Installing a cut-off wall around the excavation perimeter could reduce inflows by 30-40%, while a deeper cut-off wall could further reduce inflows. Additional site-specific hydraulic conductivity tests are recommended to refine these estimates along with details on proposed construction methodology. A Water Access Licence would likely be required for the development.

Groundwater Impact Assessment
The DMP assessed potential impacts, including:

- **Acid Sulfate Soils (ASS):** Identified at depths that may oxidize upon exposure to air when dewatering, leading to acid generation which could impact groundwater and local ecosystems.
- **Drawdown Effects:** Which are projected to have negligible effects on nearby users and groundwater-dependent ecosystems due to localized drawdown.
- **Salinity and Contamination Risks:** Risks of saltwater intrusion and aquifer contamination are considered low, though the oxidation of ASS could pose a moderate risk for groundwater quality.

Recommendations for Further Investigations
The report suggests installing monitoring bores and conducting aquifer testing and ongoing groundwater monitoring to ensure compliance with quality standards and validate the findings of this report. Additionally, preliminary water treatment measures are proposed before discharge into the local stormwater system, with trigger levels set for groundwater levels, quality, and volume to monitor potential environmental impact.

Conclusion
Temporary dewatering is considered feasible with minimal anticipated impact on the broader groundwater system, provided additional investigations and real-time monitoring are implemented. Further data will enable refined groundwater inflow projections prior to the commencement of dewatering.

[1] This executive summary must be read in the context of the full report and the attached limitations.

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1. INTRODUCTION

This report provides a preliminary Dewatering Management Plan (DMP) for the proposed Apperly Village development by Principle Living Pty Ltd. The proposed development is located within the south-western portion of Newcastle Golf Club along Nelson Bay Road, Fern Bay NSW (herein referred to as the 'site'). This assessment was carried out by Tetra Tech Coffey Pty Ltd (Tetra Tech) and commissioned by Principle Living in general accordance with our proposal (Ref. SYDGE369539-P01 Rev.00, dated 16 September 2024).

We understand that the proposed development will establish a new seniors living village located within the Newcastle Golf Club comprising of three apartment buildings and a community centre, along with a series of single-storey dwellings and associated internal roads and other structures. The apartment buildings will comprise a single storey basement which may intersect the existing groundwater table within the site area, possibly requiring dewatering works during construction of the basement.

Tetra Tech understands that, in the processing of lodging a development application, Port Stephens Council (PSC) have issued a Request for Information (RFI) which requires the submission of a DMP to support the application (Clause 9 (b)).

At present, a site-specific groundwater investigation has not been undertaken at the site to inform of groundwater levels, permeability values, or groundwater quality data to support construction water discharge. Tetra Tech understands there are currently plans for the installation of five groundwater monitoring wells and groundwater quality testing of water samples from these wells, as described in RCA (2024). Further groundwater investigations (including monitoring of groundwater levels and hydraulic conductivity testing) would likely be required to obtain the relevant data to prepare a DMP and address the requirements from Clause 9 (b) of PSC's RFI.

In lieu of site-specific data, this preliminary DMP provides this assessment based on the available geotechnical data, information obtained from literature and Tetra Tech's previous experience with groundwater conditions at similar sites. This report includes a preliminary groundwater inflow assessment and refers to a proposed groundwater investigation program that may provide the relevant additional data to meet the requirements of the RFI.

2. BACKGROUND INFORMATION

2.1 SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The site comprises a portion of the Newcastle Golf Club located at 4 and 4A Vardon Road, Fern Bay NSW situated within Lot 4 of DP 823114. The proposed seniors living development is situated over an area of approximately 6.5 hectares within the south-western portion of the golf course adjacent to Nelson Bay Road. The site is approximately 630m long and 150m wide. The location of the site is presented in Figure 1 and Figure 2 below.

The site is bounded by the existing golf course to the north and east, Nelson Bay Road to the west and residential dwellings and an existing carpark to the south. To the west of Nelson Bay Road lies further residential dwelling and mangroves which backs on to the northern channel of the Hunter River. East of the golf course lies Stockton Beach and the Pacific Ocean.

A review of the survey plans prepared by Delfs Lascelles Consulting Surveyors (Ref. 21493, Rev B, dated 21 October 2021) indicated that the southern portion of the site between the existing bitumen carpark and Nelson Bay Road is situated at a higher elevation of around 11m AHD, before falling to around 2.5m AHD at Nelson Bay Road and 8m AHD at the carpark. The site then gently grades to the north across undulating terrain and bushland, reducing in elevation to approximately 1m AHD along the northern site boundary.

Figure 1. Overall site locality

Figure 2. General locality of Senior Living Development

The architectural drawings prepared by EJE Architecture (Ref. 11589, Rev E, dated 19 July 2024) indicate that the proposed development will comprise:

- Three apartment buildings up to 5 stories tall, each with a single level basement
- A series of single storey dwellings along the eastern boundary.
- A two-storey community centre within the central area.
- New paved roads and drainage structures with access from Nelson Bay Road.

Figure 3 provides an overview of the proposed development and location of the various components along with basement floor levels (BFL).

Figure 3. Proposed development - apartment basement floor levels

Based on the ground floor and basement levels provided in the plans along with an estimate for the over excavation required for construction of the basement slab, the total Bulk Excavation Level (BEL) and depth of excavation for each apartment building is presented in Table 1.

Tetra Tech understands the construction of the development will be undertaken in stages with Apartment 1 being constructed first, followed by Apartment 3, and then Apartment 2.

Table 1. Anticipated depth of excavation for apartment buildings

Apartment No.	Ground floor level (mAHD)	Basement floor level (mAHD)	Estimated Over-excavation for basement slab and drainage (m)	Assumed bulk excavation level (mAHD)
1	3.8	0.8	0.5	0.3
2	4.7	1.7	0.5	1.2
3	6.3	3.3	0.5	2.8

2.2 GEOLOGY

The NSW seamless geology map, published on the MinView website [1] indicates that the site is situated over the following geological units:

- The southern half of the site is situated within (QH_bd) Quaternary aged coastal deposits comprising marine-deposited and aeolian-reworked coastal sand dunes.
- The northern half of the site is situated within (QH_er) Quaternary aged estuarine deposits comprising fine to medium grained sands with gravels, silt, clay and shell material.

Figure 4 provides an aerial image of the site with the local geological boundaries.

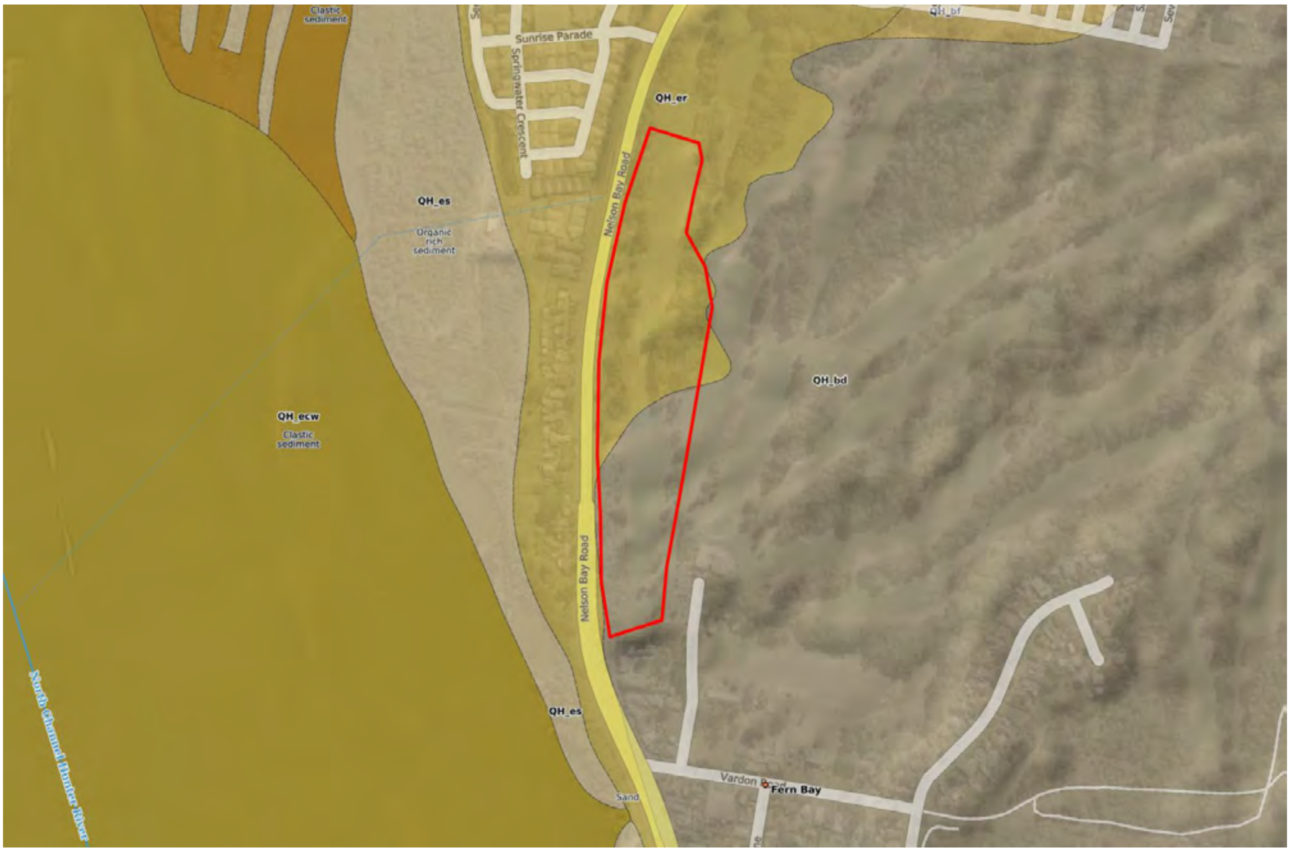


Figure 4. Published geology

2.3 ACID SULFATE SOILS

The SEED Central Resource for Sharing and Enabling Environmental Data in NSW [2] provides acid sulfate soil risk maps across the site area which is presented in Figure 5 and summarised as follows:

- An aeolian dune landform area at the southern end with a low probability of acid sulfate soils at greater than 4m below the ground surface over most of the golf course area and the southern end of the proposed seniors living development area.
- Disturbed terrain (e.g., reclaimed low lying swamps or areas which have undergone heavy ground disturbance through general urban development) requiring soil investigation to assess the presence of acid sulfate soils over the northern end of the proposed seniors living development area.
- Estuarine landform comprising mangroves, swamps and supratidal flat area further to the east with a high probability of acid sulfate soils within 1-2m of ground surface.

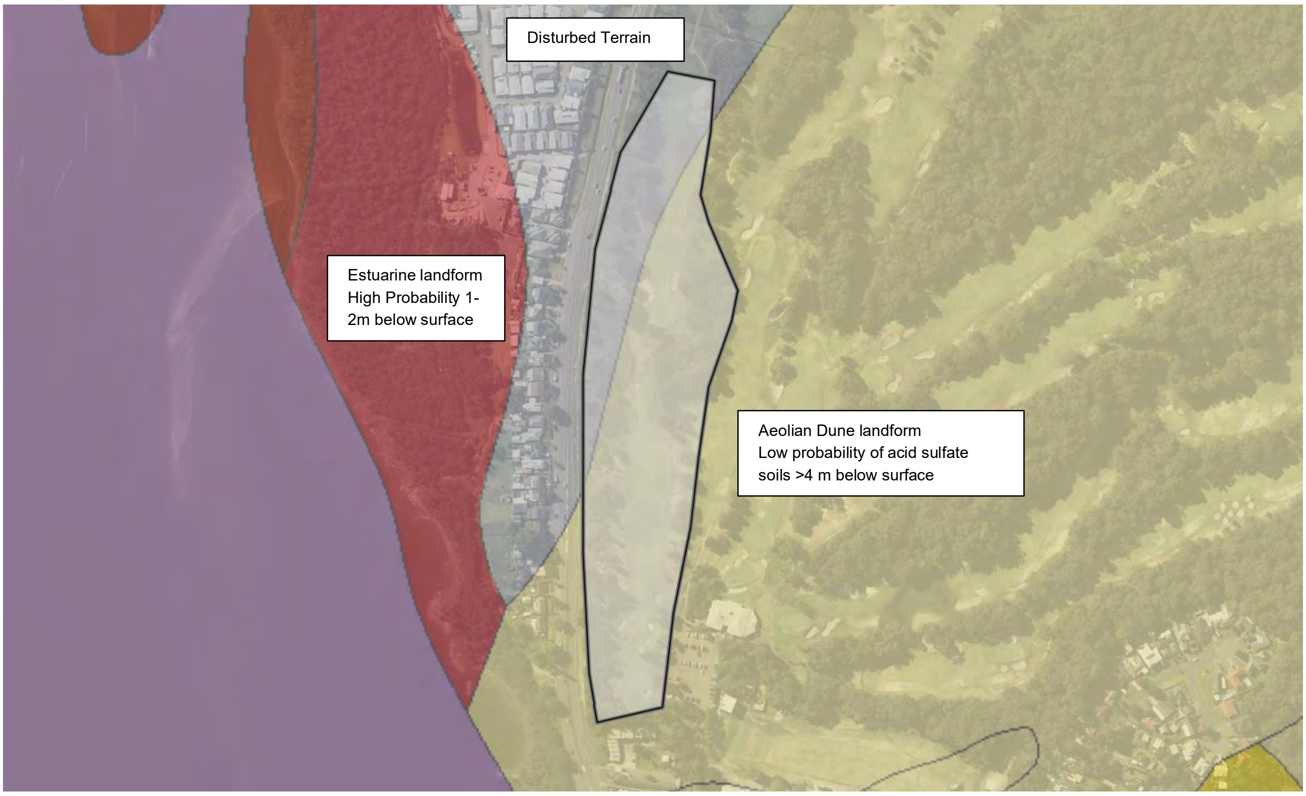


Figure 5. Acid sulfate soil risk map

2.4 REGIONAL AQUIFERS

The site is located within the Tomago-Tomaree-Stockton Sandbeds aquifer which forms part of the overall Hunter Valley Alluvium aquifer.

The NSW Government, Bioregional Assessment of the Hunter Region [3] indicates that groundwater from the Tomago-Tomaree-Stockton aquifer is used for urban water supply by the Hunter Water Corporation, providing the potable water supply to the Newcastle area, and is particularly important during droughts. However, during normal times, surface water catchments provide the majority of the potable water supplies to the Newcastle area.

The assessment estimated a transmissivity of the aquifer to be between 400 m/day to over 600 m/day with a specific yield of about 0.2.

The groundwater level in the aquifer is noted to be responsive to rainfall events, with groundwater level rises over a metre observed during individual events.

2.5 BEDROCK

No investigation data is available within the vicinity of the site to provide site-specific data for depth to bedrock. The CSIRO in conjunction with Geoscience Australia have prepared a map which outlines the depth of regolith (which is the soil and weathered material situated above the unweathered bedrock). This depth can be used as an estimate of an impermeable boundary between aquifers.

The CSIRO map is accessible through the MinView online mapping tool [1] which identifies that the site is situated in an area with an estimated depth of regolith to be in excess of 40m.

2.6 GROUNDWATER BORES

A search of WaterNSW Groundwater Bore database [4] indicates that there is one groundwater bore within 500m of the site boundaries, located approximately 50m to the west within the residential property of 1055 Nelson Bay Road identified as GW054990. Details of this bore indicate that it is used for domestic purposes, with the drillers log noting 4m of sand before termination of the bore. The current status of this bore is unknown.

2.7 PREVIOUS SITE INVESTIGATIONS

A geotechnical investigation was previously undertaken by RCA Australia (RCA, Report Ref 15442-402/3, dated November 2023) to support the design of the proposed structures within the seniors living area.

The investigation comprised four cone penetrometer tests (CPT) to depths up to 18.5m below existing ground levels (BGL) along with eleven test pits (TP) to depths up to 2.3m BGL. Perth Sand Penetrometers were undertaken at test pit location for sand consistency. Figure 6 below presents a plan showing the location of the investigations.

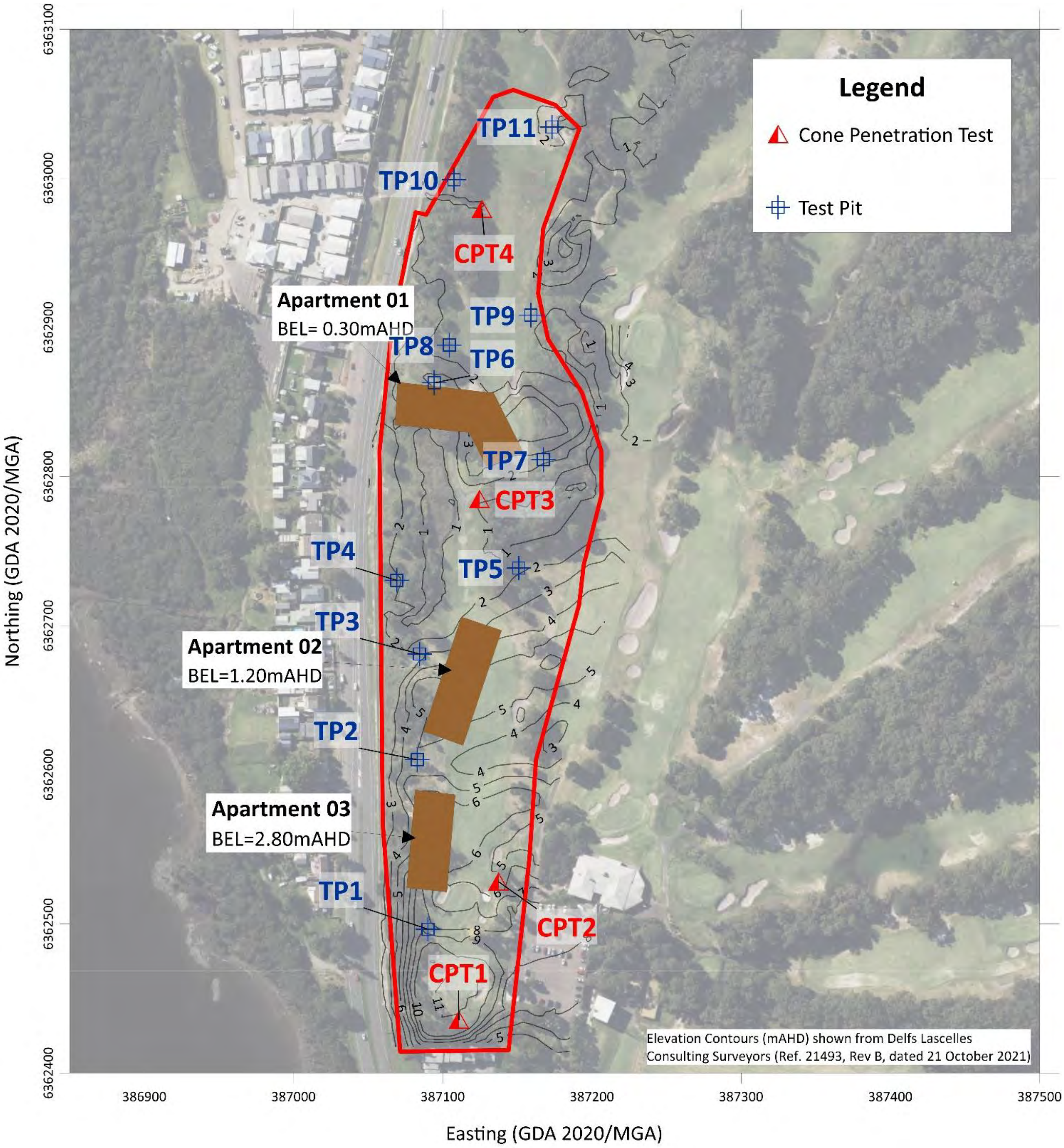


Figure 6. RCA investigation locations

The subsurface conditions encountered at the test locations were summarised as follows:

- Topsoil/fill comprising sands and silts with inclusions of shells, roots and organics were encountered to depths between 0.1 to 0.3m across the test pits with one (TP4) up to 0.9m depth.
- Natural sands were encountered below topsoil/fill and generally found to be very loose to loose up to 2.0m depth, increasing to medium dense to dense. Sporadic layers of loose sands were encountered within the CPT results along with sporadic pockets of organic or soft to firm clays at shallow depths (less than 2m).
- Groundwater was encountered at all test pits except TP1, TP2, TP3 and TP6. Groundwater was generally encountered from 1.0 to 1.6m depth.

- An assessment of CPT results indicated that CPT3 and CPT4 were inferred to encounter groundwater at approximately 1.0m depth, and CPT1 and CPT2 at depths of 10.3m and 4.4m respectively.
- Bedrock was not encountered in the investigations.

The surface elevation and coordinates were not recorded at the investigation locations. The site investigation map by RCA was overlain with the survey data prepared by Delfs Lascelles to obtain approximate coordinates and surface elevation as presented in Table 2 and Figure 7.

Table 2. Approximate investigation coordinates and surface elevations

Investigation ID	Type	Approximate Eastings GDA2020 (m)	Approximate Northings GDA2020 (m)	Approximate Surface Elevation (mAHD)	Approximate Groundwater Inflow Elevation (m AHD)
TP1	Test pit	387090.1	6362496.5	8.6	Not encountered
TP2	Test pit	387082.9	6362610.2	4.7	Not encountered
TP3	Test pit	387084.5	6362681.1	2.7	Not encountered
TP4	Test pit	387069.1	6362730.4	2.1	0.7 (s)
TP5	Test pit	387150.9	6362738.6	1.8	0.8 (s)
TP6	Test pit	387094.3	6362862.9	3.4	Not encountered
TP7	Test pit	387167.4	6362811.4	2.4	0.8 (s)
TP8	Test pit	387104.4	6362888.2	1.4	0.6
TP9	Test pit	387158.9	6362908.3	1.2	0.5
TP10	Test pit	387107.4	6362999.0	1.2	0.6
TP11	Test pit	387173.2	6363034.5	2.8	1.9
CPT1	CPT	387110.8	6362435.6	10.8	0.6
CPT2	CPT	387137.3	6362528.3	5.4	1.2
CPT3	CPT	387124.6	6362784.8	1.1	0.5
CPT4	CPT	387126.2	6362979.3	1.1	0.7

Notes:
(s) groundwater seepage only, no significant inflows

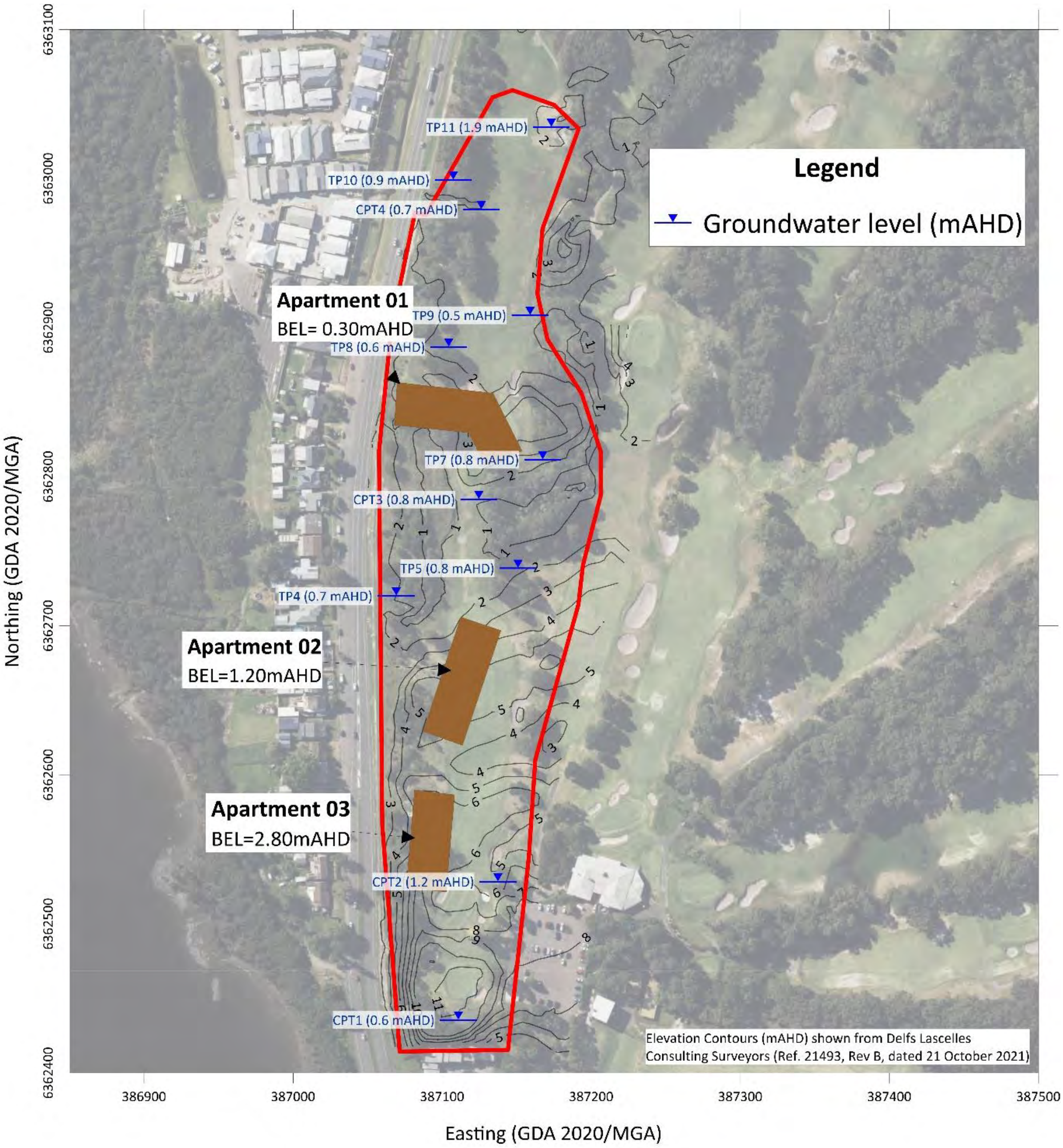


Figure 7. Elevations at which groundwater was encountered during investigations

Note that the elevations at which groundwater was encountered in the CPT and test pit investigations should be considered approximate, as readings were provided on a single date only and depend on manual measurements within test pits.

Detailed Acid Sulfate Soil (ASS) analysis undertaken on select samples within the northern half of the development area at TP5, TP7, TP8 and TP10, generally within the lower elevations across the site at elevations between 0 – 0.5m AHD. The results of the detailed ASS analysis indicated that Potential Acid Sulfate Soils (PASS) were present at all four test pit locations.

2.8 TIDE LEVELS

Given the low elevation of the site and the proximity to the Hunter River it is likely that groundwater at the site is influenced by tidal conditions.

Tidal levels are measured relative to the Lowest Astronomical Tide (LAT) at a given point, which in NSW is Fort Dennison in Sydney. The zero tide mark at fort Dennison is recorded as -0.925m AHD.

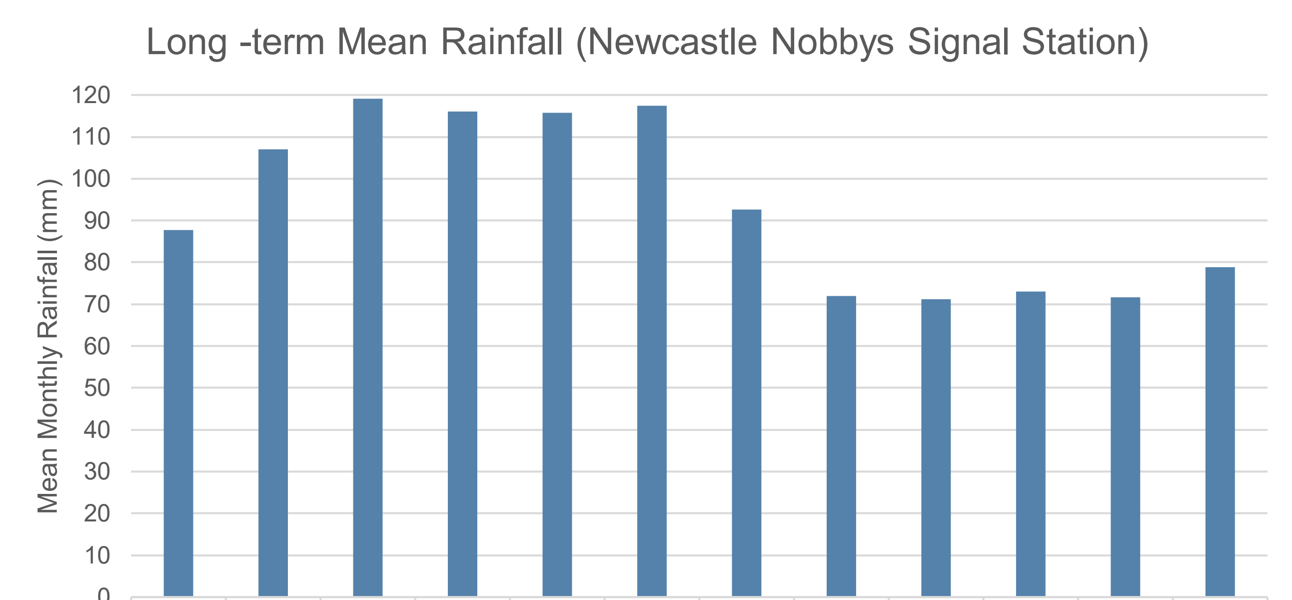
The NSW Government Tide Tables [5] note that on the day of the RCA investigation on 17 March 2021, tide levels ranged from 0.61m LAT and 1.25m LAT, which equates to tide levels of between approximately -0.3m and 0.3m AHD respectively. The tide tables for 2024-2025 indicate that tidal levels typically range between -0.4m to 0.6m AHD however can be as high as 1.2m AHD and as low as -0.8m AHD.

2.9 RAINFALL

Long term average monthly rainfall has been obtained from the Australian Bureau of Meteorology [6] for Newcastle Nobbys Signal Station (station number 61055) which is approximately 6km south of the site.

Figure 8 presents the mean monthly rainfall data at this weather station between 1862 and 2024. The mean annual rainfall is 1,117mm and the median is 1,048mm.

In the week prior to the investigation by RCA (17 May 2021) there was low rainfall reported.



	January	February	March	April	May	June	July	August	September	October	November	December
Month	87.8	107	119.1	116	115.8	117.5	92.6	72	71.2	73	71.6	78.8

Figure 8. Long-term mean rainfall data

3. SITE HYDROGEOLOGY

3.1 GEOLOGICAL MODEL

RCA presented a table of interpreted ground profiles at each investigation location based on material type and consistency. For a groundwater and hydrogeological assessment this can be further simplified by combining soils that will have similar hydraulic parameters. For this preliminary assessment, the geological profiles comprise:

- Loose to dense sands and silty sands with minor inclusions of peat and clay (Unit 1);
- Dense to very dense sands (Unit 2).

The test pits undertaken across the site typically extended to a maximum depth of 3.0m below ground and terminate within the loose to dense sands (Unit 1), however the CPT tests undertaken across the site provide information of the deeper soil profiles up to a maximum depth of 18.5m. From a review of the CPT test results along with correlation of site survey data, it was observed that the interface between Unit 1 and Unit 2 was dipping down towards the north, ranging from -4.4m AHD in the south at CPT1 to -8.4m AHD in CPT4 in the north.

From this, an approximate geological model at the location of each apartment structure can be interpreted for the purpose of this analysis. Bedrock was not encountered in the investigations and is not included in the model.

Table 3. Preliminary geological model

Description	Approximate layer depth range [Elevation]		
	Apartment 1	Apartment 2	Apartment 3
Loose to dense Sands and Silty Sands, minor inclusions of peat and clay (Unit 1)	0 – 11m BGL [3.8 to -7.2m AHD]	0 – 11m BGL [4.7 to -6.3m AHD]	0 – 12m BGL [6.3 to -5.7m AHD]
Dense to Very Dense Sands (Unit 2)	> 11m BGL	> 11m BGL	>12m BGL
Bedrock (Impermeable boundary	Not Encountered (estimated to be at around -45m AHD)		

3.2 GROUNDWATER LEVELS

At the time of reporting, no groundwater wells have been installed within the site for monitoring of groundwater levels. The extent of the available groundwater data includes:

- The depths of groundwater encountered in test pits or inferred from pore water pressure readings from CPT results as shown in Figure 7.
- Surveyed edges of small bodies of water across the golf course within the site area.

At the time of the investigation, the test pits and CPT around Apartment 1 recorded groundwater seepage and inflows at elevations between 0.6m to 0.8m AHD. Between Apartment 1 and Apartment 2 lies a small pond which forms part of the golf course. It is likely that this pond is not lined and could be connected to the groundwater table. At the time of the survey (26 May 2021), the approximate water level measured at the edges of this pond was generally between 0.3 to 0.6m AHD.

Around Apartment 2 and 3, several of the test pits did not encounter groundwater. This is likely due to the higher elevation of the existing site levels. North of Apartment 2, TP4 and TP5 encountered groundwater at 0.7m and 0.8m AHD respectively, while CPT2, south of Apartment 1, recorded pore water pressure response at an elevation of 1.2m AHD.

These groundwater measurements relate to a single measurement at a point in time and do not consider higher groundwater levels due to rainfall, tides or changes in site conditions. We anticipate that groundwater levels could potentially be higher than recorded during such events. Given the range of the observed groundwater elevations, the adoption of a groundwater level across the whole site of 0.8m AHD, plus an additional 0.5m for uncertainty in observations would be considered reasonable for the purpose of this preliminary analysis for construction dewatering.

During construction to allow for a dry working platform, average groundwater levels would typically need to be lowered to at least 0.5m below BEL.

Table 4 summarises the BEL of each apartment building with the adopted design groundwater level and anticipated depth of dewatering required for constructability (assumed as 0.5 m below BEL for the purpose of the inflow assessment). Based on these values it becomes apparent that during excavation of Apartment 3, it is not likely to impact groundwater within the proposed extent of excavation. However, the construction of Apartments 1 and 2 basements will likely require dewatering.

Table 4. Impact of excavation and dewatering to relative to groundwater levels.

Apartment	Bulk excavation level (mAHD)	Preliminary design groundwater level (mAHD)	Assumed groundwater elevation required for basement construction (mAHD) [1]	Depth of excavation below groundwater level (m)	Depth of groundwater change for basement construction (m)
Apartment 1	0.3	1.3	-0.2	1.0	1.5
Apartment 2	1.2	1.3	0.7	0.1	0.6
Apartment 3	2.8	1.3	Dewatering not likely to be required	0	0

[1] Assumed to be 0.5 m below bulk excavation level

3.3 HYDRAULIC PARAMETERS

At the time of reporting, no hydraulic conductivity testing has been undertaken on site to obtain site-specific parameters for inflow assessment. Therefore, the adopted hydraulic conductivity are based on a literature review along with correlations to grain size.

In Section 2.4, it is noted by the Bioregional Assessment that the transmissivity of the Tomago-Tomaree-Stockton Sandbeds aquifer is approximately 400 to 600m/day. The transmissivity (T) of an aquifer is calculated by the overall hydraulic conductivity (k) multiplied by the overall aquifer thickness (d). Adopting a minimum aquifer thickness of 40m as noted in Section 2.5, this would result in an average hydraulic conductivity of 1.2 to 1.7 x 10^{-4} m/s. This is an averaged hydraulic conductivity of the whole aquifer and does not account for areas of lower or higher hydraulic conductivity.

During the investigation, RCA collected samples from four test pits at shallow depths up to 1.0m and completed Particle Size Distribution (PSD) tests on the samples. The PSD test provides a grading of particle sizes in the sample. The results indicated that the samples were typically a fine to medium grained sand with low silt/clay content. An assessment of the hydraulic conductivity based on the grain size distribution was completed using the HydrogeoSieveXL spreadsheet [7].

Table 5. Calculated hydraulic conductivity based on PSD results

Investigation ID	Depth Range (mBGL)	Description on logs	Geological Unit	Calculated geometric mean hydraulic conductivity (m/s)
TP1	0.6 – 1.0	SAND, fine to medium grained	Unit 1	3.4×10^{-4}
TP3	0.3 – 0.5	SAND, fine to medium grained	Unit 1	1.2×10^{-3}
TP5	0.3 – 0.5	SAND, fine to medium grained, with silt	Unit 1	1.5×10^{-4}
TP10	0.3 – 0.4	SAND, fine to medium grained, with silt	Unit 1	6.6×10^{-5}

The average of the four PSD tests is a hydraulic conductivity of 2.5×10^{-4} m/s.

The reference *A Practical Guide to Groundwater Lowering in Construction* by P. Cashman and M. Preene [8], provides typical ranges of permeability for various soil types as reproduced in Figure 9 below.

Table 3.1 Typical values of soil permeability

Soil type	*Typical classification of permeability*	*Permeability (m/s)*
Clean gravels	High	$>1 \times 10^{-3}$
Clean sand and sand/gravel mixtures	High to moderate	1×10^{-3} to 5×10^{-4}
Fine and medium sands	Moderate to low	5×10^{-4} to 1×10^{-4}
Silty sands	Low	1×10^{-4} to 1×10^{-6}
Sandy silts, very silty fine sands and laminated or mixed strata of silt/sand/clay	Low to very low	1×10^{-5} to 1×10^{-8}
Fissured or laminated clays	Very low	1×10^{-7} to 1×10^{-9}
Intact clays	Practically impermeable	$<1 \times 10^{-9}$

Figure 9. Typical soil permeability values by P. Cashman and M. Preene

The descriptions of the site quaternary soils within the upper (Unit 1) material would most closely resemble *Fine and medium sands* which has typical permeability values ranging from 5×10^{-4} to 1×10^{-4} m/s which generally coincides with the assessment of transmissivity as well as the overall average of the PSD results.

The above tests generally do not consider the level of compaction of a given sample as the higher relative density (i.e. the more closely spaced the sand grains are), the less permeable a medium would be. Very little data is available to assess the permeability of the underlying dense to very dense soils.

For the purpose of this preliminary analysis, the following hydraulic conductivity values have been adopted, where (kx) denotes the horizontal hydraulic conductivity and (ky) the vertical hydraulic conductivity:

- Unit 1: Loose to dense sands and silty sands = 2.5×10^{-4} m/s with ky / kx = 1
- Unit 2: Dense to very dense sands = 1×10^{-4} m/s with ky / kx = 0.5

4. PRELIMINARY GROUNDWATER INFLOW AND DRAWDOWN ASSESSMENT

4.1 BASIS OF ASSESSMENT

Dewatering is likely to be required for Apartment 1 and 2 to assist with excavation below groundwater as well as providing a suitable dry working platform for constructability purposes. Apartment 3 is not anticipated to impact the groundwater table and so the inflow assessment considers only Apartments 1 and 2.

It is assumed that the basements for Apartment 1 and 2 will be tanked in the permanent case, with groundwater inflows to occur during construction only. Tetra Tech understands the construction of the basements will be staged such that no two basements will undergo dewatering simultaneously.

This analysis has considered the use of a cantilevered cut-off wall (Sheet-pile or similar) constructed around each apartment structure to assist with excavation retention as well as reducing groundwater inflows to the basement excavation during construction. A nominal wall embedment depth of twice the retained excavation height has been adopted for this analysis.

4.1.1 Model geometry and conceptual site model

A simplified geometry has been adopted based on ground conditions and proximity to nearby permanent water bodies. This includes:

- Equating each basement structure to an equivalent excavation of radius (r):
- Modelling the inflow as an axisymmetric analysis.
- Adopting a ground surface elevation across the site equivalent to the proposed ground floor level of each apartment building.

A Conceptual Site Model was then developed for each apartment structure for use in the inflow analysis. Table 6 and Figure 11 summarises the key geometries of the proposed excavations for Apartment 1 and Apartment 2.

