APPENDIX H

Groundwater Impact Assessment, Version 2

Stone Ridge Quarry

Groundwater Impact Assessment Version 2

Australian Resource Development Group

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→ **The Power of Commitment**

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Executive Summary

Australian Resource Development Group Pty Limited (ARDG) is seeking development consent for State Significant Development (SSD) 10432, a new hard rock quarry known as Stone Ridge Quarry (the project). The project is located within Wallaroo State Forest at Balickera, NSW, approximately 25 kilometres north of Newcastle. The project is seeking to access a high quality, hard rock resource suitable for producing a wide range of quarry products for the Lower Hunter, Central Coast, and northern Sydney construction materials market (ARDG 2020).

This Groundwater Impact Assessment (GIA) has been undertaken with reference to the Groundwater Assessment Toolbox for Major Projects in NSW (DPE 2022) and to address the SEARs relating to groundwater, dated 1 June 2020.

A conceptual groundwater model was developed for the project based on groundwater monitoring data, lithology logs and core photographs provided by ARDG, interpreted geology and previous hydrogeological assessments for the nearby Eagleton (proposed) and Seaham quarries. The groundwater flow system occurs in unconfined and confined fractured rock aquifers within the Eagleton Volcanics and within shallow, perched aquifers. Groundwater levels in the fractured rock aquifer reflect and are controlled by topography. Groundwater is recharged by rainfall infiltration on the upper slopes and ridgelines and flows towards lower lying areas and drainages where it is discharged. The shallow, perched aquifers on the western flank which overly a dacite aquitard are recharged via rainfall, surface runoff and shallow water movement from upslope within the regolith and soil material. Flow is topographically controlled, and disconnected from the deeper, fractured rock groundwater system.

The nearest high priority Groundwater Dependent Ecosystems (GDEs) are located near the Williams River to the north and east of Seaham, approximately eight and five kilometres from the project, respectively. With reference to the Probable Vegetation Groundwater Dependent Ecosystems – Hunter/Central Rivers dataset (DPE Water 2022), high probability GDEs are associated with Nine Mile Creek to the north and north-east of the project, and Caswell's Creek and Williams Creek to the west. Four registered bores are located within a five-kilometre radius of the project, including three basic landholder rights bores.

The project area is located within the New England Fold Belt Coast Groundwater Source which is managed by the Water Sharing Plan for the North Coast Fractured and Porous Rock Groundwater Sources. Any take of groundwater associated with the project (through passive inflow or direct take through extraction for operational purposes) will require a WAL under the WM Act. Based on water balance modelling results as part of the Surface Water Impact Assessment, ARDG requires 39 ML/year in the early stages of the project. A WAL will therefore be required for direct take prior to the commencement of the project. Based on recent (2021/2022) trades within the New England Fold Belt Coast Groundwater Source there is sufficient market depth for ARDG to obtain a licence for 39 ML/year.

Based on the results of a pumping test conducted near ARDG-P06 and forward analyses, continuous pumping at 1.25 L/s or discontinuous pumping at 2.5 L/s (assuming complete recovery following each 12-hour pumping period), is likely to be sustainable for at least a year in the fractured rock aquifer. Therefore, it is possible that the 90th percentile bore import volume could be obtained from a production bore targeting the fractured rock aquifer in the ARDG-P06 area.

Quantification of likely groundwater inflow rates and the radius of drawdown was undertaken using a steady-state analytical model. Considering that the distance to the few registered landholder bores is greater than one kilometre, the low hydraulic conductivity of the aquifer and deep groundwater levels through the rhyodacite resource that would not support GDEs, it is considered that the risk to identified groundwater receptors due to the project is low. The level of complexity of analytical equations is therefore appropriate to assess this risk.

The interpreted hydraulic conductivity at monitoring bore ARDG-P02 from rising head slug testing was assumed to represent the maximum expected hydraulic conductivity in the rhyodacite resource (K_{max} = 9.2 x 10^{-3} m/day). More likely values of hydraulic conductivity were estimated assuming 10% and 20% fracturing of the rhyodacite resource and an averaged literature value for unfractured igneous and metamorphic rock (K_{10} = 9.3 x 10^{-4} m/day and K_{20} = 1.9 x 10^{-3} m/day). Using this more likely range of hydraulic conductivity estimates, Stage 8 groundwater inflows into the pit were predicted to range from 8.7 ML/year to 14.3 ML/year, and the radius of drawdown was predicted to be between 336 m and 363 m from the centre of the pit.

The requirement for a WAL for passive take will not arise until the pit floor of the quarry progresses below the pre-quarry groundwater level. Based on the more likely inflow predictions (assuming 10% and 20% fracturing of the rhyodacite resource) for Stage 8 of the development (the final and deepest stage of quarry operations), a WAL for approximately 9 – 15 ML/year would be required. However, the 39 ML/year take is only required to meet operational demands in the early stages of the project, well before the quarry floor intercepts groundwater. Therefore, no additional licencing would be required for the pit inflows, noting that updated predictions will be obtained based on groundwater monitoring and observed inflows into the pit.

The most conservative predicted radius of drawdown for Stage 8 (468 m; Scenario 1 based on K_{max}) was used to assess the impact of the project on existing groundwater users. Landholder bores are well outside the project's radius of drawdown. No drawdown is therefore expected to occur at any of the landholder bores. Therefore, landholder bores will not be impacted by any drawdown associated with the project. Therefore, the impact of the project therefore meets the NSW Aquifer Interference Policy (AIP) Level 1 Minimal Impact Considerations for landholder bores.

Based on the most conservative predicted radius of drawdown for the potential production bore, drawdowns exceeding one metre are not expected to occur at distances beyond 600 m. Given that the nearest water supply bores (GW060834 and GW060853) are located more than one kilometre to the north-west of ARDG-P06, it is unlikely that a production bore located near ARDG-P06 will cause more than a 2 m water table decline cumulatively at any water supply work. Therefore, the impact of the potential production bore therefore meets the NSW Aquifer Interference Policy (AIP) Level 1 Minimal Impact Considerations for landholder bores.

The most conservative predicted radius of drawdown for Stage 8 (468 m; Scenario 1 based on K_{max}) was used to assess the impact of the project on GDEs. High priority GDEs identified in Section 3.6 are well outside the project's radius of drawdown. These GDEs are also well outside the conservative radius of influence of a potential production bore located near ARDG-P06. No drawdown is therefore expected to occur at any of the high priority GDEs as a result of the project or potential production bore.

Based on standing water levels measured in groundwater monitoring bores, the average depth to groundwater in the fractured rock aquifer at the project varies between 7.31 and 23.3 metres below ground level. Average groundwater levels in the area where the high probability GDEs are present, within the maximum extent of predicted drawdown (468 m; Scenario 1 based on K_{max}), are between approximately seven and 13 metres below ground level. However, groundwater in these areas is actually significantly lower (i.e., 20 m below the surface) due to the presence of a zone of low permeability dacite which acts as an aquitard, confining groundwater in the deeper, more permeable units at a depth.

A refined, conservative predicted radius of drawdown (389 m; Scenario 2 based on K_{max}) was used to assess the potential impact of the project on high probability GDEs located in the western flank area. This scenario is based on a reduced groundwater level which more closely represents impacts to the terrain surrounding the quarry footprint. Drawdowns greater than one metre are not expected to occur at distances exceeding 300 m from the centre of the pit. Therefore, drawdowns in the area of the high probability GDEs are not expected to be greater than one metre. The potential production bore could also be positioned so that drawdown in the area of the high probability GDEs would not exceed one metre.

Notwithstanding, given the depth to groundwater in this area (i.e., 20 m below surface), it is apparent that the presence of high probability GDEs is due to shallow groundwater in the overlying alluvial/colluvial material (recharged from creeks and rainfall), rather than the deeper regional groundwater table from which it is disconnected in this area. Therefore, the modelled drawdown of groundwater in the deeper, fractured rock system (including from a potential production bore) is not predicted to adversely impact these high probability GDEs.

The impacts of the project and potential production bore therefore meet the NSW Aquifer Interference Policy (AIP) Level 1 Minimal Impact Considerations for landholder bores and GDEs.

The project or a potential production bore are not expected to cause any significant change in groundwater quality or in the beneficial use of the groundwater. The increased groundwater recharge in the post closure phase may also result in a localised improvement in groundwater quality. The impacts of the project and potential production bore therefore meet the NSW Aquifer Interference Policy (AIP) Level 1 Minimal Impact Considerations for groundwater quality.

The potential production bore would only be operating during Stage 1 of the project. Groundwater inflow is only expected from Stage 5 onwards. An assessment of cumulative impacts is therefore not required.

The project is expected be completed after 30 years. With time, groundwater levels in the aquifer surrounding the project will recover until equilibrium within the system occurs, and a pit lake forms within the final voids. Once the system is in equilibrium, the flux of water within the pit lake will only be from rainfall and evaporation. During the recovery stage however, groundwater inflows will occur, and a WAL will still be required in the initial post closure phase of the project. Any enhanced recharge that occurs as a result of the quarry in the post closure phase would reduce the time required for groundwater levels to recover.

It is recommended that the existing groundwater monitoring program be continued. It is recommended that groundwater be monitored to:

– Measure dewatering performance.
– Assess potential impacts to groundwater levels and quality on other groundwater users in the vicinity.
– Identify groundwater issues such as potential large drawdowns at receptors as early as possible.
– Provide data which can be used to calibrate the analytical model and update the groundwater inflow predictions.
– Measure groundwater level recovery post closure and provide data which can be used to predict how long a WAL may be required after the project is completed.

It is recommended that the existing monitoring program be extended to include an additional monitoring bore installed approximately one kilometre from the project, to the north-west. The groundwater monitoring program should continue to monitor water levels and water quality regularly and given that the site can become inaccessible in very wet weather, it is recommended that data loggers be installed in two monitoring bores, to provide a continuous record. The groundwater monitoring program should also include pit inflow monitoring. It is recommended that the groundwater monitoring program be reviewed every two years.

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1. Introduction

1.1 Background

Australian Resource Development Group Pty Limited (ARDG) is seeking development consent for State Significant Development (SSD) 10432, a new hard rock quarry known as Stone Ridge Quarry (the project). The project is located within Wallaroo State Forest at Balickera, NSW, approximately 25 kilometres north of Newcastle. The project is seeking to access a high quality, hard rock resource suitable for producing a wide range of quarry products for the Lower Hunter, Central Coast and northern Sydney construction materials market (ARDG 2020).

1.2 Purpose of this report

The purpose of this report is to prepare a Groundwater Impact Assessment (GIA) to support the preparation of an Environmental Impact Assessment (EIS) for the project.

1.3 Scope

The scope of the GIA is as follows:

- Review available information and data, including groundwater level data, geological and exploration data, and available groundwater reports for adjacent sites (Boral and Eagleton quarries).
- Undertake searches of the registered groundwater bore and Groundwater Dependent Ecosystem (GDE) online databases and identify groundwater receptors (including basic landholder rights bores).
- Undertake a data gap analysis and identify additional monitoring and testing requirements (if any).
- Provide a description of the existing groundwater environment, including a summary of available monitoring data from site bores and adjacent quarries.
- Develop a conceptual groundwater model identifying inputs and outputs, groundwater flow systems and groundwater receptors.
- Consult with relevant agencies (DPE Water and NRAR) to discuss the proposed assessment methodology.
- Review of relevant Water Sharing Plans (WSPs) and classification of the groundwater source under the NSW Aquifer Interference Policy.
- Assess the rate of groundwater inflow and radius of drawdown due to the proposed operations using appropriate analytical methods.
- Assess potential impacts (quantity and quality) on identified groundwater receptors including assessment of impacts against the groundwater level and quality criteria in the NSW Aquifer Interference Policy (AIP).
- Identify groundwater licensing requirements under the relevant WSPs, including an assessment of market depth should a Water Access Licence (WAL) be required.
- Identify ongoing groundwater monitoring requirements.
- Document the findings of the above in a GIA report.

1.4 Limitations

This report has been prepared by GHD for Australian Resource Development Group and may only be used and relied on by Australian Resource Development Group for the purpose agreed between GHD and Australian Resource Development Group as set out in Section 1.2 of this report.

GHD otherwise disclaims responsibility to any person other than Australian Resource Development Group arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

GHD has prepared this report on the basis of information provided by Australian Resource Development Group and others who provided information to GHD (including Government authorities), which GHD has not independently verified or checked beyond the agreed scope of work. GHD does not accept liability in connection with such unverified information, including errors and omissions in the report which were caused by errors or omissions in that information.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report. GHD disclaims liability arising from any of the assumptions being incorrect.

2. Regulatory context

2.1 Legislation

2.1.1 Environmental Planning and Assessment (EP&A) Act 1979

The EP&A Act is the core legislation relating to planning and development activities in NSW and provides the statutory framework under which development proposals are assessed. The EP&A Act aims to encourage the proper management, development and conservation of resources, environmental protection and ecologically sustainable development.

2.1.2 Water Management Act 2000

The aim of the *Water Management Act 2000* (WM Act) is to ensure that water resources are conserved and properly managed for sustainable use benefiting both present and future generations. It is also intended to provide formal means for the protection and enhancement of the environmental qualities of waterways and in-stream uses as well as to provide for protection of catchment conditions.

Certain licences and approvals, including water access licences (WALs), water use approvals and water supply work approvals are issued under the WM Act.

A WAL is generally required to extract water from rivers or aquifers. The WM Act governs the issue of WALs for water sources in NSW where water sharing plans have commenced. A WAL entitles the holder to:

– Specified shares in the available water within a particular water management area or water source (the share component).
– Take water at specified times, rates or circumstances from specified areas or locations (the extraction component.

A water use approval confers a right on its holder to use water for a particular purpose at a particular location. A water supply work approval, as a specific type of water management work approval, authorises its holder to construct and use a specified water supply work at a specified location. Under sections 89 and 90 of the WM Act, water use approvals and water management work approvals do not apply for SSD that is authorised by Development Consent. Therefore, water use and water supply work approvals are not required for the project.

Landholders can take water under basic landholder rights without a water licence or approval under certain circumstances, including domestic and stock rights, native title and harvestable rights.

The WM Act defines the various offences for taking and using water from water source other than in accordance with the relevant approvals.

2.1.2.1 Water sharing plans

Fresh water sources throughout NSW are managed via water sharing plans (WSPs) under the WM Act. Provisions within WSPs provide water to support the ecological processes and environmental needs of GDEs and waterways. WSPs also regulate how the water available for extraction is shared between the environment, basic landholder rights, town water supplies and commercial uses. Key rules within the WSPs specify when licence holders can access water and how water can be traded.

The Project is located within the New England Fold Belt Coast Groundwater Source which is managed by the Water Sharing Plan for the North Coast Fractured and Porous Rock Groundwater Sources. Therefore, the interference and extraction of groundwater at the project will require a WAL under the WM Act.

2.2 Policies

2.2.1 NSW Aquifer Interference Policy

The NSW Aquifer Interference Policy (AIP) was finalised in September 2012 and clarifies water licensing and approval requirements for aquifer interference activities in NSW, including the taking of water from an aquifer while carrying out mining. Aquifer interference activities may take water from the water source in which they exist as well as connected groundwater and surface water sources.

The Policy outlines the water licensing requirements under the WM Act. A water licence is required whether water is taken for consumptive use or whether it is taken incidentally by the aquifer interference activity (such as groundwater filling a void), even where that water is not being used consumptively as part of the activity's operation. Under the WM Act, a water licence gives its holder a share of the total entitlement available for extraction from the groundwater source. The WAL must always hold sufficient share component and water allocation to account for the take of water from the relevant water source.

Sufficient access licences must be held to account for all water taken from a groundwater or surface water source resulting from aquifer interference activity, both for the life of the activity and after the activity has ceased. This also includes passive take from connected groundwater and surface water sources. The NSW AIP requires that potential impacts on groundwater sources, including users and GDEs, be assessed against minimal impact considerations, outlined in Table 1 of the Policy. If the predicted impacts meet the Level 1 minimal impact considerations, then these impacts will be considered as acceptable. The adopted Level 1 minimal impact considerations for the project are discussed in Section 6.4.

Aquifer interference approval requirements for the project have not commenced.

2.2.2 NSW Groundwater Strategy

The objective of the NSW Groundwater Strategy (December 2022) is to manage the State's groundwater resources so that they can sustain environmental, social, and economic uses for the people of NSW. The NSW Groundwater Strategy has three strategic priorities:

- Protect groundwater resources and the ecosystems that depend on them
- Build community and industry resilience through sustainable groundwater use
- Improve groundwater information and knowledge

2.2.3 Guide to Groundwater Management in NSW

The Guide to Groundwater Management in NSW (February 2023) provides details regarding the framework and regulatory context for groundwater management, as described throughout this section of this report. The Guide to Groundwater Management in NSW describes responsibilities of groundwater users, government agencies and development proponents for groundwater management using practical examples.

The Guide to Groundwater Management in NSW describes policies for groundwater management in NSW including:

- Draft NSW Groundwater Quantity Management Policy
- NSW Groundwater Quality Protection Policy
- NSW Groundwater Dependent Ecosystems Policy

Draft NSW Groundwater Quantity Management Policy

The principles of this policy include:

- Maintain total groundwater use within the sustainable yield of the aquifer from which it is withdrawn
- Groundwater extraction shall be managed to prevent unacceptable local impacts
- Provide opportunities for sustainable development that provide cultural, social, or economic benefits
- Increase community understanding of groundwater management measures

NSW Groundwater Quality Protection Policy (1998)

The objective of this policy is the ecologically sustainable management of the State's groundwater resources so as to:

- Slow, halt or reverse any degradation in groundwater resources.
- Direct potentially polluting activities to the most appropriate local geological setting so as to minimise the risk to groundwater.

NSW Groundwater Dependent Ecosystems Policy (2002)

This policy was designed to protect ecosystems that are dependent on groundwater as a primary water source so that the ecological processes and biodiversity of these ecosystems are maintained or restored for the benefit of present and future generations. It provides guidance on how to protect and manage groundwater dependent ecosystems in a practical sense.

2.3 Secretary's Environmental Assessment Requirements (SEARs)

SEARs identify key issues for consideration in the EIS. The SEARs (dated 1 June 2020) relating to groundwater, and where they are addressed in this report, or in the surface water assessment, are shown in Table 2.1.

Table 2.1 SEARs

Requirement	Comment
Identification of any licensing requirements or other approvals under the Water Act 1912 and/or Water Management Act 2000.	Addressed in Section 6.3.
A description of the measures proposed to ensure the development can operate in accordance with the requirements of any relevant WSP or water source embargo.	Addressed in Section 7.
An assessment of the likely impacts on the quality and quantity of existing surface and ground water resources, including a detailed assessment of proposed water discharge quantities and quality against receiving water quality and flow objectives.	Impacts to surface water resources and discharge assessments are addressed in the Surface Water Impact Assessment for the project. Impacts to groundwater resources are addressed in Section 6.4.
An assessment of the likely impacts of the development on aquifers, watercourses, riparian land, water-related infrastructure, the Grahamstown Dam drinking water catchment, Balickera Channel, Balickera Tunnel and any other related infrastructure, and other water users.	All impacts relating to surface water are addressed in the Surface Water Impact Assessment for the project. Impacts to groundwater are addressed in Section 6.4.
A detailed description of the proposed water management system (including sewage), water monitoring program and other measures to mitigate surface and groundwater impacts.	Addressed in the Surface Water Impact Assessment for the project. Groundwater monitoring and mitigation measures are addressed in Section 7.

2.4 DPE Water and NRAR Requirements

Additional requirements have been provided from DPE Water and NRAR and are shown in Table 2.2.

Table 2.2 DPE Water and NRAR requirements

Requirement	Comment
The identification of an adequate and secure water supply for the life of the project. This includes confirmation that water can be sourced from an appropriately authorised and reliable supply. This is also to include an assessment of the current market depth where water entitlement is required to be purchased.	Addressed in the Surface Water Impact Assessment for the project. An assessment of groundwater supply and the current market depth of the groundwater source is addressed in Section 6.3.
A detailed and consolidated site water balance.	Addressed in the Surface Water Impact Assessment for the project.
Assessment of impacts on surface and ground water sources (both quality and quantity), related infrastructure, adjacent licensed water users, basic landholder rights, watercourses, riparian land, and groundwater dependent ecosystems, and measures proposed to reduce and mitigate these impacts.	Impacts to surface water systems and mitigation measures are addressed in the Surface Water Impact Assessment for the project. Impacts to groundwater, basic landholder rights and GDEs are assessed in Section 6.4 and proposed mitigation measures are discussed in Section 7.
Proposed surface and groundwater monitoring activities and methodologies.	Surface water monitoring is addressed in the Surface Water Impact Assessment for the project. Groundwater monitoring is addressed in Section 7.
Consideration of relevant legislation, policies, and guidelines, including the NSW Aquifer Interference Policy (2012), the Guidelines for Controlled Activities on Waterfront Land (2018) and the relevant Water Sharing Plans.	Consideration of relevant legislation, policies and guidelines is addressed in Section 2, Section 6.3 and Section 6.4.

3. Regional environment

3.1 Topography and land use

The project encompasses Stone Ridge, a 1,200 m long ridge that trends north-east to south-west. The ridge consists of two rocky hills, comprised of a sequence of volcanic and sedimentary rocks of the Eagleton Volcanics Formation. The hills are separated by a low saddle. The hill in the north-east has a maximum elevation of approximately 83 m AHD and the hill to the south-west has a maximum elevation of approximately 108 m AHD. A lower, wide ridge (South Ridge) extends across the south-east of the project area and has a maximum elevation of approximately 62 m AHD. The topography of the project area is shown in Figure 3.1.

The project is located within Wallaroo State Forest. The project is mostly surrounded by rural acreages however several non-rural developments also exist within the vicinity of the project including Port Stephens Gardenland, Hunter Valley Paintball, Ringwood Park Motor Complex and Boral Quarries Seaham.

3.2 Hydrology

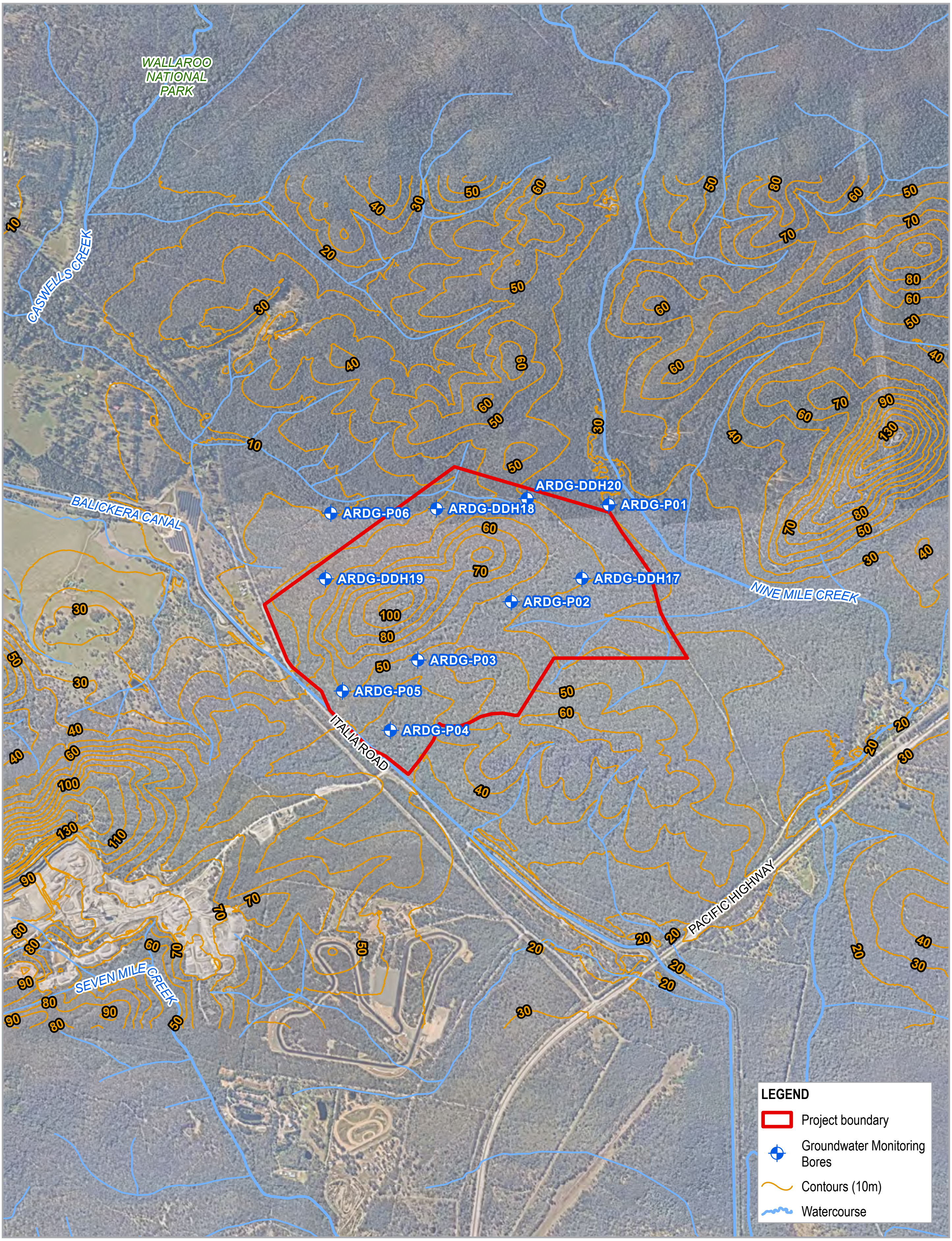
The project area is within the Grahamstown Dam drinking water catchment and extends into three sub-catchment areas: Caswell’s Creek, Nine Mile Creek and Italia Road (ARDG 2020). Sub-catchment boundaries are controlled by Stone Ridge and South Ridge (ARDG 2020). Each of these sub-catchments drain to Grahamstown Dam, via the Balickera Channel or overland flow (ARDG 2020). Grahamstown Dam is a drinking water supply for Newcastle and supplies a significant proportion of the region’s potable water requirements (URS 2014, Umwelt 2016).

The northern part of the project area drains either directly to Balickera Canal or to ephemeral tributaries of Caswell’s Creek, which subsequently drains to the Balickera Canal upstream of the Balickera Pump Station. Runoff from the eastern part of the project area drains to Nine Mile Creek, which flows under Nine Mile Road and the Pacific Highway before discharging into Grahamstown Dam approximately two kilometres south east of the Project site. Both Balickera Canal (on the eastern side of Balickera Pump Station) and Nine Mile Creek drain into the northern extent of Grahamstown Dam.