Table 6. Geometric parameters for Conceptual Site Model

Parameter	Basement Area (m^2)	Equivalent Circle Radius (m)	Ground Floor Level (mAHD)	Bulk Excavation Level (mAHD)	Groundwater level during dewatering (mAHD)	Cut-off Wall Toe (mAHD)	Assumed depth of cut-off wall below BEL (m below BEL)
Reference*	-	(r)	(a)	(b)	(c)	(d)	-
Apartment 1	2,500	28.2	3.8	0.3	-0.2	-6.7	7
Apartment 2	2,260	26.8	4.7	1.2	0.7	-5.8	7

*: Refer to Figure 11

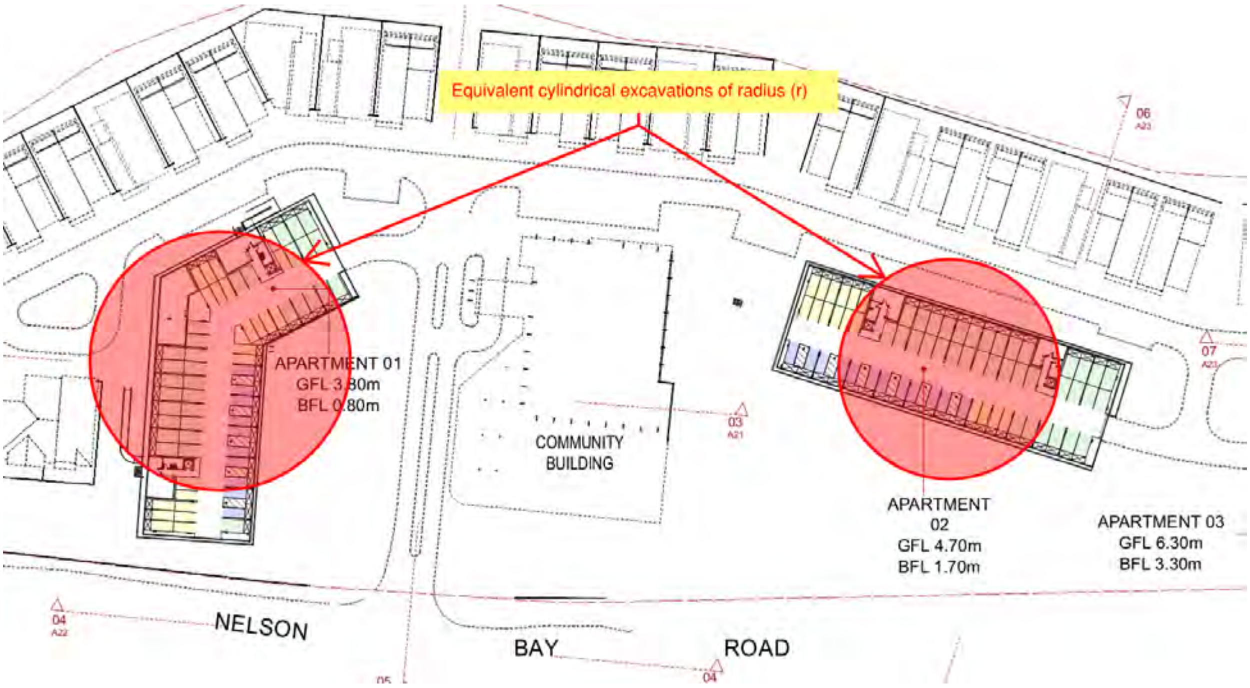


Figure 10. Equivalent radius of excavation

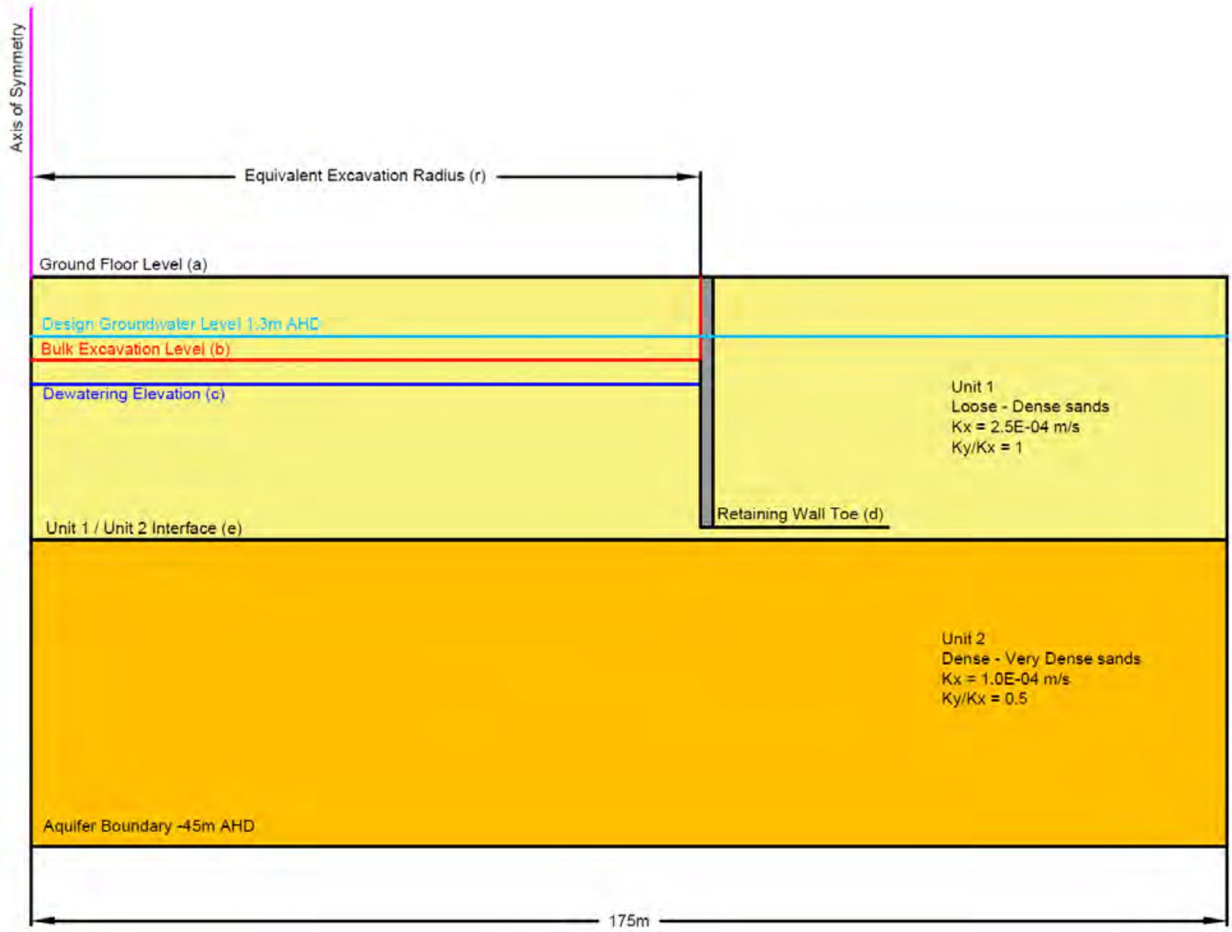


Figure 11. Conceptual Site Model (not to scale)

4.1.2 Assessment and methodology and boundary conditions

An estimation of steady state groundwater inflow rates to the proposed excavation was carried out using the proprietary software Seep/W within Geostudio Version 23.1.

The following assumptions were adopted for the inflow assessment:

- The flow to the excavation is unconfined.
- The assessment assumes a steady-state analysis.
- An estimated groundwater level across the site of 1.3m AHD has been adopted, based on initial readings taken during previous investigation including a 0.5m allowance for uncertainty. This has been set as a constant head boundary on the far side of the model.
- Groundwater levels during dewatering will be maintained at 0.5 m below BEL. This is set as a fixed head boundary in the model.
- An estimated zone of influence or distance to lateral fixed head boundaries of 175m from the edge of excavation has been adopted. This has been adopted as this is the closest distance to a point of constant groundwater level (marshlands of Hunter River to west).
- Rainfall has not been modelled in this analysis.
- The base of the aquifer is set at -45m AHD and defined as a no-flow boundary.
- The hydraulic conductivity of the upper sand layer (Unit 1) is k = 2.5 x 10^{-4} m/s with a ky / kx ratio of 1.
- The hydraulic conductivity of the lower sand layer (Unit 2) is k = 1 x 10^{-4} m/s with a ky / kx ratio of 0.5.
- The cut-off wall is modelled as an impermeably boundary.

4.2 ASSESSED INFLOWS AND GROUNDWATER TAKE

The results of the SEEP/W models are presented in Appendix D of this report. The results of the SEEP/W analysis summarised in Table 7 provide an inflow of L/s and m^3/day for options with and without a cut-off wall structure.

Table 7. Summary of Seep/W Analysis

	Analysis Condition (Cut-off Wall Toe Elevation)	Groundwater Inflow Q	
		(L/s)	(m^3/day)
Apartment 1	No cut-off wall	28	2,420
	Cut-off wall to -6.7m AHD	16	1,382
Apartment 2	No cut-off wall	7.7	665
	Cut-off wall to -5.8m AHD	5.6	484

The analysis shows that by adopting a cut-off wall to the elevations shown above, a reduction in groundwater flows of approximately 43% and 27% for Apartments 1 and Apartments 2 could be achieved.

It is estimated that the time for dewatering (which extends from excavation through the completion of the tanked basement) may be up to 6 months. During this time, it is expected that the total groundwater take will be approximately:

- Apartment 1
  - Excluding cut-off wall: 442 ML
  - Including cut-off wall: 252 ML

- Apartment 2
  - Excluding cut-off wall: 121 ML
  - Including cut-off wall: 88 ML

An assessment of groundwater within excavated soil has also been considered based on an excavated volume of soil below groundwater table of 2500m^3 for Apartment 1 and 226m^3 for Apartment 2, and sand porosity of 0.45.

For this excavation, it is expected that an additional approximately 1125m^3 (1.1 ML) and 102m^3 (0.1 ML) of additional groundwater take will be required during excavation.

Further reduction to inflow may be achieved by deepening of the cut-off wall (refer to sensitivity assessment in Section 4.4). Additional measures may be considered to reduce the aggregate groundwater take, such as staging of the dewatering and excavation to reduce the overall duration of dewatering.

It should be noted that uncertainties in the adopted predevelopment groundwater level and aquifer hydraulic conductivity mean that this assessment may be conservative. It is recommended that the predicted inflows are re-assessed once groundwater investigation data becomes available.

The assessed groundwater inflow rates are for working conditions based on information available at the time of writing this report based on limited information prior to excavation of the proposed basement and is subject to change based on future additional investigation and finalisation of basement design and construction methodology. Unforeseen ground conditions may result in inflows different to those assessed here.

4.3 ASSESSED DRAWDOWNS

Figure 12 and Figure 13 presents the assessed groundwater drawdown versus distance from the excavation face based on the Seep/W analysis for the base case conditions as presented in Table 7. Table 8 summarises the estimated distance from excavation face for the 0.5, 1.0 and 2.0m drawdown levels.

Table 8. Summary of drawdown distances

Apartment block	Analysis condition (Cut-off wall toe elevation)	Distance from excavation face for given drawdown (m)		
		0.5m drawdown	1.0m drawdown	2.0m drawdown
Apartment 1	No Cut-off Wall	45	10	Nil
	Standard Cut-off Wall (-6.7m AHD)	25	Nil	Nil
Apartment 2	No Cut-off Wall	< 5	Nil	Nil
	Standard Cut-off Wall (-5.8m AHD)	Nil (maximum 0.31m)	Nil	Nil

These assessed drawdowns are based on the analysis presented above and assumptions related to the zone of influence. We consider the assessed drawdowns represent a reasonable assessment at the pre-construction stage of the site, noting that there is an unavoidable degree of uncertainty in predictions of drawdown prior to construction observations becoming available.

This assessment considers drawdown related to the individual apartment blocks independently to each other. Should dewatering occur simultaneously at each location, the effects may result in wider cumulative drawdowns.

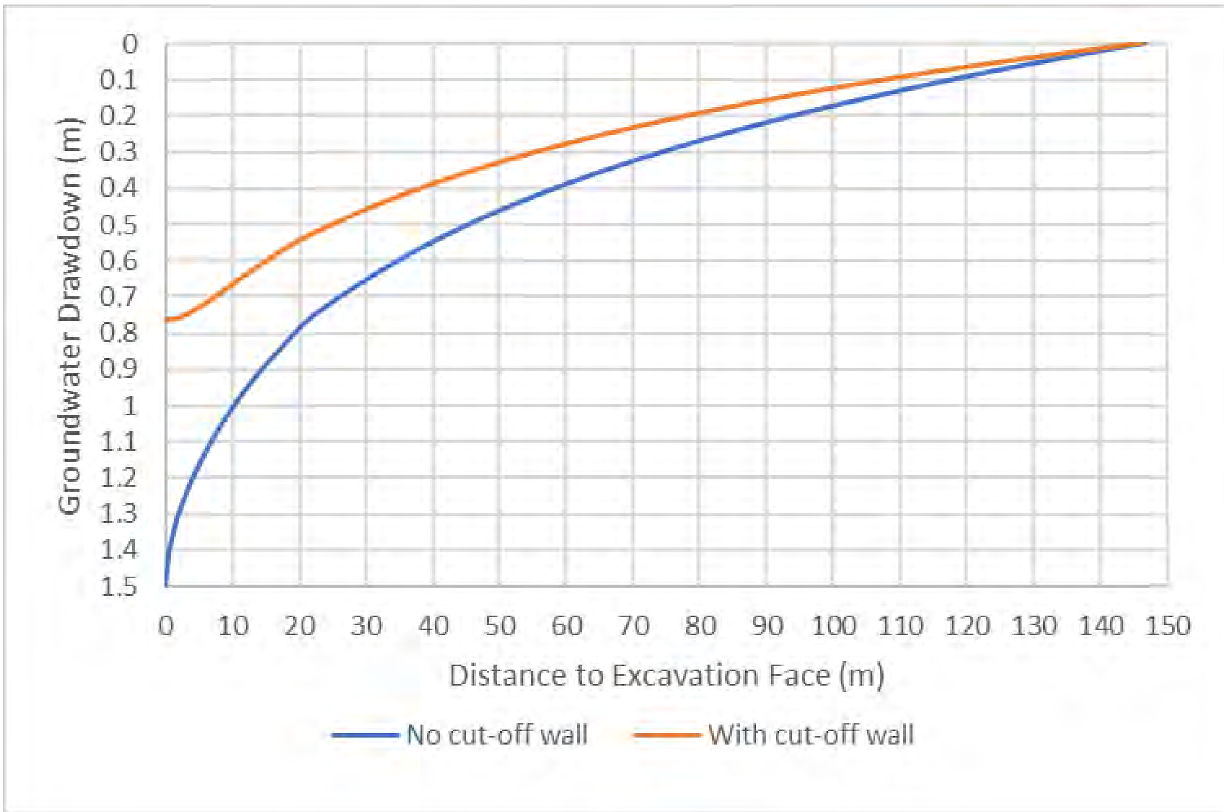


Figure 12. Drawdown curves for Apartment 1

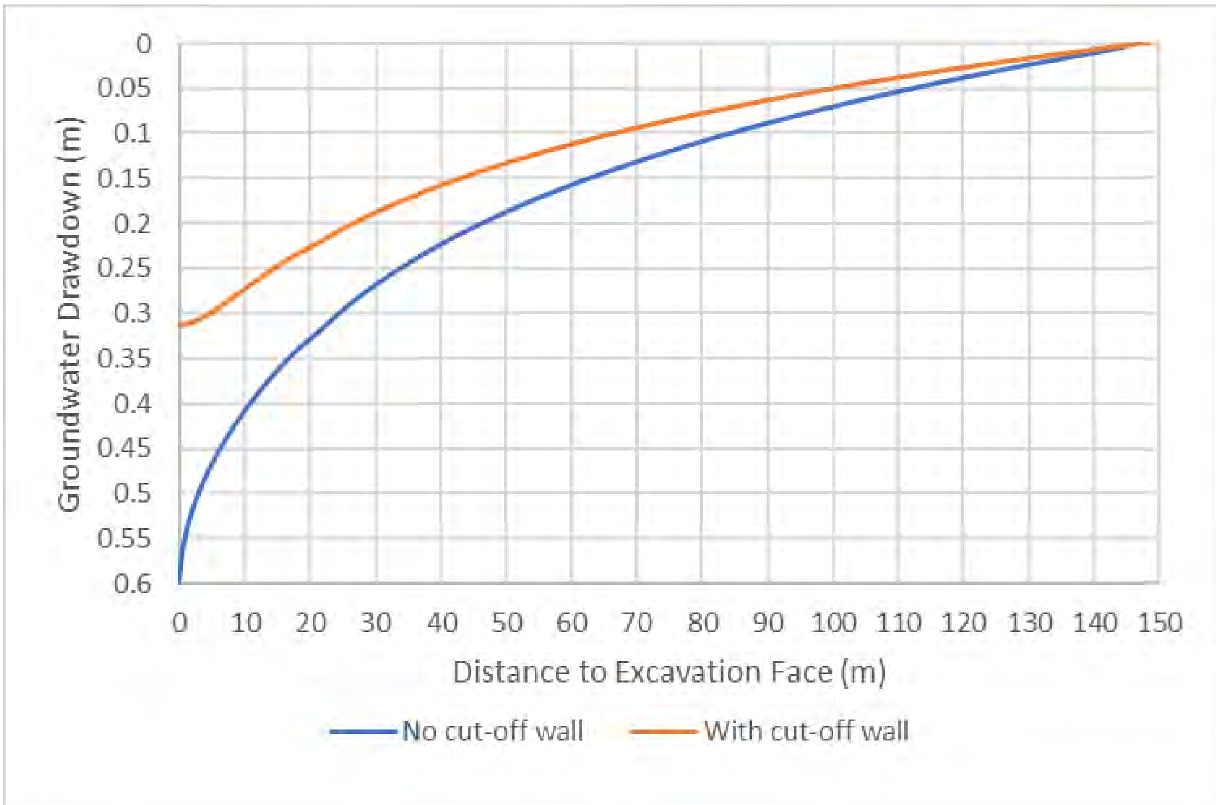


Figure 13. Drawdown curves for Apartment 2

4.4 SENSITIVITY ANALYSIS

A sensitivity analysis was undertaken to assess the affect that altering key parameters may have on the groundwater inflows. For this assessment the following parameters were adjusted:

- Reducing the design groundwater levels from 1.3mAHD to 0.8mAHD to align with the groundwater levels during the initial geotechnical investigation.
- Deepening the cut-off toe to a depth of up to three times the retained soil height, up from two times.

The results of this analysis are presented below in Table 9.

Table 9. Sensitivity Analysis

Apartment Block	Analysis Condition (Retaining Wall Toe Elevation)	Groundwater Inflow (L/s)	
		Groundwater Level at 1.3m AHD	Groundwater Level at 0.8 AHD
Apartment 1	No Retaining Wall	28	14
	Standard Cut-off Wall (-6.7m AHD)	16	10
	Deep Cut-off Wall (-10.2m AHD)	11	7
Apartment 2	No Retaining Wall	7.7	1.3
	Standard Cut-off Wall (-5.8m AHD)	5.6	0.9
	Deep Cut-off Wall (-9.3m AHD)	3.9	0.7

The sensitivity analysis shows that by adopting a deeper wall, inflows may be reduced by approximately 30% for both apartments when compared to the standard cut-off wall depth, and 50-60% when compared to no retaining wall.

If lower groundwater levels of 0.8 mAHD were found to be present on site during construction, groundwater inflows would reduce by 35-50% in Apartment 1. In Apartment 2 groundwater inflows will be reduced by a factor of 6 once a cut-off wall is installed. This is due to the groundwater level being situated below BEL (1.2mAHD) but above dewatering level (0.7mAHD), requiring only small amounts of dewatering.

It should be noted that the adoption of lower groundwater levels does not account for events such as:

- High rainfall conditions and storm events
- High-tide events if the site's groundwater levels are affected by tidal conditions.

Dewatering operations would need to be assessed by the contractor during construction to determine the appropriate amount of dewatering that would be required for constructability.

4.5 SUMMARY

The preliminary inflow analysis presented above summarises the anticipated groundwater inflow during excavation of Apartment 1 and 2 based on several different configurations of design groundwater level, retaining wall configuration and additional dewatering for constructability.

The analysis found that under typical conditions as presented in Section 4.2, inflows of between 16-28L/s are anticipated for Apartment 1, and 5.6-7.7L/s for Apartment 2. This assessment had been developed based on limited site information and should be considered as preliminary only.

Further site-specific information would be required to obtain a more accurate assessment of inflows including:

- In-situ testing to provide information on hydraulic conductivity.

- Groundwater monitoring to assess groundwater levels and impact of rain events and tides.

A discussion on a proposed investigation to provide further site information is presented in Section 6.2 of this report.

In addition, the proposed construction methodology would need to be considered which may influence the amount of groundwater inflows. It was found that by deepening the retaining wall, a reduction of inflows could be achieved, however there are likely further costs involved with deeper wall installation.

Alternatively, the extent and required depth of dewatering within the basement areas to provide a suitable working platform could be assessed.

5. GROUNDWATER IMPACT ASSESSMENT

5.1 ACID SULFATE SOILS

Acid sulfate soils (ASS) are natural sediments that contain iron sulfides. Left undisturbed they do not present any risk, however when exposed to air the iron sulfides react with oxygen to create sulfuric acid that can create very low pH levels which damages structures and adversely impacts ecosystems.

When referring to ASS, they are generally categorised as two forms, namely Potential Acid Sulfate Soils (PASS) and Actual Acid Sulfate Soils (AASS). PASS refers to undisturbed soils that contain the potential for acid generation which have not yet occurred, whereas AASS refers to soils that have started or already undergone oxidising to produce sulfuric acid.

The ASS testing undertaken by RCA comprised 20 field screening tests and four detailed ASS tests. RCA noted that the given the results of the field screening, the site soils would not be considered AASS, however there was evidence to support a potential for acid forming conditions on 13 of the 20 samples tested. These 13 samples that tested positive typically ranged at elevations between 0.5m to 1.5m AHD around Apartment 2, and between -0.1m to 1.0m AHD to the north around Apartment 1.

Detailed analysis on four samples between -0.1m and 0.6m AHD indicated that they would be considered as PASS, i.e. soil that has a high potential to form sulfuric acid when exposed to air. Oxidisation of PASS can occur without excavation through reduction of groundwater levels. Where groundwater levels are lowered, air can seep through permeable soils and begin acidifying PASS.

Given that groundwater levels observed at the time of the investigation were around 0.8m AHD, and PASS can occur at depths as shallow as 0.6m AHD, it leads to consider that:

- Historic groundwater levels across the site have not varied significantly over a considerable amount of time, otherwise AASS would be present.
- No ASS were identified above the current groundwater level, otherwise they would have been recorded as AASS.
- Changes in groundwater level have the potential to generate AASS and hence, produce sulfuric acid.

This means that drawdown below current groundwater levels may adversely affect other users and ecosystems through generation sulfuric acid that can leech into the aquifer system.

We recommend that additional investigations are undertaken to further assess the potential for ASS across the site. The location of investigations and the assessment should consider the extent of drawdown presented in this report and future revisions.

5.2 EFFECT ON NEARBY USERS

One groundwater bore was identified approximately 50m west of the site boundary, up to 175m from Apartment 1 and 125m from Apartment 2. The status of the bore is unknown. From the drawdown curves presented in Section 4.3, the likely drawdown to this groundwater bore is expected to be less than 0.1m. Given that that drawdowns of this magnitude are likely to be within natural variability the risk to existing bore users is considered to be negligible.

5.3 EFFECTS ON NEARBY GROUNDWATER DEPENDANT ECOSYSTEMS

A review of the Bureau of Meteorology Groundwater Dependent Ecosystems (GDE) Atlas [9] indicates that there are several terrestrial GDE's around the proposed site including:

- Smooth-barked Apple / Blackbutt/Old Man Banksia woodland on coastal sands to the east and south
- Grey Mangrove Low Closed Forest to the west.

The distances of these GDE's from the excavation boundary of Apartments 1 and 2 are presented in Figure 14 below.

The GDE's nearest to Apartment 1 are at least 80m away from the excavation face. At this distance the expected drawdown due to dewatering will be less than 0.3m. Given that that drawdowns of this magnitude are likely to be within natural variability, the risk to GDE's is considered to be negligible. The closest GDE's to Apartment 2 are at least 130m away, where groundwater drawdowns are predicted to be negligible.

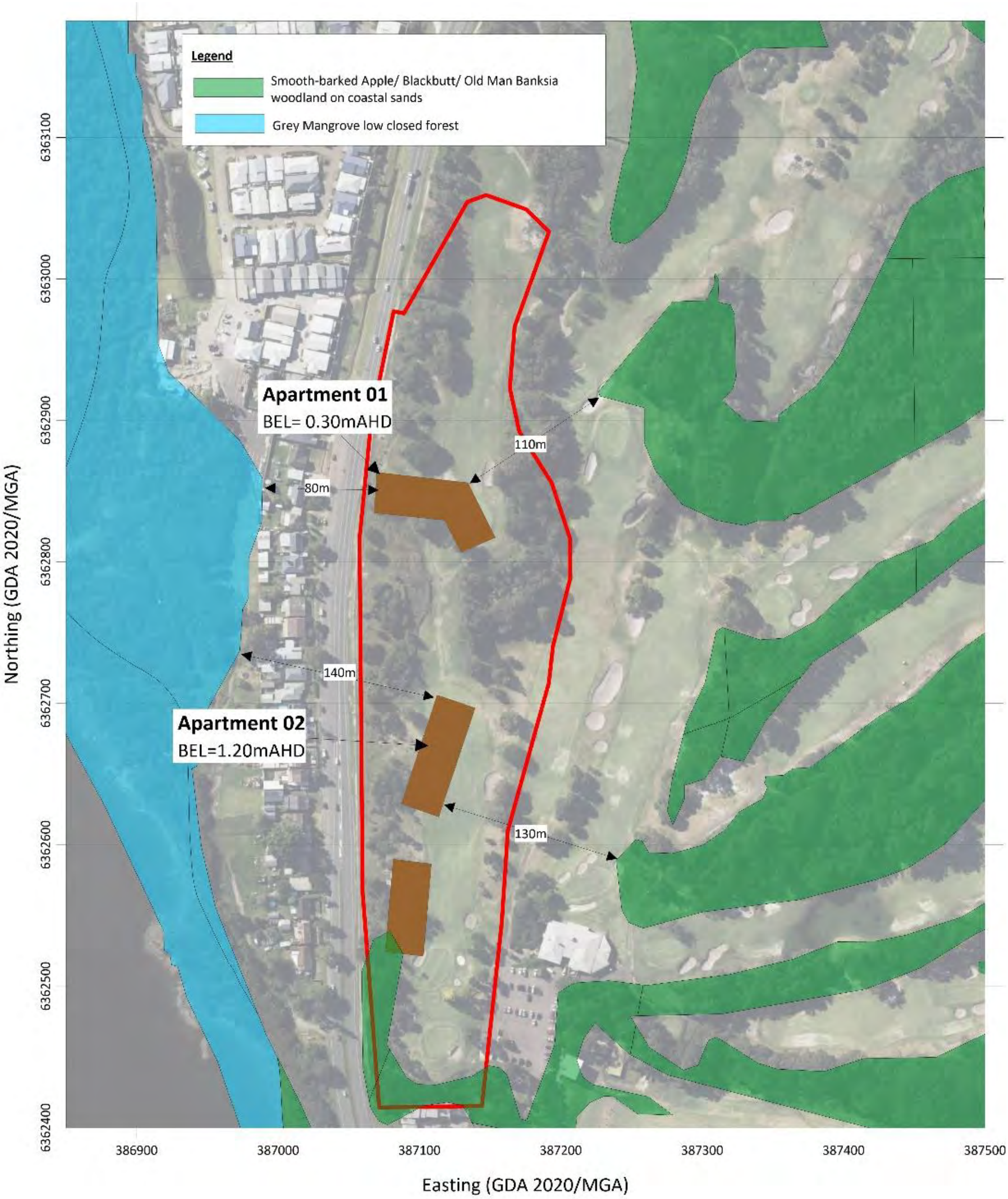


Figure 14. Nearby GDE's

5.4 RISK OF INCREASED SALINITY

The closest saltwater source to the site is the Hunter River, approximately 175m to the west of Apartment 1 and Apartment 2. Based on the predicted groundwater drawdown, we estimate that minor changes will occur to the groundwater gradient near the Hunter River. Due to this the risk of saltwater intrusion as a result of the proposed development is considered to be negligible.

5.5 RISK OF INCREASED CONTAMINATION OF AQUIFERS

The development will act as a groundwater sink, and injection/recharge of water to the groundwater system is not currently proposed. Where dewatering occurs, this may oxidise PASS nearby to the development site and draw sulfuric-acid-impacted groundwater towards the development. Once dewatering is complete, groundwater levels would rise back up, mixing with sulfuric acid within the now AASS and potentially impacting the aquifer. We would consider this a moderate risk of contamination to the aquifer due to PASS.

5.6 SURFACE SETTLEMENT

A decrease in groundwater pore pressure in the soil due to drawdown results in an increase in effective stress, which may result in surface settlement due to compression of the soil. The magnitude of settlement depends on the soil's stiffness in one-dimensional compression E'_0, the magnitude of drawdown in groundwater levels, and the saturated thickness of the compressible soil layer. Table 10 (after Preene, Roberts and Powrie, 2016) [10] shows the groundwater induced settlement for a 1 m drawdown in groundwater levels over a 1 m thick compressible soil layer.

Table 10. Groundwater induced settlement for a 1 m drawdown in groundwater levels over a 1 m thick compressible soil layer

The basic settlement is defined as the compression of a soil layer 1 m thick from an increase in vertical effective stress corresponding to a drawdown of 1 m. For a given situation, the total settlement in mm may be obtained by multiplying the basic settlement by the drawdown and the thickness of the soil layer (both in metres)

One-dimensional soil stiffness, E'_o (MPa)	1	5	10	15	20	25
Basic settlement (mm)	10.0	2.0	1.0	0.667	0.5	0.4
One-dimensional soil stiffness, E'_o (MPa)	40	50	75	100	150	200
Basic settlement (mm)	0.25	0.20	0.133	0.10	0.067	0.05

If we assume a typical saturated soil profile comprising:

- Up to 8.5m saturated thickness of Loose to dense sands (E' of 20 MPa)
- 38 m of Dense to very dense sands (E' of 75 MPa)

then the assessed groundwater induced surface settlement per metre of drawdown is less than 10 mm.

Considering the predicted drawdown of 1.5 m at the excavation face for Apartment 1, and 0.6m for Apartment 2, the predicted groundwater drawdown induced settlement is predicted to be less than 15mm and 6mm respectively, decreasing with distance from the excavation face. We consider the potential for adverse groundwater drawdown induced surface settlement impacts outside the site due to construction dewatering is low.

5.7 MINIMAL IMPACT ASSESSMENT

Table 11 presents a 'Minimal Impact Consideration' for Aquifer Interference Activities for Alluvial Water Sources following the NSW Aquifer Interference Policy (NSW Department of Primary Industries, 2012) [11]. Based on this we conclude that the development is a low risk for groundwater impacts.

Table 11. Minimal impact consideration

Consideration	Response
WATER TABLE 1. Less than or equal to a 10% cumulative variation in the water table, allowing for typical climatic "post-water sharing plan"(2) variations, 40m from any: (a) high priority groundwater dependent ecosystem; or (b) high priority culturally significant site; listed in the schedule of the relevant water sharing plan; or A maximum of a 2m decline cumulatively at any water supply work. 2. If more than 10% cumulative variation in the water table, allowing for typical climatic "post-water sharing plan" variations, 40m from any: (a) high priority groundwater dependent ecosystem; or (b) high priority culturally significant site; listed in the schedule of the relevant water sharing plan then appropriate studies(5) will need to demonstrate to the Minister's satisfaction that the variation will not prevent the long-term viability of the dependent ecosystem or significant site. If more than 2m decline cumulatively at any water supply work then make good provisions should apply.	Compliant. a) Predicted less than 0.3m variation in water table at GDE to the west of Apartment 1 which is less than a 10% cumulative variation in the water table assuming a saturated thickness of over 20 m in the coastal sands aquifer b) There is not known to be a high priority culturally significant site within 40 m from the site c) Drawdown is anticipated to be less than 2m.
WATER PRESSURE 1. A cumulative pressure head decline of not more than 40% of the "post-water sharing plan"(2) pressure head above the base of the water source to a maximum of a 2m decline, at any water supply work. 2. If the predicted pressure head decline is greater than requirement 1. above, then appropriate studies are required to demonstrate to the Minister's satisfaction that the decline will not prevent the long-term viability of the affected water supply works unless make good provisions apply.	Compliant. Drawdown is anticipated to be less than 2m.
WATER QUALITY 1. (a) Any change in the groundwater quality should not lower the beneficial use category of the groundwater source beyond 40m from the activity; and (b) No increase of more than 1% per activity in long-term average salinity in a highly connected surface water source at the nearest point to the activity. Redesign of a highly connected(3) surface water source that is defined as a "reliable water supply"(4) is not an appropriate mitigation measure to meet considerations 1.(a) and 1.(b) above. (c) No mining activity to be below the natural ground surface within 200m laterally from the top of high bank or 100m vertically beneath (or the three dimensional extent of the alluvial water source - whichever is the lesser distance) of a highly connected surface water source that is defined as a "reliable water supply". (d) Not more than 10% cumulatively of the three dimensional extent of the alluvial material in this water source to be excavated by mining activities beyond 200m laterally from the top of high bank and 100m vertically beneath a highly connected surface water source that is defined as a "reliable water supply". 2. If condition 1.(a) is not met then appropriate studies will need to demonstrate to the Minister's satisfaction that the change in groundwater quality will not prevent the long-term viability of the dependent ecosystem, significant site or affected water supply works. If condition 1.(b) or 1.(d) are not met then appropriate studies are required to demonstrate to the Minister's satisfaction that the River Condition Index category of the highly connected surface water source will not be reduced at the nearest point to the activity. If condition 1.(c) or (d) are not met, then appropriate studies are required to demonstrate to the Minister's satisfaction that: - there will be negligible river bank or high wall instability risks; - during the activity's operation and post-closure, levee banks and landform design should prevent the Probable Maximum Flood from entering the activity's site; and - low-permeability barriers between the site and the highly connected surface water source will be appropriately designed, installed and maintained to ensure their long-term effectiveness at minimising interaction between saline groundwater and the highly connected surface water supply;	Compliant. The development will act as a groundwater sink during construction, and injection/recharge of water to the groundwater system is not currently proposed. Due to the potential for PASS, there is a moderate risk of contamination to the aquifer due to oxidisation of PASS soils from drawdown. Refer to Section 5.5. The closest saltwater source to the site is over 175 m away (Hunter River). Based on the assessed groundwater levels during dewatering, which indicate negligible change to the groundwater gradient near the Hunter River, the risk of saltwater intrusion into the aquifer because of the proposed development is considered to be negligible. Refer to Section 5.4

6. PRELIMINARY DEWATERING MANAGEMENT PLAN

6.1 IDENTIFIED RISKS AND OBJECTIVES

The following risks have been identified in relation to construction dewatering for the works:

- Limited groundwater information is currently available on the site, which increases the risk of higher groundwater inflows and related impacts than anticipated, or conversely, an overestimation of likely inflows and impacts.
- No groundwater chemistry testing has been undertaken within the site, providing uncertainty of potential disposal options for the anticipated groundwater inflows.
- There is a potential of oxidising PASS during dewatering works.

The objectives of the management plan are to:

- Provide details of a proposed groundwater investigation to obtain further site-specific data for groundwater inflow analysis.
- Provide preliminary guidance around trigger levels and anticipated hold points that may be required.

6.2 FURTHER GROUNDWATER INVESTIGATIONS

Additional geotechnical and groundwater investigations would be required to provide site-specific data to support the assumptions made in this report. RCA had previously prepared a Draft Sampling Protocol for Further Targeted Assessment (Ref. 15442-404/2, dated June 2024) which outlines a proposed groundwater well installation and sampling methodology for further assessment of contamination. In addition to this, the Department of Planning and Environment (DPIE) provides guidance on the required investigation needed for groundwater assessments and dewatering plans in *Minimum requirements for building site groundwater investigations and reporting* (October 2022) . The following sections outline a suggested groundwater investigation based on these documents that would be required to satisfy the conditions of this document.

6.2.1 Monitoring bore installation

Sites require a minimum of three monitoring bores within the vicinity of the excavation area during construction to provide ongoing regular monitoring information where dewatering is required. The location of these bores should be that triangulation of groundwater elevations are possible to determine hydraulic flow direction and gradient across the excavation areas.

RCA provided locations of five proposed groundwater well installations to be installed to a depth of approximately 3.5m BGL, or a minimum 2m below groundwater levels across the site. The purpose of these wells were for groundwater sampling for contamination purposes. It is recommended that two additional groundwater wells are installed around the apartment structures and screened across the deeper Unit 2 dense sand layers below -7.0m AHD (approximately 10m below ground level). The depth of the boreholes should extend a minimum 3m below the toe of the lowest retaining wall structure. This will provide an appropriate coverage for triangulation of groundwater levels and flow direction. It will also allow for hydraulic testing of the deeper Unit 2 soils as described in Section 6.2.2 below. Figure 15 provides an overview of the proposed groundwater well locations.

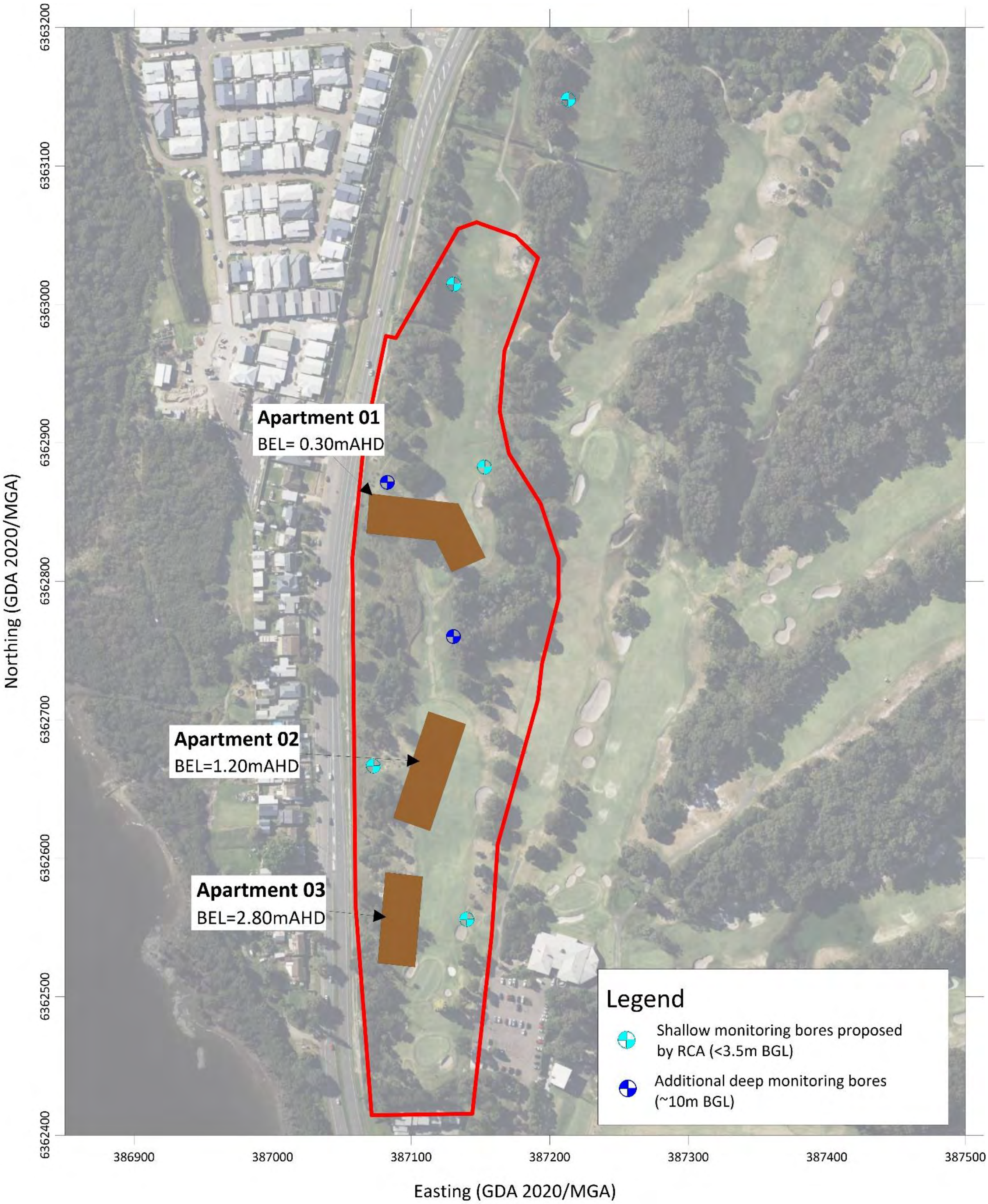


Figure 15. Proposed location of monitoring bores

Boreholes should be drilled in a manner that allows accurate logging of soil profiles along with in-situ testing comprising Standard Penetration Tests (SPT) typically at 1.5m intervals to allow for assessment of soil consistency and obtaining soil samples. The investigation should be undertaken in accordance with AS1726-

2017 with bores constructed in general accordance with *Minimum Construction Requirements for Water Bores in Australia*.

The wells should be positioned such that they are not damaged or destroyed during construction works. Where wells are damaged, they must be replaced with like-for-like wells in the same general vicinity where possible.

6.2.2 Aquifer testing

Following installation of the monitoring bores, each bore should be developed, and then site-specific aquifer testing undertaken inform the hydraulic conductivity values. This would comprise rising or falling head (slug) tests undertaken within the monitoring bores. Given the high permeability nature of the ground conditions, it is recommended to use a solid ‘slug’ to change the water level in the well, rather than pumping. Permeability tests should be undertaken at least three times in each to demonstrate repeatability.

6.2.3 Groundwater monitoring and water quality testing

Following installation of the groundwater wells, RCA provides a proposed groundwater sampling methodology for the testing of potential contaminants within the wells including TRH, BTEXN, PAH, heavy metals, nutrients, pesticides, herbicides PFAS and microorganisms. In addition to these analytes, samples should also be tested for the criteria outlined in Section 7.2 further in this report to assess suitability for discharge. Samples should be collected in accordance with the guidelines provided by the Australian Standard for water quality sampling (AS/NZS 5667.1:1998) and analysed by a NATA registered laboratory.

Upon completion of initial development and round of sampling at the wells, continuous groundwater monitoring should be undertaken to assess long-term groundwater levels. This will include installation of groundwater data loggers within the five of the wells adjacent to the apartment buildings to record changes in groundwater levels from tidal fluctuation. Groundwater monitoring should ideally occur for a minimum of three months within the six-month period prior to application for dewatering.

During the monitoring period, manual gauging of the wells to correlate with data loggers and field readings should be taken from the wells at regular intervals (i.e. monthly). Field readings should comprise the use of a water quality meter to test for pH, EC, turbidity and dissolved oxygen.

6.2.4 Reporting

Following this, an updated Dewatering Management Plan should be prepared which takes into consideration the findings of this preliminary report along with the additional investigation data collected from the site. Re-assessment of groundwater levels and inflow amount would need to be undertaken along with a comparison against regulatory bodies for groundwater chemistry and quality parameters.

6.3 DEWATERING METHODOLOGY

Details of the proposed dewatering system are not known at the time of this report, however Tetra Tech understands the proposed construction methodology will likely comprise a cantilevered continuous structural wall (e.g. secant pile wall, sheet pile wall, etc.) to retain and support the excavation.

We anticipate that the approximate dewatering methodology would be as follows:

1. Conduct additional investigation works. Obtain relevant permits and approval for works.
2. Install shoring wall around basement perimeter for geotechnical support purposes to a depth as required.

3. Install dewatering system. This may comprise a series of well spear points installed within the footprint of the excavation attached to a vacuum header pipe and a suction pump system (designed by others).
4. Begin excavation along with dewatering to lower the groundwater table below Bulk Excavation Level to a depth as required for constructability purposes. Treatment and disposal of groundwater inflows would need to be undertaken which is discussed further below.
5. Continue dewatering to maintain groundwater levels below BEL during installation of foundations, hydrostatic slab and tanked basement.
6. Once basement is tanked, switch off dewatering system.

Following the construction of the base-slab, dewatering of groundwater may need to continue until sufficient deadload (from the building) has been constructed to resist hydraulic uplift of the base-slab. The effects of groundwater uplift pressures on the slab will need to be considered by the structural designer with adequate consideration of long-term design groundwater levels, considering climate change and the effects of significant rainfall events.

It is anticipated that dewatering could comprise a wellpoint system (spears) located at regular intervals (1 to 2m spacing) outside the perimeter of the basement retaining structures. Figure 16 and Figure 17 below show typical wellpoint dewatering systems and alternative dewatering systems if required.

Figure 16. Typical wellpoint dewatering system installed inside basement footprint.

Figure 17. Typical localised pit and sump pump for additional dewatering if required.

As noted in Section 4.2, the assessed sustained inflow into the excavation where a retaining wall is adopted may be up to 16L/s for Apartment 1, and up to 5.6L/s for Apartment 2. Storage, treatment and disposal of such inflows should be considered by the dewatering contractor and may include:

- Temporary storage, treatment and disposal into local stormwater assets within Nelson Bay Road, or
- Re-injection into the ground via infiltration pits or injection system.

Where the latter is considered, further analysis would need to be undertaken to ensure that the re-injection system meets the requirements for groundwater contamination levels. The location and amount of re-injection would need to be considered in further analysis to ensure that:

- Re-injection does not influence nearby GDE's with respect to groundwater level or quality.
- The re-injection does not adversely affect the proposed excavation through higher groundwater levels or increased flows.

Dewatering should be managed by a specialist dewatering contractor. The site manager during construction and operation will be responsible for implementing the water quality management procedures described in this report.

6.4 TREATMENT OF DISCHARGED WATER

The purpose of water treatment is to prevent adverse impacts on the receiving waters by effectively treating extracted groundwater prior to pumping to the stormwater system. No groundwater testing has been undertaken to date to assess the current water quality. The implementation of the proposed investigation outlined in Section 6.2 should be undertaken to assess the baseline water quality levels prior to dewatering.

In absence of this data, it is likely that groundwater may require some level of treatment prior to discharge, usually for pH, turbidity, salinity, heavy metals, and nutrients. A groundwater filtration and pH buffering treatment system to stabilise pH and turbidity could comprise:

- A baffled treatment tank.
- Automated in-line chemical dosing systems for the addition of buffering solutions and coagulants.
- A flow metre to record extraction volumes and total flows.
- Emergency response alarms for non-routine situations; and
- A sedimentation tank to provide additional residence time and sedimentation.

Additional filtration, dosing or other methods would be required to remove other contaminants such as heavy metals and nutrients to bring groundwater levels within specified trigger values. Further to this, specific treatment may be required where PASS soils are disturbed, generating sulfuric acid which would be drawn into the excavation. A specialist dewatering contractor would need to be engaged to assess the specific needs of the system to meet the required trigger value guidelines.

Pumping to the stormwater system will be subject to the approval by Port Stevens Council. Where water quality does not meet discharge criteria, discharge must halt immediately, and further testing and remediation may be required prior to further release.

6.5 WATER ACCESS LICENSING

For basement dewatering in NSW and in accordance with the Water Management Act 2000, the following two items from WaterNSW is generally required:

1) Water Access License.
2) Water Supply Works Approval.

The site is located within the Stockton Coastal Groundwater Source as shown in Figure 18, which is covered under the Water Sharing Plan for the North Coast Coastal Sands Groundwater Sources.

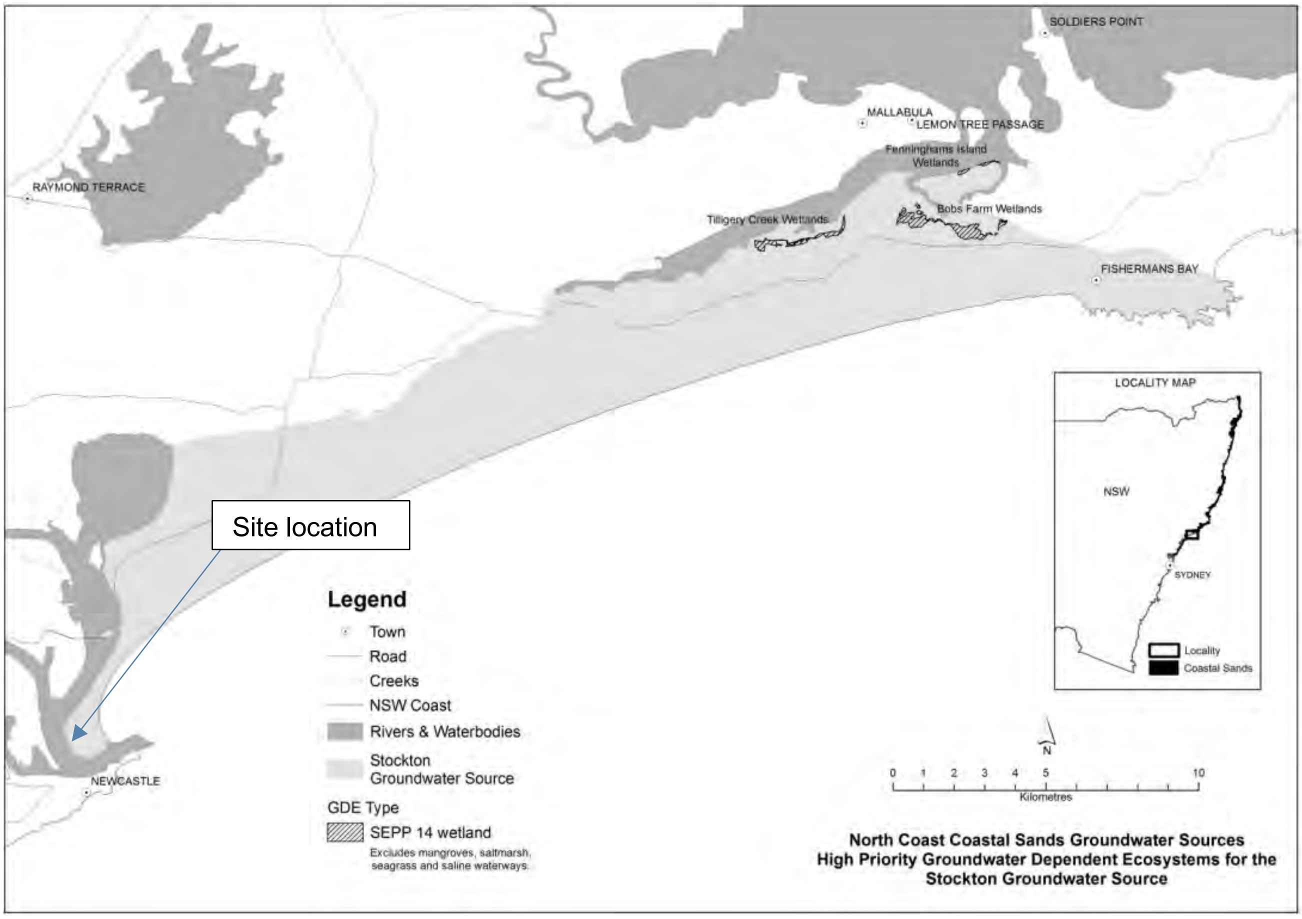


Figure 18: Stockton Groundwater Source

A water access license is required where more than 3 megalitres of groundwater are extracted through an aquifer interference activity (such as dewatering) per authorised project per water year. Considering the assessed groundwater inflows, a Water Access Licence is likely be required for these works.

7. PROPOSED TRIGGER LEVELS AND RESPONSES

During construction dewatering, a groundwater monitoring report should be prepared monthly by an experienced hydrogeologist providing factual data on the recorded dewatering volumes, quality of discharged water, and groundwater levels at monitoring piezometers. The monitoring report must also comment on a comparison of observations versus the predictions in this report, in particular highlighting, where applicable:

- Increase in risk to existing groundwater users due to higher-than-expected drawdowns.
- Whether the discharged water quality exceeding the proposed trigger levels.
- Whether observed groundwater discharge volumes are exceeding the predictions in this report.

7.1 GROUNDWATER LEVELS

Groundwater levels should be confirmed in monitoring wells prior to the commencement of dewatering activities. Groundwater levels should be:

- o Monitored to enable assessment of groundwater drawdown due to construction activities.
- o Measured at monitoring wells daily during the early stages (first week) of dewatering, and approximately weekly thereafter.
- o Reviewed monthly in conjunction with ground settlement/movement monitoring to assess groundwater drawdown and its potential impacts.

Consistent with DPI Water requirements, monitoring should be conducted through the construction period and for a period of two months following construction. Monitoring is to continue as nominated for six months after completion of tanked structure and decommissioning of the dewatering system.

Trigger levels for groundwater wells should be assessed following implementation of the additional investigations outlined in Section 6.2 along with reassessment of anticipated drawdown curves based on the adopted construction methodology including nominated retaining wall depth. Where this is undertaken, a preliminary trigger level set at 0.5m below this anticipated groundwater level for the construction monitoring bores within the site could be adopted. Where groundwater levels are deeper than this, action in accordance with Table 12 is required. Groundwater levels in the nominated piezometers will need to be checked prior to construction/dewatering.

Table 12. Required Actions upon trigger of groundwater levels

Monitoring Element	Trigger Level	Action
Groundwater levels	Monitored groundwater level is above trigger level	Continue excavation/dewatering
	Monitored groundwater level is below trigger level	Continue excavation/dewatering, notify Tetra Tech Coffey or relevant hydrogeologist. Review of monitoring results required.

7.2 GROUNDWATER QUALITY

The Australian and New Zealand Guidelines for Fresh and Marine Water Quality (ANZG 2018) [12] provide detailed approaches and advice on identifying appropriate guideline criteria for the protection of environmental receptors.

PSC's RFI stipulates that water quality monitoring is to be self-certified by an experienced groundwater specialist and is to be tested weekly (Clause 9 (b)).

For the protection of aquatic ecosystems, locally derived guideline values are most appropriate. However, in the case that these values are absent, ANZG (2018) provide default guideline values for assessing physio-chemical parameters and the impact of potential toxins on aquatic ecosystems. If default values are not available, the Australian and New Zealand Environment and Conservation Council (ANZECC) & Agriculture and Resource Management Council of Australia and New Zealand (ARMCANZ), 2000 [13] trigger values should be adopted.

We understand the point of discharge is currently being assessed, subject to discussions with dewatering contractors, adjacent property owners, and Council's stormwater system. However, we anticipate that discharge of groundwater inflows would likely be taken to Council stormwater system which would discharge to the Hunter River to the west which is an estuarine environment. In accordance with ANZECC guidelines, the Hunter River could be considered a slightly to moderately disturbed ecosystem due to upstream activities and a 95% protection criteria should be adopted.

The NSW Department of Environment and Heritage provides Water Quality Objectives (WQO) for the Hunter River based on site-specific data and ANZECC 2000 guidelines. Furthermore, Port Stephens Council has also provided minimum water quality requirements for discharge into their stormwater assets. The general hierarchy of this information for assessment against trigger values should be as following:

1. Hunter River Water WQO
2. ANZECC 2000 Guidelines
3. PSC requirements.

Table 13 below provides a summary of the trigger values adopted for aquatic ecosystems of the Hunter River within estuarine environments based on this hierarchy.

Table 13. Trigger values for groundwater discharge

Indicator	Trigger Value	Comments
Total Phosphorus	30 µg/L	Hunter River WQO
Total Nitrogen	300 µg/L	Hunter River WQO
Chlorophyll-a	4 µg/L	Hunter River WQO
Total Suspended Solids (TSS)	< 50mg/L	PSC requirements
Salinity / Electrical Conductivity	2200 µS/cm	Hunter River WQO
Dissolved Oxygen	80 – 110%	Hunter River WQO
pH	7.0 – 8.5	Hunter River WQO
Chemical Contaminants	ANZECC 2000 Guidelines, chapter 3.4 and table 3.4.1. (95% protection criteria)	Hunter River WQO
Odour	No odour or visible petrol chemical sheen	PSC requirements
Visible Debris	No visible litter or waste matter	PSC requirements

The list of chemical contaminants under ANZECC 2000 Guidelines, chapter 3.4, and table 3.4.1. are extensive and generally not considered feasible for testing of all potential contaminants. We recommend limiting the testing to the proposed list of contaminants as proposed by RCA in their Sampling Protocol.

Where concentrations are greater than (or analytes are outside the ranges of) those nominated in Table 13, action in accordance with Table 14 are required.

Table 14. Required Actions upon trigger of groundwater quality

Monitoring Element	Trigger Level	Action
Groundwater Quality – Observation	Observations of water are consistent with previous observations	Continue discharging water
	Observations of water are inconsistent with previous observations	Stop discharging, conduct water quality sampling and testing to check consistency with previous discharges
Groundwater Quality – Sampling and Analysis	Monitored groundwater quality analyte is below/within trigger level	Continue discharging water
	Monitored groundwater quality analyte is above/outside trigger level	Stop discharging, treat water further to achieve required quality. Employ alternate disposal options such as tanker or temporary storage if quality is not achieved

7.3 GROUNDWATER VOLUME

Groundwater volume should be recorded daily and checked against the predicted values as presented in this report. During the initial stages of dewatering, it is likely that the dewatering volume may be higher than anticipated as the existing soils have a large portion of retained water which will need to initially drain out before reaching a steady state.

Where groundwater inflows are consistently higher than anticipated (i.e. 20% or more), reassessment of the groundwater analysis and input parameters would be required by a hydrogeologist to re-calibrate the groundwater model and check anticipated drawdown curves along with impacts to nearby users and the environment.

Table 15. Required Actions upon trigger of groundwater volume

Monitoring Element	Trigger Level	Action
Groundwater Volume	Observations of volume are within 20% of estimated inflow rate	Continue dewatering
	Observations of volume are consistently more than 20% of estimated inflow rate	Continue excavation/dewatering. Review of inflow monitoring results by a hydrogeologist.

7.4 SUMMARY OF GROUNDWATER MONITORING REQUIREMENTS

Table 16 summarises the proposed groundwater monitoring requirements during the dewatering period.

Table 16. Groundwater Monitoring Requirements

Monitoring Element	Purpose	Assessment / Monitoring Methodology	Frequency of Monitoring	Frequency of Monitoring Data Review	Duration of Monitoring
Groundwater Levels	• Confirm dewatering is proceeding in accordance with expectations.	Measurement of groundwater levels in at least three wells outside of basement footprint (Refer to Section 6.2.1).	Daily during the early stages of dewatering, weekly thereafter.[1]	Monthly	Throughout construction and for a period of six months following switching off the dewatering system.
Discharge Water Quality[2]	• Reduce risk of discharge of unsuitable water to stormwater.	**Observation** of odours, changes to water colour, pH and turbidity. [3]	Daily	Monthly	Throughout construction up until switching off the dewatering system.
		Sampling and Analysis at a NATA registered laboratory for the analytes in Section 7.2.	Weekly, and at any time changes are observed during daily observation	Monthly, and at any time changes are observed during daily observation	Throughout construction up until switching off the dewatering system.
Volume of water Discharge	• Confirm dewatering is proceeding in accordance with expectations.	Volume of water discharged. [4]	Daily	Monthly	Throughout construction up until switching off the dewatering system.

Notes to table:

1. Continuous water level monitoring could be completed using electronic data loggers installed in boreholes. If electronic data loggers are not used, water levels must be manually measured.
2. Water quality monitoring must be self-certified by an experienced water quality expert.
3. The daily observation monitoring of water quality must be tested onsite for pH and Turbidity (NTU) using a handheld or other testing meter.
4. Where surface runoff or direct rainfall contribute to discharge volumes, a note should be made on the estimated quantity of that contribution. The volume of water removed from site must be recorded daily using calibrated flow meters attached to the dewatering system pipes.

8. CONCLUDING REMARKS

Dewatering for the proposed development will be temporary. Based on the preliminary assessment in this report, temporary dewatering will result in minimal impact to the groundwater system provided monitoring of groundwater discharge volumes and groundwater levels is carried out during construction.

The basements for Apartment 1 and Apartment 2 are expected to intercept the groundwater system within a high permeability coastal sands aquifer, and predicted dewatering inflows may be in excess of 10 L/s during construction of the basements for both Apartment 1 and Apartment 2.

This report is based on limited groundwater data, and subsurface conditions can change over relatively short distances. Groundwater monitoring conducted during construction dewatering should be used to verify the consistency of groundwater (levels, flow and quality) and ground conditions with those assumed/adopted in this assessment.

Further groundwater investigations including groundwater level monitoring, hydraulic conductivity testing, and groundwater quality testing are required prior to construction to provide a refined assessment of predicted groundwater inflows during construction.

The attached document in Appendix A titled “Important Information about your Tetra Tech Coffey Report” presents additional information about the uses and limitations of this report.

9. REFERENCES

[1] NSW Resources, “MinView,” Geosciences Asustralia, [Online]. Available: https://minview.geoscience.nsw.gov.au/.

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[4] Water NSW, “Real-time Data Groundwater Map,” [Online]. Available: https://realtimedata.waternsw.com.au/.

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[8] M. Preene and P. Cashman, Groundwater Lowering in Construction: A Practical Guide to Dewatering, CRC Press, March 2022.

[9] Bureu of Meterology, “Australian Groundwater Explorer,” Australian Government, [Online]. Available: http://www.bom.gov.au/water/groundwater/explorer/map.shtml.

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[11] NSW Department of Primary Industries, “NSW Aquifer Interference Policy,” 2012.

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APPENDIX A: LIMITATIONS

IMPORTANT INFORMATION ABOUT YOUR TETRA TECH COFFEY REPORT

As a client of Tetra Tech Coffey you should know that site subsurface conditions cause more construction problems than any other factor. These notes have been prepared by Tetra Tech Coffey to help you interpret and understand the limitations of your report.

Your report is based on project specific criteria

Your report has been developed on the basis of your unique project specific requirements as understood by Tetra Tech Coffey and applies only to the site investigated. Project criteria typically include the general nature of the project; its size and configuration; the location of any structures on the site; other site improvements; the presence of underground utilities; and the additional risk imposed by scope-of-service limitations imposed by the client. Your report should not be used if there are any changes to the project without first asking Tetra Tech Coffey to assess how factors that changed subsequent to the date of the report affect the report's recommendations. Tetra Tech Coffey cannot accept responsibility for problems that may occur due to changed factors if they are not consulted.

Subsurface conditions can change

Subsurface conditions are created by natural processes and the activity of man. For example, water levels can vary with time, fill may be placed on a site and pollutants may migrate with time. Because a report is based on conditions which existed at the time of subsurface exploration, decisions should not be based on a report whose adequacy may have been affected by time. Consult Tetra Tech Coffey to be advised how time may have impacted on the project.

Interpretation of factual data

Site assessment identifies actual subsurface conditions only at those points where samples are taken and when they are taken. Data derived from literature and external data source review, sampling and subsequent laboratory testing are interpreted by geologists, engineers or scientists to provide an opinion about overall site conditions, their likely impact on the proposed development and recommended actions. Actual conditions may differ from those inferred to exist, because no professional, no matter how qualified, can reveal what is hidden by earth, rock and time. The actual interface between materials may be far more gradual or abrupt than assumed based on the facts obtained. Nothing can be done to change the actual site conditions which exist, but steps can be taken to reduce the impact of unexpected conditions. For this reason, owners should retain the services of Tetra Tech Coffey through the development stage, to identify variances, conduct additional tests if required, and recommend solutions to problems encountered on site.

Your report will only give preliminary recommendations

Your report is based on the assumption that the site conditions as revealed through selective point sampling are indicative of actual conditions throughout an area. This assumption cannot be substantiated until project implementation has commenced and therefore your report recommendations can only be regarded as preliminary. Only Tetra Tech Coffey, who prepared the report, is fully familiar with the background information needed to assess whether or not the report's recommendations are valid and whether or not changes should be considered as the project develops. If another party undertakes the implementation of the recommendations of this report there is a risk that the report will be misinterpreted and Tetra Tech Coffey cannot be held responsible for such misinterpretation.

Your report is prepared for specific purposes and persons

To avoid misuse of the information contained in your report it is recommended that you confer with Tetra Tech Coffey before passing your report on to another party who may not be familiar with the background and the purpose of the report. Your report should not be applied to any project other than that originally specified at the time the report was issued.

Interpretation by other design professionals

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a report. To help avoid misinterpretations, retain Tetra Tech Coffey to work with other project design professionals who are affected by the report. Have Tetra Tech Coffey explain the report implications to design professionals affected by them and then review plans and specifications produced to see how they incorporate the report findings.

Data should not be separated from the report

The report as a whole presents the findings of the site assessment and the report should not be copied in part or altered in any way. Logs, figures, drawings, etc. are customarily included in our reports and are developed by scientists, engineers or geologists based on their interpretation of field logs (assembled by field personnel) and laboratory evaluation of field samples. These logs etc. should not under any circumstances be redrawn for inclusion in other documents or separated from the report in any way.

Geoenvironmental concerns are not at issue

Your report is not likely to relate any findings, conclusions, or recommendations about the potential for hazardous materials existing at the site unless specifically required to do so by the client. Specialist equipment, techniques, and personnel are used to perform a geoenvironmental assessment. Contamination can create major health, safety and environmental risks. If you have no information about the potential for your site to be contaminated or create an environmental hazard, you are advised to contact Tetra Tech Coffey for information relating to geoenvironmental issues.

Rely on Tetra Tech Coffey for additional assistance

Tetra Tech Coffey is familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a project, from design to construction. It is common that not all approaches will be necessarily dealt with in your site assessment report due to concepts proposed at that time. As the project progresses through design towards construction, speak with Tetra Tech Coffey to develop alternative approaches to problems that may be of genuine benefit both in time and cost.

Responsibility

Reporting relies on interpretation of factual information based on judgement and opinion and has a level of uncertainty attached to it, which is far less exact than the design disciplines. This has often resulted in claims being lodged against consultants, which are unfounded. To help prevent this problem, a number of clauses have been developed for use in contracts, reports and other documents. Responsibility clauses do not transfer appropriate liabilities from Tetra Tech Coffey to other parties but are included to identify where Tetra Tech Coffey's responsibilities begin and end. Their use is intended to help all parties involved to recognise their individual responsibilities. Read all documents from Tetra Tech Coffey closely and do not hesitate to ask any questions you may have.

APPENDIX B: SURVEY PLANS

NOTES:

1. FEATURES SHOWN TO SCALE ACCURACY.
2. THIS PLAN IS SUITABLE FOR DETAILED PLANNING AND DESIGN AT THE SCALE/S STATED. THE PLAN MAY NOT BE SUITABLE FOR ANY OTHER PURPOSE OR FOR USE AT ANY OTHER SCALE/S.
3. SERVICES LOCATED ONLY WHERE VISIBLE.
4. THE LOCATION OF ALL UNDERGROUND SERVICES WHETHER SHOWN ON THE PLAN OR NOT, SHOULD BE PRECISELY DETERMINED BEFORE ANY CONSTRUCTION WORK COMMENCES AND MEASURES TAKEN TO PROTECT THESE SERVICES FROM DAMAGE.
5. CONTOUR INTERVAL - 0.5m
6. THE BOUNDARIES SHOWN ARE APPROXIMATE ONLY. THE BOUNDARIES SHOWN HAVE BEEN COMPILED FROM THE RELEVANT DEPOSITED PLANS. FURTHER SURVEY WILL BE REQUIRED IF CONSTRUCTION IS TO TAKE PLACE ON OR ADJACENT TO THE BOUNDARIES.
7. LOT 105 D.P.614883 IS AFFECTED BY A RESTRICTION ON THE USE OF LAND CREATED BY D.P.250673.

TREE TABLE

TREE NO.	SPREAD	TRUNK	HEIGHT
T39	4	0.2	6
T40	6	0.4	8
T41	7	0.5	13
T90	6	0.4	9
T91	8	0.4	10
T92	12	0.6	10
T93	12	0.4	10
T94	12	0.4	10
T95	8	0.4	10
T96	4	0.6	6
T97	12	0.6	11
T98	12	0.8	14
T99	12	0.4	4
T100	6	0.3	13
T101	8	0.4	13
T102	9	0.6	11
T103	11	0.5	12
T104	4	0.2	5
T105	8	0.6	10
T106	8	0.3	10
T107	5	0.1	5
T108	6	0.1	9
T109	8	0.2	9
T110	6	0.2	8
T111	6	0.2	8
T112	6	0.2	8
T113	14	0.8	18
T114	12	0.3	16
T115	12	0.6	16
T116	12	0.6	16
T117	8	0.5	10
T118	8	0.5	12
T119	8	0.4	12
T120	8	0.3	11
T121	5	0.2	5
T122	8	0.3	9
T123	8	0.3	5
T124	9	0.5	13
T125	16	0.8	14
T126	7	0.3	8
T127	16	0.5	10
T128	16	0.8	14
T129	16	0.8	16
T130	8	0.5	10
T131	8	0.5	10
T132	8	0.5	10
T133	8	0.7	10
T134	9	0.5	11
T135	12	0.6	9
T136	9	0.5	9
T137	11	0.5	10
T138	6	0.2	9
T139	8	0.3	8
T140	10	0.4	10
T141	6	0.2	8
T142	4	0.2	6
T143	4	0.2	6
T144	8	0.3	10
T145	7	0.3	10
T146	7	0.3	10
T147	4	0.1	7
T148	4	0.1	7
T149	6	0.2	7
T150	5	0.2	5
T151	9	0.8	9
T152	7	0.2	7
T153	5	0.8	5
T154	5	0.8	5
T155	10	1.2	15
T156	10	0.8	12
T157	6	0.3	8
T158	11	0.8	12
T159	9	0.3	9
T160	7	0.4	8
T161	10	0.5	8
T162	5	0.3	4
T163	5	0.3	8
T164	5	0.5	5
T165	5	0.5	10
T166	10	0.8	8
T167	5	0.5	12
T168	8	0.7	12
T169	3	0.4	5
T171	5	0.5	4
T172	5	0.9	10
T173	4	0.5	5
T174	5	0.4	3
T175	3	0.3	4
T176	3	0.3	4
T177	3	0.3	4
T178	3	0.3	4
T180	4	0.3	4
T181	5	0.6	8
T182	4	0.3	4
T183	5	0.3	4
T184	5	0.3	3
T185	12	0.8	10
T186	10	0.8	3
T187	5	0.3	5
T188	5	0.8	5
T189	3	0.3	4
T190	6	0.2	7
T191	6	0.4	5
T192	5	0.3	6
T195	8	1.0	7
T196	4	0.5	7
T197	10	1.0	7
T198	4	0.4	4
T199	4	0.4	6
T200	4	0.3	6
T201	4	1.2	6
T202	4	0.8	6
T203	6	0.8	10
T204	7	0.4	8
T205	8	0.8	6
T206	4	0.2	4
T207	7	0.2	7
T210	6	0.3	8
T211	5	0.3	5
T212	4	0.2	6
T213	6	0.3	7
T217	12	1.5	12
T218	12	1.5	12
T219	8	0.8	10
T220	10	0.8	10
T221	10	0.5	10
T222	10	0.5	10
T223	10	0.7	10

TREE TABLE
(NO TAGS)

TREE NO.	SPREAD	TRUNK	HEIGHT
T801	8	0.4	6
T802	4	0.2	7
T803	6	0.3	6
T804	5	0.2	5
T805	7	0.4	5
T806	7	0.4	9
T807	7	0.4	9
T808	7	0.4	9
T809	7	0.4	9
T810	7	0.4	9
T811	5	0.2	8
T812	5	0.2	5
T813	5	0.2	4
T814	5	0.2	8
T815	7	0.4	9
T816	8	0.5	10
T817	8	0.5	10
T818	14	1.5	14
T819	7	0.4	9
T820	7	0.4	9
T821	7	0.4	9
T822	7	0.4	9
T823	7	0.4	9
T824	7	0.4	9
T825	7	0.4	9
T826	7	0.2	8
T827	5	0.2	8
T828	5	0.2	8
T829	5	0.2	8
T830	7	0.4	9

LEGEND

- BOUNDARY LINE
- ADJACENT BOUNDARY
- MINOR CONTOUR LINE
- MAJOR CONTOUR LINE
- RETAINING WALL
- FENCE LINE
- TOP & TOE OF BANKS
- OVERHEAD POWER LINE
- POWER POLE
- LIGHT POLE
- ELECTRICAL PILLAR
- TELSTRA PIT
- TELSTRA POST
- DRAINAGE PIT
- STOP VALVE
- HYDRANT
- WATER METER
- SEWER MAN HOLE
- DENOTES SIGNAGE
- DENOTES TREE

REV.	DATE	AMENDMENT(S)	SUR	DFT	CHK
A	08.06.21	ORIGINAL ISSUE	SG	PV	PM
B	05.11.21	ADDITIONAL DETAIL	SG	JD	PM

DELFS LASCELLES
CONSULTING SURVEYORS

260 MAITLAND ROAD,
MAYFIELD NSW 2304
ABN: 28 164 260 100

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E: admin@delacs.com.au
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CAD REF: 21493 - DET B

DETAIL SURVEY OF PART OF LOT 4 DP 823114 AND LOT 105 DP 614883

SITE ADDRESS:
4A VARDON ROAD
FERN BAY

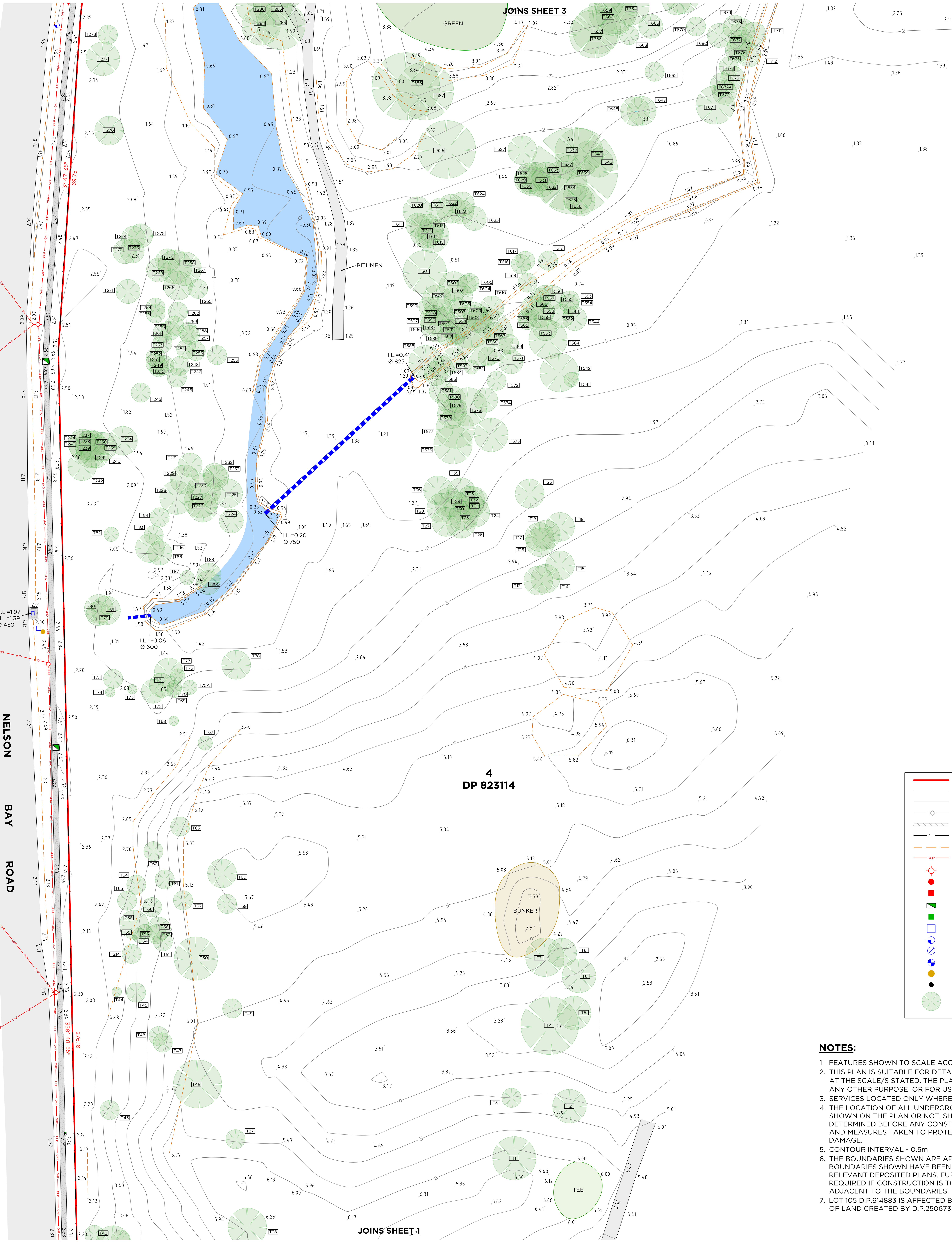
CLIENT:
AVID PROJECT MANAGEMENT SERVICES

POSITION DATUM:	PM 16471
ORIENTATION:	MGA (GROUND)
EASTING:	387 054.954
NORTHING:	6 362 579.869
CLASS: B	ORDER: -
HEIGHT DATUM:	PM 16470
CLASS: LB	ORDER: -
RL:1.974	DATE: 26.05.21

SURVEYED	DRAFTED	CHECKED
SG	JD	PM
SCALE	PAGE SIZE	DATE
1:400	A1	21.10.21
SHEET	REV.	PROJECT No.
1/4	B	21493

TREE TABLE

TREE NO.	SPREAD	TRUNK	HEIGHT
T1	9	0.7	8
T2	7	0.8	5
T3	4	0.1	4
T4	12	0.8	11
T5	7	0.6	10
T6	7	0.2	5
T7	8	0.4	6
T8	6	0.2	6
T13	6	0.5	6
T14	6	0.1	5
T15	6	0.6	10
T16	6	0.2	8
T17	6	0.2	8
T18	6	0.4	8
T19	6	0.4	8
T23	6	0.5	7
T24	6	0.2	8
T25	6	0.2	8
T26	6	0.3	8
T27	6	0.2	8
T28	6	0.3	8
T29	6	0.2	8
T30	6	0.2	8
T31	6	0.1	8
T32	6	0.2	8
T33	6	0.2	8
T35	6	0.3	8
T36	6	0.7	10
T37	4	0.3	10
T38	8	0.5	8
T42	4	0.2	6
T43	4	0.6	5
T44	2	0.3	5
T45	4	0.3	5
T46	10	0.7	9
T47	4	0.5	5
T48	4	0.5	5
T49	4	0.3	7
T50	9	0.5	8
T51	4	0.6	6
T52	4	0.3	6
T53	3	0.3	6
T54	3	0.3	6
T55	1	0.1	3
T56	5	0.2	5
T57	4	0.6	8
T58	6	0.3	7
T59	4	0.4	4
T60	6	0.8	4
T61	4	0.6	8
T62	4	0.6	8
T63	4	0.3	4
T64	4	0.5	6
T65	4	0.3	6
T66	4	0.3	6
T67	3	0.3	3
T68	2	0.2	5
T69	3	0.3	3
T70	3	0.3	3
T71	5	0.3	12
T72	5	0.6	12
T73	2	0.3	4
T74	2	0.3	4
T75	4	0.3	8
T75A	3	0.5	3
T76	3	0.5	6
T77	6	0.2	7
T78	6	0.8	6
T79	5	0.2	9
T80	6	0.3	10
T81	6	0.3	10
T82	3	0.1	4
T83	4	0.2	6
T84	4	0.3	4
T87	3	0.1	5
T88	5	0.2	5
T214	4	0.5	6
T216	6	0.2	8
T224	5	0.3	6
T225	6	0.3	7
T226	6	0.3	7
T227	7	0.7	9
T228	7	0.3	7
T229	6	0.3	7
T230	7	0.7	9
T231	5	0.2	6
T232	5	0.2	6
T233	5	0.2	6
T234	6	0.2	5
T235	6	0.2	8
T236	6	0.2	8
T237	6	0.2	8
T238	6	0.2	8
T239	6	0.2	8
T240	6	0.2	8
T241	6	0.2	8
T242	6	0.3	7
T243	6	0.2	8
T244	6	0.2	8
T245	4	0.2	5
T246	4	0.2	5
T247	4	0.2	5
T248	4	0.2	5
T249	4	0.2	5
T250	4	0.2	5
T251	4	0.2	5
T252	4	0.2	5
T253	4	0.2	5
T254	4	0.2	5
T255	4	0.2	5
T256	4	0.2	5
T257	4	0.3	5
T258	4	0.2	7
T259	4	0.2	7
T260	4	0.2	7
T261	4	0.2	7
T262	4	0.3	4
T263	4	0.2	5
T264	4	0.2	5
T265	4	0.2	5
T266	6	0.3	8
T267	7	0.8	7
T268	7	0.7	8
T269	5	0.2	7
T270	5	0.2	7
T271	5	0.2	5
T272	5	0.2	5
T273	5	0.2	5
T274	5	0.2	5
T275	5	0.2	5
T276	6	0.4	7
T277	7	0.5	8
T278	4	0.2	6
T283	10	1.2	13
T284	8	0.6	13
T285	8	0.6	13
T286	8	0.6	13
T541	3	0.1	6
T542	8	0.8	10
T544	5	0.2	12
T553	4	0.2	12
T554	4	0.2	12
T555	5	0.2	12
T556	4	0.1	12
T557	4	0.1	12
T558	4	0.2	10
T559	6	0.4	12
T560	5	0.3	12
T561	1	0.2	8
T562	5	0.2	8
T563	5	0.2	8
T564	8	0.4	12
T565	4	0.2	12
T566	4	0.2	12
T567	4	0.2	12
T568	12	1.5	12
T569	4	0.2	10
T570	4	0.2	10
T571	5	0.3	10
T572	4	0.2	7
T573	7	0.2	6
T574	5	0.3	7
T575	8	0.6	11
T576	8	0.6	11
T577	8	0.5	11
T578	6	0.4	11
T579	5	0.2	10
T580	6	0.5	11
T581	5	0.4	11
T582	3	0.1	10
T583	5	0.2	7
T584	5	0.4	9
T585	5	0.2	7
T586	14	1.2	16
T587	12	0.8	14
T588	7	0.3	11
T589	4	0.2	8
T590	4	0.2	8
T591	4	0.2	8
T592	1	0.3	6
T593	5	0.2	9
T594	9	0.4	9
T595	4	0.2	9
T596	7	0.4	12
T597	6	0.3	10
T598	6	0.4	10
T599	4	0.2	10
T600	6	0.2	8
T601	9	0.6	12
T602	6	0.3	7
T603	6	0.3	10
T604	4	0.2	10
T605	4	0.2	10
T606	4	0.3	10
T607	5	0.3	8
T608	5	0.3	8
T609	5	0.3	8
T610	5	0.2	8
T611	5	0.1	8
T612	7	0.5	12
T613	5	0.3	10
T614	5	0.2	10
T615	7	0.5	12
T616	4	0.2	8
T617	4	0.2	8
T618	4	0.2	8
T619	4	0.2	8
T620	6	0.5	8
T621	5	0.2	8
T622	6	0.5	10
T623	5	0.3	8
T624	5	0.3	8
T625	5	0.3	8
T626	12	0.8	10
T627	6	0.2	6
T628	6	0.4	8
T629	6	0.4	8
T630	6	0.2	8
T631	6	0.2	8
T632	6	0.6	8
T633	6	0.2	8
T634	10	0.8	12
T635	6	0.2	10
T636	6	0.2	10
T637	5	0.2	10
T638	10	0.7	10
T639	18	1.4	16
T642	6	0.4	8
T643	6	0.4	8
T648	6	0.4	10
T649	6	0.4	10
T656	10	0.4	9
T657	10	0.4	9
T659	10	0.8	12
T660	8	0.2	9
T662	4	0.2	5
T663	4	0.2	6
T664	8	0.3	8
T666	5	0.2	6
T670	4	1.0	4
T671	8	0.5	5
T672	5	0.2	8
T672A	5	0.2	8
T673	5	0.2	8
T674	4	0.1	8
T675	5	0.2	8
T676	5	0.5	8
T677	5	0.2	8
T678	5	0.2	7
T679	5	0.1	7
T680	5	0.2	7
T711	5	0.3	7
T712	5	0.2	7



TREE TABLE
(NO TAGS)

TREE NO.	SPREAD	TRUNK	HEIGHT
T800	5	0.2	6

LEGEND

- BOUNDARY LINE
- ADJACENT BOUNDARY
- MINOR CONTOUR LINE
- MAJOR CONTOUR LINE
- RETAINING WALL
- FENCE LINE
- TOP & TOE OF BANKS
- OVERHEAD POWER LINE
- POWER POLE
- LIGHT POLE
- ELECTRICAL PILLAR
- TELSTRA PIT
- TELSTRA POST
- DRAINAGE PIT
- STOP VALVE
- HYDRANT
- WATER METER
- SEWER MAN HOLE
- DENOTES SIGNAGE
- DENOTES TREE

NOTES:

1. FEATURES SHOWN TO SCALE ACCURACY.
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3. SERVICES LOCATED ONLY WHERE VISIBLE.
4. THE LOCATION OF ALL UNDERGROUND SERVICES WHETHER SHOWN ON THE PLAN OR NOT, SHOULD BE PRECISELY DETERMINED BEFORE ANY CONSTRUCTION WORK COMMENCES AND MEASURES TAKEN TO PROTECT THESE SERVICES FROM DAMAGE.
5. CONTOUR INTERVAL - 0.5m
6. THE BOUNDARIES SHOWN ARE APPROXIMATE ONLY. THE BOUNDARIES SHOWN HAVE BEEN COMPILED FROM THE RELEVANT DEPOSITED PLANS. FURTHER SURVEY WILL BE REQUIRED IF CONSTRUCTION IS TO TAKE PLACE ON OR ADJACENT TO THE BOUNDARIES.
7. LOT 105 D.P.614883 IS AFFECTED BY A RESTRICTION ON THE USE OF LAND CREATED BY D.P.250673.