All drainages within the area surrounding the project are ephemeral however Nine Mile Creek located to the north-east of the project tends to permanently have water within scattered waterholes along its length, depending on rainfall conditions (ARDG 2020).

Hunter Water Corporation operates the Balickera Channel which transports water from the Williams River to Grahamstown Dam. The Balickera Channel is primarily an open canal approximately 2.7 km long cut into the surrounding land (ARDG 2020). Water sourced from the Williams River is pumped into the Balickera Canal at Seaham Weir and then raised approximately 15 m at the Balickera Pump Station into the eastern end of Balickera Canal which drains into the northern end of Grahamstown Dam. The Balickera Channel is outside the project area.

The hydrology of the project area is shown in Figure 3.1.



Paper Size ISO A4

0 180 360 540 720

Metres

Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

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Hydrography and Topography

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FIGURE 3.1

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Data source: Groundwater bores, project boundary, contours - ARDG 2022. General topography - LPI 2017. Nearmap WMS Server: - Imagery (date extracted: 16/02/2024).
Created by: tmorton

3.3 Climate

Climate data were obtained as SILO Patched Point Data from the Science Division of the Queensland Government's Department of Environment and Science. SILO Patched Point Data are based on historical data from a particular Bureau of Meteorology station with missing data "patched in" by interpolating with data from nearby stations. For this assessment SILO data were obtained from Williamtown RAAF (61078). The station was chosen based on the length and quality of the data record, and proximity to the project (approximately 13 km). The monitoring period selected was from January 1960 to April 2022.

3.3.1 Rainfall

Annual totals are shown in Figure 3.2 and rainfall statistics are summarised below:

- Minimum annual rainfall – 541 mm in 1980
- Maximum annual rainfall – 1794 mm in 1963
- Average annual rainfall – 1129 mm
- Median annual rainfall – 1090 mm

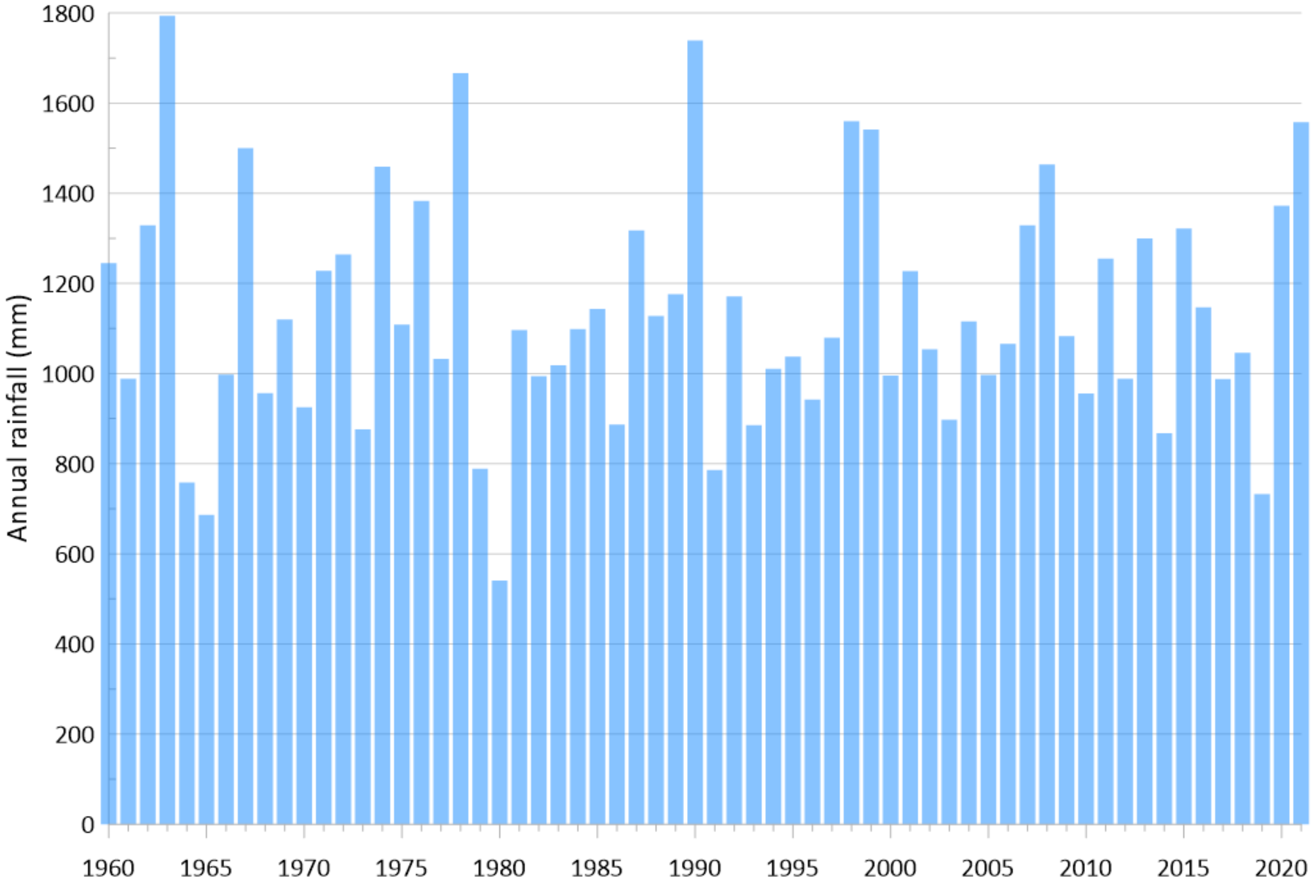


Figure 3.2 Annual rainfall at Williamtown RAAF Station (61078)

Rainfall is variably distributed throughout the year, with the highest rainfall occurring between February and June, as shown in Figure 3.3.

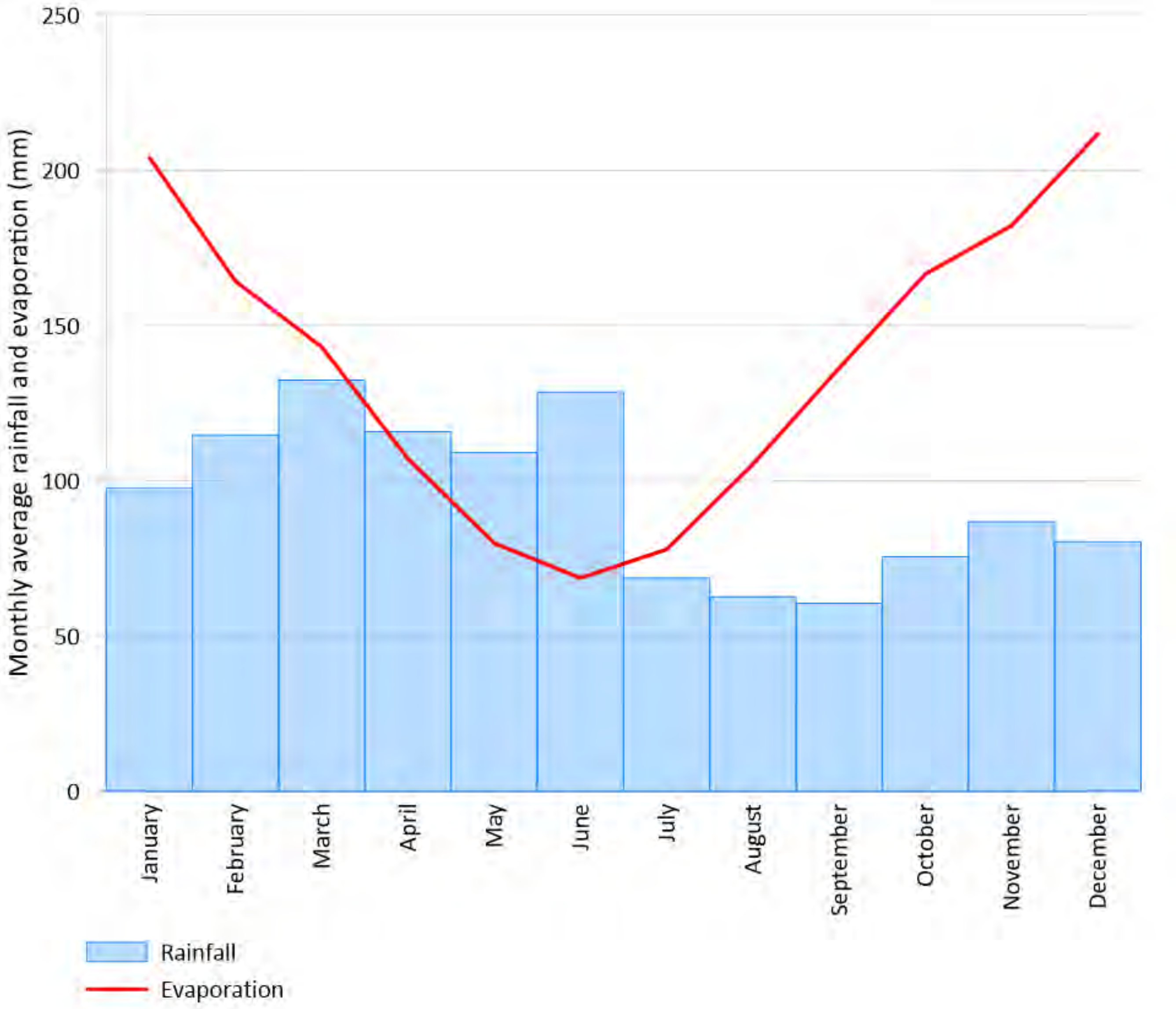


Figure 3.3 Monthly average rainfall and evaporation at Williamtown RAAF Station (61078)

The SILO dataset was used to generate a Cumulative Rainfall Departure (CRD) curve (accumulative annual residual rainfall). CRD is the monthly accumulation of the difference between the observed monthly rainfall and the long-term average monthly rainfall. Any increase in the CRD reflects above average rainfall while a decrease in CRD reflects below average rainfall. The CRD curve only deviates from zero due to atypical (above and below average) rainfall. The CRD over the period 1960 to 2021 is shown in Figure 3.4. Since mid-2020 the CRD shows an overall increasing trend, indicating above average rainfall conditions.

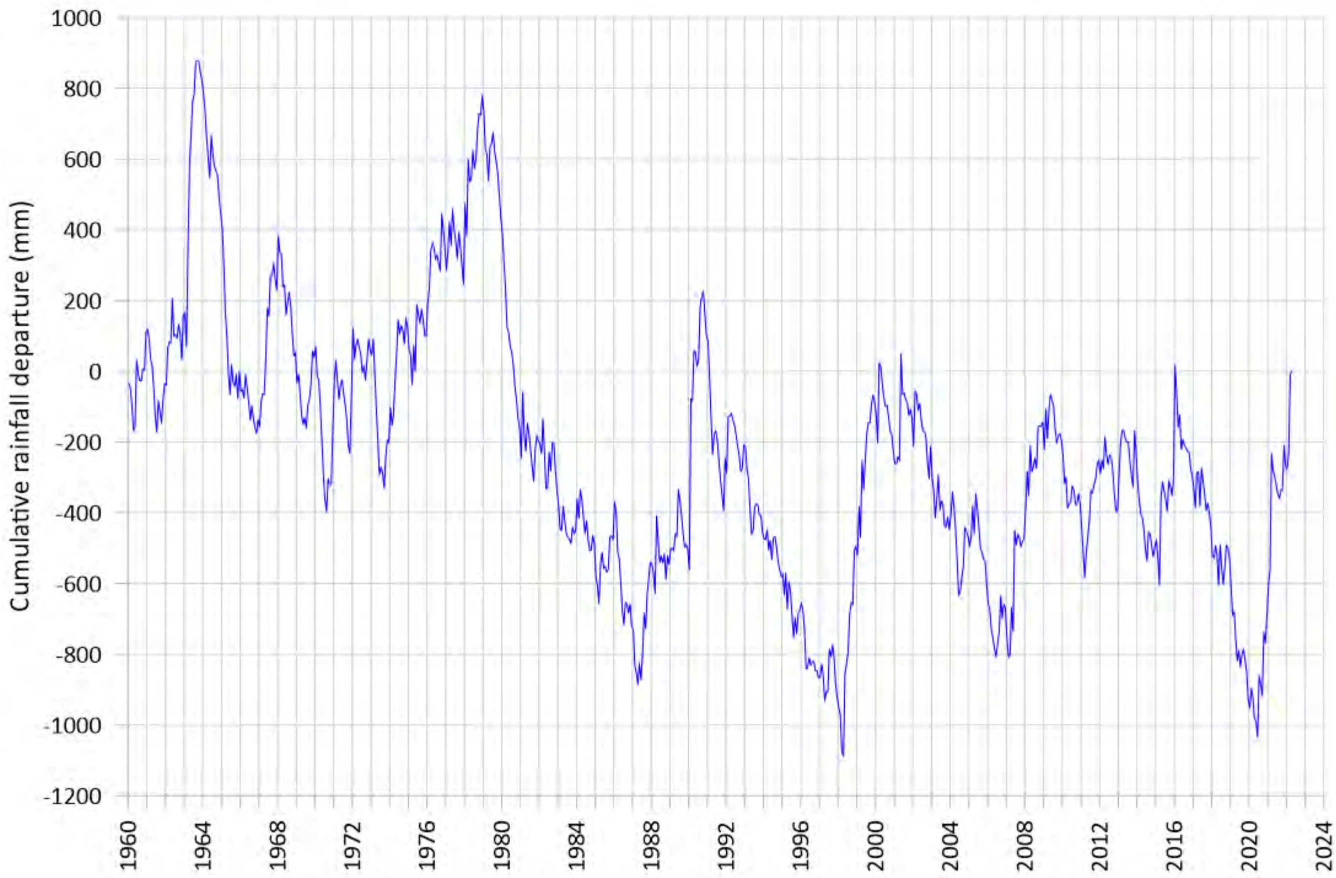


Figure 3.4 ***Cumulative Rainfall Departure curve for Williamtown RAFF Station (61078)***

3.3.2 Evaporation

Average annual evaporation is 1,651 mm. Average monthly evaporation varies between 69 mm in June to 212 mm in December. Evaporation exceeds rainfall for all months of the year except for April, May and June, as shown in Figure 3.3.