REV.	DATE	AMENDMENT(S)	SUR	DFT	CHK
A	08.06.21	ORIGINAL ISSUE	SG	PV	PM
B	05.11.21	ADDITIONAL DETAIL	SG	JD	PM

DELFS LASCELLES
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delacs.com.au

CAD REF: 21493 - DET B

DETAIL SURVEY OF PART OF LOT 4 DP 823114 AND LOT 105 DP 614883

SITE ADDRESS:
4A VARDON ROAD
FERN BAY

CLIENT:
AVID PROJECT MANAGEMENT SERVICES

POSITION DATUM: PM 16471
ORIENTATION: MGA (GROUND)
EASTING: 387 054.954
NORTHING: 6 362 579.869
CLASS: B
HEIGHT DATUM: PM 16470
CLASS: LB
RL:1.974
ORDER: -
ORDER: -
DATE: 26.05.21

SURVEYED	DRAFTED	CHECKED
SG	JD	PM

SCALE	PAGE SIZE	DATE
1:400	A1	21.10.21

SHEET	REV.	PROJECT No.
2/4	B	21493

TREE TABLE

TREE NO.	SPREAD	TRUNK	HEIGHT
T279	6	0.3	7
T280	8	0.5	10
T281	4	0.2	7
T282	8	0.8	11
T287	9	0.8	13
T288	9	0.8	13
T289	10	1.5	13
T290	10	0.8	13
T291	5	0.4	7
T292	5	0.2	7
T293	5	0.2	7
T294	5	0.3	7
T295	4	0.2	6
T296	5	0.2	6
T297	5	0.2	6
T298	5	0.2	6
T299	4	0.1	6
T300	5	0.2	6
T301	5	0.4	7
T302	4	0.2	6
T303	4	0.1	6
T304	5	0.2	6
T305	6	0.5	8
T306	6	0.5	8
T307	6	0.5	8
T308	5	0.2	7
T309	5	0.2	7
T310	5	0.3	7
T311	5	0.2	7
T312	4	0.1	6
T313	6	0.3	7
T314	4	0.1	7
T315	7	0.3	8
T316	5	0.2	7
T317	5	0.2	7
T318	6	0.3	7
T319	5	0.2	7
T320	5	0.2	7
T321	5	0.2	7
T322	5	0.2	7
T323	5	0.2	7
T324	4	0.2	6
T325	5	0.2	7
T326	5	0.2	7
T327	5	0.2	7
T328	5	0.2	7
T329	4	0.1	5
T330	5	0.2	6
T331	5	0.2	4
T332	6	0.2	4
T333	12	1.1	12
T334	3	0.1	5
T335	4	0.3	7
T336	4	0.4	6
T337	3	0.1	4
T338	4	0.1	5
T339	4	0.2	5
T340	5	0.2	6
T341	6	0.4	7
T342	5	0.2	7
T343	5	0.2	7
T344	5	0.2	7
T345	7	0.2	7
T346	2	0.2	7
T347	6	0.4	7
T348	6	0.4	7
T349	6	0.4	7
T350	6	0.4	7
T351	6	0.4	7
T352	5	0.2	5
T353	4	0.2	4
T354	5	0.2	5
T355	5	0.2	4
T356	4	0.2	5
T357	5	0.3	8
T358	4	0.1	5
T359	4	0.2	5
T360	4	0.2	5
T361	4	0.2	6
T362	4	0.1	5
T363	4	0.2	4
T376	9	0.8	10

TREE NO.	SPREAD	TRUNK	HEIGHT
T377	7	0.4	9
T378	7	0.4	9
T379	4	0.1	5
T380	12	0.8	8
T381	8	0.4	8
T382	8	0.4	8
T383	8	0.6	8
T384	7	0.4	9
T385	6	0.4	8
T387	8	0.5	10
T388	8	0.4	10
T389	6	0.4	6
T390	6	0.3	9
T391	4	0.2	6
T392	7	0.7	10
T393	4	0.2	9
T394	7	0.4	9
T395	7	0.6	9
T396	5	0.2	8
T397	10	1.0	11
T398	4	0.3	11
T399	4	0.2	10
T400	4	0.2	10
T401	2	0.1	6

TREE NO.	SPREAD	TRUNK	HEIGHT
T402	2	0.1	6
T403	2	0.1	6
T404	2	0.1	6
T405	2	0.1	6
T406	2	0.1	6
T407	4	0.2	6
T408	4	0.2	6
T409	4	0.2	6
T410	4	0.2	6
T411	4	0.1	6
T412	4	0.1	6
T413	4	0.1	6
T414	3	0.2	5
T415	3	0.1	6
T416	5	0.2	10
T417	4	0.2	6
T418	2	0.2	6
T419	2	0.2	6
T420	5	0.2	10
T421	5	0.2	10
T422	5	0.2	10
T423	5	0.2	10
T424	5	0.2	10
T425	5	0.2	10

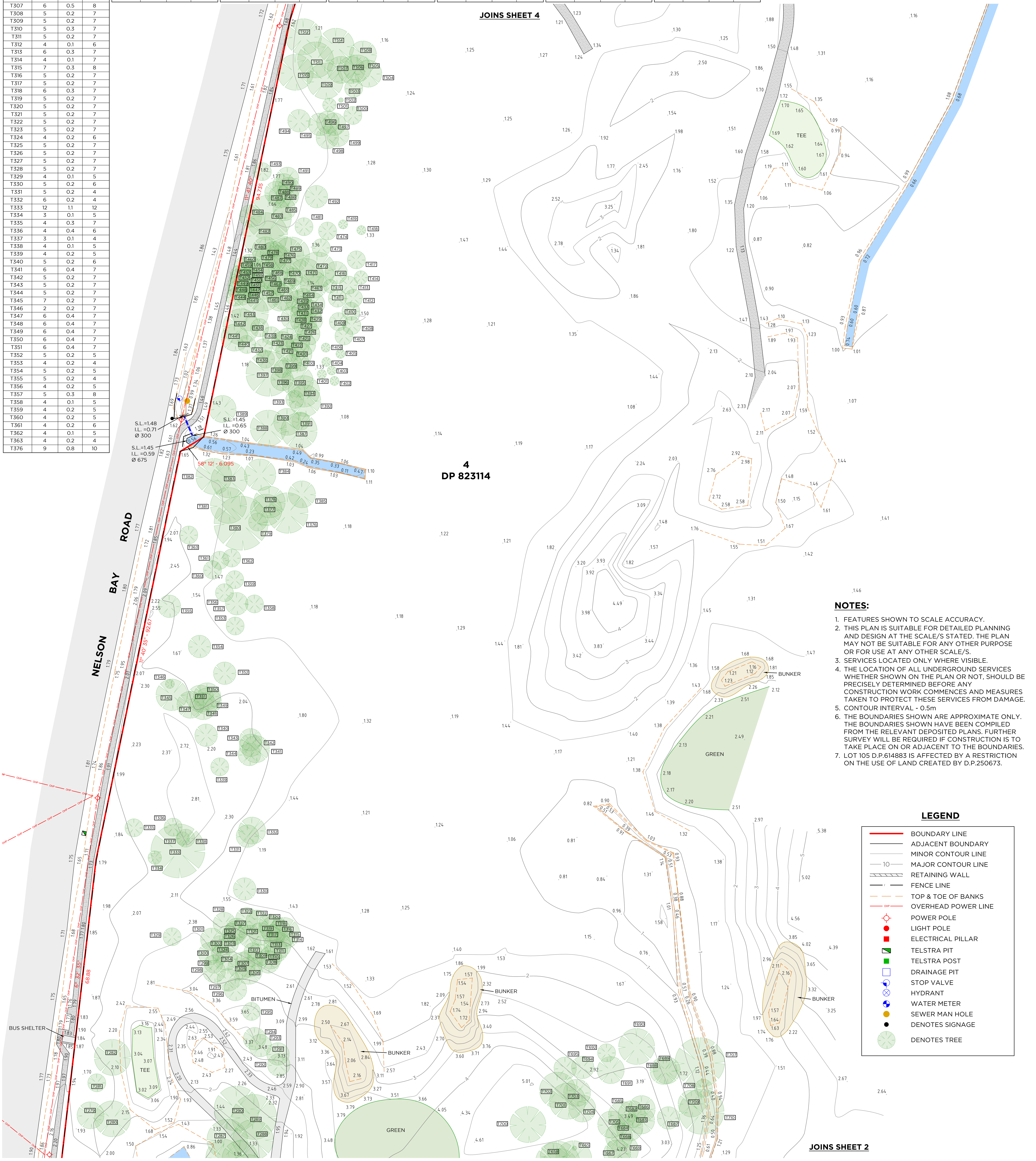
TREE NO.	SPREAD	TRUNK	HEIGHT
T426	5	0.2	10
T427	5	0.2	10
T428	5	0.2	10
T429	5	0.3	10
T430	5	0.3	10
T431	5	0.3	10
T432	5	0.3	10
T433	5	0.1	10
T434	5	0.3	10
T435	5	0.3	10
T436	4	0.2	10
T437	4	0.2	10
T438	4	0.2	10
T439	4	0.1	8
T440	4	0.2	10
T441	6	0.3	10
T442	8	0.8	10
T443	8	0.6	10
T444	4	0.2	5
T445	4	0.2	7
T446	4	0.3	10
T447	4	0.2	8
T448	4	0.2	5
T449	4	0.2	5

TREE NO.	SPREAD	TRUNK	HEIGHT
T450	6	0.3	10
T451	4	0.2	8
T452	4	0.2	5
T453	4	0.2	5
T454	4	0.2	8
T455	8	0.3	10
T456	8	0.3	10
T457	8	0.3	10
T458	4	0.1	8
T459	4	0.1	5
T460	4	0.1	5
T461	6	0.2	7
T462	6	0.2	7
T463	6	0.2	7
T464	4	0.1	6
T465	4	0.3	10
T467	4	0.3	10
T468	4	0.2	10
T469	4	0.1	10
T470	4	0.5	10
T471	8	0.5	10
T472	8	0.4	10
T473	4	0.2	7
T474	4	0.1	6

TREE NO.	SPREAD	TRUNK	HEIGHT
T475	8	0.7	10
T476	8	0.5	10
T477	6	0.2	10
T478	6	0.2	10
T479	7	0.4	10
T480	6	0.2	7
T481	4	0.2	10
T482	10	1.0	10
T483	10	0.1	10
T484	5	0.2	5
T485	5	0.2	8
T486	5	0.2	8
T487	5	0.2	8
T488	5	0.2	8
T489	5	0.2	8
T490	5	0.2	8
T491	5	0.2	8
T492	5	0.2	6
T493	4	0.2	5
T494	5	0.2	10
T495	5	0.2	10
T496	5	0.2	10
T497	5	0.2	10
T498	5	0.2	10

TREE NO.	SPREAD	TRUNK	HEIGHT
T499	5	0.2	10
T500	4	0.1	10
T501	3	0.1	8
T502	1	0.1	4
T503	4	0.2	10
T504	6	0.3	10
T505	6	0.3	10
T506	6	0.3	10
T507	6	0.3	10
T508	6	0.3	10
T509	8	1.0	10
T510	8	1.0	10
T511	8	0.8	10
T512	8	0.8	10
T514	8	0.8	10
T658	6	0.2	8
T665	1	0.3	6
T667	6	0.4	6
T668	6	0.4	6
T669	7	0.2	5
T682	6	0.2	6
T683	6	0.2	6
T684	6	0.2	6
T685	6	0.1	6

TREE NO.	SPREAD	TRUNK	HEIGHT
T686	6	0.1	6
T687	5	0.3	7
T688	6	0.4	6
T689	6	0.4	6
T690	8	0.4	10
T691	6	0.2	7
T692	4	0.1	5
T694	5	0.2	6
T695	5	0.2	6
T700	8	0.4	5
T701	8	0.5	10
T702	8	0.5	10
T703	6	0.3	6
T704	6	0.3	6
T705	6	0.1	6
T707	11	1.0	11
T708	4	0.2	8
T709	6	0.3	8
T710	10	0.8	10



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7. LOT 105 D.P.614883 IS AFFECTED BY A RESTRICTION ON THE USE OF LAND CREATED BY D.P.250673.

LEGEND

- BOUNDARY LINE
- ADJACENT BOUNDARY
- MINOR CONTOUR LINE
- MAJOR CONTOUR LINE
- RETAINING WALL
- FENCE LINE
- TOP & TOE OF BANKS
- OVERHEAD POWER LINE
- POWER POLE
- LIGHT POLE
- ELECTRICAL PILLAR
- TELSTRA PIT
- TELSTRA POST
- DRAINAGE PIT
- STOP VALVE
- HYDRANT
- WATER METER
- SEWER MAN HOLE
- DENOTES SIGNAGE
- DENOTES TREE

REV.	DATE	AMENDMENT(S)	SUR	DFT	CHK
A	08.06.21	ORIGINAL ISSUE	SG	PV	PM
B	05.11.21	ADDITIONAL DETAIL	SG	JD	PM

DELFS LASCELLES
CONSULTING SURVEYORS

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MAYFIELD NSW 2304
ABN: 28 164 260 100

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E: admin@delacs.com.au
delacs.com.au

CAD REF: 21493 - DET B

DETAIL SURVEY OF PART OF LOT 4 DP 823114 AND LOT 105 DP 614883

SITE ADDRESS:
4A VARDON ROAD
FERN BAY

CLIENT:
AVID PROJECT MANAGEMENT SERVICES

POSITION DATUM:	PM 16471
ORIENTATION:	MGA (GROUND)
EASTING:	387 054.954
NORTHING:	6 362 579.869
CLASS: B	ORDER: -
HEIGHT DATUM:	PM 16470
CLASS: LB	ORDER: -
RL:1.974	DATE: 26.05.21

SURVEYED	DRAFTED	CHECKED
SG	JD	PM
SCALE	**PAGE SIZE**	**DATE**
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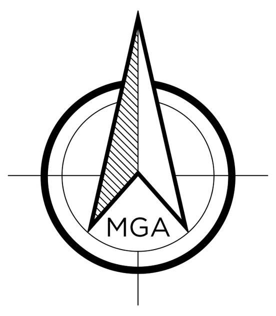
SHEET	**REV.**	**PROJECT No.**
3/4	B	21493



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5. CONTOUR INTERVAL - 0.5m
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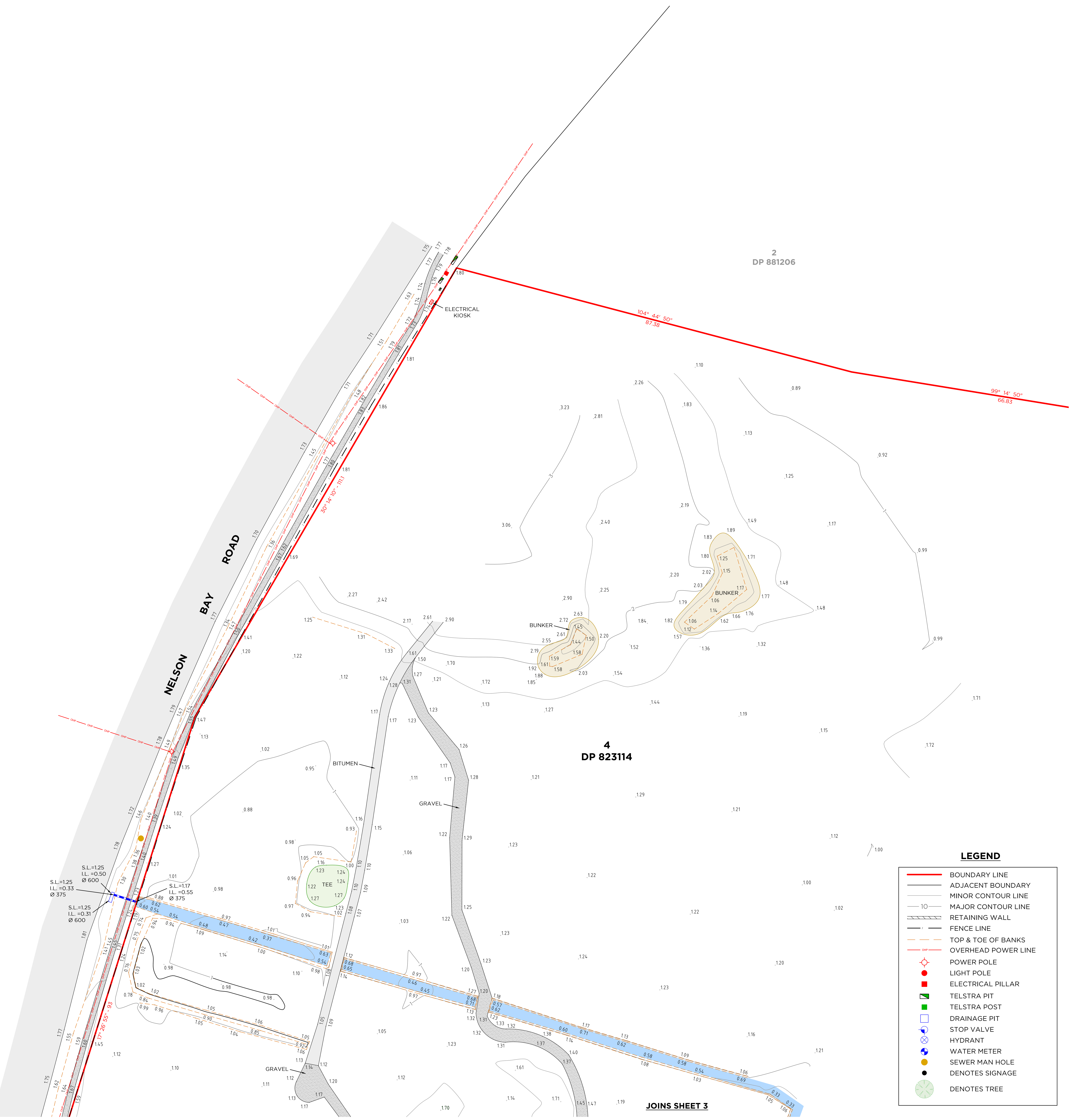
7. LOT 105 D.P.614883 IS AFFECTED BY A RESTRICTION ON THE USE OF LAND CREATED BY D.P.250673.



LEGEND

- BOUNDARY LINE
- ADJACENT BOUNDARY
- MINOR CONTOUR LINE
- 10 MAJOR CONTOUR LINE
- RETAINING WALL
- FENCE LINE
- TOP & TOE OF BANKS
- OHP OVERHEAD POWER LINE
- POWER POLE
- LIGHT POLE
- ELECTRICAL PILLAR
- TELSTRA PIT
- TELSTRA POST
- DRAINAGE PIT
- STOP VALVE
- HYDRANT
- WATER METER
- SEWER MAN HOLE
- DENOTES SIGNAGE
- DENOTES TREE

REV.	DATE	AMENDMENT(S)	SUR	DFT	CHK
A	08.06.21	ORIGINAL ISSUE	SG	PV	PM
B	05.11.21	ADDITIONAL DETAIL	SG	JD	PM

DELFS LASCELLES
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MAYFIELD NSW 2304
ABN: 28 164 260 100

T: (02) 4964 4886
E: admin@delacs.com.au
delacs.com.au

CAD REF: 21493 - DET B

DETAIL SURVEY OF PART OF LOT 4 DP 823114 AND LOT 105 DP 614883

SITE ADDRESS:
4A VARDON ROAD
FERN BAY

CLIENT:
AVID PROJECT MANAGEMENT SERVICES

POSITION DATUM:	PM 16471
ORIENTATION:	MGA (GROUND)
EASTING:	387 054.954
NORTHING:	6 362 579.869
CLASS: B	ORDER: -
HEIGHT DATUM:	PM 16470
CLASS: LB	ORDER: -
RL:1.974	DATE: 26.05.21

SURVEYED	DRAFTED	CHECKED
SG	JD	PM
SCALE	PAGE SIZE	DATE
1:400	A1	21.10.21
SHEET	REV.	PROJECT No.
4/4	B	21493

APPENDIX C: PREVIOUS INVESTIGATIONS

GEOTECHNICAL INVESTIGATION

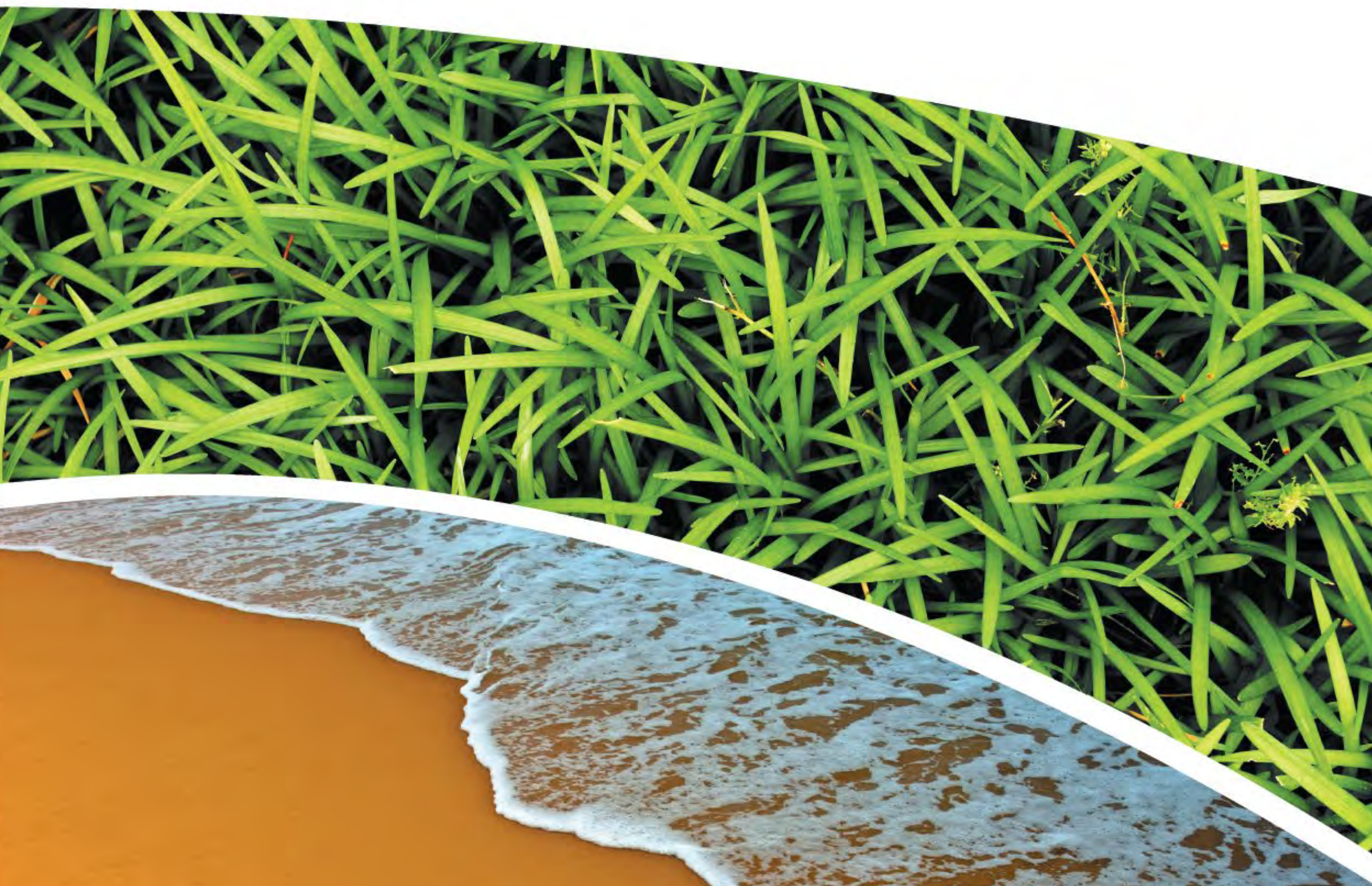
NEWCASTLE GOLF CLUB

Prepared for PRINCIPLE LIVING PTY LTD

Prepared by RCA Australia

RCA ref 15442-402/3

NOVEMBER 2023



RCA Australia
ABN 53 063 515 711
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				Name	**Signature**	**Date**
/0	Draft	R Cater	M Allman	M Allman		24.06.21
/1	Final	R Cater	M Allman	M Allman		3.12.21
/2	Revised Final	R Cater	M Allman	M Allman		19.10.23
/3	Revised for adjusted wording regarding development	R Cater	M Allman	M Allman		30.11.23

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/0	1	Electronic (email)	Avid Project Management Pty Ltd – Chris Old – chris.old@avidpm.com.au	24.06.21
/0	1	Electronic report	RCA – job archive	24.06.21
/1	1	Electronic (email)	Avid Project Management Pty Ltd – Chris Old – chris.old@avidpm.com.au	3.12.21
/1	1	Electronic report	RCA – job archive	3.12.21
/2	1	Electronic (email)	Principle Living Pty Ltd – Chris Old – chris.old@principleliving.com.au	19.10.23
/2	1	Electronic report	RCA – job archive	19.10.23
/3	1	Electronic (email)	Principle Living Pty Ltd – Chris Old – chris.old@principleliving.com.au	30.11.23
/3	1	Electronic report	RCA – job archive	30.11.23

ISO 9001 Quality Management Systems CERTIFIED
ISO 14001 Environmental Management CERTIFIED
ISO 45001 Occupational Health and Safety Management CERTIFIED

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RCA ref 15442-402/3

GEOTECHNICAL ENGINEERING • ENVIRONMENTAL

Geotechnical Engineering
Engineering Geology
Environmental Engineering
Hydrogeology
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Environmental Monitoring
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Occupational Hygiene

30 November 2023

Principle Living Pty Ltd
34 Western Road
Medowie NSW 2318

Attention: Mr Chris Old

GEOTECHNICAL INVESTIGATION
NEWCASTLE GOLF CLUB

1 INTRODUCTION

This report presents the findings of a geotechnical investigation undertaken at Newcastle Gold Club, 4 and 4A Vardon Road, Fern Bay. The investigation was undertaken at the request of David Rosewarne of Avid Project Management Pty Ltd in 2021.

It is understood that Newcastle Golf Club and Principle Living have entered into an agreement to develop a portion of the site for seniors living and undertake upgrades in other areas of the existing golf course.

The following are the development's key components:

- Site preparation & establishment activities – clearing existing vegetation, demolition of existing golf course via earthworks, bulk earthworks.
- Establishment of vehicular access from Nelson Bay Road.
- Construction and occupancy of a seniors living development comprising:
    - Three (3) apartment buildings containing 125 serviced self-care dwellings.
    - Forty seven (47) single storey (villas) serviced self-care dwellings.

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- Carparking – 295 spaces across the site with each villa being provided with a double garage (94 spaces) and 201 basement carparking spaces within the three (3) apartment buildings.

- Provision of pedestrian and vehicular access to and from the site.

- Establishment of a Community centre & administration building.

- Pickle ball courts, lawn bowls facility, open space, landscaping, picnic shelter, public art, open lawn area for passive recreational activities and formal striking planting.

- Civil works including internal access roads, pedestrian linkages to Nelson Bay Road and the golf club.

- Connection to Country 'Keeping Place'.

- Extension and enhancement of physical infrastructure utilities as needed.

It is understood that this investigation is required to provide geotechnical data for design of the structures forming the development.

The layout of the proposed development has been modified since the 2021 RCA geotechnical investigation studies and this revised report provides updated references, discussion and comments based on the current development layout. It is noted that further field geotechnical investigations will be required based on the new development layout.

This report provides the following:

- Site description.

- Details of fieldwork and laboratory testing undertaken for the investigation.

- Description of the subsurface conditions encountered at the site.

- Site classification in accordance with AS2870 - Residential Slabs and Footings.

- Discussion on earthworks including site preparation, excavation conditions and the suitability of the site soils for use as fill and fill construction/compaction procedures.

- Alternative footing types and founding levels, including recommendations as to allowable bearing pressure and probable settlements.

- Pavement design with comments on construction methods, material specification and compaction requirements, drainage and subgrade preparation.

- Information on soil and groundwater aggressivity to buried structural elements.

- A summary of further geotechnical investigation expected to be required.

2 SITE DESCRIPTION

The site comprises Newcastle Gold Club, 4 and 4A Vardon Road, Fern Bay, NSW, Lot 105 DP 614883 and Lot 4 DP 823114. The site locality and an aerial image of the site are shown on **Drawing 1** in **Appendix A**, together with the geotechnical investigation test locations. The supplied Newcastle Golf Club Masterplan Drawing prepared by EJE Architecture is shown on **Drawing 2** in **Appendix A** and the supplied cut fill plan is shown on **Drawing 3** in **Appendix A.**

The proposed seniors living development is located over an area of about 6.5ha along the western side of the golf course fronting Nelson Bay Road. The golf course area comprises maintained lawns over gently undulating terrain with some areas of trees and bushland.

The proposed seniors living development covers an area of higher elevation at the southern end and lower elevation at the northern end. Reference to elevation data published by the NSW Government Spatial Services indicates the elevation of the site ranges from about 1m AHD in low lying areas at the southern end to around 11m AHD at the northern end of the site. The elevation data obtained from NSW Government Spatial Services is shown on **Drawing 1** in **Appendix A.**

A significant proportion of drainage at the site is expected to occur through infiltration through the sandy soils at the site. An existing drain/watercourse is located at the northern end of the proposed seniors living development area.

Site conditions at the time of the fieldwork are shown in **Photograph 1** to **Photograph 16**. Approximate photograph locations and directions are shown on **Drawing 1** in **Appendix A**.

Photograph 1 *Looking south east at the southern end of the site*

Photograph 2 *Looking east towards club house building from TP1*

Photograph 3 *Looking north east from near TP1*

Photograph 4 *Looking south west at CPT2 location*

Photograph 5 *Looking north from between CPT2 and TP2*

Photograph 6 *Looking north west towards Nelson Bay Road from near TP2*

Photograph 7 *Looking north from between TP4 and TP5, long grass present on the eastern side of the site and fill overlying peat soils encountered in TP4 to the east of the photograph location*

Photograph 8 *Looking north east from TP5*

Photograph 9 *Looking east from near TP6*

Photograph 10 *Looking south from TP9*

Photograph 11 *Looking north from near CPT4*

Photograph 12 *Looking south west at TP10, drain/watercourse and Nelson Bay Road*

Photograph 13 *Looking south from TP11*

Photograph 14 *Looking west from TP11*

Photograph 15 *Looking north west from TP11*

Photograph 16 *Looking south from the northern end of the site, the golf cart acess pathway crosses a watercourse and ponding water is present in low lying area near the watercourse*

3 FIELD AND LABORATORY INVESTIGATIONS

Fieldwork was conducted on the 17th of May 2021 and included the following:

- A visual appraisal and mapping of site features.
- Cone penetrometer tests (CPTs) at four locations to depths of 18.5m (CPT1), 16.6m (CPT2), 10.3m (CPT3) and 11.6m (CPT4) to provide a continuous profile of soil type and strength with depth. CPTs were undertaken with a four tonne self-anchoring track mounted Geoprobe 6625 CPT rig. Tip resistance, sleeve friction, pore pressure and inclination readings were electronically recorded at 20mm depth intervals.
- Excavation and logging of eleven test pits (TP1 to TP11) using a 3.5 tonne tracked excavator with a 450mm wide bucket to depths ranging from 1.1m to 2.3m.
- Obtaining disturbed samples and bulk disturbed samples from the test pits for laboratory testing.
- Perth sand penetrometer testing at all test pit locations to depths of 2-3m.

All fieldwork was carried out by and in the presence of RCA Australia (RCA) personnel. Approximate test locations are shown on **Drawings 1 to 3** in **Appendix A**.

The test locations were set out by hand held GPS. The CPT and test pit depths were recorded relative to the existing ground surface at the time of investigation.

All test pits were backfilled on completion.

The CPT results sheets, engineering logs of the test pits and test pit photographs are presented in **Appendix B** together with explanatory notes.

Groundwater conditions/levels at the time of fieldwork are noted on the test pit logs and the CPT pore pressures are shown on the CPT results sheets. Fluctuations in groundwater conditions may be expected due to variations in rainfall and site conditions.

Laboratory testing of samples recovered during fieldwork consisted of:

- Four particle size distribution tests for soil classification.
- Two Atterberg limits tests for soil classification.
- Six California bearing ratio (CBR) tests to assess subgrade strength.
- Five soil and three groundwater pH, electrical conductivity, sulfate and chloride tests to assess aggressivity to buried structural elements.
- Twenty acid sulfate screening tests to assess the presence of acid sulfate soils.
- Four acid sulfate soil analysis tests by the Suspension Peroxide Oxidation Combined Acidity and Sulfate (SPOCAS) and chromium reducible sulfur (CRS) methods.

The laboratory test results are summarised in Section 4.4 and the laboratory test report sheets are attached in **Appendix C**.

4 INVESTIGATION FINDINGS

4.1 PUBLISHED GEOLOGY MAPS

The NSW Seamless Geology Map published by the Department of Regional NSW indicates that the site overlies the following geological units:

- QH_bd - Marine deposited and aeolian reworked coastal sand dunes over most of the golf course area and the southern end of the proposed seniors living development area.
- QH_er - Fine to medium grained lithic quartz sand (fluvially deposited), very fine to fine grained lithic carbonate quartz sand (marine deposited), polymictic gravel, silt, clay, shell material over the northern end of the proposed seniors living development area.

An aerial image of the site together with an overlay of the NSW Seamless Geology Map units is shown in **Figure 1**.

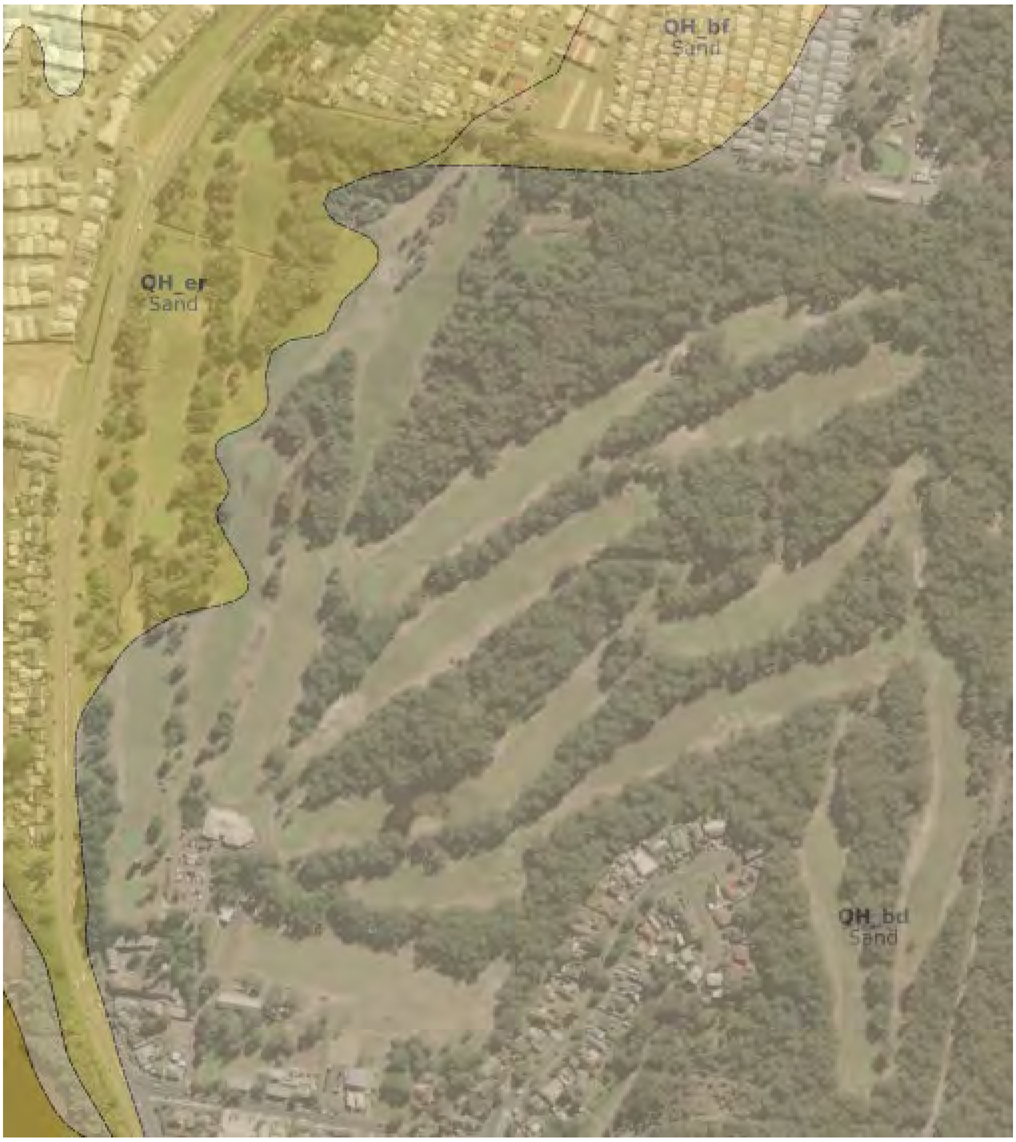


Figure 1 *Aerial image of the site with the NSW Seamless Geology Map units*

4.2 ACID SULFATE SOIL RISK MAP

The Williamtown Acid Sulfate Soil Risk Map - Edition Two published by the Department of Land and Water Conservation indicates the proposed seniors living development area overlies the following mapped regions:

- An aeolian dune landform area at the southern end with a low probability of acid sulfate soils at greater than 3m below the ground surface over most of the golf course area and the southern end of the proposed seniors living development area.

- Disturbed terrain (e.g., reclaimed low lying swamps or areas which have undergone heavy ground disturbance through general urban development) requiring soil investigation to assess the presence of acid sulfate soils over the northern end of the proposed seniors living development area.

An aerial image of the site overlain with the Williamtown Acid Sulfate Soil Risk Map - Edition 2 boundaries and region labels is shown in **Figure 2**.

Figure 2 *Aerial image of the site with Williamtown Acid Sulfate Soil Risk Map - Edition 2 boundaries (green lines) and region labels (white text)*

4.3 SUBSURFACE CONDITIONS

The subsurface conditions encountered at the site are detailed on the CPT results sheets and test pit logs which are attached in **Appendix B** together with test pit photographs and explanatory notes.

Plots of tip resistance, sleeve friction and pore pressure from the CPTs are shown on the CPT results sheets attached in **Appendix B**. The CPT cone resistance profiles are plotted in **Figure 3**.

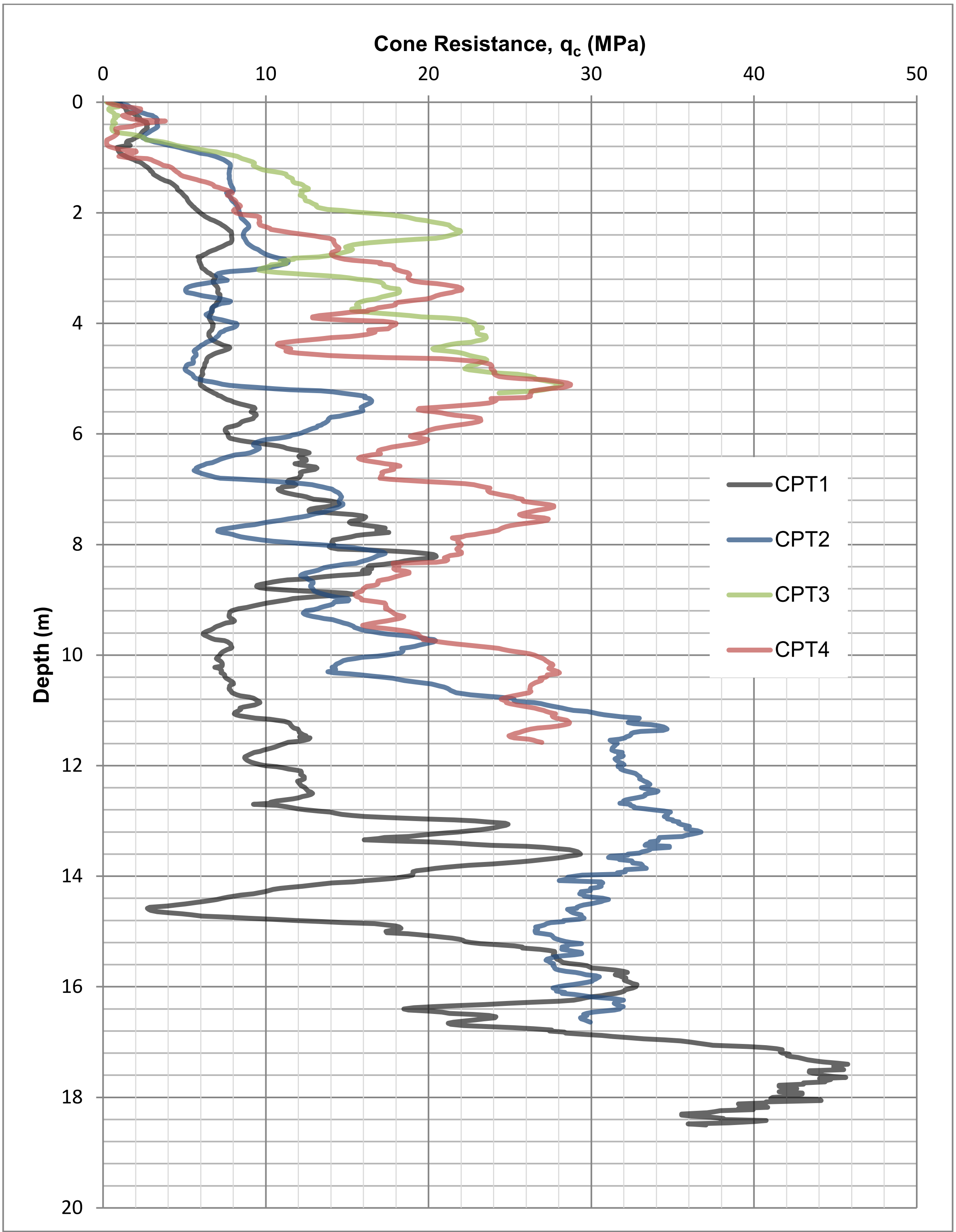


Figure 3 *CPT plots of cone resistance (q_c)*

The results of the dynamic penetrometer testing undertaken at the test pit locations are shown on the test pit logs in **Appendix B** and graphically in **Figure 4**. A large scatter can be observed in the results shown in **Figure 4**, particularly between 1m and 2m depth where blow counts ranged from 1-15 per 100mm penetration.

The relative density curves and the densities shown in **Figure 4** are based on a published calibration chart for the blunt-tipped, Perth sand penetrometer, in silica sands (Ref [1]). Based on previous experience in similar geological environments (and on the results of this geotechnical investigation) the relative densities based on Ref [1] (indicated in **Figure 4**) are often higher than those assessed based on results of other test methods such as CPTs or standard penetration tests (SPTs).

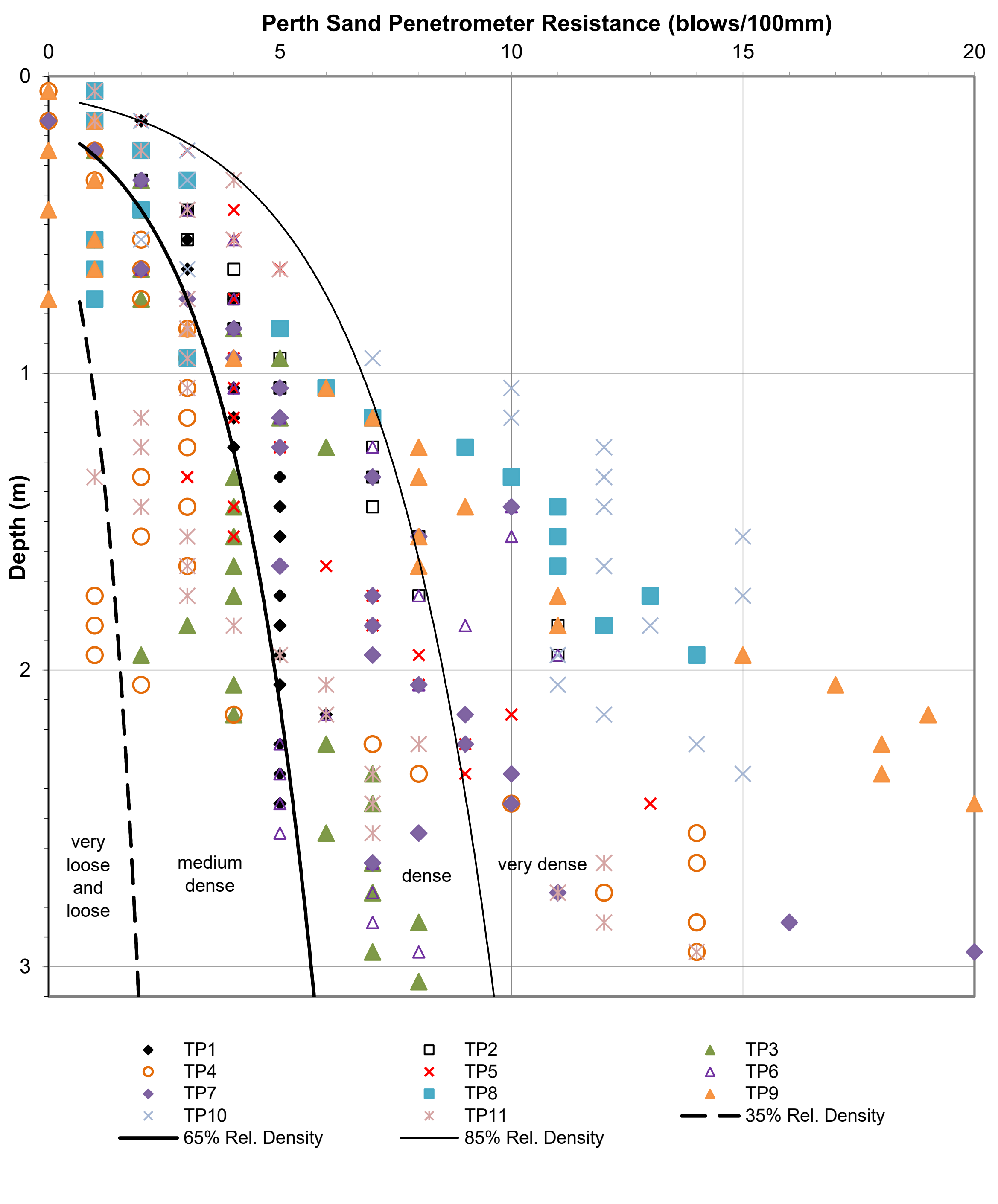


Figure 4 *Dynamic penetrometer test results plotted with calibration chart for the blunt-tipped, Perth sand penetrometer, in silica sands (Ref [1])*

The subsurface conditions at the test locations can generally be summarised as follows:

- Topsoil comprising sand or silt materials with roots/rootlets was encountered to depths of between 0.1 and 0.3m in the test pits.