3.4 Geology

The project is located just to the north of the north-east extent of the Sydney Basin. The project area is within the New England Orogen Tectonic Province and is underlain by a sequence of volcanic and sedimentary rocks of the Eagleton Volcanics Formation.

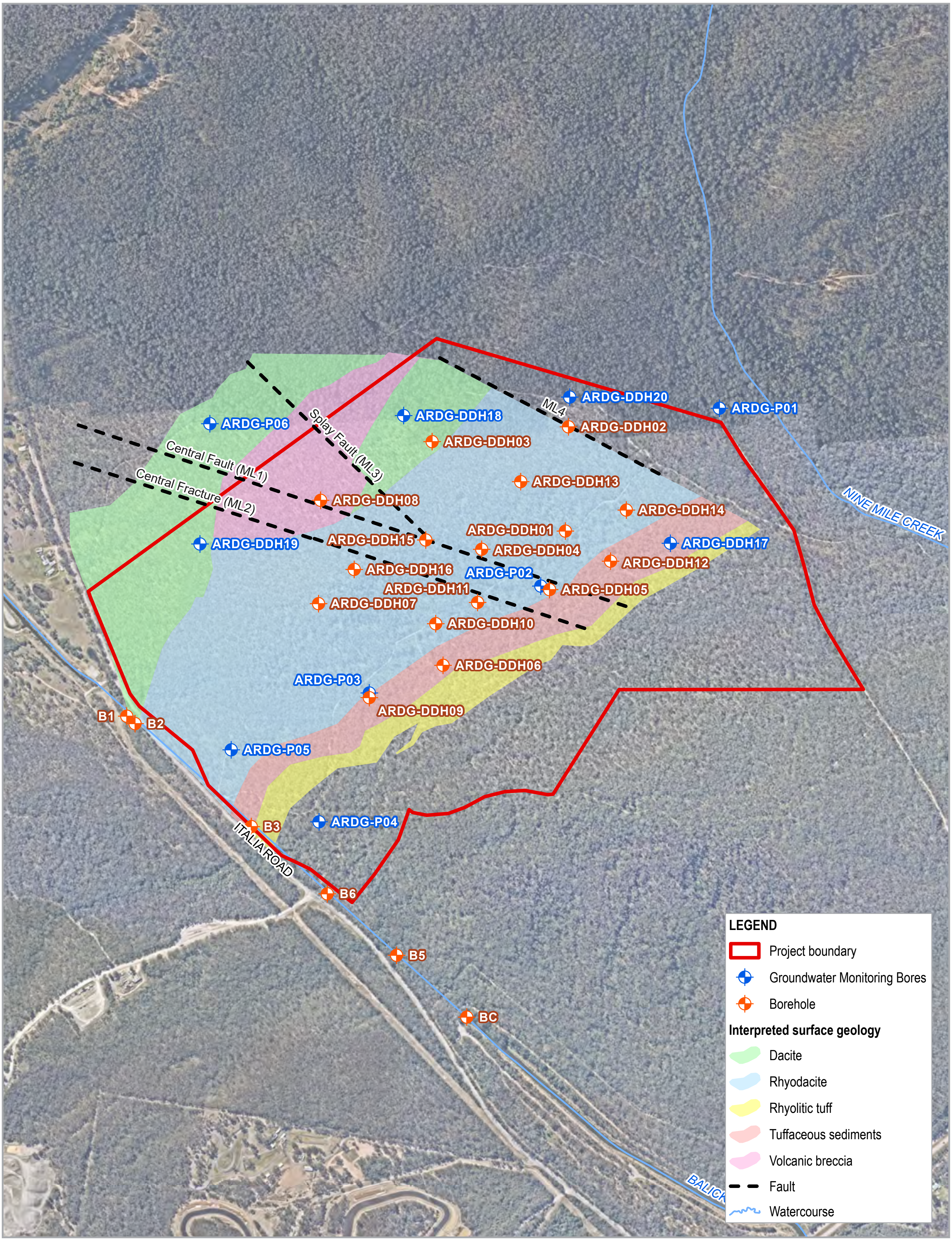
Detailed surface mapping, surface and downhole geophysics, and extensive diamond drilling undertaken by ARDG from 2016 to 2019 has confirmed that the Eagleton Volcanics within the project area is a bedded sequence of rocks that strikes northeast southwest and dips at approximately 35 degrees to the southeast (ARDG 2020). The dominant and most relevant rocks from a quarrying perspective are rhyodactic and dacitic rocks of the Eagleton Volcanics Formation (ARDG 2020).

Massive tuffs and lavas of rhyodactic composition outcrop extensively across Stone Ridge above an elevation of approximately 50 m AHD. The true thickness of this unit is interpreted to range from 200 to 275 m. The rhyodacites are underlain by an interbedded sequence of dacite, with lesser andesitic lithic fragmental tuff, volcanic breccia and rhyolitic vitric-crystal tuff. Based on the interpreted strike and dip of the volcanic stratigraphy, these units are interpreted to underlie the rhyodacite and extend to depth beneath the axis of Stone Ridge. They are interpreted to have a true thickness of approximately 280 m. Directly overlying the rhyodacite on the southeast flank of Stone Ridge is an interbedded sequence of moderately to highly weathered volcanic sandstone, siltstone, clayey tuff and rhyolitic tuff. It has a true thickness of approximately 50 m (ARDG 2020).

Soil profile development over the Eagleton Volcanics is very poor to non-existent (ARDG 2020). Along the crest and flanks of Stone Ridge soils are generally less than 0.3 m in depth and are typically weakly structured, sandy loams (ARDG 2020). In the south-east of the project area, borehole logs indicate a surficial clay layer varying in thickness from 1.0 m to at ARDG-P03 to 11 m thick at DDH17.

Vertical to sub-vertical faults occur in the project area. The most fractured and altered rhyodacite within the project area was observed along the Central Fault.

Interpreted surface geology, structures and drilled boreholes within the project area are shown in Figure 3.5.



Paper Size ISO A4

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Metres

Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

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Interpreted Geology and Drilling Investigations

FIGURE 3.5

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Data source: Groundwqater bores, project boundary - ARDG 2022. General topography - LPI 2017. Nearmap WMS Server: - Imagery (date extracted: 16/02/2024) .
Created by: tmorton

3.5 Hydrogeology

The project is located within the New England Fold Belt Coast Groundwater Source. The New England Fold Belt Coast Groundwater Source is a fractured aquifer system with groundwater contained within and moving through fractures in the rock that have occurred due to folding and faulting of the rock formations (NSW DPI 2016). Yields within the groundwater source are generally low, around 1 L/s, however, yields up to 10 L/s may be obtained from highly fractured fault systems (NSW DPI 2016).

Regional groundwater flow in the fractured rock aquifers is anticipated to be southeast towards Grahamstown Dam and the South Pacific Ocean (URS 2014). Regional gradients in the aquifers are typically less than 1% (URS 2014).

The local flow system occurs in unconfined and confined fractured rock aquifers within the Eagleton Volcanics. Where the fractured rock unit outcrops and where surficial clay layers are thin, the aquifer is unconfined. To the south-east of the project area, depending on the lateral continuity of the clay layer, the fractured rock may behave as a confined aquifer.

At the nearby Eagleton Quarry, David (2016) obtained the piezometric surface from four bores installed within the shallow unconfined zone and reported that groundwater levels appear to reflect topography with depth to water greatest at elevated areas and closer to the surface in low lying areas. This is consistent with most groundwater levels for the project (refer Section 4.2.1).

The Eagleton Volcanics has a very low primary porosity with groundwater flow occurring within secondary porosity features such as fractures or along contact boundaries between the volcanic rock and igneous dykes (EMM 2019). Hydraulic conductivity of rock generally decreases with burial depth as joints close and become less frequent (David 2016). However, in volcanic rocks weathering on the surface results in an increase in clay content which therefore lowers hydraulic conductivity compared with sub cropping rocks (David 2016).

Recharge to the fractured rock aquifers occurs via rainfall in the elevated areas where the rock sub-crops or outcrops. EMM (2019) reported that the establishment of quarries in the area has likely increased the overall recharge. Discharge is likely to occur at relatively lower lying areas. David (2016) reported that even though groundwater levels are sustained by rainfall infiltration, they are controlled by topography, geology and surface water levels in creeks, rivers and dams. Locally groundwater mounds beneath hills and discharges to creeks and is lost by evaporation where the water table is near the ground surface (David 2016).

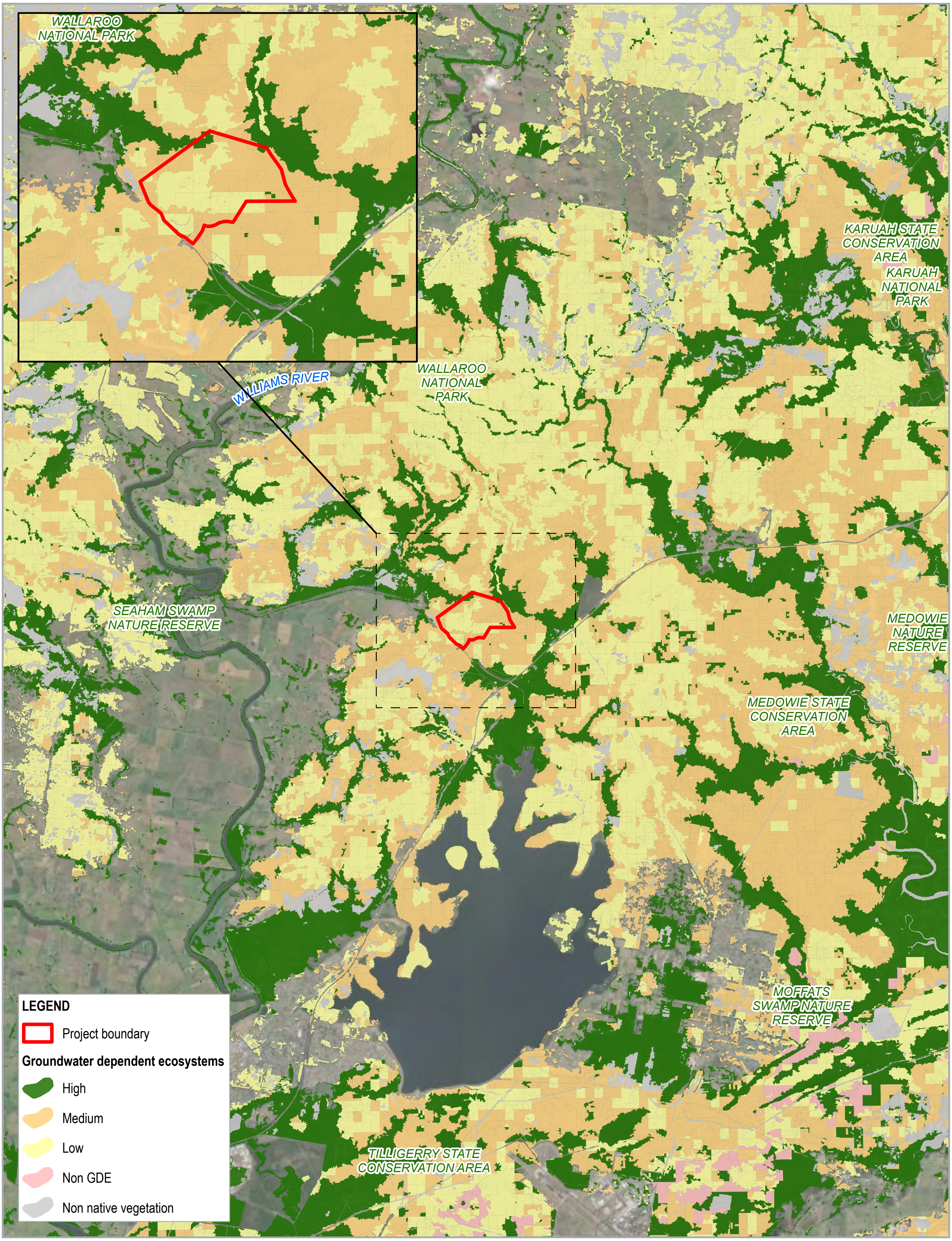
3.6 Environmental values of groundwater

3.6.1 Groundwater dependent ecosystems

The *Probable Vegetation Groundwater Dependent Ecosystems – Hunter/Central Rivers* dataset (DPE Water 2022) was reviewed to identify highly probable groundwater dependent ecosystems (GDEs) in the vicinity of the project. The dataset developed by DPE Water identifies vegetation communities that have a probability of being a groundwater dependent ecosystem within NSW. The dataset has been divided into catchment management areas. For this project, the Hunter-Central Rivers dataset was used.