- Fill comprising silty sand with inclusions of shells, organics and glass was encountered in TP4 to 0.9m depth.

- Natural sands were encountered below topsoil/fill in all test pits. Very loose or loose relative density sands were inferred from penetrometer test results at all test locations except TP1, TP2, TP5 and TP6 (as noted in **Table 1**). The very loose or loose relative density sands (where encountered) extended to depths of between 0.5m and 2.0m and were generally underlain by medium dense or better sands. A deeper layer of loose sand was inferred between 14.4-14.7m depth in CPT1.

- Peat layers were encountered between 1.7-2.0m in TP4 and 0.5-0.8m in TP9. Shallow clay layers generally of soft to firm consistency were inferred in CPT3 and CPT4 extending to 0.5m in CPT3 and 1.0m in CPT4.

- Groundwater/seepage was recorded in CPT3, CPT4, TP4, TP5 and TP7-TP11 at depths of between 0.6-1.6m. Groundwater was recorded at deeper depths of 4.4m in CPT2 and 10.3m in CPT1.

It is noted that groundwater levels are likely to fluctuate with variations in climatic and site conditions and the possibility of the groundwater level rising above that noted at the time of the field investigation should be allowed for in the design. It is understood form golf course staff that the lower lying northern areas of the development site are prone to becoming waterlogged during wetter periods.

A general summary of the subsurface conditions encountered at the test locations is shown in **Table 1**.

Table 1 *Summary of subsurface conditions encountered in the test pits and inferred subsurface conditions from penetrometer test results.*

Layer/Feature	Depth encountered (m)														
	CPT1	**CPT2**	**CPT3**	**CPT4**	**TP1**	**TP2**	**TP3**	**TP4**	**TP5**	**TP6**	**TP7**	**TP8**	**TP9**	**TP10**	**TP11**
Topsoil/Fill	-	-	-	0 - 0.4	0 - 0.15	0 - 0.2	0 - 0.1	0 - 0.9	0 - 0.3	0 - 0.15	0 - 0.1	0 - 0.15	0 - 0.2	0 - 0.25	0 - 0.15
Peat	-	-	-	-	-	-	-	2.0 - 2.3	-	-	-	-	0.5 - 0.8	-	-
Clay (generally soft to firm consistency)	-	-	0 - 0.5	0.4 - 1.0											
Very Loose or Loose Sand	0 - 2.0	0 - 1.0	0.5 - 1.0	1.0 - 1.4	-	-	0.1 - 0.6	0.9 - 2.0	-	-	0.1 - 0.6	0.15 - 0.8	0.2 - 0.5	-	0.15 - 1.5
Medium Dense or medium dense to dense sand	2.0 - 15.2*	1.0 - 11.0	1.0 - 7.8	1.4 - 9.5	0.15 - 2.5	0.2 - 2.0	0.6 - 3.1*	2.3 - 2.8	0.3 - 2.5	0.15 - 3.0	0.6 - 2.5	0.8 - 2.0	0.8 - 2.5	0.25 - 2.5	1.5 - 3.0
Dense or Very Dense Sand	15.2 - 18.5	11.0 - 16.6	7.8 - 10.3	9.5 - 11.6	-	-	-	-	-	-	-	-	-	-	-
Groundwater/ seepage	10.3	4.4	1.0	1.1	-	-	-	1.4	1.0	-	1.6	0.8	0.7	0.6	0.9

* loose sand layers from 14.4-14.7m in CPT1 and 1.8-2.0m in TP3

4.4 LABORATORY TEST RESULTS

The laboratory test results are provided on the laboratory report sheets attached in **Appendix C**. The laboratory test results are summarised in **Table 2** to **Table 7**.

Table 2 *Summary of Particle Size Distribution Test Results*

Test Pit	Depth (m)	Soil Type	% Fines	% Fine to Medium Grained SAND	% Coarse Grained SAND
TP1	0.6 - 1.0	SAND	3	96	1
TP3	0.3 - 0.5	SAND	3	91	6
TP5	0.3 - 0.5	SAND with silt	8	92	0
TP10	0.3 - 0.4	SAND with silt	8	89	7

Table 3 *Summary of Atterberg Limits Test Results*

Test Pit	Depth (m)	Soil Type	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
TP4	2.2 - 2.3	PEAT	Not Obtainable	Not Obtainable	Non Plastic
TP9	0.5 - 0.7	PEAT	Not Obtainable	Not Obtainable	Non Plastic

Table 4 *Summary of Compaction and CBR Test Results*

Test Pit	Depth (m)	Soil Type	FMC (%)	MDD (t/m^3)	SOMC (%)	CBR (%)
TP1	0.6 - 1.0	SAND	2.9	1.608	10.9	20
TP4	0.3 - 0.5	FILL, Silty SAND	14.4	1.554	23.1	5
TP5	0.3 - 0.5	SAND with silt	12.8	1.631	18.1	9
TP6	0.2 - 0.5	Silty SAND	3.3	1.672	11.1	14
TP9	0.5 - 0.7	PEAT	101.0	1.059	62.5	2
TP11	0.3 - 0.5	SAND	6.8	1.632	9.6	16

NOTES: FMC – field moisture content
MDD – maximum dry density (Standard compaction)
SOMC – Standard optimum moisture content
CBR – California bearing ratio, penetration 2.5 / 5.0mm

Table 5 *Summary of Soil and Groundwater Chemistry Test Results*

Test Pit	Depth/ Description	pH	Chloride (ppm)	Sulphate (ppm)	Conductivity (µS/cm)	AS2159-2009 Exposure Classification	
						Concrete	Steel
TP3	1.3-1.5/SAND	9.1	<10	<10	42	Non-aggressive	Non-aggressive
TP4	2.2-2.3/PEAT	6.6	170	530	133	Mild	Non-aggressive
TP6	0.7-0.8/SAND	7.6	<10	<10	22	Non-aggressive	Non-aggressive
TP8	Groundwater	6.53	220	<1	988	Mild	Moderate
TP9	0.2-0.4/SAND	6.5	160	10	96	Non-aggressive	Non-aggressive
TP9	Groundwater	6.20	210	<1	790	Mild	Moderate
TP10	Groundwater	5.64	404	<1	1330	Mild	Severe
TP11	1.0-1.1/SAND	6.6	<10	<10	9	Mild	Non-aggressive

Table 6 *Acid Sulfate Screening Test Results*

Test Pit	Depth (m)	Soil Type	pH_F	pH_{FOX}	$pH_F - pH_{FOX}$	Reaction Rate
TP1	0.8-1.0	SAND	5.99	4.45	1.54	Slight
TP2	0.8-1.0	SAND	5.98	5.09	0.89	Slight
TP3	1.3-1.5	SAND	6.11	5.11	1.00	Slight
TP3	2.0-2.2	SAND	6.75	5.79	0.96	Slight
TP4	1.2-1.3	SAND	6.77	6.13	0.64	Slight
TP5	0.3-0.5	SAND with silt	5.54	3.49	2.05	Slight
TP5	1.3-1.5	SAND, trace silt	5.74	1.45	4.29	Moderate
TP6	0.3-0.5	Silty SAND	6.05	5.18	0.87	Slight
TP6	1.9-2.0	SAND	6.25	5.50	0.75	Slight
TP7	1.4-1.5	SAND	6.18	4.55	1.63	Slight
TP7	1.8-2.0	SAND	6.48	1.95	4.53	Very Vigorous
TP8	0.4-0.5	Silty SAND	7.65	5.63	2.02	Moderate
TP8	1.3-1.5	SAND	6.99	1.76	5.23	Very Vigorous
TP9	0.5-0.7	PEAT	6.26	4.22	2.04	High
TP9	1.2-1.4	SAND	6.34	1.66	4.68	Very Vigorous
TP10	0.1-0.2	Silty SAND	4.04	2.60	1.44	Moderate
TP10	0.8-1.0	SAND	5.61	1.56	4.05	Very Vigorous
TP10	1.1-1.3	SAND	5.09	1.58	3.51	Very Vigorous
TP11	0.3-0.4	SAND	6.22	5.42	0.80	Slight
TP11	1.0-1.2	SAND	6.42	5.64	0.78	Slight

Note: Results in shaded cells may indicate potential acid sulfate soils as discussed in ASSMAC (Ref [4]).

Table 7 *Summary of SPOCAS and CRS Test Results*

Location and Depth (m)	Soil Type	Texture	pH_{KCl}	Acid Trail (mole H+/t)			S_{CRS}	S_{POS}	S_{Net}	Liming Rate
				TAA	TPA	TSA	(%S)	(%S)	(%S)	(kg CaCO3/t)
TP5 1.3-1.5	SAND, trace silt	Coarse	6.0	3	60	58	0.088	0.127	0.13	6
TP7 1.8-2.0	SAND	Coarse	6.3	<2	24	24	0.046	0.061	0.06	3
TP8 1.3-1.5	SAND	Coarse	5.3	5	93	88	0.249	0.333	0.34	16
TP10 0.8-1.0	SAND	Coarse	5.0	10	80	70	0.155	0.264	0.28	13

Note: Results in shaded cells exceed ASSMAC (Ref [4]) action criteria and trigger requirement for acid sulfate soil management plan.

5 DISCUSSION AND COMMENTS

5.1 KEY GEOTECHNICAL ISSUES

The geotechnical investigation findings summarised in Section 4 of this report indicate the following key geotechnical issues will need to be addressed:

- Laboratory testing has shown that potential acid sulfate soils are present at the site and site works will need to be carried out in accordance with an acid sulfate soils management plan. Further testing will be required within the compensatory cut area to the north of the proposed seniors living development area to investigate acid sulfate soils within this area.

- Unsuitable compressible soils including soft to firm clay and peat soils were inferred/encountered in CPT3, CPT4, TP4 and TP9 and these soils will need to be removed prior to filling lower elevation areas of the site.

- Very loose/loose relative density sands were inferred/encountered in the upper soil profile in ten of the fourteen geotechnical investigation test locations. Works to increase the density and reduce the variation in density of the upper soil profile would be required to reduce the risk of settlement (both total and differential) of the very loose/loose relative density sands under footing loads or dynamic loads (e.g. vibrations from earthquakes).

- Groundwater was encountered at shallow depths of less than 1m in lower elevation areas of the site. The proposed basement level for Apartment Building 1 is expected to be below groundwater level. Further groundwater level monitoring will need to be carried out to confirm groundwater levels for input into design of the three proposed apartment building basements. It is expected that basement excavations below, or in the vicinity of, the groundwater level will need to be designed as fully tanked.

- The proposed multi storey concrete framed buildings will require further investigation expected to comprise additional cone penetration testing for foundation design.
- Filling is expected to be required close to and below the groundwater level in some areas (e.g., low lying areas or where excavation to remove unsuitable compressible soils has been undertaken).

These issues are discussed further in the following sections of this report. A summary of further geotechnical input expected to be required is provided in Section 5.6.

5.2 SITE EARTHWORKS

5.2.1 EXCAVATIONS

Excavations should be carried out in accordance with an acid sulfate soil management plan as discussed in Section 5.5.

Excavations are expected to be required for regrading works, installation of underground services/drainage, boxing out for pavement construction, construction of footings etc.

It is generally expected that the soil profiles encountered at the site could be excavated by conventional earthmoving equipment such as bobcats, backhoes and excavators.

It is unlikely that the sand soils above groundwater level would be stable at angles greater than about 30° (about 1.75H:1V) and allowance should be made for either battering back or support of the sides of excavations above groundwater level during construction.

Unsupported excavations could be expected to undergo slumping into the excavation where seepage or groundwater is encountered (groundwater was encountered on the site at between 0.6m and 10.3m below ground level at the time of the field investigation work). Where excavations are proposed below the groundwater level, the sand strata encountered at the site will not be stable and excavations below the groundwater level are expected to require support/shoring together with groundwater control/dewatering.

All long-term excavations should either be supported by properly designed and constructed retaining walls or alternatively battered at 2H:1V or flatter. Any basement, excavations, if/where proposed, are expected to require support by properly designed and constructed retaining walls. It is expected that basement excavations below, or in the vicinity of, the groundwater level will need to be designed as fully tanked.

The soils encountered at the site are judged to be susceptible to erosion and should be protected by vegetation or similar, together with adequate drainage where exposed.

5.2.2 FILLING

It is noted that where subgrade formation level is below or in the vicinity of the groundwater level, adequate compaction of the subgrade and subsequent fill and pavement layers is expected to be difficult. Owing to the presence of high groundwater levels across the low-lying areas of the site (refer to Section 5.5) excavation and removal of unsuitable materials is likely to result in a subgrade formation level below or in the vicinity of the groundwater level which may result in deformation of the subgrade during compaction. Options to address this issue include the following:

- Provision of a bridging layer (e.g. rock fill covered with geotextile) in the vicinity of the water table to provide a stable base for compaction of overlying fill. This is a common method to facilitate construction of pavements where subgrade formation level is in vicinity of the water table. It is noted that provision of rockfill below areas of proposed residential lots may not be acceptable to the client and would present a constraint to future construction of deep foundations (e.g. screw piles).

  The required thickness of the bridging layer is unknown, however is likely to be in the range of 0.5m to 1m, and could be expected to be dependent on factors such as subgrade strength and climatic conditions at the time of construction. The actual thickness of the bridging layer required should be based on the achievement of a stable working platform for the support of construction equipment and compaction of the subsequent fill and pavement layers. Depending on the grading of the bridging layer material, a geotextile may be required to separate the bridging layer and overlying fill materials to prevent migration of these materials into the bridging layer.

- Place and compact sand fill in the vicinity of the water table. This option is expected to be more difficult from a constructability perspective than the option of a bridging layer. Discussions should be held with contractors regarding the methods and previous experience/verification in compacting/densifying sand fill below and in the vicinity of the water table where this option is adopted.

- Deep ground improvement options such as vibro compaction to densify the sands below and in the vicinity of the water table. This is expected to be the least economical option and would not be required where a bridging layer or sand fill are able to be properly constructed and verified.

The above options would require an appropriate methodology for verification of works to confirm that very loose and loose sands have been densified to reduce the risk of future settlements.

The soft to firm clay and peat soils inferred/encountered in in CPT3, CPT4, TP4 and TP9 are considered to be unsuitable and should not be left in situ below any engineered fill to support structures, pavements or movement sensitive elements and these soils must be removed prior to filling. Refer to **Table 1** or the logs for the depths these materials were encountered. Soft to firm clay and peat soils were inferred/encountered in lower lying areas in CPT3, CPT4, TP4 and TP9 and as such, do not appear laterally continuous across the entire development footprint. Accurate estimation of the likely volume of these unsuitable soils requiring removal is expected to require further investigation at an appropriate stage of the development. Removal of the soft to firm clay and peat soils in low lying area of the site is likely to disturb potential acid sulfate soils and should be carried out in accordance with an acid sulfate soil management plan.

Any proposed filling on the site should be placed and compacted in accordance with *AS 3798-2007, Guidelines on Earthworks for Commercial and Residential Developments* (Ref [2]).

Site preparation for the placement of fill should include the following:

- Removal of any existing fill such as silty sand with inclusions of shells, organics and glass was encountered in TP4 to 0.9m depth, topsoil and deleterious soils together with any surface vegetation, e.g., grass/weeds, and heavily root affected soils, to expose a clean sand subgrade.

- Proof rolling of the exposed sand subgrade.

Where site levels need to be raised, clean sand fill should be placed in layers and compacted to achieve the following minimum density index (*AS 1289.5.6.1*):

- 70% as general site fill.

- 75% beneath structures and in areas of proposed pavements

It is recommended that fill material comprising sand with little or no fines be used for any proposed filling at the site. The use of sand with little or no fines as fill will promote infiltration drainage to the underlying sand subgrade and prevent the fill from holding water.

The sand soils at the site (excluding topsoil, near surface heavily root affected or organic soils and sandy fill with a high silt content/organic content such as encountered in TP4 between 0-0.9m depth), are generally expected to be suitable for re-use as fill provided that any deleterious material is removed prior to incorporation of the material into fill earthworks. The particle size distribution test results summarised in **Table 2** indicate that the soils on which particle size distribution testing was carried out are suitable for re use as fill material at the site.

Owing to the presence of the sands (including loose sands) together with groundwater, the effects of vibrations associated with proof rolling and compaction should be taken into consideration, with particular care given to the choice of compaction equipment and method. Observation and monitoring of any existing adjacent development or structures for any signs of settlement or distress should be undertaken in conjunction with any proposed proof rolling and compaction.

All fill should be supported by properly designed and constructed retaining walls or else battered at 2H:1V or flatter and protected against erosion by vegetation or similar and the provision of adequate drainage.

5.3 FOUNDATIONS

5.3.1 GENERAL

At the time of preparation of this report no details in relation to proposed structural loadings were available.

It is understood that proposed structures include:

- Three 5 storey concrete frame apartment buildings over one below ground basement level. A two storey community centre and care facility building.

- Single storey dwellings/duplexes.

Site classification and discussion on foundation alternatives is provided in the following sections.

Further investigation expected to comprise additional cone penetration testing would be required to confirm suitable founding strata at the locations of the apartment and community centre buildings.

5.3.2 SITE CLASSIFICATION

As discussed in Section 4.3, the subsurface conditions encountered at the test locations were generally characterised by sand and the CPT and dynamic penetrometer test results indicated the presence of very loose or loose sands within the upper subsurface profile. Compressible soils comprising soft to firm clay and peat soils were inferred in CPT3 and CPT4 and encountered in TP4 and TP9.

Based on the presence of loose sand and compressible soils within the soil profiles encountered at the site and in accordance with *AS 2870-2011, Residential slabs and Footings* (Ref [3]) the site in its existing condition would be classified as a Class P site. It is noted that reclassification of the site in accordance with AS 2870-2011 will be required following site regrade and development.

5.3.3 HIGH LEVEL FOUNDATIONS

High-level footing alternatives for proposed structures could be expected to include slabs on ground with edge beams or pad footings for the support of concentrated loads.

Compressible soils such as the soft to firm clay and peat soils inferred in CPT3 and CPT4 and encountered in TP4 and TP9 are not suitable for support of footing loads and must be removed from below areas where structures are proposed. As discussed in Section 5.2.2, accurate estimation of the extent and volume of these unsuitable soils requiring removal is expected to require further investigation at an appropriate stage of the development. Removal of the soft to firm clay and peat soils in low lying area of the site is likely to disturb potential acid sulfate soils (see Section 5.5) and must be carried out in accordance with an acid sulfate soil management plan.

Owing to the presence of very loose or loose sands within the subsurface profile at the site as previously discussed, the option of high-level footings is considered to carry a risk of poor performance associated with settlement (both total and differential) of high-level foundations and subsequent distress of proposed structures. Owing to the presence of loose sands it is also considered that there could be the potential for settlement under dynamic loading, e.g., vibrations from earthquakes, etc. Earthquake or other induced vibrations may have the potential to induce densification of the sands resulting in unexpected settlements and subsequent distress of proposed structures. Variations in the density of the sand soils at the site could also be expected to result in differential settlement of high-level foundations.

In consideration of the risk associated with settlement of high-level footings, high-level footings are not recommended unless works are undertaken to increase the density and reduce the variation in density of the sand soils within the upper part of the subsurface profile at the site.

To increase the density and reduce the variation in density of the sand soils in areas above the groundwater table, such in the area of CPT2 where very loose to loose sands were inferred to 1m depth and groundwater was encountered at 4.4m depth, the very loose and loose sands could be excavated, moisture conditioned and recompacted in layers to achieve the relative density requirements in Section 5.2.2 of this report.

Options to reduce the variation in density of the sand soils within the upper part of the subsurface profile where the very loose or loose sands are present close to or below the groundwater table are discussed in Section 5.2.2.

Where options to increase the density and reduce the variation in density of the sand soils within the upper part of the subsurface profile are adopted the works would require the development of an appropriate methodology for verification of works and confirmation that the improved sand subgrade was suitable for the support of structural loads.

Following works to increase the density and reduce the variation in density of the sand soils within the upper part of the subsurface profile at the site and confirmation of the density of the sands within the proposed building area it is suggested that high-level foundations may be proportioned based on an allowable bearing pressure of 150kPa. Settlement of high-level foundations under the above bearing pressures is estimated to be in the order of 5-10mm and could be expected to occur upon loading.

Lots will require reclassification following earthworks but as a guide it is suggested that on-ground slabs for the proposed residential units could be designed as Class M slabs in accordance with *AS 2870-2011, Residential Slabs and Footings* (Ref [3]) where founding conditions comprise medium dense or better sands. It is noted that the scope of *AS 2870-2011* applies to single dwelling, townhouse or similar residential structures and the proposed apartments are expected to lie outside the scope of *AS 2870-2011*.

Excavations in the sand soils at the site are unlikely to stand steeper than about 30° in the short-term and consequently the support of footing excavation faces may generally be expected to be required. Accordingly, it is suggested that allowance should be made during construction for the support of footing excavations.

The base of all footing excavations should be cleaned of fall-in prior to formation of the footing and inspection of the base of footing excavations should be undertaken during construction to confirm founding conditions.

The alternative to high-level footings is piered or piled foundations, which are discussed in the following sections.

5.3.4 DEEP FOUNDATIONS

Piered or piled foundations could be considered as an alternative to high-level footings. Options include:

- bored piers;
- driven displacement piles;
- screw piles.

A suitable founding stratum for piered or piled foundations is expected to comprise the medium dense or better sands below depths of about 2m.

Suggested soil parameters for analysis/design based on the subsurface soil profile encountered at the site are shown on **Table 8**. The assessed boundaries between the various strata are shown in **Table 1**.

Table 8 *Suggested Parameters for Analysis/Design of Deep Foundations*

Strata	Effective Bulk Unit Weight (kN/m^3)	Drained Angle of Friction (degrees)	Drained Cohesion (kPa)	Young's Modulus (kPa)
Very loose and loose sand	18 above water table 8 below water table	30	0	5,000

Strata	Effective Bulk Unit Weight (kN/m^3)	Drained Angle of Friction (degrees)	Drained Cohesion (kPa)	Young's Modulus (kPa)
Medium dense or medium dense to dense sand	20 above water table 10 below water table	35	0	35,000
Dense and very dense sand	12 below water table	38	0	50,000

In undertaking analysis / design using the parameters shown in **Table 8** the appropriate strength reduction factors should be applied in accordance with the relevant applicable standards.

Bored piers and piled foundations are discussed in the following sections.

5.3.4.1 BORED PIERS/CAST IN SITU PILES

Bored piers and cast in situ piles such as grout injected piles could be designed based on the design parameters shown on **Table 9**.

Table 9 *Design Parameters for Bored/Grout Injected Piers*

Founding Strata	End Bearing Pressure (kPa)		Ultimate Shaft Adhesion in Compression (kPa)
	Ultimate(1)	Serviceability(2)	
Medium dense or better sands below a depth of about 2m	1500	500	50

(1) Ultimate values occur at large settlement (>5% of minimum footing dimensions).
(2) End bearing pressure to cause settlement of <1% of minimum footing dimension.
(3) Parameters for piers assume L>4D (L= pile length, D = pile diameter).

The support of pier holes could be expected to be required and accordingly it is recommended that allowance should be made during construction for the support of pier holes. Inspection of the base of piered footing excavations should be undertaken during construction to confirm founding conditions. The base of all footings should be cleaned of fall-in prior to formation of the footing.

Due to the sandy surface profile and founding levels potentially being below the groundwater level it is expected that bored piers would be uneconomical when compared to other available alternatives, e.g. driven piles or screw piles, owing to the difficulty in forming bored piers in the subsurface conditions encountered.

5.3.4.2 DRIVEN PILES

Driven piles such as treated timber piles or driven steel or concrete piles could be considered with piles being driven into the medium dense or better sands.

Suitable alternatives for piled foundations include driven treated timber mini-piles. Treated timber mini-piles of nominal 125mm to 150mm diameter can be driven to depths up to 5 to 6m and where driven into the medium dense to dense sands at the site, may have an allowable bearing capacity in the range of 50 to 80kN. Actual pile load capacities should be based on the pile driving records in conjunction with an appropriate recognised dynamic pile driving formulae.

Where increased load capacities were required (e.g., the two proposed 4/5 storey buildings) consideration could be given to the use of larger diameter hardwood timber piles. The down thrust load capacity of timber piles driven to practical refusal in the dense sands with appropriately sized equipment could be expected to approach the structural capacity of the pile.

Steel and concrete pile capacity can be significantly greater than that available from timber piles. The down thrust load capacity of steel or concrete piles driven to practical refusal in the dense sands with appropriately sized equipment could be expected to approach the structural capacity of the pile.

It would generally be expected that driven piles with solid cross-sections such as timber or concrete piles would achieve practical refusal in the dense sands. Driven steel piles with a relatively small cross-section, eg, steel H sections, are likely to penetrate the dense sands to some degree.

Ultimate pile skin friction design parameters may be calculated based on the parameters provided in **Table 8**.

The capacity of driven piles should be confirmed by use of a suitable dynamic pile analysis package. Settlements of piles founded at practical refusal in the dense sands and with capacity confirmed by use of a suitable dynamic pile analysis package are likely to be within tolerable limits.

Where the pile toe is terminated in dense sand overlying a loose sand layer (such as encountered in CPT1 from 14.4-4.7m depth) there is a risk of the pile punching through to the loose layer. The pile toe should be terminated a minimum of 10 pile diameters above the loose layer, or founded below the loose layer, to avoid a reduction in pile base resistance capacity due to punching.

The effects of vibrations associated with any proposed pile driving would need to be considered, particularly if/where driving piles in proximity to any existing adjacent development, buried services or structures and consultations should be held with piling contractors in this regard. Observation and monitoring of any existing adjacent development and structures for any signs of vibration related distress should be undertaken in conjunction with any proposed pile driving.

5.3.4.3 *Screw Piles*

Screw piles are an alternative to driven piles. Screw piles are end-bearing displacement piles that are screwed to founding depths by excavator or backhoe with the pile capacity determined from installation torque. The advantage of screw piles is the minimal noise and vibration levels during installation and being able to add extension lengths as required. It is expected that screw piles could be seated in the medium dense or better sands below about 2m depth. However, screw piles are likely to be more expensive than driven piles. A specialist screw pile contractor should be contacted to determine suitable foundation depth for screw piles.

5.3.5 Earthquake Design

In accordance with *AS 1170.4-2007* the site is classified as a sub-soil Class D_e – deep or soft soil site.

5.3.6 Foundation Durability

AS 2159–2009, Piling Design and Installation provides recommendations for exposure classification for piles.

A range of soil and groundwater samples were tested to assess aggressivity and the results are attached in **Appendix C** and summarised in **Table 5**. Exposure classifications for buried steel and concrete elements based on the test results were assessed in accordance with *AS 2159-2009, Piling – Design and Installation*, and are also shown in **Table 5**.

Based on the laboratory test results recommended exposure classifications are as follows:

- A mild exposure classification to concrete (governed by the presence of high permeability sandy soils at the site and the presence of groundwater).
- A severe exposure classification for steel (due to high electrical conductivity of the groundwater and hence low resistivity, indicating that the groundwater is saline).

Assessment of exposure classifications using another method/standard (if required) could be based on the laboratory test attached in Appendix C.

5.4 Pavement Design

5.4.1 Design Traffic Loading

A design traffic loading of 1×10^6 ESAs has been adopted for pavement design purposes for the access roads for the proposed development. The guidance provided in the Port Stephens Council *Development Design Specification, D2, Pavement Design* indicates that this is a suitable design traffic loading for a Urban Residential - Local Street type.

If advice indicates different traffic loading to the above the pavement design presented in this report may need to be reviewed.

5.4.2 *SUBGRADE CONDITIONS*

Based on the subsurface conditions encountered at the site, subgrade conditions for pavements are generally expected to comprise sand soils.

The results of the laboratory CBR tests undertaken on samples of the natural sand subgrade materials encountered at the site are summarised in **Table 4** indicated soaked CBR values of 20%, 9%, 14% and 16% for the natural sand subgrade materials tested. A soaked CBR value of 5% was recorded for the sample sandy fill encountered in TP4 between 0-0.9m depth.

The soaked CBR values of 5% and 9% are lower than the typical presumptive CBR values for a sand subgrade provided in Austroads Guide to Pavement Technology Part 2: Pavement Structural Design (Ref [3]).

In consideration of the above, a subgrade CBR of 5% has been adopted for pavement design purposes for proposed pavements at the site.

5.4.3 *PAVEMENT COMPOSITION*

A suitable pavement composition for an unbound flexible pavement for the proposed pavements at the site is shown in **Table 10**.

Table 10 *Pavement Compositions for Unbound Flexible Pavement*

Pavement Course	Thickness of Pavement Course
Wearing course	40mm dense graded AC14
Basecourse	150mm
Subbase	200mm
Total pavement thickness	390mm

A 7mm primer seal should be placed over the basecourse prior to placement of the asphaltic concrete wearing course.

It is noted that the thickness of the asphaltic concrete wearing course shown on **Table 10** is based on the asphaltic concrete wearing course thicknesses typically indicated in Port Stephens Council engineering guidelines and is considered to be the minimum required wearing course thickness.

In our experience there is a risk with the use of a thin asphaltic concrete wearing course associated with deformations/distress of the asphaltic concrete wearing course, particularly from turning/screwing loads at locations such as corners and intersections. Accordingly, periodic maintenance/rehabilitation of the asphaltic concrete wearing course may be required and it is suggested that allowance should be made in this regard. Alternatively, consideration could be given to the use of a thicker asphaltic concrete wearing course in order to increase the life and improve the performance of the asphaltic concrete wearing course, with the use of a suitably toughened or heavy duty asphaltic concrete wearing course in areas, e.g., intersections, which may be subject to turning/screwing loads.

It is noted that previous experience has indicated difficulties can be encountered constructing pavements over sands including subgrade deforming during the placement of the subbase resulting in an uneven subbase thickness across the pavement width. Recommendations to limit this include:

- Keeping the sand subgrade wet prior to placement of the subbase.
- The dozer tracking the subbase over the subgrade should work at the pavement sides rather than the centre to minimise the shoving of the subgrade toward the pavement edge.

5.4.4 PAVEMENT MATERIALS AND COMPACTION REQUIREMENTS

Pavement material specifications and compaction requirements for unbound pavement materials are shown on **Table 11**.

Table 11 *Pavement Materials and Compaction Requirements*

Pavement Course	Material Specification	Compaction Requirements
Basecourse High quality crushed rock or base quality gravel	Material complying with Transport for NSW Specification (Ref [4]) CBR ≥ 80% PI ≥2% and PI ≤ 6%	Min 98% Modified (AS 1289 5.2.1)
Subbase Subbase quality gravel	Material complying with Transport for NSW Specification (Ref [4]) CBR ≥ 30% PI ≤ 10%	Min 95% Modified (AS 1289 5.2.1)
Upper 0.5m of Fill or subgrade replacement	CBR ≥ 10%	Min 100% Standard (AS 1289 5.1.1) Min 75% density index (AS 1289 Cl 5.6.1)
Fill below the upper 0.5m of fill	CBR ≥ 10%	Min 95% Standard (AS 1289 5.1.1) Min 70% density index (AS 1289 Cl 5.6.1)
Sand Subgrade		Min 80% density index (AS 1289 Cl 5.6.1)

CBR – California bearing ratio, PI – Plasticity index.

5.4.5 PAVEMENT DRAINAGE

The moisture regime associated with a pavement has a major influence on pavement performance since the stiffness/strength of the pavement materials and subgrade is highly dependent on the moisture content of the materials. Accordingly, to protect the pavement materials and subgrade from wetting up and softening, particular care would be required to provide a waterproof seal for the pavement materials and adequate surface and sub-surface drainage of the pavement and adjacent area.

It is recommended that subsoil drains should be provided as follows:

- At the interface between any sections of different types of pavements.
- Along the upslope (high) side of any road which are aligned across a slope.
- Along both sides of proposed roads if/where boxed construction is used or the road is in cut.

5.4.6 SUBGRADE PREPARATION

Subgrade preparation for pavement formation could generally be expected to comprise the following:

- Removal of any compressible soils (such as soft to firm clay and peat soils inferred in CPT3 and CPT4 and encountered in TP4 and TP9), topsoil, near surface heavily root affected or organic soils and existing fill and excavation (where required) to subgrade formation level, with the spoiling of any deleterious material.

  As discussed in Section 5.5.2 removal of the soft to firm clay and peat soils in low lying area of the site is likely to disturb potential acid sulfate soils and must be carried out in accordance with an acid sulfate soil management plan.

- Removal of any deleterious material exposed at subgrade level to expose a clean sand subgrade.
- Proof rolling of the exposed subgrade with a heavy (minimum 10 tonne static) roller. Soft or weak areas detected during the proof rolling should be excavated and replaced with compacted fill/subgrade replacement, i.e., select subgrade filling comprising material with a CBR $>$ 10%.
- Compaction of the sand subgrade to achieve a minimum density index of 75% (*AS 1289.5.6.1*).
- Placement and compaction of fill (if/where required) to 100% Standard (*AS 1289.5.1.1*) or a minimum density index of 75% (*AS 1289.5.6.1*) as appropriate.
- Formation of the pavement in accordance with the recommendations and specifications in this report.

Particular care should be taken in the choice of compaction equipment and methods where pavement construction is to be undertaken in the vicinity of any existing adjacent development or buried services. Observation and monitoring of existing adjacent development for any signs of distress should be undertaken in conjunction with proof rolling and compaction of the subgrade and pavement materials.

5.5 ACID SULFATE SOILS

5.5.1 ASSESSMENT CRITERIA

Reference to the ASSMAC Acid Sulfate Soil Manual 1998 (Ref [2]) indicates the soil action criteria for soils according to their texture and the combined existing and potential acidity of the material. The action criteria also take into account the volume of soil to be disturbed, as shown in **Table 12**.

Table 12 *Texture Based Acid Sulfate Action Criteria* (Ref [12])

Type of Material		**Action Criteria if 1 to 1000 Tonnes of material is Disturbed**		**Action Criteria > 1000 Tonnes of material is Disturbed**	
Soil Texture	Approx. Clay Content (%)	Equivalent Sulphur (%S)	Equivalent Acidity (mol H^+/tonne)	Equivalent Sulphur (%S)	Equivalent Acidity (mol H^+/tonne)
Coarse (silty sand to sands)	≤5	0.03	18	0.03	18
Medium (sandy loam-light clay)	5-40	0.06	36	0.03	18
Fine (Medium to heavy clays and silty clays)	≥40	0.1	62	0.03	18

5.5.2 ANALYSIS OF TEST RESULTS

Acid sulfate screening tests were undertaken on twenty soil samples recovered from the test pits and the results of the acid sulfate screening are detailed on the laboratory test reports attached in **Appendix C** and are summarised in **Table 6**.

The field pH (pH_F) of all soil samples tested was greater than 4 and, as such, the soils would not be classified as actual acid sulfate soils.

The ASSMAC guidelines (1998) (Ref [2]) indicate that there is a high level of certainty that potential acid sulfate soil conditions are present where the pH of soil in peroxide is less than 3 and/or the pH change during the test is greater than 1. Based on the ASSMAC guidelines (1998) and the results of the screening tests, there appeared to be a potential for acid forming conditions upon oxidation for thirteen of the twenty samples tested.

Four samples were selected for further analysis by the Suspension Peroxide Oxidation Combined Acidity and Sulfate (SPOCAS) and Chromium Reducible Sulfur (CRS) methods based on the results of the screening testing and the results of this analysis are detailed on the laboratory test reports attached in **Appendix C** and are summarised in **Table 7**.

The laboratory test results indicate that potential acid sulfate soils are present and all four samples tested exceed the action criteria from Ref [2] shown in **Table 12** and an acid sulfate soil management plan (ASSMP) is required. Earthworks and dewatering works would need to be undertaken in accordance with the ASSMP.

5.6 FURTHER GEOTECHNICAL INPUT

Further geotechnical input expected to be required includes the following:

- Further cone penetration testing for input into foundation design for the apartment buildings and community centre and care building.
- Groundwater investigation and monitoring including installation of piezometers to establish groundwater levels at the apartment building locations for design of the proposed basements.
- Detailed investigation to better understand the extent of unsuitable/compressible soils (i.e. peat) requiring removal or treatment.
- Detailed acid sulfate soil investigations of cut areas (e.g. basements, compensatory cut area).

6 LIMITATIONS

This report has been prepared for Principle Living Pty Ltd in accordance with the agreement with RCA. The services performed by RCA have been conducted in a manner consistent with that generally exercised by members of its profession and consulting practice.

This report has been prepared for the sole use of Principle Living Pty Ltd for the specific purpose and the specific development described in the report. The report may not contain sufficient information for purposes or developments other than that described in the report or for parties other than Principle Living Pty Ltd. This report shall only be presented in full and may not be used to support objectives other than those stated in the report without permission.

The information in this report is considered accurate at the date of issue with regard to the current conditions of the site. The conclusions drawn in the report are based on interpolation between boreholes or test pits. Conditions can vary between test locations that cannot be explicitly defined or inferred by investigation.

Yours faithfully
RCA AUSTRALIA

Robert Cater
Senior Geotechnical Engineer

Dr Mark Allman
Principal Geotechnical Engineer

REFERENCES

[1] Fityus, SG, *Calibration of the Blunt Tipped Dynamic Penetrometer for Silica Sands*, proceedings of the First International Conference on Site Characterisation - ISC'98, Vol. 1, Atlanta, Georgia, USA, 1998.

[2] Stone, Y, Ahern, CR, and Blunden, B, *Acid Sulfate Soil Manual 1998*, ASSMAC, Wollongbar, NSW, 1998.

[3] Austroads, *Guide to Pavement Technology Part 2: Pavement Structural Design*, Austroads Publication No. AGPT02-17, Sydney, 2017.

[4] Transport for NSW, *Granular Pavement Base and Subbase Materials*, QA Specification 3051, Edition 7/Revision 1, June 2020.

Appendix A

Drawings

LEGEND

Approximate site boundary

Approximate test pit location

Approximate CPT location

Approximate photograph location and direction

Note: Aerial image taken from Nearmap, 19 August 2023 (used in accordance with commercial licence)
Contours produced from NSW Government Spatial Services LiDAR (extracted from elevation.fdsf.org,au)

N

0 15 30 60 90 120
metres

RCA AUSTRALIA
GEOTECHNICAL • ENVIRONMENTAL

SITE AND TEST LOCATION PLAN
NEWCASTLE GOLF CLUB
VARDON ROAD
FERN BAY

CLIENT	Principle Living Pty Ltd			RCA Ref	15442-402/3		
DRAWN BY	RC	SCALE	1:2,500 (A3)	DRAWING No	1	REV	0
APPROVED BY	MA	DATE	30/11/2023	OFFICE	NEWCASTLE		

LEGEND

Approximate site boundary

Approximate test pit location

Approximate CPT location

Aerial image taken from Nearmap, 19 August 2023
(used in accordance with commercial licence)
Proposed development drawing supplied by Principle Living Pty Ltd
(Drawn by Northrop, DWG No. NL166557-DA Civil)

N

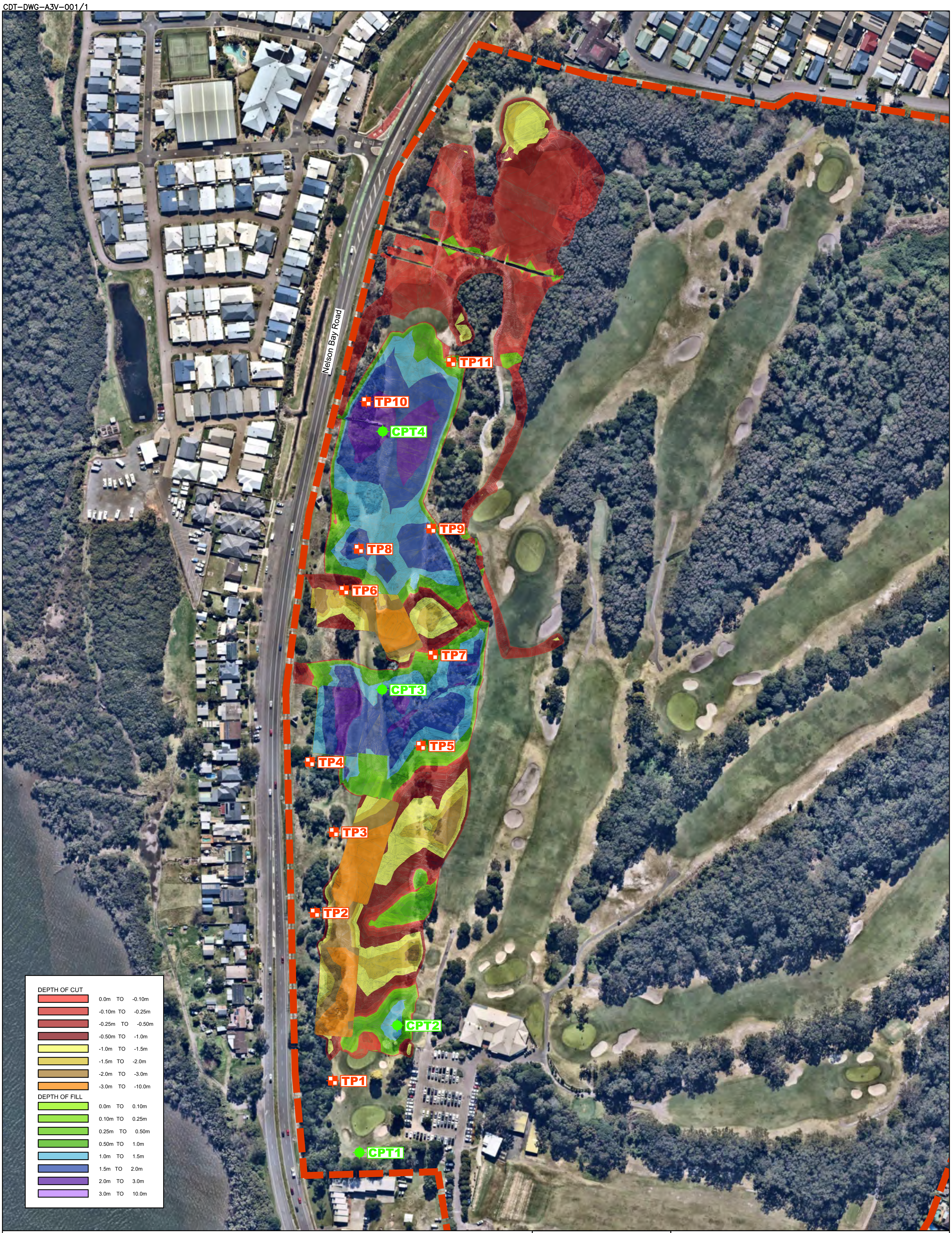
0 15 30 60 90 120
metres

RCA AUSTRALIA
GEOTECHNICAL • ENVIRONMENTAL

GEOTECHNICAL INVESTIGATION
TEST LOCATION PLAN
NEWCASTLE GOLF CLUB
VARDON ROAD, FERN BAY

CLIENT	Principle Living Pty Ltd			RCA Ref	15442-402/3
DRAWN BY	RC	SCALE	1:2,500 (A3)	DRAWING No 2	REV 0
APPROVED BY	MA	DATE	30/11/2023	OFFICE	NEWCASTLE

CDT-DWG-A3V-001/1



LEGEND

Approximate site boundary

Approximate test pit location

Approximate CPT location

NOTE: Aerial image taken from Nearmap, 19 August 2023
(used in accordance with commercial licence)

Drawing adapted from drawing supplied and drawn by
Northrop Consulting Engineers,
(Dwg No. N466557, BEL Shading High Contrast)

0 20 40 60 80 100
metres

N

RCA AUSTRALIA
GEOTECHNICAL • ENVIRONMENTAL

CUT/FILL PLAN
GEOTECHNCIAL INVESTIGATION
NEWCASTLE GOLF CLUB
VARDON ROAD, FERN BAY

CLIENT	Principle Living Pty Ltd			RCA Ref	15442-402/3		
DRAWN BY	RC	SCALE	1:2500 (A3)	DRAWING No	3	REV	0
APPROVED BY	MA	DATE	30/11/2023	OFFICE	NEWCASTLE		

Appendix B

CPT Results

Test Pit Logs

Test Pit Photographs

Explanatory Notes

GEOTECHNICAL TEST PIT LOG
TP1
SHEET 1 OF 1

RCA AUSTRALIA
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PROJECT No: 15442
CLIENT: Principle Living
PROJECT: Geotechnical Investigation
LOCATION: Newcastle Golf Club, Vardon Road, Fern Bay

DATE: 17/05/2021
SURFACE RL:
COORDS:
EXCAVATION METHOD: 3.5t Excavator

Test Pit Information

Depth interval (m)	Dynamic Penetrometer
0.0–0.1	1
0.1–0.2	2
0.2–0.3	1
0.3–0.4	3
0.4–0.5	3
0.5–0.6	3
0.6–0.7	3
0.7–0.8	4
0.8–0.9	4
0.9–1.0	4
1.0–1.1	4
1.1–1.2	4
1.2–1.3	4
1.3–1.4	5
1.4–1.5	5
1.5–1.6	5
1.6–1.7	5
1.7–1.8	5
1.8–1.9	5
1.9–2.0	5
2.0–2.1	5
2.1–2.2	6
2.2–2.3	5
2.3–2.4	5
2.4–2.5	5

WATER: Not Encountered

SAMPLES: 0.30m D 0.50m; 0.60m B (0.6-1.0m) D (0.8-1.0m) 1.00m

Field Material Information

Depth (m)	Classification Symbol	Description	Moisture/Weathering	Consistency/Relative Density/Strength	Structure and Additional Observations
0.00–0.15	SP	TOPSOIL, SAND, fine to medium grained, grey	M		TOPSOIL Grass cover
0.15–1.60	SP	SAND, fine to medium grained, grey		MD MD - D	AEOLIAN
1.60		TEST PIT TP1 TERMINATED AT 1.60 m Test pit sides collapsing			

DESCRIPTION: (SOIL NAME; plasticity/grain size, colour, particle shape, secondary components, minor constituents) (ROCK NAME; grain size, colour, minor constituents)

LOGGED: RC
CHECKED: MA
DATE: 18/06/2021


GREY SCALE
C M Y
R G B
PROJECT No: 15442
PROJECT: Geotechnical Investigation
LOCATION: Newcastle Golf Club,
Vardon Road, Fern Bay
TEST PIT No: TP1
DEPTH: 1.60m
DATE: 17/05/2021

TP1

RCA AUSTRALIA
GEOTECHNICAL • ENVIRONMENTAL

GEOTECHNICAL TEST PIT LOG
TP2

PROJECT No: 15442
CLIENT: Principle Living
PROJECT: Geotechnical Investigation
LOCATION: Newcastle Golf Club, Vardon Road, Fern Bay

DATE: 17/05/2021
SURFACE RL:
COORDS:
EXCAVATION METHOD: 3.5t Excavator

Test Pit Information					Field Material Information					
WATER	DYNAMIC PENETROMETER	FIELD TEST	SAMPLE	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	DESCRIPTION (SOIL NAME; plasticity/grain size, colour, particle shape, secondary components, minor constituents) (ROCK NAME; grain size, colour, minor constituents)	MOISTURE/ WEATHERING	CONSISTENCY/ RELATIVE DENSITY/ STRENGTH	STRUCTURE AND ADDITIONAL OBSERVATIONS
Not Encountered	1, 1			0.20		SM	TOPSOIL, Silty SAND, fine to medium grained, brown	M		TOPSOIL
	2, 2, 3, 3, 4, 4, 4, 5, 5		0.30m D 0.50m; 0.80m D 1.00m	1.10		SM	Silty SAND, fine to medium grained, brown		MD - D	AEOLIAN
	7, 7, 7, 7, 8, 11		1.30m D 1.50m	1.70		SP	SAND, fine to medium grained, pale grey		D	
	8, 11, 11						TEST PIT TP2 TERMINATED AT 1.70 m Test pit sides collapsing			

LOGGED: RC CHECKED: MA DATE: 18/06/2021


RCA
AUSTRALIA
GEOTECHNICAL • ENVIRONMENTAL
GREY SCALE
C M Y
R G B
PROJECT No: 15442
PROJECT: Geotechnical Investigation
LOCATION: Newcastle Golf Club,
Vardon Road, Fern Bay
TEST PIT No: TP2
DEPTH: 1.70m
DATE: 17/05/2021

TP2

GEOTECHNICAL TEST PIT LOG
TP3
SHEET 1 OF 1

RCA AUSTRALIA
GEOTECHNICAL • ENVIRONMENTAL

PROJECT No: 15442
CLIENT: Principle Living
PROJECT: Geotechnical Investigation
LOCATION: Newcastle Golf Club, Vardon Road, Fern Bay

DATE: 17/05/2021
SURFACE RL:
COORDS:
EXCAVATION METHOD: 3.5t Excavator

Depth (m)	Dynamic Penetrometer
0.0–0.1	1
0.1–0.2	1
0.2–0.3	1
0.3–0.4	2
0.4–0.5	2
0.5–0.6	1
0.6–0.7	2
0.7–0.8	2
0.8–0.9	4
0.9–1.0	5
1.0–1.1	6
1.1–1.2	5
1.2–1.3	6
1.3–1.4	4
1.4–1.5	4
1.5–1.6	4
1.6–1.7	4
1.7–1.8	4
1.8–1.9	3
1.9–2.0	2
2.0–2.1	4
2.1–2.2	4
2.2–2.3	6
2.3–2.4	7
2.4–2.5	7
2.5–2.6	6
2.6–2.7	7
2.7–2.8	7
2.8–2.9	8
2.9–3.0	7
3.0–3.1	8

Water	Field Test	Sample	Depth (m)	Classification Symbol	Description	Moisture/Weathering	Consistency/Relative Density/Strength	Structure and Additional Observations
Not Encountered			0.00–0.10	SM	TOPSOIL, Silty SAND, fine to medium grained, brown	M		TOPSOIL
		0.30m B/D 0.50m; 1.30m D 1.50m; 2.00m D 2.20m	0.10–2.20	SP	SAND, fine to medium grained, pale grey	M (0.10–2.0); M-W (2.0–2.2)	L - MD (0.10–0.8); MD (0.8–1.8); L (1.8–2.0); MD (2.0–2.2)	AEOLIAN
			2.20		TEST PIT TP3 TERMINATED AT 2.20 m Test pit sides collapsing			

LOGGED: RC | CHECKED: MA | DATE: 18/06/2021

TP3

Client: Principle Living	**RCA Australia**
Project: Geotechnical Investigation	
Location: Newcastle Golf Club, Vardon Road, Fern Bay	**RCA ref:** 15442-401/0

GEOTECHNICAL TEST PIT LOG
TP4
SHEET 1 OF 1

RCA AUSTRALIA
GEOTECHNICAL • ENVIRONMENTAL

PROJECT No: 15442
CLIENT: Principle Living
PROJECT: Geotechnical Investigation
LOCATION: Newcastle Golf Club, Vardon Road, Fern Bay

DATE: 17/05/2021
SURFACE RL:
COORDS:
EXCAVATION METHOD: 3.5t Excavator

Test Pit Information: WATER, DYNAMIC PENETROMETER, FIELD TEST, SAMPLE, DEPTH (m). Field Material Information: GRAPHIC LOG, CLASSIFICATION SYMBOL, DESCRIPTION, MOISTURE/WEATHERING, CONSISTENCY/RELATIVE DENSITY/STRENGTH, STRUCTURE AND ADDITIONAL OBSERVATIONS.

DEPTH (m)	CLASSIFICATION SYMBOL	DESCRIPTION (SOIL NAME; plasticity/grain size, colour, particle shape, secondary components, minor constituents) (ROCK NAME; grain size, colour, minor constituents)	MOISTURE/ WEATHERING	CONSISTENCY/ RELATIVE DENSITY/ STRENGTH	STRUCTURE AND ADDITIONAL OBSERVATIONS
0.00–0.90	SM	FILL, Silty SAND, fine to medium grained, dark brown, with shells and organics	M		FILL Grass cover Glass bottle at base of fill
0.90–2.00	SP	SAND, fine to medium grained, pale brown	M; W (from 1.4)	L - MD; VL (from 1.7)	AEOLIAN Sides of test pit caving from 1.4m
2.00–2.30	Pt	PEAT, dark brown, pseudo-fibrous	w>PL	F	SWAMP DEPOSIT Hydrogen sulfide odour at 2.0m
2.30		TEST PIT TP4 TERMINATED AT 2.30 m Test pit sides caving in			

Samples:

Depth	Sample
0.30m – 0.50m	B/D/ES
1.20m – 1.30m	D/ES
2.20m – 2.30m	D

Water: Seepage at approx. 1.4 m

Dynamic Penetrometer (blows per 100 mm, from surface): 0, 0, 1, 1, 2, 2, 2, 2, 3, 3, 3, 3, 3, 2, 3, 2, 3, 1, 1, 1, 2, 4, 7, 8, 10, 14, 14, 12, 14, 14

LOGGED: RC | CHECKED: MA | DATE: 18/06/2021

GREY SCALE

C M Y R G B

PROJECT No: 15442
PROJECT: Geotechnical Investigation
LOCATION: Newcastle Golf Club,
Vardon Road, Fern Bay
TEST PIT No: TP4
DEPTH: 2.30m
DATE: 17/05/2021

TP4

GEOTECHNICAL TEST PIT LOG
TP5
SHEET 1 OF 1

RCA AUSTRALIA
GEOTECHNICAL • ENVIRONMENTAL

PROJECT No: 15442
CLIENT: Principle Living
PROJECT: Geotechnical Investigation
LOCATION: Newcastle Golf Club, Vardon Road, Fern Bay

DATE: 17/05/2021
SURFACE RL:
COORDS:
EXCAVATION METHOD: 3.5t Excavator

Test Pit Information					Field Material Information					
WATER	DYNAMIC PENETROMETER	FIELD TEST	SAMPLE	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	DESCRIPTION (SOIL NAME; plasticity/grain size, colour, particle shape, secondary components, minor constituents) (ROCK NAME; grain size, colour, minor constituents)	MOISTURE/ WEATHERING	CONSISTENCY/ RELATIVE DENSITY/ STRENGTH	STRUCTURE AND ADDITIONAL OBSERVATIONS
	1, 2, 3		0.30m	0.30		SP	TOPSOIL, SAND, fine to medium grained, grey-brown, with silt	M		TOPSOIL Grass cover
	3, 4, 4		B/D 0.50m	0.5, 0.60		SP	SAND, fine to medium grained, grey-brown, with silt	M	MD	AEOLIAN Slight Hydrogen sulfide odour at 0.3m
Seepage (1.0m)	5, 4, 5, 4, 4, 4, 5		0.90m D 1.00m; 1.30m	1.0, 1.30		SP	SAND, fine to medium grained, grey	M-W		Sides of test pit caving from 1.0m
	3, 4, 4, 6		D 1.50m	1.5, 1.70		SP	SAND, fine to medium grained, brown, trace of silt	W		Slight Hydrogen sulfide odour at 1.3m
	7, 7, 8, 8, 10, 9, 9, 13			2.0, 2.5, 3.0, 3.5			TEST PIT TP5 TERMINATED AT 1.70 m Test pit sides caving in			

LOGGED: RC | CHECKED: MA | DATE: 18/06/2021

GREY SCALE

C M Y R G B

PROJECT No: 15442
PROJECT: Geotechnical Investigation
LOCATION: Newcastle Golf Club, Vardon Road, Fern Bay
TEST PIT No: TP5
DEPTH: 1.70m
DATE: 17/05/2021

TP5

GEOTECHNICAL TEST PIT LOG

TP6

RCA AUSTRALIA
GEOTECHNICAL • ENVIRONMENTAL

PROJECT No: 15442	DATE: 17/05/2021
CLIENT: Principle Living	SURFACE RL:
PROJECT: Geotechnical Investigation	COORDS:
LOCATION: Newcastle Golf Club, Vardon Road, Fern Bay	EXCAVATION METHOD: 3.5t Excavator

Test Pit Information / **Field Material Information**

WATER	SAMPLE	DEPTH (m)	CLASSIFICATION SYMBOL	DESCRIPTION (SOIL NAME; plasticity/grain size, colour, particle shape, secondary components, minor constituents) (ROCK NAME; grain size, colour, minor constituents)	MOISTURE/ WEATHERING	CONSISTENCY/ RELATIVE DENSITY/ STRENGTH	STRUCTURE AND ADDITIONAL OBSERVATIONS
Not Encountered		0.00–0.15		TOPSOIL, Silty SAND, fine to medium grained, brown	M		TOPSOIL Grass/shrub cover
	0.20m B (0.2-0.5m) D (0.3-0.5m) 0.50m	0.15–0.60	SM	Silty SAND, fine to medium grained, brown		MD	AEOLIAN
	0.70m D 0.80m 1.90m D 2.00m	0.60–2.10	SP	SAND, fine to medium grained, pale grey		MD (to approx. 1.2 m), D	
		2.10		TEST PIT TP6 TERMINATED AT 2.10 m Slow progress due to sides of test pit collapsing			

DYNAMIC PENETROMETER (blows per 0.1 m, from surface):

Depth interval (m)	Blows
0.0–0.1	1
0.1–0.2	1
0.2–0.3	2
0.3–0.4	2
0.4–0.5	3
0.5–0.6	4
0.6–0.7	2
0.7–0.8	4
0.8–0.9	4
0.9–1.0	4
1.0–1.1	4
1.1–1.2	5
1.2–1.3	7
1.3–1.4	8
1.4–1.5	10
1.5–1.6	10
1.6–1.7	8
1.7–1.8	8
1.8–1.9	9
1.9–2.0	11
2.0–2.1	8
2.1–2.2	6
2.2–2.3	5
2.3–2.4	5
2.4–2.5	5
2.5–2.6	5
2.6–2.7	7
2.7–2.8	7
2.8–2.9	7
2.9–3.0	8

FIELD TEST: (none recorded)

LOGGED: RC	CHECKED: MA	DATE: 18/06/2021

GREY SCALE

C M Y R G B

PROJECT No: 15442
PROJECT: Geotechnical Investigation
LOCATION: Newcastle Golf Club,
Vardon Road, Fern Bay
TEST PIT No: TP6
DEPTH: 2.10m
DATE: 17/05/2021

TP6

GEOTECHNICAL TEST PIT LOG
TP7
SHEET 1 OF 1

RCA AUSTRALIA
GEOTECHNICAL • ENVIRONMENTAL

PROJECT No: 15442
CLIENT: Principle Living
PROJECT: Geotechnical Investigation
LOCATION: Newcastle Golf Club, Vardon Road, Fern Bay

DATE: 17/05/2021
SURFACE RL:
COORDS:
EXCAVATION METHOD: 3.5t Excavator

Test Pit Information / Field Material Information

DEPTH (m)	CLASSIFICATION SYMBOL	DESCRIPTION (SOIL NAME; plasticity/grain size, colour, particle shape, secondary components, minor constituents) (ROCK NAME; grain size, colour, minor constituents)	MOISTURE/ WEATHERING	CONSISTENCY/ RELATIVE DENSITY/ STRENGTH	STRUCTURE AND ADDITIONAL OBSERVATIONS
0.00–0.10		TOPSOIL, SAND, fine to medium grained, grey	D-M		TOPSOIL Grass cover
0.10–2.10	SP	SAND, fine to medium grained, grey	D-M (to 1.4m); M-W (from 1.4m)	VL - L (to 0.6m); MD (0.6–1.0m); MD - D (from 1.0m)	AEOLIAN; Slight Hydrogen sulfide odour at 1.4m
2.10		TEST PIT TP7 TERMINATED AT 2.10 m Slow progress due to sides of test pit collapsing			

WATER: Seepage at approx. 1.6m

SAMPLES: 0.10m D 0.20m; 0.70m D 0.80m; 1.40m D 1.50m; 1.80m D 2.00m

DYNAMIC PENETROMETER (per 0.1 m):

Depth interval (m)	Blows
0.0–0.1	1
0.1–0.2	0
0.2–0.3	1
0.3–0.4	2
0.4–0.5	2
0.5–0.6	1
0.6–0.7	2
0.7–0.8	3
0.8–0.9	4
0.9–1.0	4
1.0–1.1	5
1.1–1.2	5
1.2–1.3	5
1.3–1.4	7
1.4–1.5	10
1.5–1.6	8
1.6–1.7	5
1.7–1.8	7
1.8–1.9	7
1.9–2.0	7
2.0–2.1	8
2.1–2.2	9
2.2–2.3	9
2.3–2.4	10
2.4–2.5	10
2.5–2.6	8
2.6–2.7	7
2.7–2.8	11
2.8–2.9	16
2.9–3.0	20

LOGGED: RC | CHECKED: MA | DATE: 18/06/2021

TP7

Client: Principle Living

RCA Australia

Project: Geotechnical Investigation

Location: Newcastle Golf Club, Vardon Road, Fern Bay

RCA ref: 15442-401/0

GEOTECHNICAL TEST PIT LOG
TP8
SHEET 1 OF 1

RCA AUSTRALIA
GEOTECHNICAL • ENVIRONMENTAL

PROJECT No: 15442
CLIENT: Principle Living
PROJECT: Geotechnical Investigation
LOCATION: Newcastle Golf Club, Vardon Road, Fern Bay

DATE: 17/05/2021
SURFACE RL:
COORDS:
EXCAVATION METHOD: 3.5t Excavator

Depth (m)	Dynamic Penetrometer	Sample	Classification Symbol	Description	Moisture/Weathering	Consistency/Relative Density/Strength	Structure and Additional Observations
0.0–0.15	1		SM	TOPSOIL, Silty SAND, fine to medium grained, dark brown	M		TOPSOIL Grass cover
0.15–0.60	1, 2, 3, 2, 1	0.20m B (0.2-0.5m) D (0.4-0.5m) 0.50m	SM	Silty SAND, fine to medium grained, black, trace of root fibres	M	L	ALLUVIUM
0.60–1.50	1, 1, 5, 3, 6, 7, 9, 10, 11	1.30m D/GW 1.50m	SP	SAND, fine to medium grained, pale brown	M-W, W	MD - D	AEOLIAN
1.50	11, 11, 13, 12, 14			TEST PIT TP8 TERMINATED AT 1.50 m Sides of test pit collapsing below groundwater table			

Water: groundwater observed at approx. 0.8 m, 17/05/21

LOGGED: RC
CHECKED: MA
DATE: 18/06/2021


RCA AUSTRALIA
GREY SCALE
C M Y
R G B
PROJECT No: 15442
PROJECT: Geotechnical Investigation
LOCATION: Newcastle Golf Club,
Vardon Road, Fern Bay
TEST PIT No: TP8
DEPTH: 1.50m
DATE: 17/05/2021

TP8

GEOTECHNICAL TEST PIT LOG

TP9

SHEET 1 OF 1

RCA AUSTRALIA
GEOTECHNICAL • ENVIRONMENTAL

PROJECT No: 15442
CLIENT: Principle Living
PROJECT: Geotechnical Investigation
LOCATION: Newcastle Golf Club, Vardon Road, Fern Bay

DATE: 17/05/2021
SURFACE RL:
COORDS:
EXCAVATION METHOD: 3.5t Excavator

Test Pit Information | Field Material Information

WATER	DYNAMIC PENETROMETER	FIELD TEST	SAMPLE	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	DESCRIPTION (SOIL NAME; plasticity/grain size, colour, particle shape, secondary components, minor constituents) (ROCK NAME; grain size, colour, minor constituents)	MOISTURE/ WEATHERING	CONSISTENCY/ RELATIVE DENSITY/ STRENGTH	STRUCTURE AND ADDITIONAL OBSERVATIONS
	0, 1		0.20m	0.20		OH	TOPSOIL, ORGANIC, Sandy SILT, high plasticity, dark brown			TOPSOIL Grass cover
	0, 1, 0		D 0.40m; 0.50m	0.50		SP	SAND, fine to medium grained, brown, with silt	M	VL	AEOLIAN
17/05/21 (groundwater)	1, 1, 0	PP 60 - 90kPa	D (0.5-0.7m) U50 (0.6-0.8m) 0.80m	0.80		Pt	PEAT, black, with timber fragments, pseudo-fibrous	w>PL	F	SWAMP DEPOSIT
	3, 4, 6, 7, 8, 8		1.20m D/GW 1.40m	1.0, 1.40		SP	SAND, fine to medium grained, pale grey	M	MD; MD - D	AEOLIAN
	9, 8, 8, 11, 11, 15, 17, 19, 18, 18, 20			1.5, 2.0, 2.5, 3.0, 3.5			TEST PIT TP9 TERMINATED AT 1.40 m Sides of test pit collapsing below groundwater table			

LOGGED: RC | CHECKED: MA | DATE: 18/06/2021


RCA AUSTRALIA
GEOTECHNICAL • ENVIRONMENTAL
GREY SCALE
C M Y
R G B
PROJECT No: 15442
PROJECT: Geotechnical Investigation
LOCATION: Newcastle Golf Club,
Vardon Road, Fern Bay
TEST PIT No: TP9
DEPTH: 1.40m
DATE: 17/05/2021

TP9

GEOTECHNICAL TEST PIT LOG

TP10

SHEET 1 OF 1

RCA AUSTRALIA
GEOTECHNICAL • ENVIRONMENTAL

PROJECT No: 15442
CLIENT: Principle Living
PROJECT: Geotechnical Investigation
LOCATION: Newcastle Golf Club, Vardon Road, Fern Bay

DATE: 17/05/2021
SURFACE RL:
COORDS:
EXCAVATION METHOD: 3.5t Excavator

Test Pit Information					Field Material Information					
WATER	DYNAMIC PENETROMETER	FIELD TEST	SAMPLE	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	DESCRIPTION (SOIL NAME; plasticity/grain size, colour, particle shape, secondary components, minor constituents) (ROCK NAME; grain size, colour, minor constituents)	MOISTURE/ WEATHERING	CONSISTENCY/ RELATIVE DENSITY/ STRENGTH	STRUCTURE AND ADDITIONAL OBSERVATIONS
	1, 2, 3		0.10m D 0.20m	0.25			TOPSOIL, Silty SAND, fine to medium grained, dark brown	M		TOPSOIL
17/05/21 (at 0.6m)	3, 3, 3, 2, 3, 3, 3, 7, 10, 10, 12		0.30m D 0.40m; 0.80m D 1.00m; 1.10m GW (1.1-1.2m); D (1.1-1.3m) 1.30m	0.5, 1.0, 1.30		SP	SAND, fine to medium grained, grey-brown, with silt Becoming grey	M M-W W	MD D	AEOLIAN Hydrogen sulfide odour Seepage observed when digging at 1.1m, water level rises to 0.6m over ~15 minutes
	12, 12, 15, 12, 15, 13, 11, 11, 12, 14, 15, 21			1.5, 2.0, 2.5, 3.0, 3.5			TEST PIT TP10 TERMINATED AT 1.30 m Sides of test pit collapsing below groundwater table			

LOGGED: RC | CHECKED: MA | DATE: 18/06/2021


RCA AUSTRALIA
GEOTECHNICAL • ENVIRONMENTAL
GREY SCALE
C M Y
R G B
PROJECT No: 15442
PROJECT: Geotechnical Investigation
LOCATION: Newcastle Golf Club,
Vardon Road, Fern Bay
TEST PIT No: TP10
DEPTH: 1.30m
DATE: 17/05/2021

TP10

GEOTECHNICAL TEST PIT LOG
TP11
SHEET 1 OF 1

RCA AUSTRALIA
GEOTECHNICAL • ENVIRONMENTAL

PROJECT No: 15442
CLIENT: Principle Living
PROJECT: Geotechnical Investigation
LOCATION: Newcastle Golf Club, Vardon Road, Fern Bay

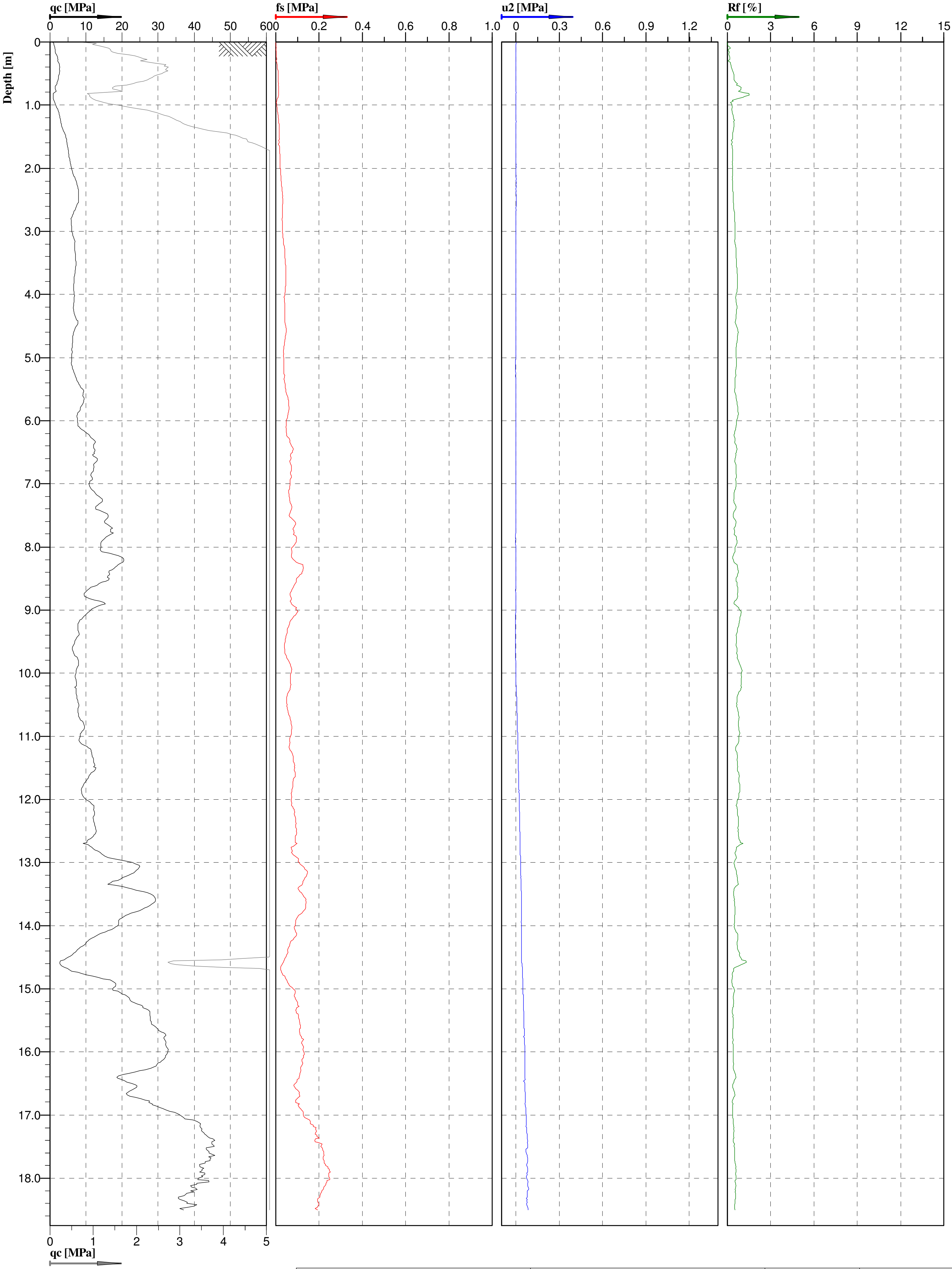
DATE: 17/05/2021
SURFACE RL:
COORDS:
EXCAVATION METHOD: 3.5t Excavator

Test Pit Information					Field Material Information					
WATER	DYNAMIC PENETROMETER	FIELD TEST	SAMPLE	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	DESCRIPTION (SOIL NAME; plasticity/grain size, colour, particle shape, secondary components, minor constituents) (ROCK NAME; grain size, colour, minor constituents)	MOISTURE/ WEATHERING	CONSISTENCY/ RELATIVE DENSITY/ STRENGTH	STRUCTURE AND ADDITIONAL OBSERVATIONS
	1, 1			0.15		SP	TOPSOIL, SAND, fine to medium grained, grey	M		TOPSOIL
17/05/21 (at ~0.85 m)	2, 4, 3, 4, 5, 3, 3, 3, 3		0.30m D (0.3-0.4m) B (0.3-0.5m) 0.50m; 1.00m D 1.10m	0.5, 1.0, 1.10		SP	SAND, fine to medium grained, grey Becoming grey and brown Becoming pale grey Becoming orange-brown	M W	L - MD	AEOLIAN
	2, 2, 1, 2, 3, 3, 3, 4, 5, 6, 6, 8, 7, 7, 7, 12, 11, 12, 14			1.5, 2.0, 2.5, 3.0, 3.5			TEST PIT TP11 TERMINATED AT 1.10 m Sides of test pit collapsing below groundwater table Test pit collapsed to 0.8m depth following excavation			

LOGGED: RC
CHECKED: MA
DATE: 18/06/2021


GREY SCALE
C M Y
R G B
PROJECT No: 15442
PROJECT: Geotechnical Investigation
LOCATION: Newcastle Golf Club,
Vardon Road, Fern Bay
TEST PIT No: TP11
DEPTH: 1.10m
DATE: 17/05/2021

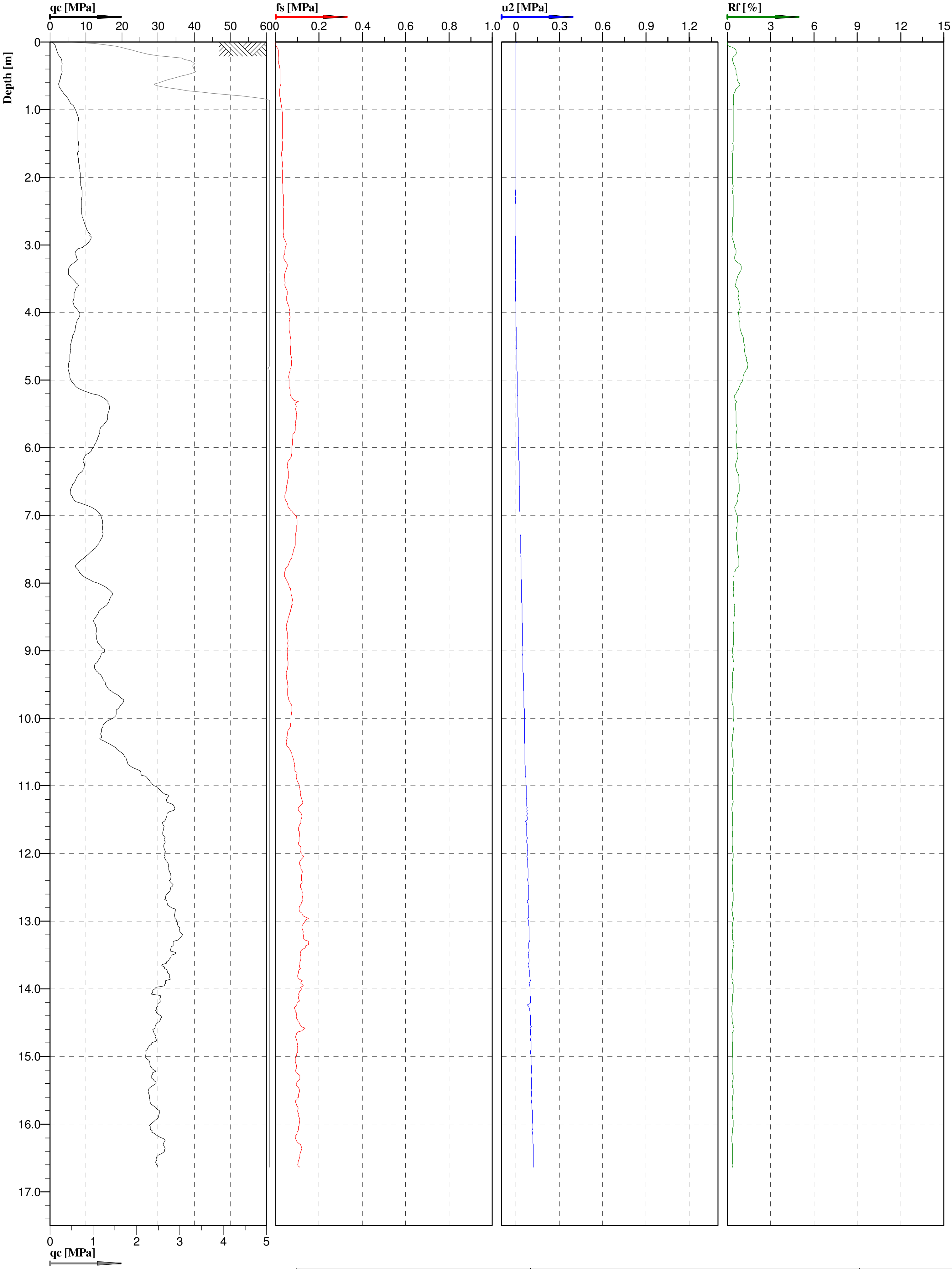
TP11

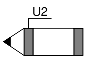


stratacore

Cone No: 4254
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: Newcastle Golf Club	Position: X: 0.00 m, Y: 0.00 m	Ground level: 0.00	Test no: CPT1
Project ID: 15442	Client: RCA Australia	Date: 17/05/2021	Scale: 1 : 75
Project: Newcastle Golf Club		Page: 1/1	Fig:
		File: CPT1.cpt	

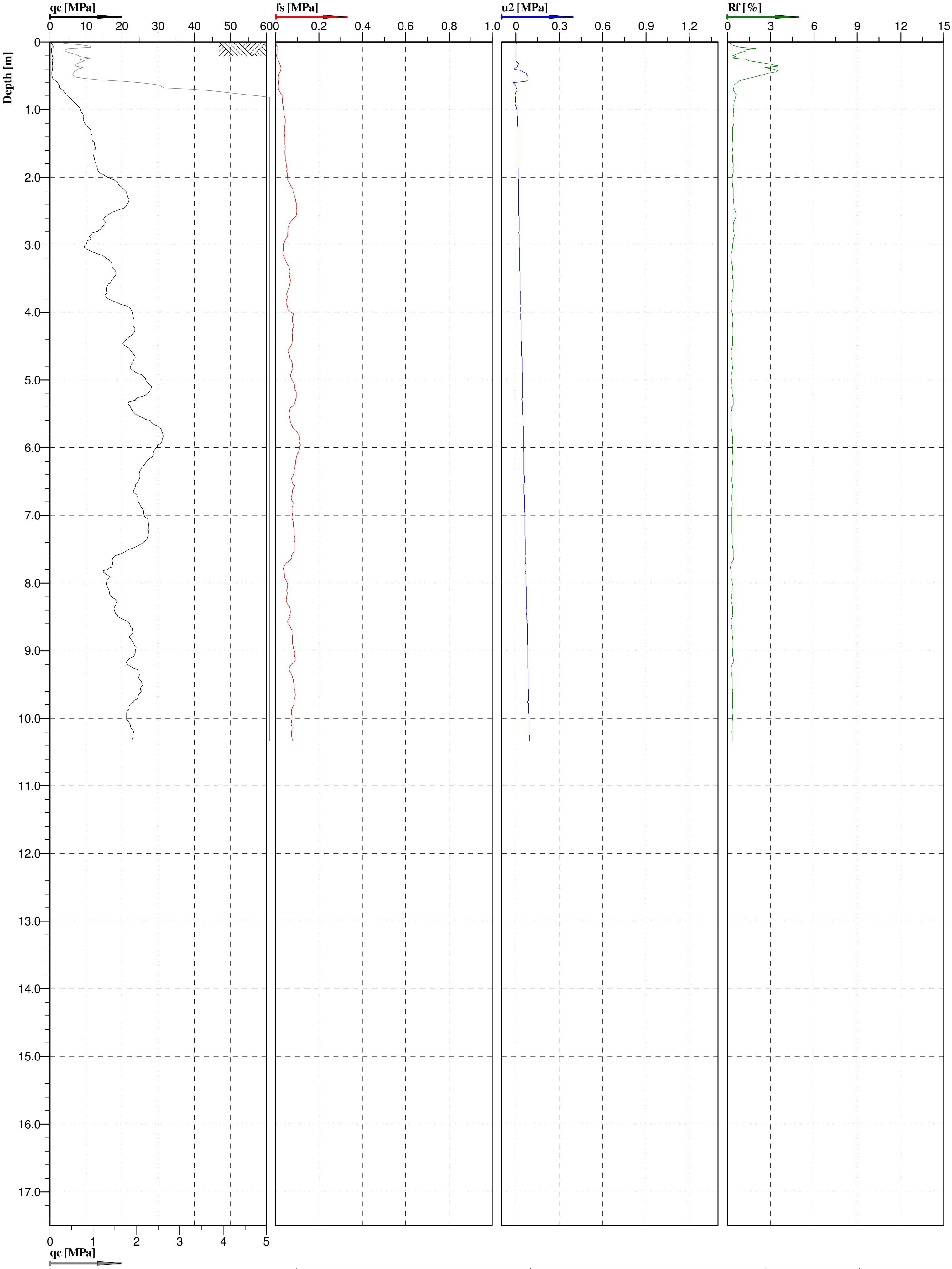




stratacore

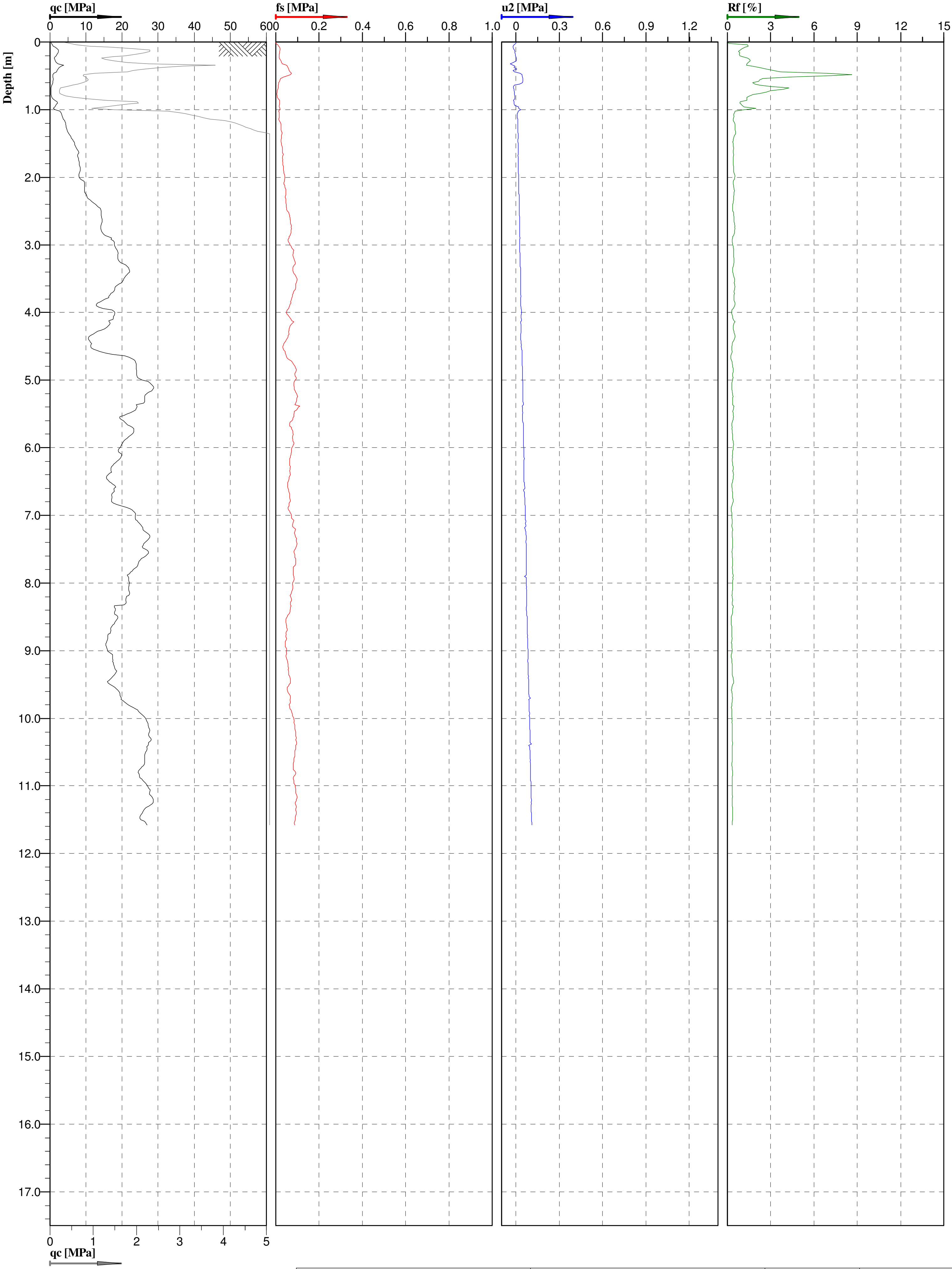
Cone No: 4254
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: Newcastle Golf Club	Position: X: 0.00 m, Y: 0.00 m	Ground level: 0.00	Test no: CPT2
Project ID: 15442	Client: RCA Australia	Date: 17/05/2021	Scale: 1 : 70
Project: Newcastle Golf Club		Page: 1/1	Fig:
		File: CPT2.cpt	



Cone No: 4254
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: Newcastle Golf Club	Position: X: 0.00 m, Y: 0.00 m	Ground level: 0.00	Test no: CPT3
Project ID: 15442	Client: RCA Australia	Date: 17/05/2021	Scale: 1 : 70
Project: Newcastle Golf Club		Page: 1/1	Fig:
		File: CPT3.cpt	



stratacore

Cone No: 4254
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: Newcastle Golf Club	Position: X: 0.00 m, Y: 0.00 m	Ground level: 0.00	Test no: CPT4
Project ID: 15442	Client: RCA Australia	Date: 17/05/2021	Scale: 1 : 70
Project: Newcastle Golf Club		Page: 1/1	Fig:
		File: CPT4.cpt	

Explanatory Notes – Soil Description

In engineering terms, soil includes every type of uncemented or partially cemented material found in the ground. In practice, if the material can be remoulded by hand in its field condition or in water it is described as a soil. The dominant soil constituent is given in capital letters, with secondary textures in lower case. The dominant feature is assessed from *AS 1726:2017 – Geotechnical Site Investigations* and a soil symbol is used to define a soil layer.