The dataset indicates there are areas of high, medium, and low probability GDEs in the area (within 10 km of the project), as shown in Figure 3.6. High probability GDEs are associated with Nine Mile Creek to the north and north-east of the project, and Caswell's Creek and Williams Creek to the west.

The background document for the WSP for the North Coast Fractured and Porous Rock Groundwater Sources 2016 (DPI 2016a) refers to the High Priority Groundwater Dependent Ecosystem Map (DPI 2016b) which was reviewed to identify any high priority GDEs within the New England Fold Belt Coast Groundwater Source. The nearest high priority GDEs are located near the Williams River to the north and east of Seaham, approximately eight and five kilometres from the project, respectively. The GDEs are classified as wetlands.



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Groundwater Dependent Ecosystems

FIGURE 3.6

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Data source: Groundwater dependent ecosystems - BoM 2022. Project boundary - ARDG 2022. General topography - LPI 2017. Sixmaps/DCS - Imagery (date extracted: 16/02/2024) World Imagery: Earthstar Geographics. Created by: tmorton

3.6.2 Landholder bores

A search of the Australian Groundwater Explorer (BOM 2022) and Water NSW (2022) was undertaken to identify registered bores in the vicinity of the project. The search identified four bores within a five-kilometre radius. GW060834, GW060853 and GW066683 were reported to be water supply bores (stock and domestic) and are likely to be basic landholder rights bores. Reported yields range from 0.63 L/s to 0.90 L/s. The nearest private landholder bore (GW060834) is located approximately 1.7 km from the project area.

Registered bore details are summarised in Table 3.1.

Table 3.1 Registered private landholder bores within approximately five kilometres of the project

Bore	Depth (m)	Purpose	Drilled date	Screened interval (m depth)	Water bearing zone (m)	Yield (L/s)
GW060834	30.5	Water supply – stock and domestic	01/02/1985	Unknown	12.2 – 12.6	0.70
GW060853	24.3	Water supply – stock and domestic	01/02/1985	6.9 – 18	11.2 – 12.4	0.63
GW066683	35.0	Water supply – stock and domestic	06/02/1991	15 – 22	14 – 15, 20 – 21	0.90
GW079737	20.0	Unknown	20/10/1999	Unknown	Unknown	Unknown

GW060834 and GW060853 are located to the northwest of the project, in sedimentary units associated with the Wallaringa Formation. GW079737 is located to the southwest of the project, adjacent to Boral Quarries Seaham. GW066683 is located further to the southwest of the project.

Registered bore locations are shown in Figure 3.7 and are summarised in Table 3.2, including the approximate distance from the bore to the centre of the project area.

Table 3.2 Registered bore locations and distance from the project

Bore	Easting (MGA Zone 56)	Northing (MGA Zone 56)	Distance from the project (m)[1]
GW060834	386765	6386090	1,720
GW060853	386296	6386054	2,110
GW066683	385987	6381215	4,550
GW079737	386046	6383849	2,550

1 Approximate distance from the centre of the project area

Groundwater level observations (depth to groundwater) for GW060853 and GW060834 are summarised in Table 3.3 and Table 3.4, respectively. Average groundwater levels at these bores are 4.15 and 3.7 m below ground level, respectively.

An electrical conductivity (EC) of 1,255 µS/cm was reported at GW060853 on 10/09/1988 and an EC of 8,000 µS/cm was reported at GW060834 on 2/11/1987.

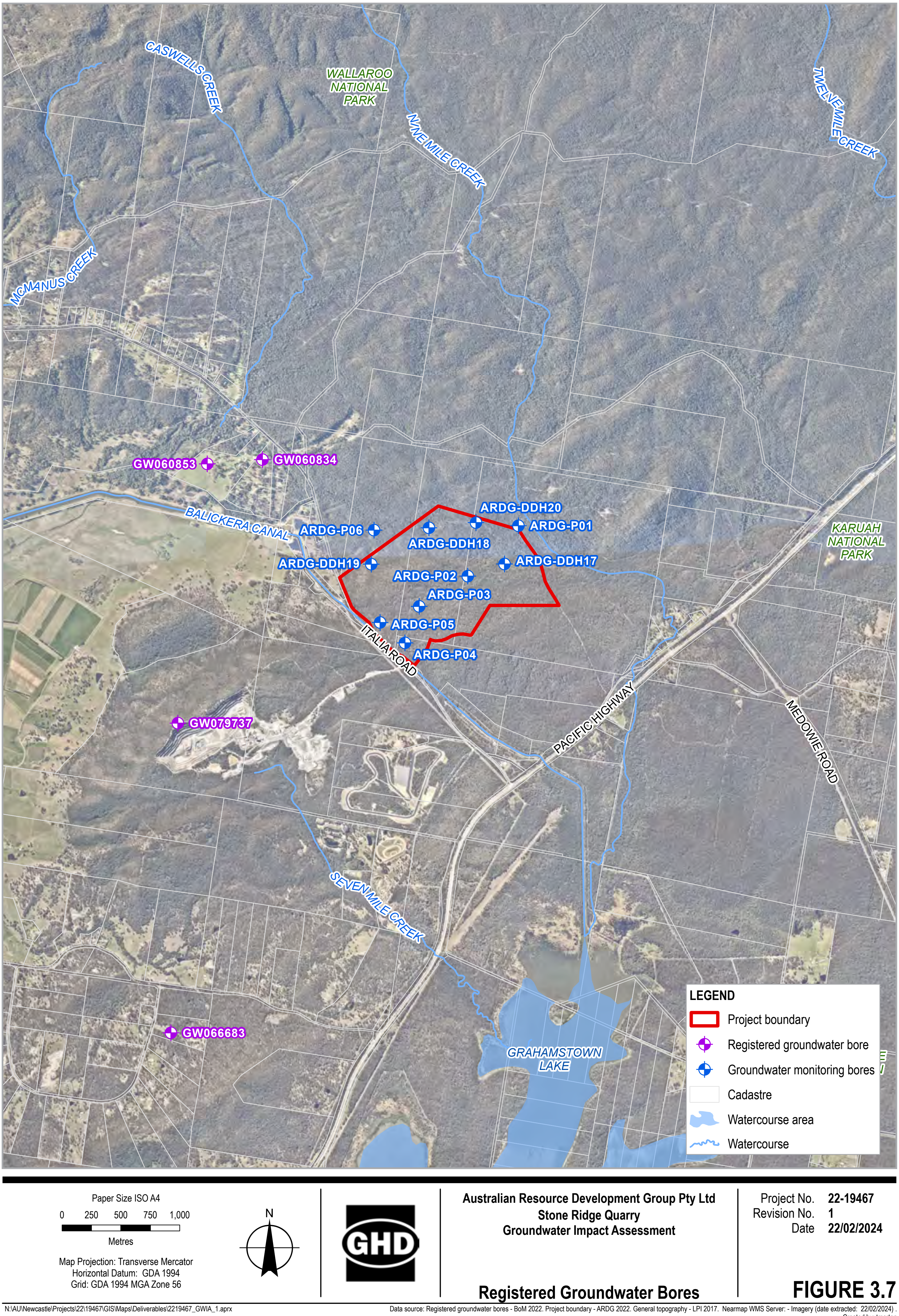
Table 3.3 Groundwater level observations for GW060853

Bore ID	Date	Standing water level (m below ground level)
GW060853.1.1	01-01-1985	4.20
GW060853.1.1	01-01-1994	4.26
GW060853.1.1	01-01-1995	4.00
GW060853.1.1	01-06-1995	4.00
GW060853.1.1	10-09-1998	4.29

Table 3.4 Groundwater level observations for GW060834

Bore ID	Date	Standing water level (m below ground level)
GW060834.1.1	01-01-1985	4.20
GW060834.1.1	01-01-1994	3.59
GW060834.1.1	01-01-1995	3.55
GW060834.1.1	01-06-1995	3.55

DPE Water advised that there is a water supply bore (Works Approval 20WA214724) located 1.5 km to the south west of the project. This bore was not identified in the bore search. Licence information indicates that this bore is located within Lot 66, DP 753200.



Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

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Registered Groundwater Bores

FIGURE 3.7

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Data source: Registered groundwater bores - BoM 2022. Project boundary - ARDG 2022. General topography - LPI 2017. Nearmap WMS Server: - Imagery (date extracted: 22/02/2024).
Created by: tmorton

3.7 Surrounding operations

3.7.1 Boral Quarries Seaham

Boral Quarries Seaham is located immediately southwest of the project. The quarry has operated as a hard rock quarry since 1991 (Boral 2019).

EMM (2019) conducted a groundwater assessment on the existing groundwater monitoring network to establish accurate groundwater levels and regional flow. EMM (2019) reported that the groundwater levels have been stable since monitoring began in 2015, with minor fluctuations generally less than 0.5 m. Groundwater levels measured at the Seaham Quarry in July 2019 are shown in Table 3.5.

Table 3.5 Groundwater levels at Seaham Quarry July 2019, reproduced from EMM (2019)

Monitoring bore	Bore elevation (m AHD)	Groundwater elevation (m AHD)
MW1	131.70	105.10
MW2	44.00	23.60
MW3	72.00	58.80
MW4	66.89	64.50
MW5	109.43	78.32
MW6	105.10	43.80
MW7	94.40	62.00
MW8	53.90	44.00

EMM (2019) analysed slug test data to estimate hydraulic conductivity values for the water bearing fractures screened by the monitoring bores. EMM (2019) reported that the water bearing fractures only represent a small proportion of the total rock mass which is most likely a low permeability igneous ignimbrite and rhyolite. Estimated hydraulic conductivities reported by EMM (2019) are shown in Table 3.6. For the analysis EMM (2019) assumed an unconfined aquifer.

Table 3.6 Estimated hydraulic conductivities, reproduced from EMM (2019)

Monitoring bore	Hydraulic conductivity (m/day)
MW4	1.2×10^{-1}
MW6	7.0×10^{-4}
MW7	2.5×10^{-1}
MW8	1.0×10^{-3}

Using analytical modelling (Marinelli and Nicolli (2000)), EMM predicted a maximum groundwater inflow to the quarry of 15 ML/year and a maximum radius of drawdown of 383 m. These results were based on a final quarry floor elevation of 45 m AHD, a water table elevation of 60 m AHD and a hydraulic conductivity of 1×10^{-1} m/day.

Groundwater quality at Boral Quarries Seaham was reported by EMM (2019) to be brackish to saline with electrical conductivity ranging from 3,200 µS/cm to 10,000 µS/cm. Groundwater pH was reported to be between 7 and 7.1 (EMM 2019).

3.7.2 Eagleton Quarry

The proposed Eagleton Quarry is located approximately 600 m to the south of Boral Quarries Seaham. URS (2014) conducted a hydrogeological investigation for the Eagleton Quarry. As part of the investigation five monitoring bores were installed, including a large 100 mm test well (GWB05). Groundwater levels ranged from eight metres below top of casing levels in the east to 40 m below top of casing levels in the steeper hills of the western portion of the site.

URS (2014) conducted falling head slug tests on all five wells at the Eagleton Quarry site to establish estimates for hydraulic conductivity. A groundwater flow model using Visual MODFLOW Pro was used to evaluate the potential impact of dewatering at the proposed Eagleton Quarry on the baseflow to adjacent creeks and groundwater users in the area (URS 2014). The following hydrogeological units and hydraulic conductivities (K) were adopted by URS (2014) for the Eagleton groundwater flow model:

– Volcanic/volcaniclastic rocks: variably weathered and fractured water table aquifer; K = 0.0004 m/day to 0.2 m/day.
– Sedimentary rocks: variably weathered and fractured water table aquifer; K = 0.02 m/day to 0.2 m/day.
– Estuarine/alluvial sediments: porous media water table aquifer; K = 0.1 m/day to 10 m/day.

The groundwater flow model predicted that quarry dewatering under steady-state conditions would reduce baseflow to the surrounding creeks between 12% and 68% (URS 2014).

A revised groundwater flow model and conceptual model was prepared by Katarina David on behalf of Umwelt, to support the EIS for the development of the Eagleton Quarry. The revised model estimated groundwater inflow to be 2.9 to 7.7 ML per year over the 30 years of the project. The revised groundwater modelling showed limited drawdown impact outside of the project area with a maximum impact on the south-western boundary of the site. The modelling indicated a drawdown of less than one metre is likely to extend to approximately 200 m to the west, north and south of the quarry excavation area. Modelling results indicated minor baseflow loss to Seven Mile Creek with a decrease of 0.75 m^3/day over the 30 years of the project. (Umwelt 2016).

David (2016) defined the hydrogeological regime within the project area as being comprised of two main systems:

– Alluvial/colluvial aquifer system mainly found in the west and low-lying areas. This unconsolidated sediment also includes windblown sand associated with Stockton sand dunes to the east and alluvial sediments associated with Williams River to the west. The alluvial/colluvial system is considered mostly hydraulically independent from the sedimentary/volcanic sequence.
– Sedimentary and volcanic sequence, although of different lithology, due to low permeability and porosity is one groundwater flow unit.

Groundwater quality at the proposed Eagleton Quarry was reported as being fresh to slightly saline (465 to 6,060 µS/cm) with neutral to alkaline pH (7.47 to 10.2) (URS 2014).