METHOD

Method	Description
AD/T	Auger Drilling with tungsten carbide bit
AD/V	Auger Drilling with V Bit
AS	Auger Screwing
AT	Air Track
BH	Backhoe
CT	Cable Tool Rig
DB	Washbore Drag Bit
DT	Diatube
E	Excavator
EH	Excavator with Hammer
HA	Hand Auger
HQ	Diamond Core-63mm diameter
N	Natural Exposure
NMLC	Diamond Core-52mm diameter
NQ	Diamond Core-47mm diameter
Percussion	Percussion Drilling
PT	Push Tube
RR	Rock Roller
V	Vacuum Excavation
WS	Washbore
X	Existing Excavation

WATER

Water level at date shown

Seepage

NOT ENCOUNTERED: The borehole/test pit was dry soon after excavation. Inflow may have been observed had the borehole/test pit been left open for a longer period.

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

SAMPLING

Sample	Description
B	Bulk Disturbed Sample
D	Disturbed Sample
SPT	Standard Penetration Test
U50	Undisturbed Sample - 50mm diameter
U75	Undisturbed Sample - 75mm diameter
ES	Soil Sample, Environmental
EW	Water Sample, Environmental
G	Gas Sample

SOIL CLASSIFICATION

The appropriate symbols are selected based on the result of visual examination, field tests and available laboratory test results, such as particle size analysis, liquid limit and plasticity index.

Group Symbol	Description
GW	Well graded gravel
GP	Poorly graded gravel
GM	Silty gravel
GC	Clayey gravel
SW	Well graded sand
SP	Poorly graded sand
SM	Silty sand
SC	Clayey sand
ML	Silt of low plasticity
CL	Clay of low plasticity
OL	Organic soil of low plasticity
CI	Clay of medium plasticity
MH	Silt of high plasticity
CH	Clay of high plasticity
OH	Organic soil of high plasticity
Pt	Peat, highly organic soil

MOISTURE CONDITION

For coarse grained soils, the following terms are used

Dry - Non-cohesive and free-running

Moist - Soil feels cool, darkened in colour
- Soil tends to stick together

Wet - Soil feels cool, darkened in colour
- Soil tends to stick together, free water forms when handling

For fine grained soils, the following moisture content (w) terms are used:

$w < PL$ - Moist, dry of plastic limit

$w \approx PL$ - Moist, near plastic limit.

$w > PL$ - Moist, wet of plastic limit.

$w \approx LL$ - Wet, near liquid limit.

$w > LL$ - Wet, wet of liquid limit

PLASTICITY

Soil plasticity is a measure of the range of water content over which a soil exhibits plastic properties. The classification of the degree of plasticity in terms of the Liquid Limit (LL) is as follows.

Description of Plasticity	Range of Liquid Limit for Silt	Range of Liquid Limit for Clay
Non-plastic	Not applicable	Not applicable
Low plasticity	≤50	≤35
Medium plasticity	Not applicable	>35 and ≤50
High plasticity	>50	>50

COHESIVE SOILS – CONSISTENCY

The consistency of a cohesive soil is defined by descriptive terminology such as very soft, soft, firm, stiff, very stiff and hard. These terms are assessed by the shear strength of the soil as observed visually, by hand penetrometer, dynamic cone penetrometer or vane shear values and by resistance to deformation to hand moulding.

A hand penetrometer may be used in the field or the laboratory to provide an approximate assessment of the unconfined compressive strength (UCS) of cohesive soils. Undrained shear strength $c_u = 0.5 \times UCS$. Undrained shear strength values are recorded in kPa as follows:

Strength	Symbol	Indicative Undrained Shear Strength, c_u (kPa)
Very Soft	VS	≤12
Soft	S	>12 and ≤25
Firm	F	>25 and ≤50
Stiff	St	>50 and ≤100
Very Stiff	VSt	>100 and ≤200
Hard	H	>200
Friable	Fr	—

COHESIONLESS SOILS – RELATIVE DENSITY

Relative density terms such as very loose, loose, medium dense, dense and very dense are used to describe silty and sandy material, and these are usually based on resistance to drilling penetration, Standard Penetration Test (SPT) N values or Perth Sand Penetrometer resistance.

Term	Symbol	Density Index
Very Loose	VL	0 to 15
Loose	L	15 to 35
Medium Dense	MD	35 to 65
Dense	D	65 to 85
Very Dense	VD	>85

SOIL PARTICLE SIZE DESCRIPTIVE TERMS

Fraction	Name	Subdivision	Size (mm)
Oversize	Boulders		>200
	Cobbles		63 to 200
Coarse grained soil	Gravel	Coarse	19 to 63
		Medium	6.7 to 19
		Fine	2.36 to 6.7
	Sand	Coarse	0.6 to 2.36
		Medium	0.21 to 0.6
		Fine	0.075 to 0.21
Fine grained soil	Silt		0.002 to 0.075
	Clay		<0.002

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Explanatory Notes - Rock Description

METHOD

Refer to soil description sheet.

WATER

Refer to soil description sheet.

ROCK QUALITY

The defect spacing is shown where applicable and the Rock Quality Designation (RQD) and Total Core Recovery (TCR) for each core run is given where:

$$\text{TCR} = \frac{\text{Length of core recovered}}{\text{Length of core run}} \times 100\%$$

$$\text{RQD} = \frac{\text{Sum of axial length of sound core pieces >100mm long}}{\text{Length of core run}} \times 100\%$$

ROCK MATERIAL WEATHERING

Rock material weathering is described using the abbreviations and definitions used in *AS1726:2017– Geotechnical Site Investigations*.

<table>
<tr><th colspan="2">Term</th><th colspan="2">Abbreviation</th><th>Definition</th></tr>
<tr><td colspan="2">Residual Soil</td><td colspan="2">RS</td><td>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.</td></tr>
<tr><td colspan="2">Extremely weathered</td><td colspan="2">XW</td><td>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.</td></tr>
<tr><td>Highly Weathered</td><td rowspan="2">Distinctly Weathered</td><td>HW</td><td rowspan="2">DW</td><td>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.</td></tr>
<tr><td>Moderately Weathered</td><td>MW</td><td>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.</td></tr>
<tr><td colspan="2">Slightly Weathered</td><td colspan="2">SW</td><td>Rock is partially discoloured with staining or bleaching along joints but shows little or no change of strength from fresh rock.</td></tr>
<tr><td colspan="2">Fresh</td><td colspan="2">FR</td><td>Rock shows no sign of decomposition of individual minerals or colour changes.</td></tr>
</table>

Where it is not practicable to distinguish between 'Highly Weathered' and 'Moderately Weathered' rock the term 'Distinctly Weathered' may be used. 'Distinctly Weathered' is defined as follows: '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 weathering products in the pores'. There is some change in rock strength.

ROCK MATERIAL STRENGTH

Rock strength is described using *AS1726:2017– Geotechnical Site Investigations* and *ISRM – Commission on Standardisation of Laboratory and Field Tests, 'Suggested method of determining the Uniaxial Compressive Strength of Rock materials and the Point Load Index'* as follows:

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

◀ Diametral Point Load Index test.

▼ Axial Point Load Index test.

DEFECT SPACING/BEDDING THICKNESS

Depending on the project, may be either described as mean perpendicular spacing within a set of defects or bedding, or as the spacing between all defects within the rock mass.

Term	Defect Spacing	Bedding
Extremely closely spaced	<6 mm	Thinly laminated
	6 to 20 mm	Laminated
Very closely spaced	20 to 60 mm	Very thin
Closely spaced	0.06 to 0.2 m	Thin
Moderately widely spaced	0.2 to 0.6 m	Medium
Widely spaced	0.6 to 2.0 m	Thick
Very widely spaced	>2 m	Very thick

DEFECT DESCRIPTION

Type	Definition
JT	Joint
BP	Bedding Parting
CO	Contact
CS	Clay Seam
CZ	Crush Zone
DK	Dyke
DZ	Decomposed Zone
FC	Fracture
FZ	Fracture Zone
FL	Foliation
FLT	Fault
VN	Vein
SM	Seam
IS	Infilled Seam
SZ	Shear Zone

Planarity	Roughness
PR – Planar	VR – Very Rough
CU – Curved	RF – Rough
U – Undulating	S – Smooth
ST – Stepped	POL – Polished
IR – Irregular	SL – Slickensided

Symbol	Coating or Infill
CA	Calcite
Clay	Clay
CN	Clean
Fe	Iron oxide
KT	Chlorite
Qz	Quartz
X	Carbonaceous
SN	Stain
VNR	Veneer

The inclinations of defects are measured from perpendicular to the core axis.

Appendix C

Laboratory Test Results

Appendix C1

Particle Size Distribution, Atterberg Limits and CBR Laboratory Test Results

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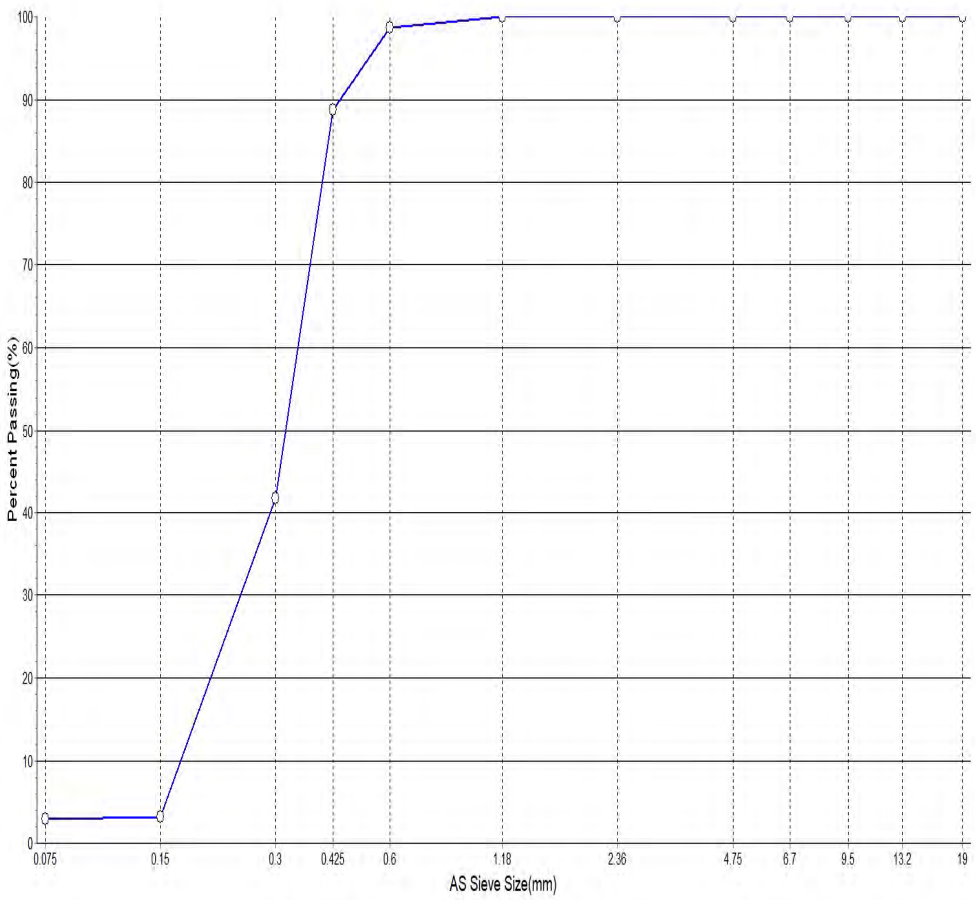
Particle Size Distribution Report

Client :	**Principle Living**	Report Number:	**15442 - 001**
Address :	**PO Box 206, Carrington, NSW, 2294**	Report Date :	**8/06/2021**
Project Name :	**Geotechnical Investigation**	Order Number :	
Project Number :	**15442**	Test Method :	**AS 1289.3.6.1 (washed)**
Location:	**Newcastle Golf Club Vardon Road , Fern Bay**		**Page 1 of 4**

Sample Number :	21-1087	SAMPLE LOCATION	
Sampling Method :	AS SUPPLIED	**TP1**	
Sampled By :	RCA Geotech	**0.6-1.0m**	
Date Sampled :	17/05/2021		
Date Tested :	20/05/2021		
Material Type :		Test Number :	
Material Source :		Lot Number :	
Remarks :		Specification Number :	

AS Sieve Size(mm)	Percent Passing	Specification Limits
100		
75		
63		
53		
37.5		
26.5		
19.0		
13.2		
9.5		
6.7		
4.75		
2.36		
1.18	**100**	
0.600	**99**	
0.425	**89**	
0.300	**42**	
0.150	**3**	
0.075	**3**	



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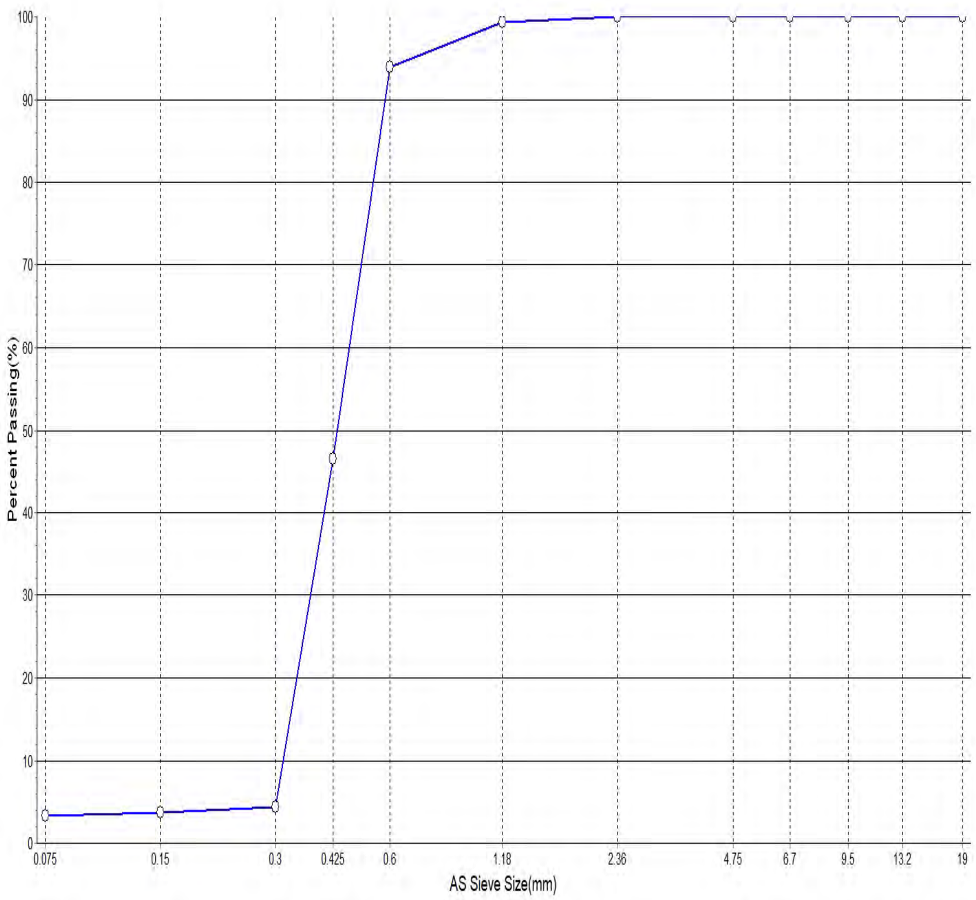
Particle Size Distribution Report

Client :	**Principle Living**	Report Number:	**15442 - 001**
Address :	**PO Box 206, Carrington, NSW, 2294**	Report Date :	**8/06/2021**
Project Name :	**Geotechnical Investigation**	Order Number :	
Project Number :	**15442**	Test Method :	**AS 1289.3.6.1 (washed)**
Location:	**Newcastle Golf Club Vardon Road , Fern Bay**		**Page 2 of 4**

Sample Number :	21-1088	SAMPLE LOCATION	
Sampling Method :	AS SUPPLIED	**TP3**	
Sampled By :	RCA Geotech	**0.3-0.5m**	
Date Sampled :	17/05/2021		
Date Tested :	20/05/2021		
Material Type :		Test Number :	
Material Source :		Lot Number :	
Remarks :		Specification Number :	

AS Sieve Size(mm)	Percent Passing	Specification Limits
100		
75		
63		
53		
37.5		
26.5		
19.0		
13.2		
9.5		
6.7		
4.75		
2.36	**100**	
1.18	**99**	
0.600	**94**	
0.425	**46**	
0.300	**4**	
0.150	**4**	
0.075	**3**	



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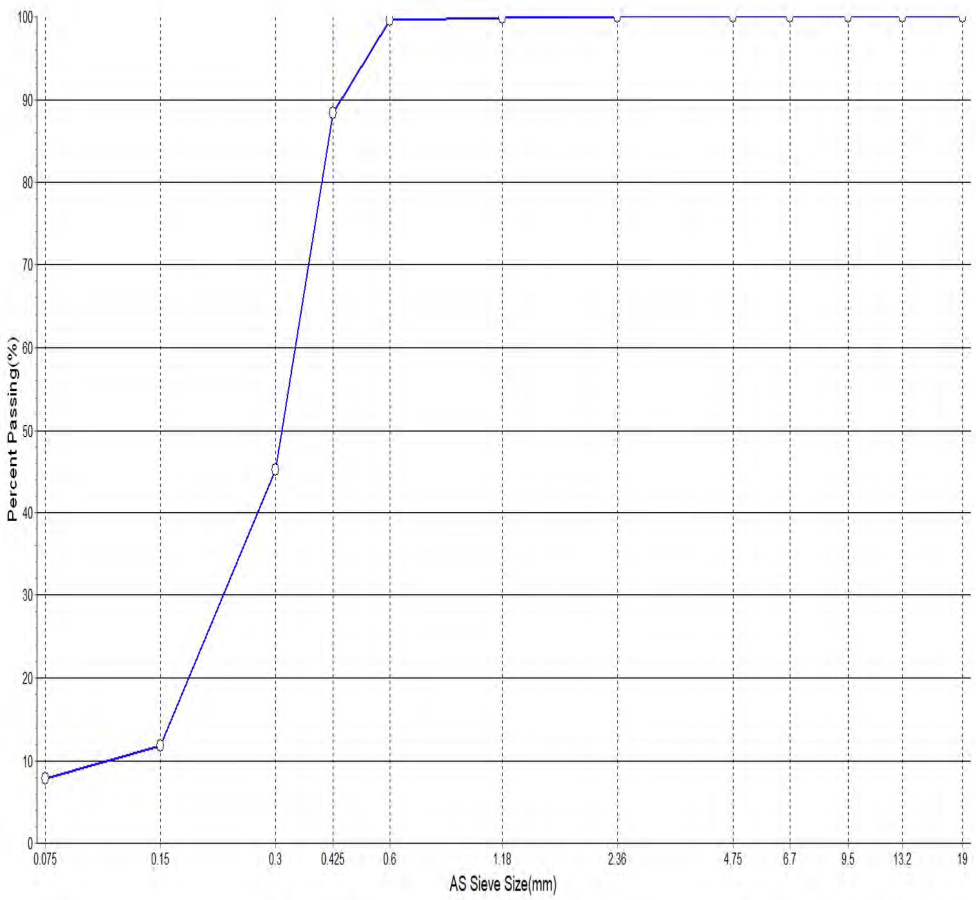
Particle Size Distribution Report

Client :	**Principle Living**	Report Number:	**15442 - 001**
Address :	**PO Box 206, Carrington, NSW, 2294**	Report Date :	**8/06/2021**
Project Name :	**Geotechnical Investigation**	Order Number :	
Project Number :	**15442**	Test Method :	**AS 1289.3.6.1 (washed)**
Location:	**Newcastle Golf Club Vardon Road , Fern Bay**		**Page 3 of 4**

		SAMPLE LOCATION	
Sample Number :	21-1090	**TP5**	
Sampling Method :	AS SUPPLIED	**0.3-0.5m**	
Sampled By :	RCA Geotech		
Date Sampled :	17/05/2021		
Date Tested :	20/05/2021		
Material Type :		Test Number :	
Material Source :		Lot Number :	
Remarks :		Specification Number :	

AS Sieve Size(mm)	Percent Passing	Specification Limits
100		
75		
63		
53		
37.5		
26.5		
19.0		
13.2		
9.5		
6.7		
4.75		
2.36	**100**	
1.18	**100**	
0.600	**100**	
0.425	**88**	
0.300	**45**	
0.150	**12**	
0.075	**8**	



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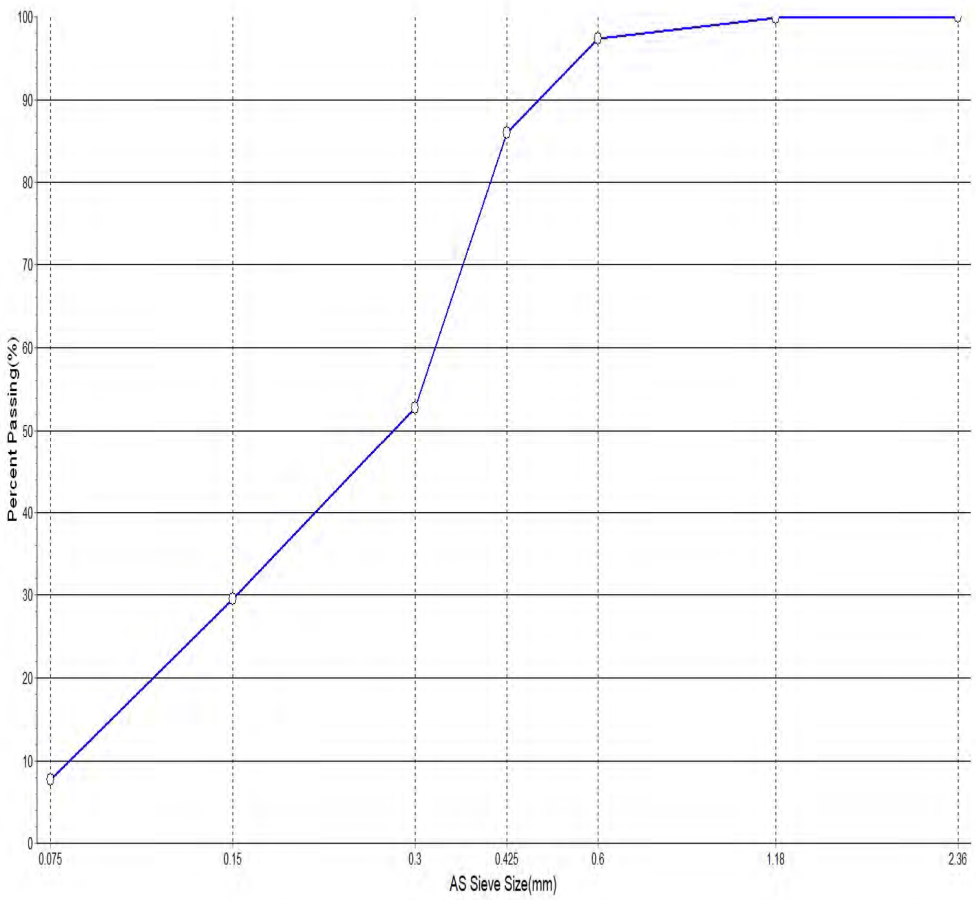
Particle Size Distribution Report

Client :	**Principle Living**	Report Number:	**15442 - 001**
Address :	**PO Box 206, Carrington, NSW, 2294**	Report Date :	**8/06/2021**
Project Name :	**Geotechnical Investigation**	Order Number :	
Project Number :	**15442**	Test Method :	**AS 1289.3.6.1 (washed)**
Location:	**Newcastle Golf Club Vardon Road , Fern Bay**		**Page 4 of 4**

Sample Number :	21-1096	SAMPLE LOCATION	
Sampling Method :	AS SUPPLIED	**TP10**	
Sampled By :	RCA Geotech	**0.3-0.4m**	
Date Sampled :	17/05/2021		
Date Tested :	20/05/2021		
Material Type :		Test Number :	
Material Source :		Lot Number :	
Remarks :		Specification Number :	

AS Sieve Size(mm)	Percent Passing	Specification Limits
100		
75		
63		
53		
37.5		
26.5		
19.0		
13.2		
9.5		
6.7		
4.75		
2.36	**100**	
1.18	**100**	
0.600	**97**	
0.425	**86**	
0.300	**53**	
0.150	**29**	
0.075	**8**	



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Atterberg Limits Report

Client :	**Principle Living**	Report Number:	**15442 - 002**
Address :	**PO Box 206, Carrington, NSW, 2294**	Report Date :	**8/06/2021**
Project Name :	**Geotechnical Investigation**	Order Number :	
Project Number :	**15442**	Test Method :	
Location:	**Newcastle Golf Club Vardon Road , Fern Bay**		**AS1289.3.1.1, 3.2.1, 3.3.1**

Sample Number :	21-1093	21-1095		
Test Number :				
Date Sampled :	17/05/2021	17/05/2021		
Date Tested :	29/05/2021	29/05/2021		
Sampled By :	RCA Geotech	RCA Geotech		
Sampling Method :	AS SUPPLIED	AS SUPPLIED		
Material Source :				
Material Type :				
Sample Location :	TP9 0.5-0.7m	TP4 2.2-2.3m		
Lot Number :				
Moisture Method :	AS 1289.2.1.1	AS 1289.2.1.1		
Sample History :	Oven dried prep (50°C)	Oven dried prep (50°C)		
Sample Preparation :	Dry	Dry		
Notes :				
Mould Length (mm) :				
Liquid Limit (%) :	Not Obtainable	Not Obtainable		
Plastic Limit (%) :	Not Obtainable	Not Obtainable		
Plasticity Index (%) :	NP (Non Plastic)	NP (Non Plastic)		
Linear Shrinkage (%) :				
SPECIFICATION DETAILS				
Specification Number :				
Liquid Limit - Max :				
Plasticity Index - Max :				
Linear Shrinkage - Max :				
Remarks :	-			

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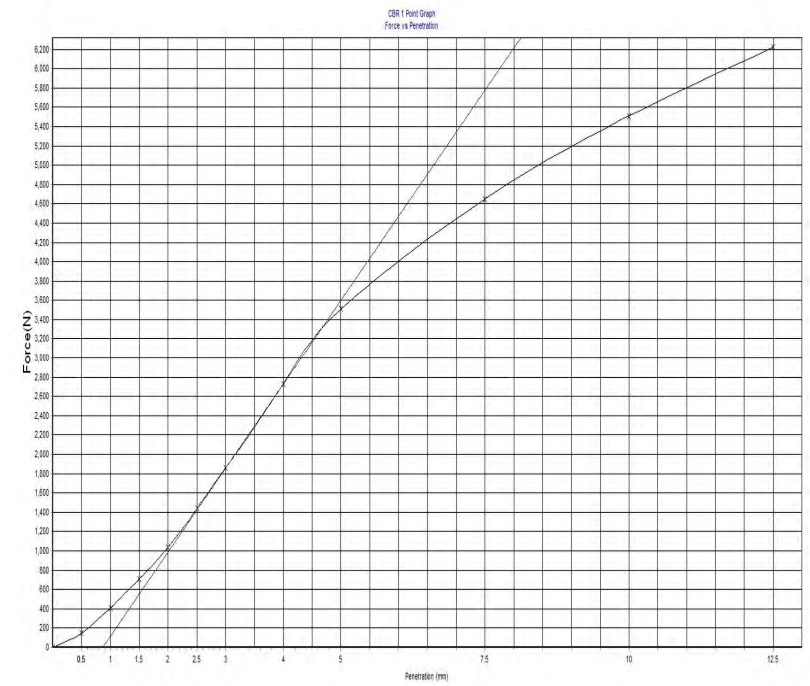
California Bearing Ratio Report (1 Point)

Client :	**Principle Living**	Report Number:	**15442 - 003**
Address :	**PO Box 206, Carrington, NSW, 2294**	Report Date :	**8/06/2021**
Project Number :	**15442**	Order Number :	
Project Name :	**Geotechnical Investigation**	Test Method :	**AS 1289.6.1.1**
Location:	**Newcastle Golf Club Vardon Road , Fern Bay**		

Sample Number :	21-1087	SAMPLE LOCATION
Date Sampled :	17/05/2021	TP1
Date Tested :	25/05/2021	0.6-1.0m
Sampled By :	RCA Geotech	
Sampling Method :	AS SUPPLIED	
Material Source :		Lot Number :
Material Type :		Test Number :
Remarks :		#REF!

Moisture Method :	AS 1289.2.1.1
Maximum Dry Density (t/m³) :	1.608
Optimum Moisture Content (%) :	10.9
Compactive Effort :	Standard
Nominated Percentage of MDD :	100
Nominated Percentage of OMC :	100
Achieved Percentage of MDD :	100
Achieved Percentage of OMC :	101.0
Dry Density Before Soak (t/m³) :	1.607
Dry Density After Soak (t/m³) :	1.607
Moisture Content Before Soak (%) :	11.0
Moisture Content After Soak (%) :	11.4
Density Ratio After Soak (%) :	100
Field Moisture Content (%) :	2.9
Top Moisture Content - After Penetration (%) :	12.1
Total Moisture Content - After Penetration (%) :	11.7
Soak Condition :	Soaked
Soak Period (days) :	4
Swell (%) :	0.0



CBR Surcharge (kg) :	4.5	CBR 2.5mm (%) :	16
Oversize (%) :	0	CBR 5.0mm (%) :	**20**
Oversize Material Replaced (%) :	Excluded	**CBR Value (%) :**	**20**

Site Selection :	
Soil Description :	SAND

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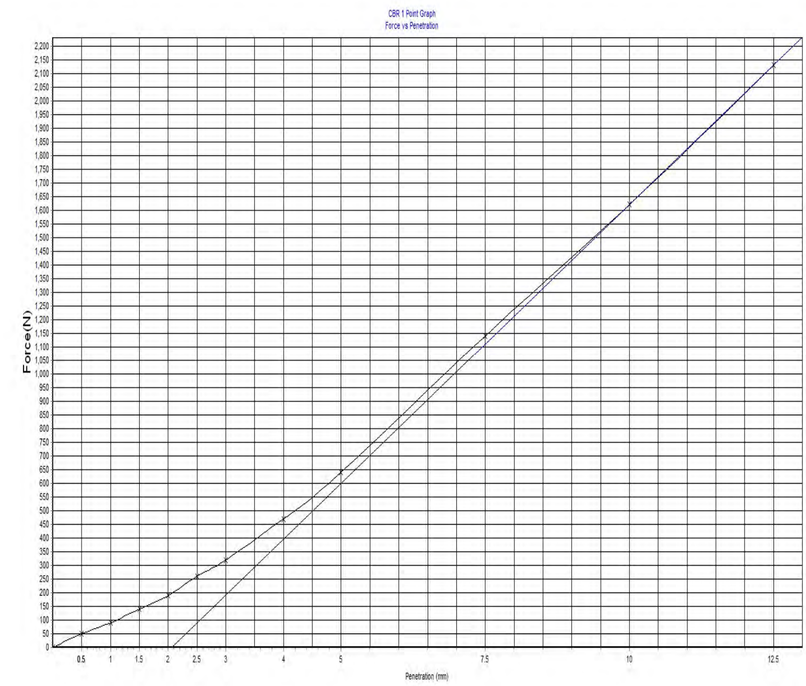
California Bearing Ratio Report (1 Point)

Client :	**Principle Living**	Report Number:	**15442 - 003**
Address :	**PO Box 206, Carrington, NSW, 2294**	Report Date :	**8/06/2021**
Project Number :	**15442**	Order Number :	
Project Name :	**Geotechnical Investigation**	Test Method :	**AS 1289.6.1.1**
Location:	**Newcastle Golf Club Vardon Road , Fern Bay**		

Sample Number :	21-1089	SAMPLE LOCATION
Date Sampled :	17/05/2021	TP4
Date Tested :	25/05/2021	0.3-0.5m
Sampled By :	RCA Geotech	
Sampling Method :	AS SUPPLIED	
Material Source :		Lot Number :
Material Type :		Test Number :
Remarks :		

Moisture Method :	AS 1289.2.1.1
Maximum Dry Density (t/m³) :	1.554
Optimum Moisture Content (%) :	23.1
Compactive Effort :	Standard
Nominated Percentage of MDD :	100
Nominated Percentage of OMC :	100
Achieved Percentage of MDD :	98
Achieved Percentage of OMC :	99.0
Dry Density Before Soak (t/m³) :	1.528
Dry Density After Soak (t/m³) :	1.528
Moisture Content Before Soak (%) :	22.9
Moisture Content After Soak (%) :	23.3
Density Ratio After Soak (%) :	98
Field Moisture Content (%) :	14.4
Top Moisture Content - After Penetration (%) :	23.8
Total Moisture Content - After Penetration (%) :	23.4
Soak Condition :	Soaked
Soak Period (days) :	4
Swell (%) :	0.0



CBR Surcharge (kg) :	4.5	CBR 2.5mm (%) :	4.5
Oversize (%) :	0	CBR 5.0mm (%) :	**5**
Oversize Material Replaced (%) :	Excluded	**CBR Value (%) :**	**5**

Site Selection :	
Soil Description :	Silty SAND

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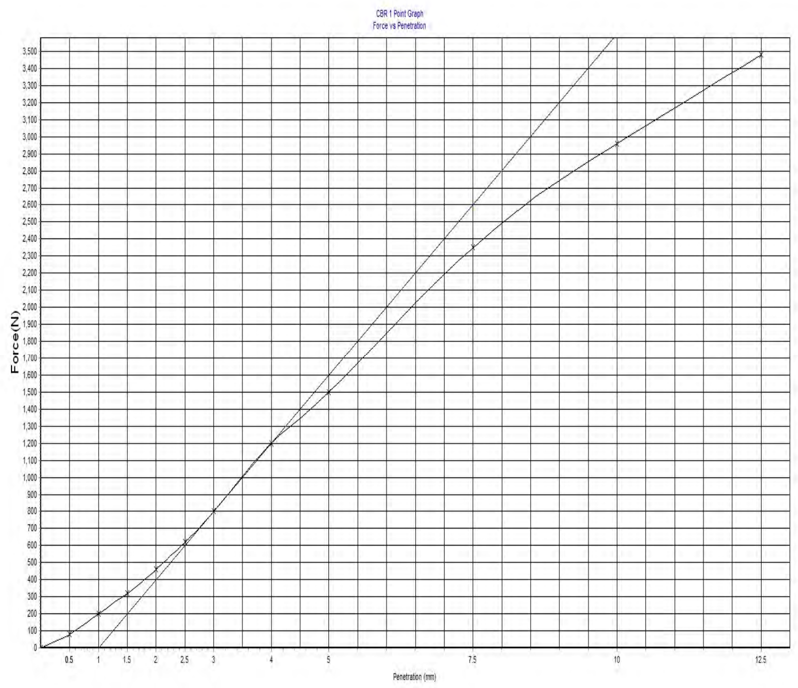
California Bearing Ratio Report (1 Point)

Client :	**Principle Living**	Report Number:	**15442 - 003**
Address :	**PO Box 206, Carrington, NSW, 2294**	Report Date :	**8/06/2021**
Project Number :	**15442**	Order Number :	
Project Name :	**Geotechnical Investigation**	Test Method :	**AS 1289.6.1.1**
Location:	**Newcastle Golf Club Vardon Road , Fern Bay**		**Page 3 of 6**

Sample Number :	21-1090	SAMPLE LOCATION
Date Sampled :	17/05/2021	TP5
Date Tested :	25/05/2021	0.3-0.5m
Sampled By :	RCA Geotech	
Sampling Method :	AS SUPPLIED	
Material Source :		Lot Number :
Material Type :		Test Number :
Remarks :		#REF!

Moisture Method :	AS 1289.2.1.1
Maximum Dry Density (t/m³) :	1.631
Optimum Moisture Content (%) :	18.1
Compactive Effort :	Standard
Nominated Percentage of MDD :	100
Nominated Percentage of OMC :	100
Achieved Percentage of MDD :	100
Achieved Percentage of OMC :	98.0
Dry Density Before Soak (t/m³) :	1.632
Dry Density After Soak (t/m³) :	1.631
Moisture Content Before Soak (%) :	17.8
Moisture Content After Soak (%) :	18.1
Density Ratio After Soak (%) :	100
Field Moisture Content (%) :	12.8
Top Moisture Content - After Penetration (%) :	19.0
Total Moisture Content - After Penetration (%) :	18.3
Soak Condition :	Soaked
Soak Period (days) :	4
Swell (%) :	0.0



CBR Surcharge (kg) :	4.5	CBR 2.5mm (%) :	8
Oversize (%) :	0	CBR 5.0mm (%) :	**9**
Oversize Material Replaced (%) :	Excluded	**CBR Value (%) :**	**9**

Site Selection :	
Soil Description :	Silty SAND

NATA
WORLD RECOGNISED ACCREDITATION

Accredited for compliance with ISO/IEC 17025 - Testing.

APPROVED SIGNATORY

Timothy Baker - Senior Soil Technician
NATA Accreditation Number : 9811

RCA LABORATORIES
• ENVIRONMENTAL •
CONSTRUCTION MATERIALS TESTING

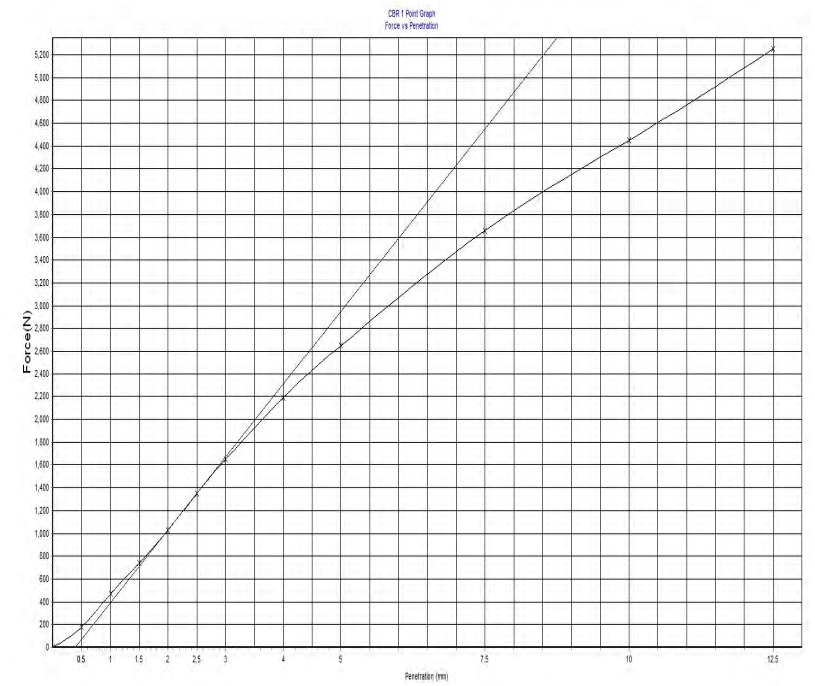
92 Hill St Carrington Newcastle NSW 2294
Ph +61 2 4902 9200
Web www.rca.com.au
ABN 53 063 515 711
NATA Accredited Laboratory: 9811
Corporate Site No: 9804
Construction Materials Testing

California Bearing Ratio Report (1 Point)

Client :	**Principle Living**	Report Number:	**15442 - 003**
Address :	**PO Box 206, Carrington, NSW, 2294**	Report Date :	**8/06/2021**
Project Number :	**15442**	Order Number :	
Project Name :	**Geotechnical Investigation**	Test Method :	**AS 1289.6.1.1**
Location:	**Newcastle Golf Club Vardon Road , Fern Bay**		**Page 4 of 6**

Sample Number :	21-1091	SAMPLE LOCATION
Date Sampled :	17/05/2021	TP6
Date Tested :	25/05/2021	0.2-0.5m
Sampled By :	RCA Geotech	
Sampling Method :	AS SUPPLIED	
Material Source :		Lot Number :
Material Type :		Test Number :
Remarks :		#REF!

Moisture Method :	AS 1289.2.1.1
Maximum Dry Density (t/m³) :	1.672
Optimum Moisture Content (%) :	11.1
Compactive Effort :	Standard
Nominated Percentage of MDD :	100
Nominated Percentage of OMC :	100
Achieved Percentage of MDD :	100
Achieved Percentage of OMC :	100.0
Dry Density Before Soak (t/m³) :	1.667
Dry Density After Soak (t/m³) :	1.667
Moisture Content Before Soak (%) :	11.1
Moisture Content After Soak (%) :	12.5
Density Ratio After Soak (%) :	100
Field Moisture Content (%) :	3.3
Top Moisture Content - After Penetration (%) :	13.6
Total Moisture Content - After Penetration (%) :	12.5
Soak Condition :	Soaked
Soak Period (days) :	4
Swell (%) :	0.0



CBR Surcharge (kg) :	4.5	CBR 2.5mm (%) :	12
Oversize (%) :	0	CBR 5.0mm (%) :	**14**
Oversize Material Replaced (%) :	Excluded	**CBR Value (%) :**	**14**

Site Selection :	
Soil Description :	Silty SAND

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APPROVED SIGNATORY

Timothy Baker - Senior Soil Technician
NATA Accreditation Number : 9811

Document Code RF39-10

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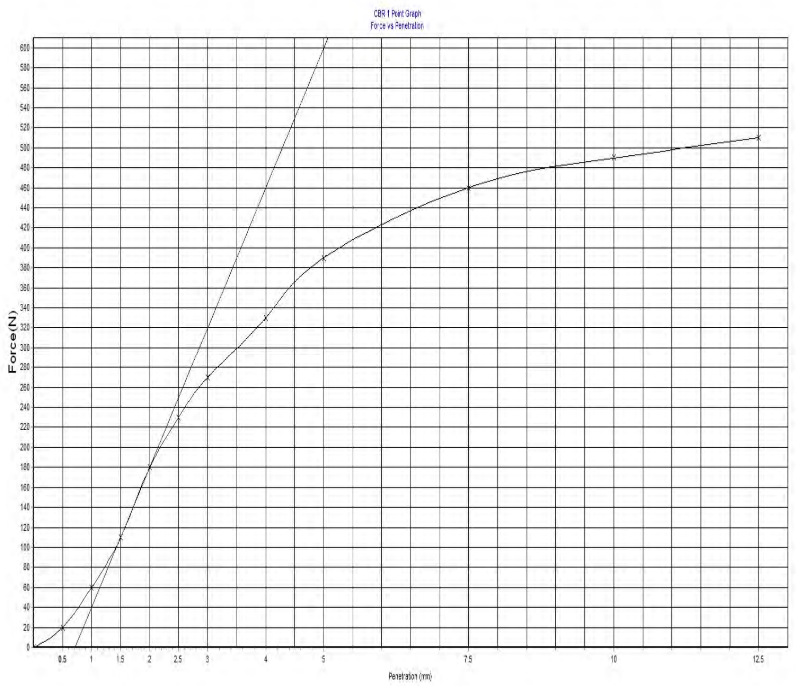
NATA Accredited Laboratory: 9811
Corporate Site No: 9804
Construction Materials Testing

California Bearing Ratio Report (1 Point)

Client :	**Principle Living**	Report Number:	**15442 - 003**
Address :	**PO Box 206, Carrington, NSW, 2294**	Report Date :	**8/06/2021**
Project Number :	**15442**	Order Number :	
Project Name :	**Geotechnical Investigation**	Test Method :	**AS 1289.6.1.1**
Location:	**Newcastle Golf Club Vardon Road , Fern Bay**		

Sample Number :	21-1093	SAMPLE LOCATION
Date Sampled :	17/05/2021	TP9
Date Tested :	7/06/2021	0.5-0.7m
Sampled By :	RCA Geotech	
Sampling Method :	AS SUPPLIED	
Material Source :		Lot Number :
Material Type :		Test Number :
Remarks :		#REF!