4. Groundwater management and monitoring

4.1 Groundwater monitoring network

Ten groundwater monitoring bores were installed at the project by ARDG in 2019 and 2020. All monitoring bores except for ARDG-P06 were constructed with a three-metre blank casing with base cap, below a three-metre laser cut slotted screen (1 mm aperture), and blank casing to the surface. ARDG-P06 was constructed with the three-metre slotted screen at the base of the monitoring bore. Each hole was backfilled with 10 mm aggregate and sealed with three metres of bentonite at the top. ARDG-P01 was blocked off during construction before adequate bentonite seal could be established. Monitoring bore construction details are summarised in Table 4.1 and locations are shown in Figure 4.1.

The groundwater monitoring bores are screened in different lithologies within the Eagleton Volcanics. ARDG-DDH18 was drilled deeper than the proposed Stage 8 pit floor. The borehole intersected the volcanic breccia unit which underlies the rhyodacite. Site communications indicated that groundwater flowed into the borehole when the breccia was intersected. ARDG-DDH18 is also located close to a natural drainage line. ARDG-P02 is located near borehole ARDG-DDH05 and the Central Fault Zone, which is where the most significant fracturing was encountered at the project.

Table 4.1 Monitoring bore construction details

Monitoring bore	Easting MGA Zone 56	Northing MGA Zone 56	Elevation – top of casing (m AHD)	Hole depth (m)	Screened elevation (m AHD)	Screened lithology
ARDG-DDH17	388,823.5	6,385,202.5	41.41	39.60	4.81 to 7.81	Conglomerate, sandstone and siltstone
ARDG-DDH18	388181	6385510.1	32.34	54.65	-19.31 to -16.31	Dacite and volcanic breccia
ARDG-DDH19	387690.4	6385200.8	36.77	36.00	3.77 to 6.77	Volcanic breccia
ARDG-DDH20	388,581.1	6,385,554.6	40.65	36.50	7.15 to 10.15	Sandstone and siltstone
ARDG-P01	388,941.6	6,385,527.7	29.86	21.00	11.86 to 14.86	Dacite
ARDG-P02	388,511.3	6,385,099.4	45.13	24.40	23.73 to 26.73	Dolerite
ARDG-P03	388098.2	6384841.9	46.60	24.50	25.1 to 28.1	Rhyodacite
ARDG-P04	387977.2	6384531.4	34.23	23.80	13.43 to 16.43	Tuff
ARDG-P05	387765.6	6384704.9	49.46	23.80	28.66 to 31.66	Rhyodacite
ARDG-P06	387,714.2	6,385,490.5	20.06	21.00	-0.94 to 2.06	Dacite

Paper Size ISO A4

0 100 200 300 400

Metres

Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

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Groundwater Monitoring Bores

FIGURE 4.1

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Data source: Groundwqater bores, project boundary - ARDG 2022. General topography - LPI 2017. Nearmap WMS Server: - Imagery (date extracted: 16/02/2024) Nearmap WMS Server: .
Created by: tmorton

4.2 Monitoring results

4.2.1 Groundwater levels

Groundwater levels have been monitored manually by ARDG since June 2019 in ARDG-DDH bores and since March 2020 in ARDG-P bores. Groundwater level hydrographs are shown in Figure 4.2. Individual groundwater level hydrographs are shown in Appendix A.

Average groundwater levels in the fractured rock aquifer range from 12.7 m AHD at ARDG-P06 to 28.2 m AHD at ARDG-P02, and in general, reflect topography.

On the lower western flanks of Stone Ridge, the average groundwater level at ARDG-P06, recorded as standing water levels in the monitoring bore, do not reflect the actual depth to groundwater at this location. In this area, which surrounds the ephemeral tributaries of Caswells Creek, the results of drilling and geophysical investigations undertaken for the project indicate the presence of a zone of low permeability dacite which overlies deeper, more permeable fractured rock. Where present, the low permeability dacite acts as an aquitard, confining groundwater in the deeper more permeable units at a depth.

During drilling of ARDG-P06, groundwater was intersected at a depth of 20 m below surface (i.e., 0 m AHD), at the contact between the overlying dacite and the underlying more permeable fractured rock. The standing water level in ARDG-P06 was observed to stabilise at approximately 12.7 m AHD. The actual depth to groundwater within the fractured rock aquifer is greater than the standing water level recorded ARDG-P06 due to the confining effect of the low permeability dacite (massive unfractured rock mass) (refer Figure 5.3). Therefore, the standing water level in monitoring bore ARDG-P06 represents the potentiometric surface of the deeper, confined, fractured rock aquifer rather than a water table. The low permeability dacite also acts as an aquitard between the shallow weathered material and soils and the deeper, fractured groundwater system. Unconfined, perched aquifers are present in some areas on the western flank. The perched aquifer system is disconnected from the deeper, fractured groundwater and is recharged primarily from rainfall, surface runoff and shallow water movement from upslope within the regolith and soil material overlying the low permeability dacite.

Groundwater levels in the fractured rock aquifer generally reflect rainfall conditions, with an increasing trend observed over the last few years as a result of above average rainfall. Groundwater levels at ARDG-DDH19 fluctuate by several metres between measurements and may be influenced by surface water flowing into the borehole or are potentially associated with the shallow perched aquifer system present in this area.

There was a lack of groundwater level monitoring conducted in 2021 and early 2022 due to access restrictions resulting from both wet weather and COVID issues.

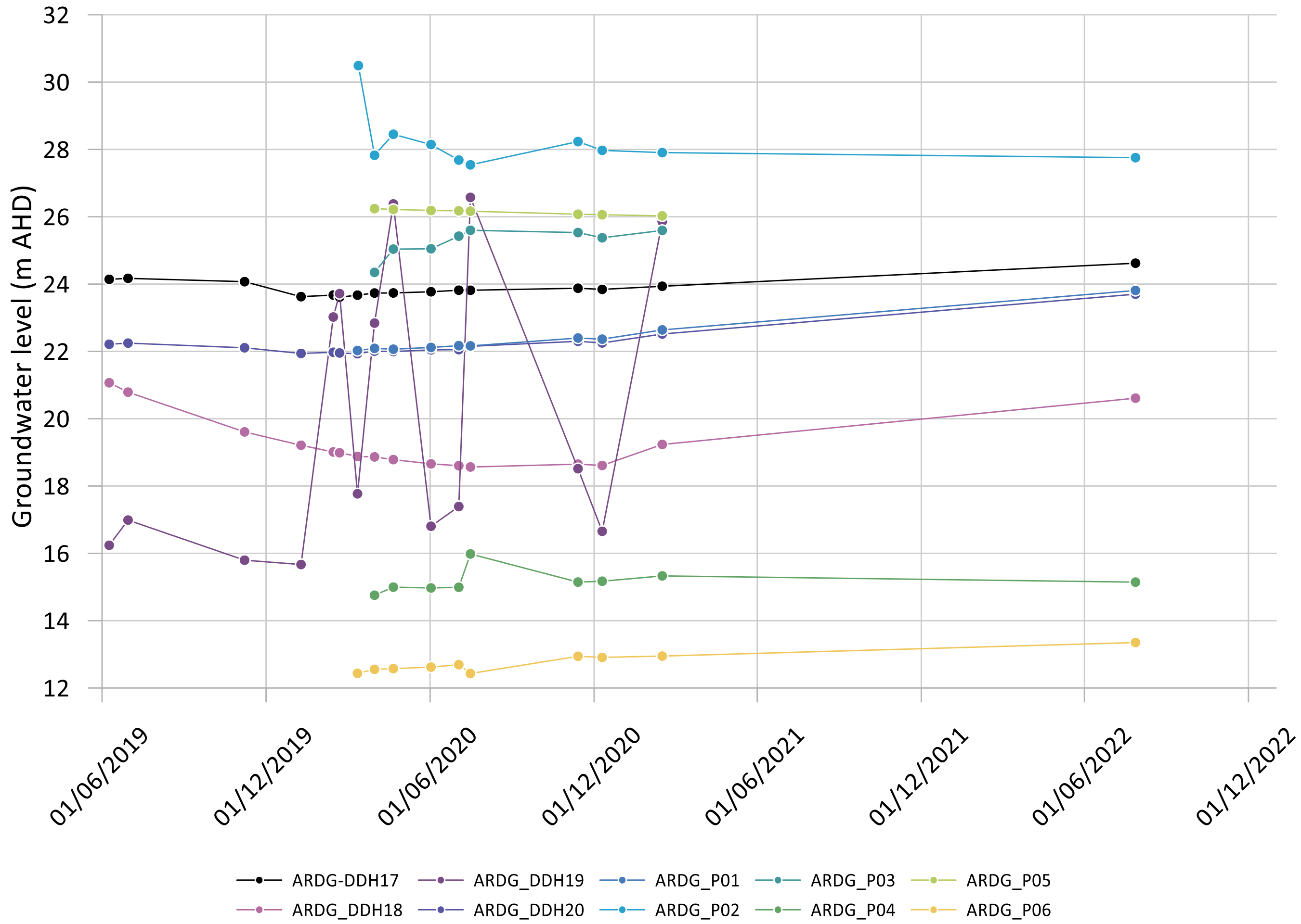
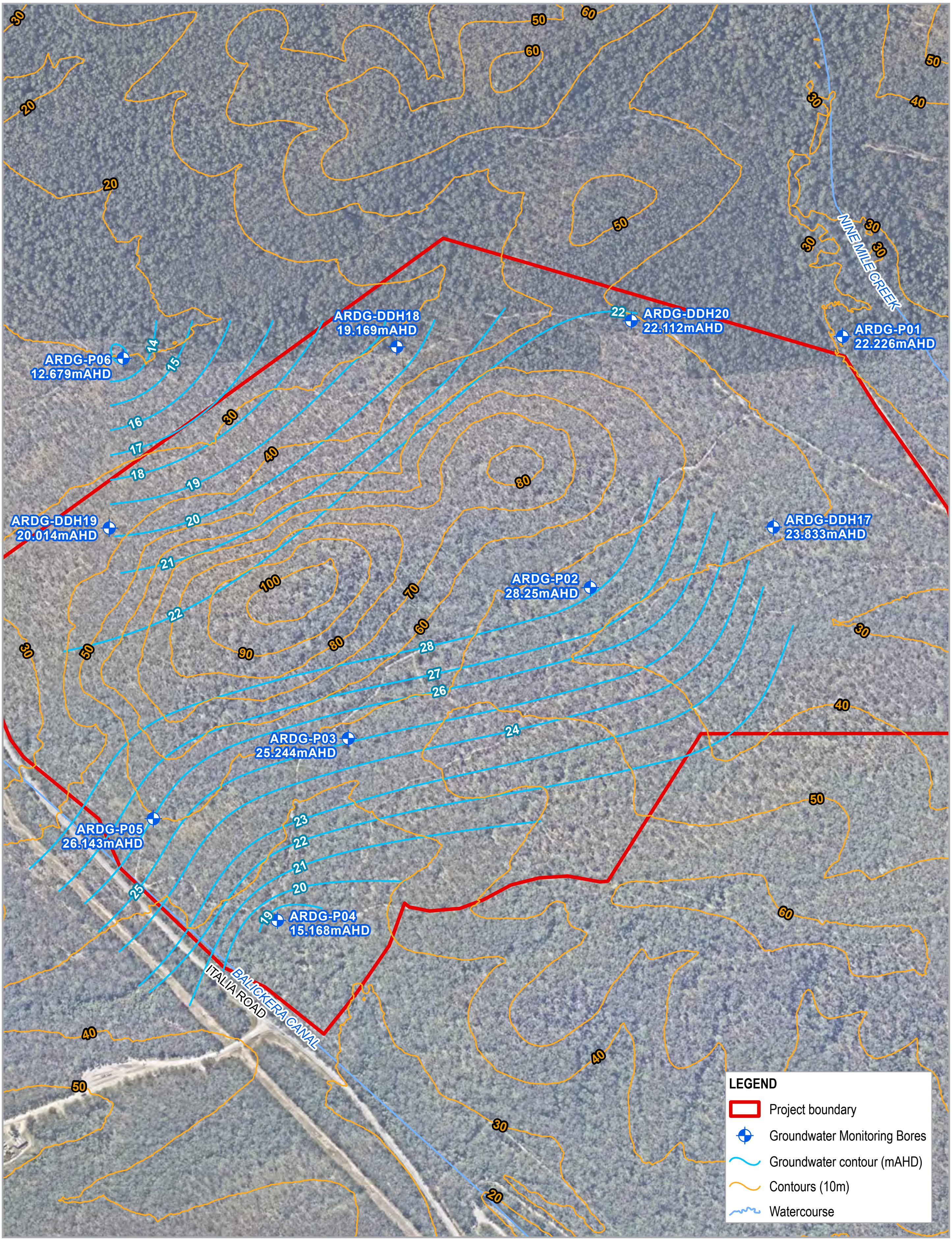


Figure 4.2 Groundwater level hydrographs

Average groundwater levels and interpreted contours are shown in Figure 4.3. Average groundwater level elevation and depth are summarised in Table 4.2. The groundwater level elevation at the project is higher than the registered private landholder bores located to the north-west. The private bores are therefore located hydraulically down gradient of the project.