Moisture Method :	AS 1289.2.1.1
Maximum Dry Density (t/m³) :	1.059
Optimum Moisture Content (%) :	62.5
Compactive Effort :	Standard
Nominated Percentage of MDD :	100
Nominated Percentage of OMC :	100
Achieved Percentage of MDD :	100
Achieved Percentage of OMC :	100.0
Dry Density Before Soak (t/m³) :	1.058
Dry Density After Soak (t/m³) :	1.028
Moisture Content Before Soak (%) :	62.4
Moisture Content After Soak (%) :	66.2
Density Ratio After Soak (%) :	97
Field Moisture Content (%) :	101.0
Top Moisture Content - After Penetration (%) :	68.3
Total Moisture Content - After Penetration (%) :	65.5
Soak Condition :	Soaked
Soak Period (days) :	4
Swell (%) :	3.0



CBR Surcharge (kg) :	4.5	CBR 2.5mm (%) :	2
Oversize (%) :	0	CBR 5.0mm (%) :	**2**
Oversize Material Replaced (%) :	Excluded	**CBR Value (%) :**	**2**

Site Selection :	
Soil Description :	PEAT

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APPROVED SIGNATORY

Timothy Baker - Senior Soil Technician
NATA Accreditation Number :
9811

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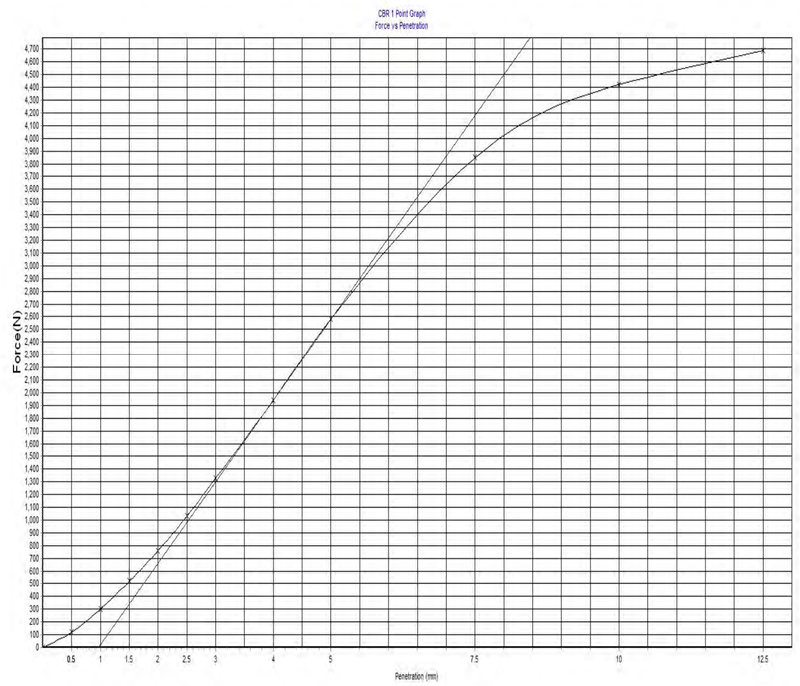
Corporate Site No: 9804
Construction Materials Testing

California Bearing Ratio Report (1 Point)

Client :	**Principle Living**	Report Number:	**15442 - 003**
Address :	**PO Box 206, Carrington, NSW, 2294**	Report Date :	**8/06/2021**
Project Number :	**15442**	Order Number :	
Project Name :	**Geotechnical Investigation**	Test Method :	**AS 1289.6.1.1**
Location:	**Newcastle Golf Club Vardon Road , Fern Bay**		

Sample Number :	21-1094	SAMPLE LOCATION
Date Sampled :	17/05/2021	TP11
Date Tested :	25/05/2021	0.3-0.5m
Sampled By :	RCA Geotech	
Sampling Method :	AS SUPPLIED	
Material Source :		Lot Number :
Material Type :		Test Number :
Remarks :		

Moisture Method :	AS 1289.2.1.1
Maximum Dry Density (t/m³) :	1.632
Optimum Moisture Content (%) :	9.6
Compactive Effort :	Standard
Nominated Percentage of MDD :	100
Nominated Percentage of OMC :	100
Achieved Percentage of MDD :	100
Achieved Percentage of OMC :	101.0
Dry Density Before Soak (t/m³) :	1.635
Dry Density After Soak (t/m³) :	1.633
Moisture Content Before Soak (%) :	9.7
Moisture Content After Soak (%) :	10.5
Density Ratio After Soak (%) :	100
Field Moisture Content (%) :	6.8
Top Moisture Content - After Penetration (%) :	11.3
Total Moisture Content - After Penetration (%) :	10.6
Soak Condition :	Soaked
Soak Period (days) :	4
Swell (%) :	0.0



CBR Surcharge (kg) :	4.5	CBR 2.5mm (%) :	12
Oversize (%) :	0	CBR 5.0mm (%) :	**16**
Oversize Material Replaced (%) :	Excluded	**CBR Value (%) :**	**16**

Site Selection :	
Soil Description :	SAND

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APPROVED SIGNATORY

Timothy Baker - Senior Soil Technician
NATA Accreditation Number :
9811

Appendix C2

Soil and Groundwater Aggressivity Laboratory Test Results

CERTIFICATE OF ANALYSIS

Work Order	: **ES2118460**	Page	: 1 of 4
Client	: **ROBERT CARR & ASSOCIATES P/L**	Laboratory	: Environmental Division Sydney
Contact	: MR ROBERT CATER	Contact	: Grace White
Address	: PO BOX 175 CARRINGTON NSW, AUSTRALIA 2294	Address	: 277-289 Woodpark Road Smithfield NSW Australia 2164
Telephone	: +61 02 49029200	Telephone	: +61 2 8784 8555
Project	: 15442 Geotech Investigation	Date Samples Received	: 17-May-2021 16:34
Order number	: 15442	Date Analysis Commenced	: 17-May-2021
C-O-C number	: ----	Issue Date	: 24-May-2021 12:57
Sampler	: Rob Cater		
Site	: ----		
Quote number	: SYBQ/400/18		
No. of samples received	: 8		
No. of samples analysed	: 8		

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

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

This Certificate of Analysis contains the following information:

- General Comments
- Analytical Results

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

Signatories

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

Signatories	*Position*	*Accreditation Category*
Ankit Joshi	Inorganic Chemist	Sydney Inorganics, Smithfield, NSW
Celine Conceicao	Senior Spectroscopist	Sydney Inorganics, Smithfield, NSW
Neil Martin	Team Leader - Chemistry	Chemistry, Newcastle West, NSW

General Comments

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

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

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

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

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

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

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

Analytical Results

Sub-Matrix: SOIL (Matrix: SOIL)			Sample ID	TP3 1.3-1.5	TP4 2.2-2.3	TP6 0.7-0.8	TP9 0.2-0.4	TP11 1.0-1.1
			Sampling date / time	17-May-2021 00:00	17-May-2021 00:00	17-May-2021 00:00	17-May-2021 00:00	17-May-2021 00:00
Compound	*CAS Number*	*LOR*	*Unit*	ES2118460-001	ES2118460-002	ES2118460-003	ES2118460-006	ES2118460-008
				Result	Result	Result	Result	Result
EA002: pH 1:5 (Soils)								
pH Value	----	0.1	pH Unit	**9.1**	**6.6**	**7.6**	**6.5**	**6.6**
EA010: Conductivity (1:5)								
Electrical Conductivity @ 25°C	----	1	µS/cm	**42**	**133**	**22**	**96**	**9**
EA055: Moisture Content (Dried @ 105-110°C)								
Moisture Content	----	1.0	%	**3.3**	**74.0**	**3.5**	**27.8**	**6.2**
ED040S : Soluble Sulfate by ICPAES								
Sulfate as SO4 2-	14808-79-8	10	mg/kg	<10	**530**	<10	**10**	<10
ED045G: Chloride by Discrete Analyser								
Chloride	16887-00-6	10	mg/kg	<10	**170**	<10	**160**	<10

Analytical Results

Sub-Matrix: WATER (Matrix: WATER)			Sample ID	TP8	TP9	TP10	----	----
			Sampling date / time	17-May-2021 00:00	17-May-2021 00:00	17-May-2021 00:00	----	----
Compound	CAS Number	LOR	Unit	ES2118460-004	ES2118460-005	ES2118460-007	--------	--------
				Result	Result	Result	----	----
EA005: pH								
pH Value	----	0.01	pH Unit	**6.53**	**6.20**	**5.64**	----	----
EA010P: Conductivity by PC Titrator								
Electrical Conductivity @ 25°C	----	1	µS/cm	**988**	**790**	**1330**	----	----
ED041G: Sulfate (Turbidimetric) as SO4 2- by DA								
Sulfate as SO4 - Turbidimetric	14808-79-8	1	mg/L	<1	<1	<1	----	----
ED045G: Chloride by Discrete Analyser								
Chloride	16887-00-6	1	mg/L	**220**	**210**	**404**	----	----

Inter-Laboratory Testing

Analysis conducted by ALS Newcastle - Water, NATA accreditation no. 825, site no. 1656 (Chemistry) 9854 (Biology).

(WATER) EA005: pH

Appendix C3

Acid Sulfate Soils Laboratory Test Results

RCA Australia Trading as RCA Laboratories – Environmental
92 Hill Street PO Box 175, Carrington NSW 2294
ABN 53 063 515 711 Ph 02 4902 9200
Email: administrator@rca.com.au Web www.rca.com.au

NATA Accredited Laboratory 9811
Corporate Site Number 18077
Accredited for compliance with ISO/IEC 17025 - Testing

Robert Carr & Associates
92 Hill Street
Carrington NSW 2287

Attention: Robert Cater

Project:	RCA ref 15442-701/0		
Date:	18/05/2021		
Client reference:	Newcastle Golf Club		
Received date:	18/05/2021	**Number of samples:**	20
Client order number:	N/A	**Testing commenced:**	18/05/2021

CERTIFICATE OF ANALYSIS

1 ANALYTICAL TEST METHODS

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ANALYSIS	METHOD	UNITS	ANALYSING LABORATORY	NATA ANALYSIS / NON NATA	Measurement of Uncertainty Coverage Factor 2
Acid Sulfate Soil	ENV-LAB032	pH	RCA Laboratories - Environmental	NATA	±0.54

***** The analytical procedures used by RCA Laboratories - Environmental are based on established internationally recognised procedures such as APHA and Australian Standards

****** Indicates NATA accreditation does not cover the performance of this service

2 RESULTS

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ANALYSIS	UNITS	TP1 0.8-1.0	TP2, 0.8-1.0	TP3, 1.3-1.5	TP3, 2.0-2.2	TP4 1.2-1.3
Acid Sulfate Soil Screening Test						
Sample Number	-	052115442001	052115442002	052115442003	052115442004	052115442005
Date Sampled	-	17/05/2021	17/05/2021	17/05/2021	17/05/2021	17/05/2021
pHF	pH unit	5.99	5.98	6.11	6.75	6.77
pHFOX		4.45	5.09	5.11	5.79	6.13
pHF – pHFOX		1.54	0.89	1.00	0.96	0.64
Reaction Rate^	-	1	1	1	1	1
Soil Type	-	Not Supplied	Not Supplied	Not Supplied	Not Supplied	Not Supplied

ANALYSIS	UNITS	TP5, 0.3-0.5	TP5, 1.3-1.5	TP6, 0.3-0..5	TP6, 1.9-2.0	TP7, 1.4-1.5
Acid Sulfate Soil Screening Test						
Sample Number	-	052115442006	052115442007	052115442008	052115442009	052115442010
Date Sampled	-	17/05/2021	17/05/2021	17/05/2021	17/05/2021	17/05/2021
pHF	pH unit	5.54	5.74	6.05	6.25	6.18
pHFOX		3.49	1.45	5.18	5.50	4.55
pHF – pHFOX		2.05	4.29	0.87	0.75	1.63
Reaction Rate^	-	1	2	1	1	1
Soil Type	-	Not Supplied	Not Supplied	Not Supplied	Not Supplied	Not Supplied

ANALYSIS	UNITS	TP7, 1.8-2.0	TP8, 0.4-0.5	TP8, 1.3-1.5	TP9 0.5-0.7	TP9, 1.2-1.4
Acid Sulfate Soil Screening Test						
Sample Number	-	052115442011	052115442012	052115442013	052115442014	052115442015
Date Sampled	-	17/05/2021	17/05/2021	17/05/2021	17/05/2021	17/05/2021
pHF	pH unit	6.48	7.65	6.99	6.26	6.34
pHFOX		1.95	5.63	1.76	4.22	1.66
pHF – pHFOX		4.53	2.02	5.23	2.04	4.68
Reaction Rate^	-	4	2	4	3	4
Soil Type	-	Not Supplied	Not Supplied	Not Supplied	Not Supplied	Not Supplied

ANALYSIS	UNITS	TP10, 0.1-0.2	TP10, 0.8-1.0	TP10, 1.1-1.3	TP11, 0.3-0.4	TP11, 1.0-1.2
Acid Sulfate Soil Screening Test						
Sample Number	-	052115442016	052115442017	052115442018	052115442019	052115442020
Date Sampled	-	17/05/2021	17/05/2021	17/05/2021	17/05/2021	17/05/2021
pHF	pH unit	4.04	5.61	5.09	6.22	6.42
pHFOX		2.60	1.56	1.58	5.42	5.64
pHF – pHFOX		1.44	4.05	3.51	0.80	0.78
Reaction Rate^	-	2	4	4	1	1
Soil Type	-	Not Supplied	Not Supplied	Not Supplied	Not Supplied	Not Supplied

** Indicates NATA accreditation does not cover the performance of this service

Acid Sulfate Soil Screening

Note: This screening test only provides an indication of the likely presence and severity of Acid Sulfate Soils. This test should not be used as a substitute for laboratory analysis which would positively identify the presence of Acid Sulfate Soils (ASS) for assessment purposes.

NATA Scope of Accreditation does not cover the sampling of soils by the client or by RCA Employee's.

Analysis for pH and Acid Sulfate Screen Testing is covered by RCA Laboratories - Environmental NATA Scope of Accreditation.

Analysis on samples is on an as received basis.

Acid Soil Screening Test Reaction Rate

^Reaction Rate: 1 = Slight, 2 = Moderate, 3 = High, 4 = Very Vigorous

Note: Due to the subjectivity the assessment of the Reaction Rate is not covered by our NATA Scope of Accreditation.

3 QUALITY CONTROL RESULTS

Acid Sulfate Soil Screening Test Quality Control

DATE	ANALYSIS	METHOD	UNITS	QUALITY CONTROL STANDARD VALUE	QUALITY CONTROL ACCEPTANCE CRITERIA	QUALITY CONTROL STANDARD RESULT
18/05/2021	pH – Acid Sulfate Soil	ENV-LAB032	pH	7.00	6.95 - 7.05	7.01

Acid Sulfate Soil Screening Test Duplicate Analysis

SAMPLE NUMBER	DATE	ANALYSIS	METHOD	UNITS	LOR	SAMPLE RESULT	SAMPLE DUPLICATE RESULT
052115442001	18/05/2021	pH – Acid Sulfate Soil	ENV-LAB032	pH	N/A	5.99	5.94
052115442011	18/05/2021	pH – Acid Sulfate Soil	ENV-LAB032	pH	N/A	6.48	6.51

Please contact the undersigned if you have any queries.

Yours sincerely

Laura Schofield
Environmental Laboratory Manager
Robert Carr & Associates Pty Ltd Trading as
RCA Laboratories – Environmental
Approved Signatory Approved

Neena Tewari
Senior Environmental Microbiologist
Robert Carr & Associates Pty Ltd Trading as
RCA Laboratories - Environmental

RCA Internal Quality Review

General

1. Laboratory QC results for Method Blanks, Duplicates and Laboratory Control Samples are included in this QC report where applicable. Additional QC data maybe available on request.
2. RCA QC Acceptance / Rejection Criteria are available on request.
3. Proficiency Trial results are available on request.
4. Actual PQLs are matrix dependant. Quoted PQLs may be raised where sample extracts are diluted due to interferences.
5. When individual results are qualified in the body of a report, refer to the qualifier descriptions that follow.
6. Samples were analysed on an 'as received' basis.
7. Sampled dates in this report are those listed on the COC or sample jars; if no sample dates are noted, the date the samples are received at the laboratory have been used.
8. All soil results are reported on a dry basis, unless otherwise stated. (ACID SULFATE SOILS)
9. This report replaces any interim results previously issued.

Holding Times.

For samples received on the last day of holding time, notification of testing requirements should have been received at least 6 hours prior to sample receipt deadlines as stated on the Sample

Receipt Acknowledgment.

If the Laboratory did not receive the information in the required timeframe, and regardless of any other integrity issues, suitably qualified results may still be reported.

Holding times apply from the date of sampling, therefore compliance to these may be outside the laboratory's control.

##NOTE: pH duplicates are reported as a range NOT as RPD

QC - ACCEPTANCE CRITERIA

RPD Duplicates: Global RPD Duplicates Acceptance Criteria is 30% however the following acceptance guidelines are equally applicable:

Results <10 times the LOR: No Limit

Results between 10-20 times the LOR: RPD must lie between 0-50%

Results >20 times the LOR: RPD must lie between 0-30%

QC DATA GENERAL COMMENTS

1. Where a result is reported as a less than (<), higher than the nominated LOR, this is due to either matrix interference, extract dilution required due to interferences or contaminant levels within the sample, high moisture content or insufficient sample provided.

2. Duplicate data shown within this report that states the word "BATCH" is a Batch Duplicate from outside of your sample batch, but within the laboratory sample batch at a 1:10 ratio. The Parent and Duplicate data shown is not data from your samples.

3. Duplicate RPD's are calculated from raw analytical data thus it is possible to have two sets of data.

Glossary

UNITS

mg/kg: milligrams per Kilogram

ug/L: micrograms per litre

ppm: Parts per million

ppb: Parts per billion

%: Percentage

org/100ml: Organisms per 100 millilitres

NTU: Units

MPN/100mL: Most Probable Number of organisms per 100 millilitres

mg/L: milligrams per Litre

TERMS

Dry Where moisture has been determined on a solid sample the result is expressed on a dry basis.

LOR Limit of Reporting.

RPD Relative Percent Difference between two Duplicate pieces of analysis can be obtained upon request.

QCS Quality Control Sample - reported as value recovery

Method Blank In the case of solid samples these are performed on laboratory certified clean sands.

In the case of water samples these are performed on de-ionised water.

Duplicate A second piece of analysis from the same sample and reported in the same units as the result to show comparison.

Batch Duplicate A second piece of analysis from a sample outside of the clients batch of samples but run within the laboratory batch of analysis.

USEPA United States Environment Protection Authority

APHA American Public Health Association

COC Chain of Custody

CP Client Parent - QC was performed on samples pertaining to this report

NCP Non-Client Parent - QC performed on samples not pertaining to this report, QC is representative of the sequence or batch that client samples were analysed within

< indicates less than

> Indicates greater than

ND Not Detected

CERTIFICATE OF ANALYSIS

Work Order	: **EB2114672**	Page	: 1 of 4
Client	: **ROBERT CARR & ASSOCIATES P/L**	Laboratory	: Environmental Division Brisbane
Contact	: MR ROBERT CATER	Contact	: Customer Services EB
Address	: PO BOX 175 CARRINGTON NSW, AUSTRALIA 2294	Address	: 2 Byth Street Stafford QLD Australia 4053
Telephone	: +61 02 49029200	Telephone	: +61-7-3243 7222
Project	: 15442 Geotechnical Investigation	Date Samples Received	: 27-May-2021 08:30
Order number	: ----	Date Analysis Commenced	: 02-Jun-2021
C-O-C number	: ----	Issue Date	: 03-Jun-2021 12:55
Sampler	: ROBERT CATER		
Site	: ----		
Quote number	: SYBQ/400/18		
No. of samples received	: 4		
No. of samples analysed	: 4		

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ISO/IEC 17025 - Testing

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

This Certificate of Analysis contains the following information:

- General Comments
- Analytical Results

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

Signatories

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

Signatories	*Position*	*Accreditation Category*
Ben Felgendrejeris	Senior Acid Sulfate Soil Chemist	Brisbane Acid Sulphate Soils, Stafford, QLD

General Comments

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

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

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

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

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

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

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

- ASS: EA029 (SPOCAS): Retained Acidity not required because pH KCl greater than or equal to 4.5
- ASS: EA029 (SPOCAS): Excess ANC not required because pH OX less than 6.5.
- ASS: EA029 (SPOCAS): Liming rate is calculated and reported on a dry weight basis assuming use of fine agricultural lime ($CaCO_3$) and using a safety factor of 1.5 to allow for non-homogeneous mixing and poor reactivity of lime. For conversion of Liming Rate from kg/t dry weight to kg/m3 in-situ soil, multiply reported results x wet bulk density of soil in t/m3.

Analytical Results

Sub-Matrix: SOIL (Matrix: SOIL)			Sample ID	TP5, -1.3-1.5	TP7, 1.8-2.0	TP8, 1.3-1.5	TP10, 0.8-1.0	----
			Sampling date / time	17-May-2021 00:00	17-May-2021 00:00	17-May-2021 00:00	17-May-2021 00:00	----
Compound	CAS Number	LOR	Unit	EB2114672-001	EB2114672-002	EB2114672-003	EB2114672-004	--------
				Result	Result	Result	Result	----
EA026 : Chromium Reducible Sulfur								
Chromium Reducible Sulphur	----	0.005	%	**0.088**	**0.046**	**0.249**	**0.155**	----
EA029-A: pH Measurements								
pH KCl (23A)	----	0.1	pH Unit	**6.0**	**6.3**	**5.3**	**5.0**	----
pH OX (23B)	----	0.1	pH Unit	**2.9**	**3.2**	**2.4**	**2.7**	----
EA029-B: Acidity Trail								
Titratable Actual Acidity (23F)	----	2	mole H+ / t	**3**	<2	**5**	**10**	----
Titratable Peroxide Acidity (23G)	----	2	mole H+ / t	**60**	**24**	**93**	**80**	----
Titratable Sulfidic Acidity (23H)	----	2	mole H+ / t	**58**	**24**	**88**	**70**	----
sulfidic - Titratable Actual Acidity (s-23F)	----	0.020	% pyrite S	<0.020	<0.020	<0.020	<0.020	----
sulfidic - Titratable Peroxide Acidity (s-23G)	----	0.020	% pyrite S	**0.097**	**0.039**	**0.149**	**0.128**	----
sulfidic - Titratable Sulfidic Acidity (s-23H)	----	0.020	% pyrite S	**0.092**	**0.039**	**0.141**	**0.112**	----
EA029-C: Sulfur Trail								
KCl Extractable Sulfur (23Ce)	----	0.020	% S	<0.020	<0.020	<0.020	<0.020	----
Peroxide Sulfur (23De)	----	0.020	% S	**0.127**	**0.061**	**0.333**	**0.264**	----
Peroxide Oxidisable Sulfur (23E)	----	0.020	% S	**0.127**	**0.061**	**0.333**	**0.264**	----
acidity - Peroxide Oxidisable Sulfur (a-23E)	----	10	mole H+ / t	**79**	**38**	**208**	**164**	----
EA029-D: Calcium Values								
KCl Extractable Calcium (23Vh)	----	0.020	% Ca	**0.027**	<0.020	<0.020	**0.021**	----
Peroxide Calcium (23Wh)	----	0.020	% Ca	**0.027**	<0.020	<0.020	**0.022**	----
Acid Reacted Calcium (23X)	----	0.020	% Ca	<0.020	<0.020	<0.020	<0.020	----
acidity - Acid Reacted Calcium (a-23X)	----	10	mole H+ / t	<10	<10	<10	<10	----
sulfidic - Acid Reacted Calcium (s-23X)	----	0.020	% S	<0.020	<0.020	<0.020	<0.020	----
EA029-E: Magnesium Values								
KCl Extractable Magnesium (23Sm)	----	0.020	% Mg	<0.020	<0.020	<0.020	<0.020	----
Peroxide Magnesium (23Tm)	----	0.020	% Mg	<0.020	<0.020	<0.020	<0.020	----
Acid Reacted Magnesium (23U)	----	0.020	% Mg	<0.020	<0.020	<0.020	<0.020	----
Acidity - Acid Reacted Magnesium (a-23U)	----	10	mole H+ / t	<10	<10	<10	<10	----
sulfidic - Acid Reacted Magnesium (s-23U)	----	0.020	% S	<0.020	<0.020	<0.020	<0.020	----
EA029-H: Acid Base Accounting								
ANC Fineness Factor	----	0.5	-	**1.5**	**1.5**	**1.5**	**1.5**	----
Net Acidity (sulfur units)	----	0.02	% S	**0.13**	**0.06**	**0.34**	**0.28**	----

Analytical Results

Sub-Matrix: SOIL (Matrix: SOIL)			Sample ID	TP5, -1.3-1.5	TP7, 1.8-2.0	TP8, 1.3-1.5	TP10, 0.8-1.0	----
			Sampling date / time	17-May-2021 00:00	17-May-2021 00:00	17-May-2021 00:00	17-May-2021 00:00	----
Compound	CAS Number	LOR	Unit	EB2114672-001	EB2114672-002	EB2114672-003	EB2114672-004	--------
				Result	Result	Result	Result	----
EA029-H: Acid Base Accounting - Continued								
Net Acidity (acidity units)	----	10	mole H+ / t	**82**	**38**	**212**	**174**	----
Liming Rate	----	1	kg CaCO3/t	**6**	**3**	**16**	**13**	----
Net Acidity excluding ANC (sulfur units)	----	0.02	% S	**0.13**	**0.06**	**0.34**	**0.28**	----
Net Acidity excluding ANC (acidity units)	----	10	mole H+ / t	**82**	**38**	**212**	**174**	----
Liming Rate excluding ANC	----	1	kg CaCO3/t	**6**	**3**	**16**	**13**	----

Appendix D

CSIRO Information Sheet BTF 18

Foundation Maintenance and Footing Performance: A Homeowner's Guide

CSIRO

BTF 18 replaces Information Sheet 10/91

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

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

Soil Types

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

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

Causes of Movement

Settlement due to construction

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

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

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

Erosion

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

Saturation

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

Seasonal swelling and shrinkage of soil

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

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

Shear failure

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

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

GENERAL DEFINITIONS OF SITE CLASSES

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

Tree root growth

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

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

Unevenness of Movement

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

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

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

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

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

Effects of Uneven Soil Movement on Structures

Erosion and saturation

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

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

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

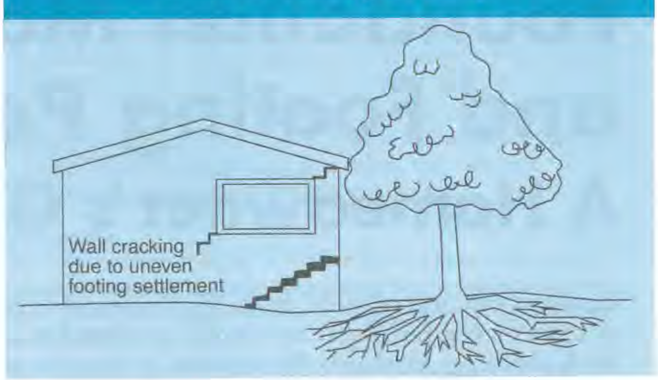
Seasonal swelling/shrinkage in clay

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

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

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

Trees can cause shrinkage and damage



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

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

Movement caused by tree roots

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

Complications caused by the structure itself

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

Effects on full masonry structures

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

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

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

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

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

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

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

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

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

Water Service and Drainage

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

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

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

Seriousness of Cracking

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

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

Prevention/Cure

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

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

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

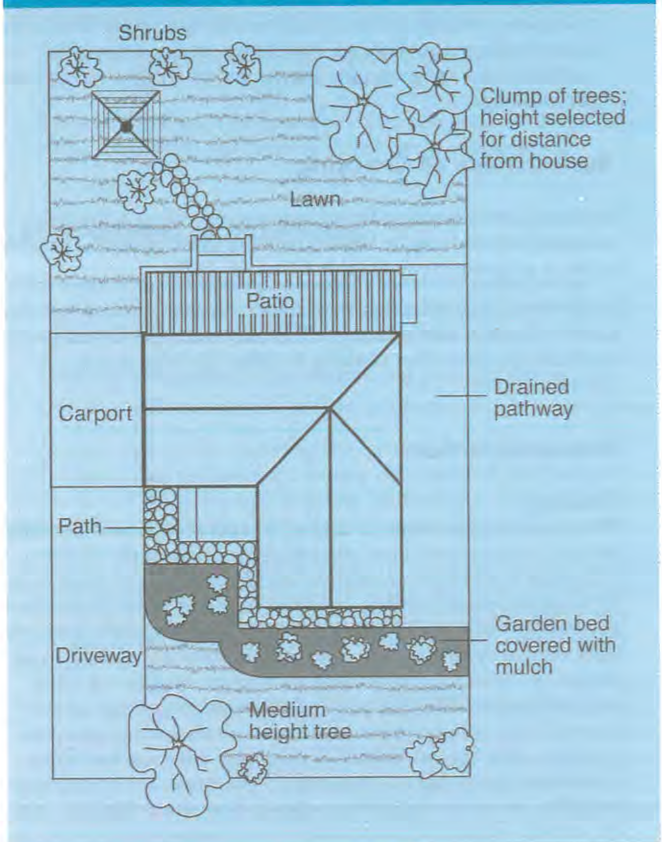
Protection of the building perimeter
It is essential to remember that the soil that affects footings extends well beyond the actual building line. Watering of garden plants, shrubs and trees causes some of the most serious water problems.

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

CLASSIFICATION OF DAMAGE WITH REFERENCE TO WALLS

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

Gardens for a reactive site



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

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

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

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

Condensation

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

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

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

The garden

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

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

Existing trees

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

Information on trees, plants and shrubs

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

Excavation

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

Remediation

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

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

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

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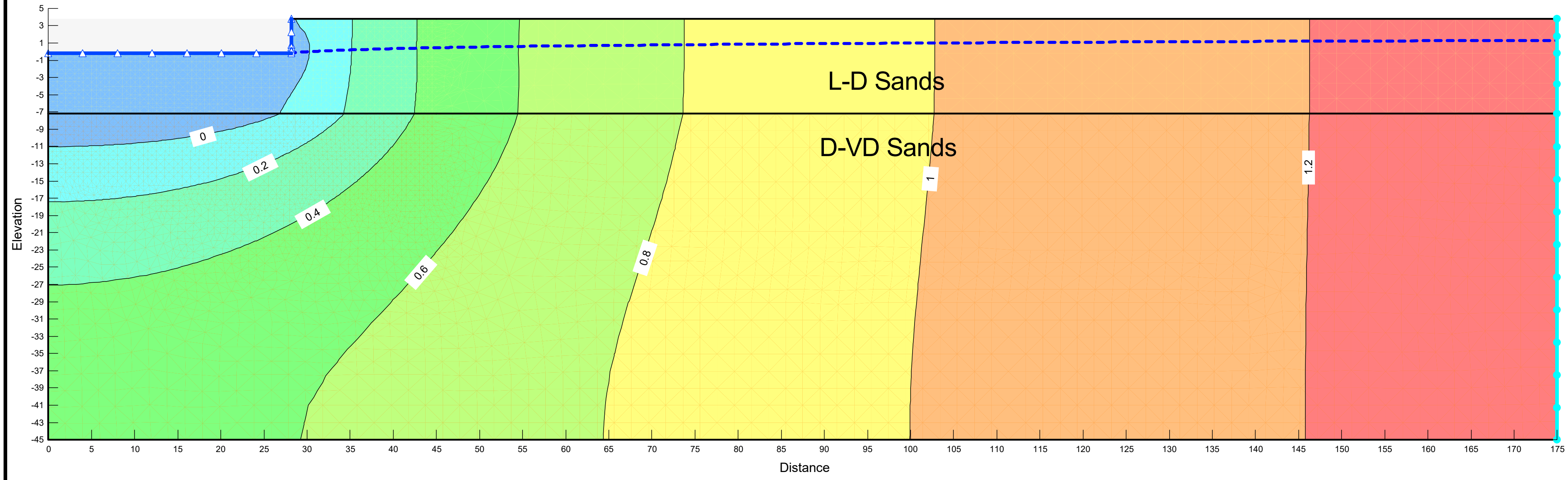
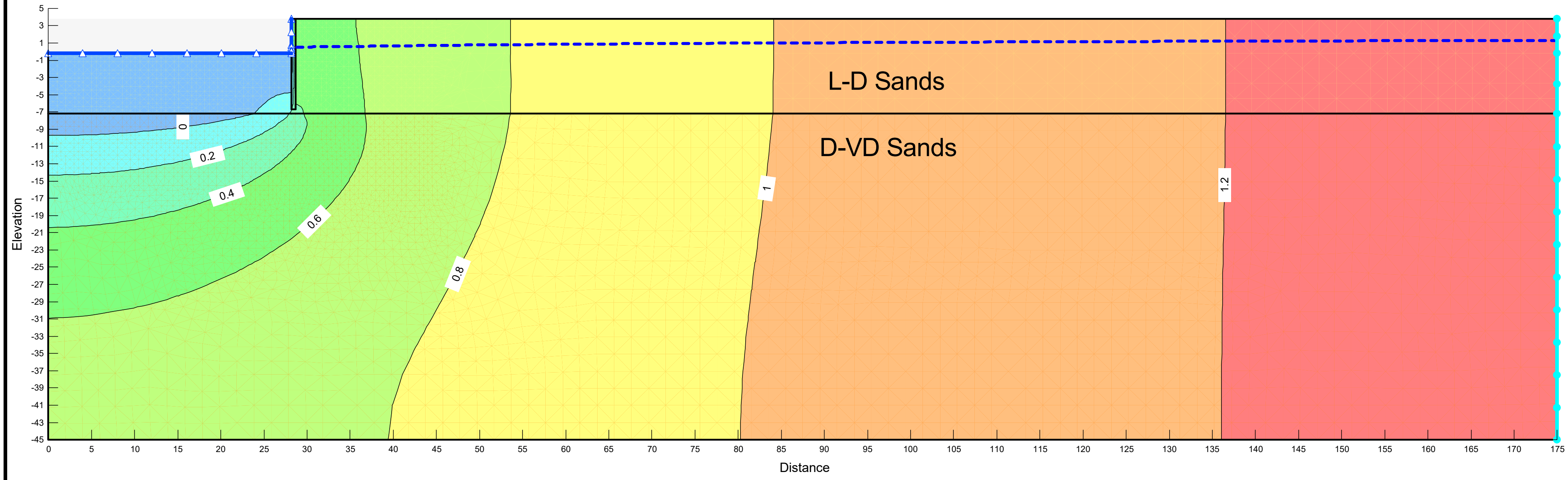
APPENDIX D: SEEP/W ANALYSIS OUTPUTS



Color	Name	Hydraulic Material Model	Vol. WC. Function	K-Function	Sat Kx (m/sec)	Ky'/Kx' Ratio	Rotation (°)	Volumetric Water Content	Compressibility (/kPa)
	D-VD Sands	Saturated Only			0.0001	0.5	0	0.4	1e-05
	L-D Sands	Saturated / Unsaturated	L-D Sands VWC	L-D Sands HC		1	0		

Color	Name	Category	Kind	Parameters
	Hunter River 1.3mAHD	Hydraulic	Water Total Head	1.3 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec

Apartment 1 - No cut-off wall - 1.3m AHD Ground Water Level	
Inflow Analysis_0.5m BEL DW.gsz	
08/11/2024	1:700



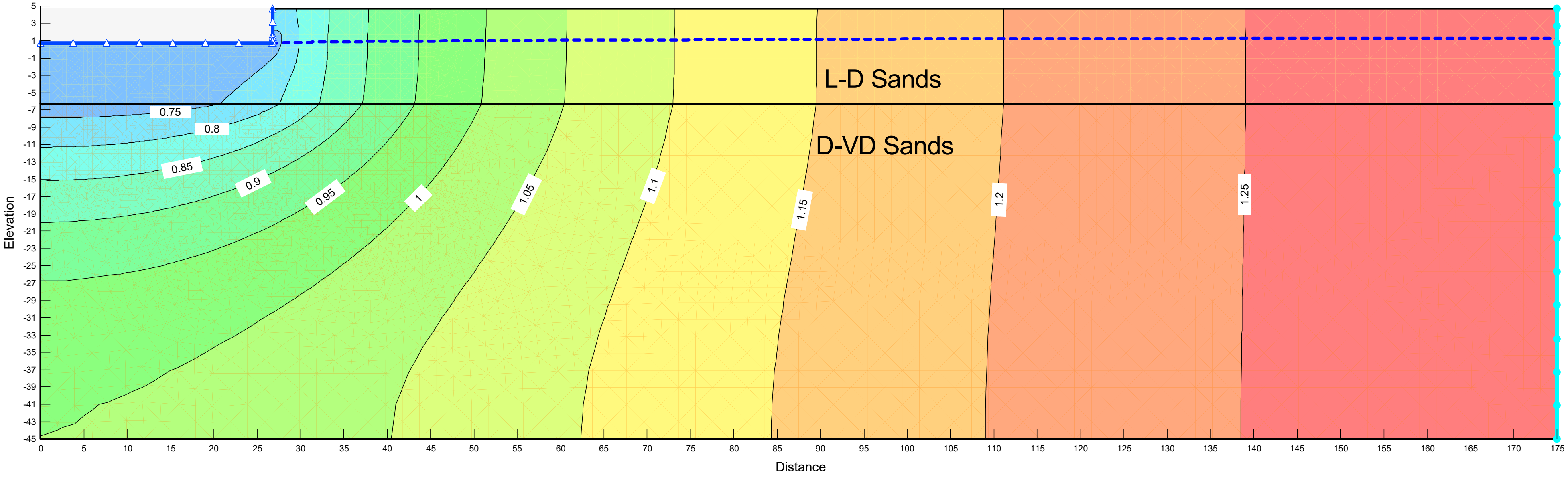
Color	Name	Hydraulic Material Model	Vol. WC. Function	K-Function	Sat Kx (m/sec)	Ky'/Kx' Ratio	Rotation (°)	Volumetric Water Content	Compressibility (/kPa)
	Cutoff Wall	Saturated Only			1e-08	1	0	0	0
	D-VD Sands	Saturated Only			0.0001	0.5	0	0.4	1e-05
	L-D Sands	Saturated / Unsaturated	L-D Sands VWC	L-D Sands HC		1	0		

Color	Name	Category	Kind	Parameters
	Hunter River 1.3mAHD	Hydraulic	Water Total Head	1.3 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec

Apartment 1 - Standard cut-off wall - 1.3m AHD Ground Water Level

Inflow Analysis_0.5m BEL DW.gsz

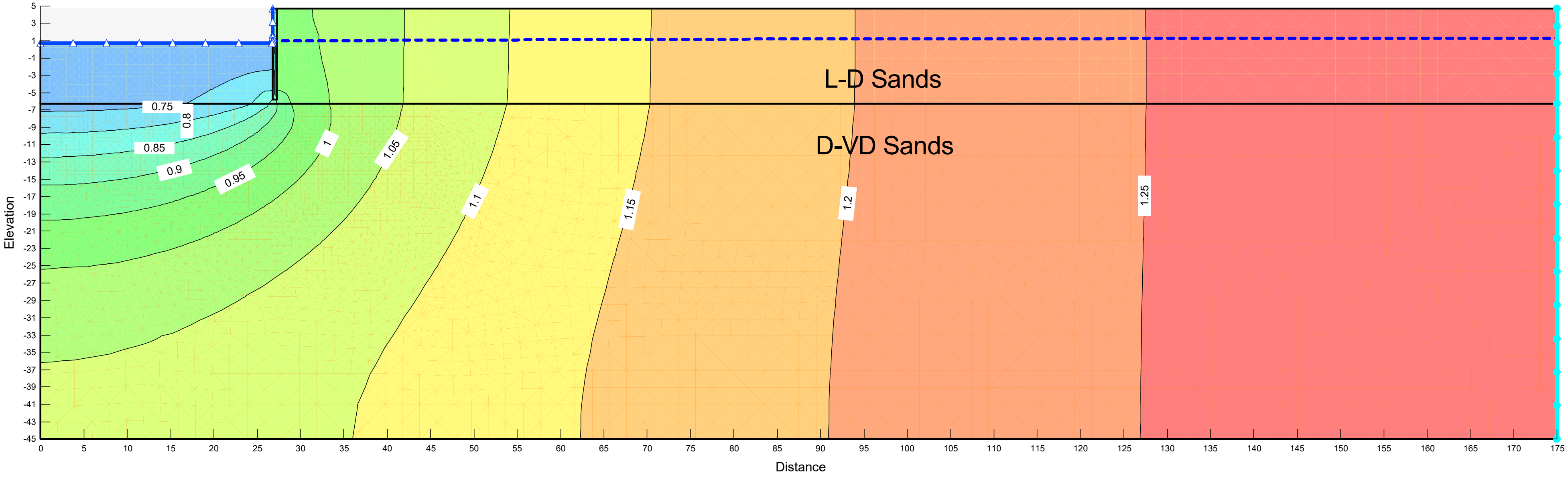
08/11/2024 1:700



Color	Name	Hydraulic Material Model	Vol. WC. Function	K-Function	Sat Kx (m/sec)	Ky'/Kx' Ratio	Rotation (°)	Volumetric Water Content	Compressibility (/kPa)
	D-VD Sands	Saturated Only			0.0001	0.5	0	0.4	1e-05
	L-D Sands	Saturated / Unsaturated	L-D Sands VWC	L-D Sands HC		1	0		

Color	Name	Category	Kind	Parameters
	Hunter River 1.3mAHD	Hydraulic	Water Total Head	1.3 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec

Apartment 2 - No cut-off wall - 1.3m AHD Ground Water Level
Inflow Analysis_0.5m BEL DW.gsz
08/11/2024   1:700



Color	Name	Hydraulic Material Model	Vol. WC. Function	K-Function	Sat Kx (m/sec)	Ky'/Kx' Ratio	Rotation (°)	Volumetric Water Content	Compressibility (/kPa)
	Cutoff Wall	Saturated Only			1e-08	1	0	0	0
	D-VD Sands	Saturated Only			0.0001	0.5	0	0.4	1e-05
	L-D Sands	Saturated / Unsaturated	L-D Sands VWC	L-D Sands HC		1	0		

Color	Name	Category	Kind	Parameters
	Hunter River 1.3mAHD	Hydraulic	Water Total Head	1.3 m
	Seepage Face	Hydraulic	Water Rate	0 m³/sec

Apartment 2 - Standard cut-off wall - 1.3m AHD Ground Water Level	
Inflow Analysis_0.5m BEL DW.gsz	
08/11/2024	1:700