Table 4.2 Average groundwater level elevation and depth

Monitoring bore	Elevation – top of casing (m AHD)	Average groundwater level (m AHD)	Average depth to groundwater (m below ground level)
ARDG-DDH17	41.41	23.88	17.53
ARDG-DDH18	32.34	19.26	13.08
ARDG-DDH19	36.77	20.01	16.76
ARDG-DDH20	40.65	22.21	18.44
ARDG-P01	29.86	22.38	7.48
ARDG-P02	45.13	28.20	16.93
ARDG-P03	46.6	25.24	21.36
ARDG-P04	34.23	15.17	19.06
ARDG-P05	49.46	26.14	23.32
ARDG-P06	20.06	12.75	7.310



Paper Size ISO A4

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Metres

Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

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Average Groundwater Level

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FIGURE 4.3

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Data source: Groundwqater bores, project boundary - ARDG 2022. General topography - LPI 2017. Nearmap WMS Server: - Imagery (date extracted: 16/02/2024) .
Created by: tmorton

4.2.2 Groundwater flow

Groundwater levels in the deeper, fractured rock aquifer generally reflect topography. The highest groundwater levels are observed where the ground surface is elevated, and lower groundwater levels occur at relatively lower elevations. Based on the average groundwater levels shown in Figure 4.3, groundwater flow in the deeper, fractured rock aquifer is controlled by topography, with Stone Ridge and South Ridge acting as local groundwater divides. Groundwater flows from topographically elevated areas towards lower lying areas.

The shallow, perched aquifers on the lower western flank which overly the dacite aquitard are recharged via rainfall, surface runoff and shallow water movement from upslope within the regolith and soil material. Flow is topographically controlled, and is isolated from the deeper, fractured rock groundwater system.

Nine Mile Creek, Balickera Channel and the tributary of Caswell’s Creek located to the north-west of the project are likely too high in elevation relative to the groundwater levels to be points of discharge for groundwater from within the project area. The project is therefore unlikely to have an impact on aquatic GDEs or baseflow, as the groundwater elevation is already below the creeks in the area.

4.2.3 Groundwater quality

Groundwater was sampled by ARDG at monitoring bores ARDG-DDH17, ARDG-DDH18, ARDG-DDH20, ARDG-P01, ARDG-P02 and ARDG-P06 on 28 July 2022 and analysed for major ions, nutrients, dissolved metals, pH and EC. Results are summarised in Table 4.3.

pH varied between 5.64 and 7.83, with an average value near neutral (6.88). EC varied between 198 µS/cm at ARDG-P02 and 5,820 µS/cm at ARDG-P01. The variability in EC, and overall chemistry is likely related to differences in geology and the degree of fracturing within the aquifer systems. ARDG-P01 and ARDG-P06 are both screened in the dacite and show similar chemistry, with elevated EC measurements. ARDG-DDH18 and ARDG-DDH20 have similar chemistry. These bores are located adjacent to one another, although they are in different geological units. ARDG-P02 and ARDG-DDH17 have very similar chemistry and low EC. These bores are also located close together, although they are in different units. The low EC at ARDG-P02 may be a result of the high degree of fracturing at this location, and potentially increased rainfall recharge through the fracture network.

The EC of groundwater in ARDG-P01 (5,820 µS/cm), located near Nine Mile Creek is much higher than in the creek itself (139 – 297 µS/cm at locations upstream and downstream of the project), which suggests the systems are unlikely to be connected.

The variability in EC, and elevated observations are consistent with monitoring undertaken by Boral and Eagleton in the Eagleton Volcanics.

Table 4.3 Groundwater quality monitoring

Analyte	Units	Limit of reporting	ARDG-DDH17	ARDG-DDH18	ARDG-DDH20	ARDG-P01	ARDG-P02	ARDG-P06
Physical parameters								
pH	-	0.01	6.86	7.83	7.11	6.87	5.64	6.94
EC	µS/cm	1	211	804	777	5820	198	5770
Major ions								
Sodium	mg/L	1	34	129	168	944	41	903
Potassium	mg/L	1	2	3	<1	3	<1	2
Calcium	mg/L	1	9	26	16	56	<1	183
Magnesium	mg/L	1	3	10	12	144	<1	184
Chloride	mg/L	1	34	195	155	1740	43	1800
Sulfate as SO4	mg/L	1	6	16	12	145	4	194
Hydroxide alkalinity as CaCO3	mg/L	1	<1	<1	<1	<1	<1	<1
Carbonate alkalinity as CaCO3	mg/L	1	<1	<1	<1	<1	<1	<1
Bicarbonate alkalinity as CaCO3	mg/L	1	58	143	237	242	28	475
Total alkalinity as CaCO3	mg/L	1	58	143	237	242	28	475
Nutrients								
Ammonia as N	mg/L	0.01	0.33	1	0.17	0.02	0.04	0.08
Nitrite as N	mg/L	0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
Nitrate as N	mg/L	0.01	0.01	0.03	0.02	3.44	<0.01	<0.01
Nitrite + Nitrate as N	mg/L	0.01	0.01	0.03	0.02	3.44	<0.01	<0.01
Total Kjeldahl Nitrogen as N	mg/L	0.1	3	6.2	8.2	0.3	20.2	0.3
Total Nitrogen as N	mg/L	0.1	3	6.2	8.2	3.7	20.2	0.3
Reactive Phosphorus as P	mg/L	0.01	<0.01	0.02	0.02	<0.01	0.01	<0.01
Total Phosphorus as P	mg/L	0.01	0.02	0.15	0.16	0.94	25.8	0.11

Analyte	Units	Limit of reporting	ARDG-DDH17	ARDG-DDH18	ARDG-DDH20	ARDG-P01	ARDG-P02	ARDG-P06
Dissolved metals								
Aluminium	mg/L	0.01	0.05	<0.01	0.01	<0.01	0.23	<0.01
Arsenic	mg/L	0.001	<0.001	<0.001	0.002	<0.001	<0.001	0.002
Cadmium	mg/L	0.0001	<0.0001	<0.0001	<0.0001	<0.0001	<0.0001	<0.0001
Chromium	mg/L	0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001
Copper	mg/L	0.001	<0.001	<0.001	<0.001	<0.001	0.002	<0.001
Iron	mg/L	0.05	0.31	0.07	0.87	<0.05	0.32	<0.05
Manganese	mg/L	0.001	0.034	0.047	0.104	0.45	0.007	0.589
Nickel	mg/L	0.001	0.003	<0.001	0.002	0.005	<0.001	<0.001
Lead	mg/L	0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001
Zinc	mg/L	0.005	0.019	<0.005	0.006	0.008	<0.005	<0.005
Mercury	mg/L	0.0001	<0.0001	<0.0001	<0.0001	<0.0001	<0.0001	<0.0001

4.3 Slug testing

Slug testing involves adding (or removing) a small slug of water to a well and monitoring the subsequent rise or fall of the water level. From these measurements the aquifer's transmissivity or hydraulic conductivity can be determined.

4.3.1 Falling head slug tests

ARDG conducted falling head slug tests on monitoring bores ARDG-DDH17, ARDG-DDH18, ARDG-DDH20, ARDG-P01, ARDG-P02 and ARDG-P06 on 23 August 2022. ARDG-P03, ARDG-P04 and ARDG-P05 were not tested because the water levels were either below or within the screened section of the monitoring bore. ARDG-DDH19 was not tested as the water levels appear to be influenced by surface water flow.

The slug test procedure involved introducing a 20 L slug into each monitoring bore and recording the water level at 20 second intervals. Only 11-12 L was introduced into ARDG-P01 as the water level reached the top of the borehole casing. The time taken to introduce the slug varied between 1.2 to 1.7 minutes, and the first water level measurement was taken at the two-minute mark. Ideally, the slug should be introduced rapidly into the borehole, and the water level recorded immediately after the slug has been added. Since the slug was introduced relatively slowly, it is likely the test was initiated non-instantaneously, however, once monitoring began at the two-minute mark, the data had already stabilised, so no corrections were required prior to interpretation.

Slug test data were analysed using the Bouwer-Rice (1976, 1989) method using pump test software Aqtesolv. Bouwer-Rice (1976, 1989) is the preferred semi-analytical method for analysing slug test data for a single-well slug test in an unconfined aquifer with a partially penetrating well. The slug test data for ARDG-P06 could not be validly interpreted as the introduced slug caused only a small initial change in head (0.075 m).

Interpreted slug test results are summarised in Table 4.4.

Plots and further details are included in Appendix B. The *recommended head ranges* reported by Butler (1998) were superimposed on the plots to obtain the most reliable matching results for the solutions (assuming a steady-state representation of flow for a slug test).

Table 4.4 Falling head slug test results

Bore	Hydraulic conductivity (m/day) Bouwer-Rice (1976)	Screened lithology
ARDG-DDH17	6.47×10^{-2}	Conglomerate, sandstone and siltstone
ARDG-DDH18	3.80×10^{-1}	Dacite and volcanic breccia
ARDG-DDH20	1.72×10^{-1}	Sandstone and siltstone
ARDG-P01	3.20×10^{-2}	Dacite
ARDG-P02	2.92×10^{-2}	Dolerite
ARDG-P06	Not interpreted	Dacite

The interpreted hydraulic conductivity estimates are within the range of literature values reported by Domenico and Schwartz (1990) for fractured igneous and metamorphic rocks (6.91×10^{-4} to 2.59×10^{1} m/day).

Slug test interpretation requires aquifer conceptualisation, including defining aquifer saturated thickness. For the slug test interpretation, it was assumed that the fractured rock aquifer extended to a depth of 60 m below ground level. This was based on the observed fracturing depth in boreholes located along the Central Fault zone. For example, strong to intense fracturing was observed at the following depths in ARDG-DDH04, ARDG-DDH05 and ARDH-DDH15:

– ARDG-DDH05 at 40 – 43 m, 50 – 54 m and 58 – 60 m
– ARDG-DDH04 at 54 – 55 m
– ARDG-DDH15 at 44.7 – 57 m

4.3.2 Rising head slug tests

ARDG conducted rising head slug tests on monitoring bores ARDG-P02 and ARDG-P06 on 1 December 2023. The slug test procedure involved removing (by airlifting) the entire volume of water within the monitoring bore and then recording the subsequent rise in the groundwater level. The first water level measurements were taken at the one-minute mark.

Slug test data were analysed using the Bouwer-Rice (1976, 1989) method using pump test software Aqtesolv. The slug test data for ARDG-P06 could not be validly interpreted as the majority of recovery occurred within the first minute of testing.

Interpreted slug test results are summarised in Table 4.5. Plots and further details are included in Appendix B.

Table 4.5 Rising head slug test results

Bore	Hydraulic conductivity (m/day) Bouwer-Rice (1976)	Screened lithology
ARDG-P02	9.2×10^{-3}	Dolerite
ARDG-P06	Not interpreted	Dacite

The interpreted hydraulic conductivity estimate is within the range of literature values reported by Domenico and Schwartz (1990) for fractured igneous and metamorphic rocks (6.91×10^{-4} to 2.59×10^{1} m/day).

4.3.3 Discussion

The rising head slug test result for ARDG-P02 (9.2×10^{-3} m/day) was almost an order of magnitude less than the falling head slug test result for the same monitoring bore (2.92×10^{-2} m/day). All monitoring bores were backfilled with 10 mm aggregate and the bentonite seal was placed at the top of the hole. The backfill would have a higher hydraulic conductivity compared to the fractured rock aquifer. It is likely that the falling head test may have over-estimated the hydraulic conductivity because for a period of time the introduced slug would be causing the water level within the backfill to rise above the initial groundwater level, rather than the slug directly entering the fractured rock aquifer. Therefore, the interpreted result from the rising head slug test is likely to be more representative of the fractured rock aquifer than the result obtained from the falling head test.

4.4 Pump testing

Based on water balance modelling results from the Surface Water Impact Assessment, ARDG requires 39 ML/year in the early stages of quarrying (i.e. Stage 1), assuming a very dry year (90th percentile bore import volume). Water is required for dust suppression, prior to the establishment of surface water capture. During Stage 1, approximately 2.5 L/s of groundwater will be required (assuming 12 hours pumping per day, over one year).

The rising head slug test carried out in ARDG-P06 (refer to Section 4.3) showed relatively rapid groundwater level recovery. Therefore, the area to the north-west of the quarry footprint was targeted for potential groundwater supply. A short duration pump test was conducted in an exploration hole near ARDG-P06 to determine the aquifer's hydraulic characteristics and to provide a preliminary indication of yield.

A 4.5 m PVC screen and small pump (Grundfos SQ7-40N) were installed in the 100 mm diameter exploration hole. A seven-hour pump test was conducted, followed by a one-hour recovery. The pump intake was set at 22.5 m depth, and the standing water level prior to the test was recorded at 6.09 m.

The flow rate for the pumping test varied between 2.4 L/s and 2.5 L/s for the initial 175 minutes of pumping. The pump stopped for approximately six minutes after 175 minutes, and then the test continued at a pump rate of 2.2 L/s. After seven hours pumping, the water level in the bore has been drawn down by 6.2 m, and the rate of drawdown had declined to 0.001 m per minute. Based on this rate, the extrapolated drawdown after 12 hours pumping is 6.54 m (based on a continued pumping rate of 2.2 L/s).

Pump test data was analysed using pump test software Aqtesolv. An average pumping rate of 2.4 L/s was used. As a result of the pump test stopping for six minutes, curve matching was weighted to the data prior to 175 minutes and the recovery period. Given that the fractured rock aquifer in this immediate area is confined (refer Section 5) and the pump test duration was relatively short, the aquifer was modelled as confined.

Pumping test results are summarised in Table 4.6 and the best match is highlighted in bold.

Plots and further details are included in Appendix C.

Table 4.6 ARDG-P06 pump test details and results

Method	Model	Transmissivity (m^2/day)	Comment
Theis/Hantush	Confined	12.06	Reasonable match to late time data prior to 175 minutes, and recovery.
Cooper-Jacob	Confined	11.61	Good match to late time data prior to 175 minutes.
Papadopulos-Cooper	**Confined**	**9.67**	**Good match to late time data prior to 175 minutes, and recovery.**

Based on an aquifer saturated thickness of 20 m and the transmissivity values shown in Table 4.6, the hydraulic conductivity of the fractured dacite in this area varies between 0.48 m/day and 0.60 m/day. These results are of the same order of magnitude as the interpreted slug test result at ARDG-DDH18 (0.38 m/day), which is also located to the north-west of the quarry footprint.

The main disadvantage of a short-term pumping test is that the results need to be extrapolated to estimate the long-term yield of the bore over months and years. The accuracy of the extrapolation can be affected by the intersection of aquifer boundary conditions during long term pumping. For example, yield may decline once the radius of influence of the pumping bore intersects the less transmissive rocks within the quarry footprint.

A forward analysis was carried out using Aqtesolv based on the Theis/Hantush method to predict the likely drawdown in a 100 mm diameter production bore, pumping at 1.25 L/s for 365 days (assuming continuous pumping, 24 hours per day). An aquifer transmissivity of 10 m^2/day (best estimate, refer Table 4.6) and saturated thickness of 20 m were assumed. True aquifer thickness is likely greater, however reducing the aquifer thickness allows for a more realistic drawdown prediction since the production bore only partially penetrates the aquifer. This extrapolation does not consider the intersection of aquifer boundary conditions as previously discussed.

Approximate drawdown and the percentage of available drawdown are shown in Table 4.7 for storativity values 0.1 m/m and 0.01 m/m. Storativity values for unconfined aquifers are typically between 0.1 and 0.3, and for the majority of the project site, the aquifer is considered unconfined. A storativity value one order of magnitude lower has also been modelled to consider the confined nature of the aquifer in the immediate vicinity of the potential production bore (refer Section 5).

Plots and further details are included in Appendix C.

Table 4.7 Forward solution based on Q = 1.25 L/s and T = 10 m^2/day

Storativity (m/m)	Approximate drawdown at 365 days (m)	Percent of available drawdown[1]
0.1	14	85%
0.01	16	98%

Note: 1 Available drawdown is 16.41 m assuming pump intake positioned at 22.5 m depth.

Depending on the efficiency of the production bore, and actual aquifer parameters (confirmed by a longer duration pumping test in the actual production bore), it is possible that 1.25 L/s continuous pumping could be sustainable for at least a year in the fractured rock aquifer. For a storativity of 0.01, the available drawdown is almost exhausted by one year. These results however are based on a slotted PVC test bore which only extends to 27.5 m depth. The design of the actual production would aim to maximise the efficiency (i.e. reduce drawdown) of the bore by optimising depth, screen length, screen interval and slot aperture. Therefore, it is still possible that 1.25 L/s could be sustained, even with a lower storativity value.

The forward analysis is limited to continuous pumping. If the bore was pumped at 2.5 L/s for 12 hours only, a 12-hour recovery would follow each pumping period. Recovery data was only collected for one hour and therefore it is uncertain as to whether groundwater levels would completely recover before the next pumping period started. Extrapolation of the data however suggests full recovery in 5.5 hours. This is based on an average pumping rate of 2.4 L/s over a seven-hour pumping duration. The extrapolated drawdown after 12 hours pumping was only 6.54 m (compared with 6.2 m after seven hours). Even though the pump rate had reduced to 2.2 L/s towards the end of the pumping test, it is still likely that if the bore was pumped at 2.5 L/s for 12 hours, it would completely recover in the following 12-hour recovery period.

Therefore, based on the results of a pumping test conducted near ARDG-P06 and forward analyses, continuous pumping at 1.25 L/s or discontinuous pumping at 2.5 L/s (assuming complete recovery following each 12-hour pumping period), is likely to be sustainable for at least a year in the fractured rock aquifer. Therefore the 90th percentile bore import volume is likely to be obtained from a production bores targeting the fractured rock aquifer in the ARDG-P06 area.

5. Conceptual hydrogeological model

The conceptual model is based on groundwater monitoring data, lithology logs and core photographs provided by ARDG, interpreted geology, and previous hydrogeological assessments for the nearby Eagleton (proposed) and Seaham quarries.

The groundwater flow systems occur in unconfined and confined fractured rock aquifers within the Eagleton Volcanics and within shallow, perched aquifers. Where fractured rock unit outcrops (for example at ARDG-P05, ARDG-DDH20, and most of the quarry area) and where surficial clay layers are thin (less than 1 m), the aquifer is likely unconfined. To the south-east of the project area, depending on the lateral continuity of the clay layer, the fractured rock may behave as a confined aquifer. On the lower western flanks of Stone Ridge, low permeability dacite acts as an aquitard, confining the fractured rock aquifer in this area. Shallow, perched aquifers overlying the dacite aquitard also occur in the lower western flank area.

The conceptual hydrogeological model consists of five hydrostratigraphic layers:

– Clay layer, up to 11 m thick, discontinuous across the project area
– Shallow perched aquifer system (on the lower western flanks)
– Low permeability dacite aquitard (on the lower western flanks)
– Unconfined fractured rock aquifer (Eagleton Volcanics)
– Confined fractured rock aquifer (Eagleton Volcanics)

The volcanic and sedimentary sequences within the Eagleton Volcanics which form the fractured rock aquifer have been considered as one hydrostratigraphic unit. The hydraulic conductivity of this unit varies spatially within several orders of magnitude (EMM 2019, URS 2014).

Groundwater levels in the fractured rock aquifer reflect and are controlled by topography, with Stone Ridge and South Ridge acting as local groundwater divides. Groundwater is recharged by rainfall infiltration on the upper slopes, ridgelines and hilltops in the landscape where the rock sub-crops or outcrops. Groundwater flows from topographically elevated areas towards lower lying areas and drainages where is it discharged.

The shallow, perched aquifers on the western flank which overly the dacite aquitard are recharged via rainfall, surface runoff and shallow water movement from upslope within the regolith and soil material. Flow is topographically controlled, and isolated from the deeper, fractured rock groundwater system.

Average groundwater levels in the fractured rock aquifer range from 12.7 m AHD at ARDG-P06 to 28.2 m AHD at ARDG-P02. On the lower western flanks of the main ridge, average groundwater levels (as recorded as standing water levels) do not reflect the actual depth to groundwater. The actual depth to groundwater within the fractured rock aquifer is approximately 20 m below surface which is greater than the standing water level recorded at ARDG-P06 (7.3 m below surface) due to the confining effect of the low permeability dacite.

Nine Mile Creek, Balickera Channel and the tributary of Caswell’s Creek located to the north-west of the project are likely too high in elevation relative to the groundwater levels to be points of discharge for groundwater from within the project area. The project is therefore unlikely to have an impact on aquatic GDEs or baseflow, as the groundwater elevation is already below the creeks in the area. The EC of groundwater in ARDG-P01 located near Nine Mile Creek is much higher than in the creek itself (139 – 297 µS/cm at locations upstream and downstream of the project), which also suggests the systems are unlikely to be connected.

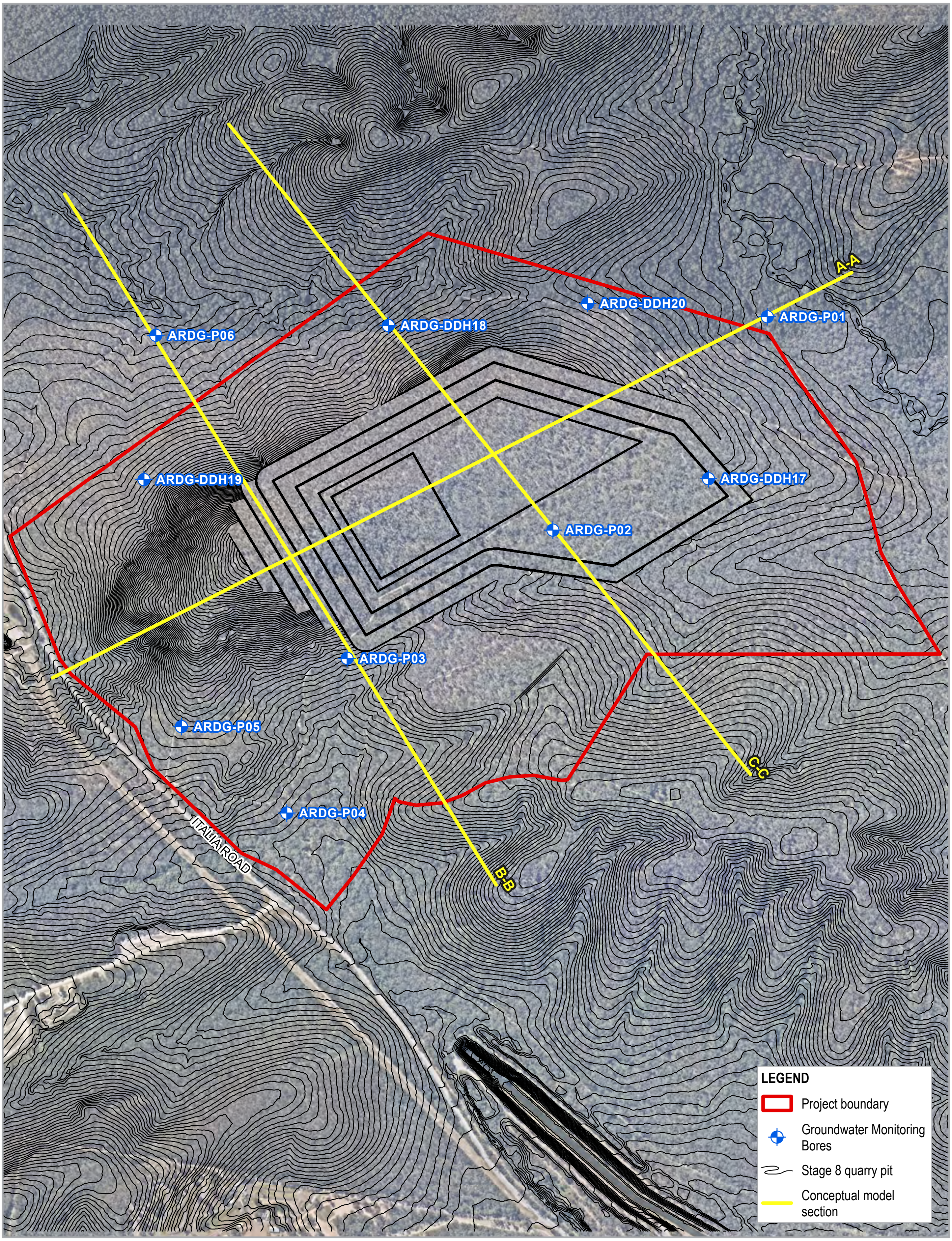
The groundwater level elevation within the fractured rock aquifer at the project is higher than the registered private landholder bores located to the north-west. The private bores are therefore located hydraulically down-gradient of the project.

The Eagleton Volcanics has a very low primary porosity with groundwater flow occurring within secondary porosity features such as fractures and contact boundaries of igneous dykes. In the project area, the fractured rock aquifer has been assumed to extend to approximately 60 m below ground level. The majority of fracturing in the project area has been observed along the Central Fault zone.

Groundwater quality in the project area is highly variable. This likely relates to the differences in geology and the degree of fracturing within the aquifer systems. The EC values measured at the project (198 to 5,820 µS/cm) are within the EC range observed at the nearby Seaham Quarry (3,200 to 10,000 µS/cm) (EMM 2019) and at the proposed Eagleton Quarry (465 – 6,060 µS/cm) (URS 2014).

The conceptual hydrogeological model is shown as three cross sections (A-A, B-B and C-C) through the project area. Cross section locations and the proposed pit are shown in Figure 5.1. Cross section A-A trends northeast to southwest, extending past Nine Mile Creek. Cross Section B-B trends northwest to southeast, through ARDG-P03 and ARDG-P06. Cross Section C-C trends northwest to southeast, through ARDG-DDH18 and ARDG-P02. Cross sections are shown in Figure 5.2, Figure 5.3 and Figure 5.4.

The Stage 8 quarry pit shell is shown in all cross-section figures. The groundwater levels shown in Figure 5.2, Figure 5.3 and Figure 5.4 represent pre-quarrying conditions, based on average groundwater levels (blue dashed line). During development, groundwater inflow into the quarry pit will cause groundwater drawdown in the vicinity of the project. Predicted groundwater level drawdown (based on the conservative potential radius of influence of 468 m, refer Section 6.1.5.1) at the end of Stage 8 development is shown as a red dashed line in Figure 5.2, Figure 5.3 and Figure 5.4. Predicted groundwater level drawdown (based on the conservative potential radius of influence of 389 m, refer Section 6.1.5.2) in the vicinity of high probability GDE’s at the end of Stage 8 development is shown as a purple dashed line in Figure 5.3 and Figure 5.4 only.



Paper Size ISO A4

0 100 200 300

Metres

Map Projection: Transverse Mercator
Horizontal Datum: GDA 1994
Grid: GDA 1994 MGA Zone 56

N

GHD

Australian Resource Development Group Pty Ltd
Stone Ridge Quarry
Groundwater Impact Assessment

Conceptual Model
Section Locations

Project No. 22-19467
Revision No. 1
Date 16/02/2024

FIGURE 5.1

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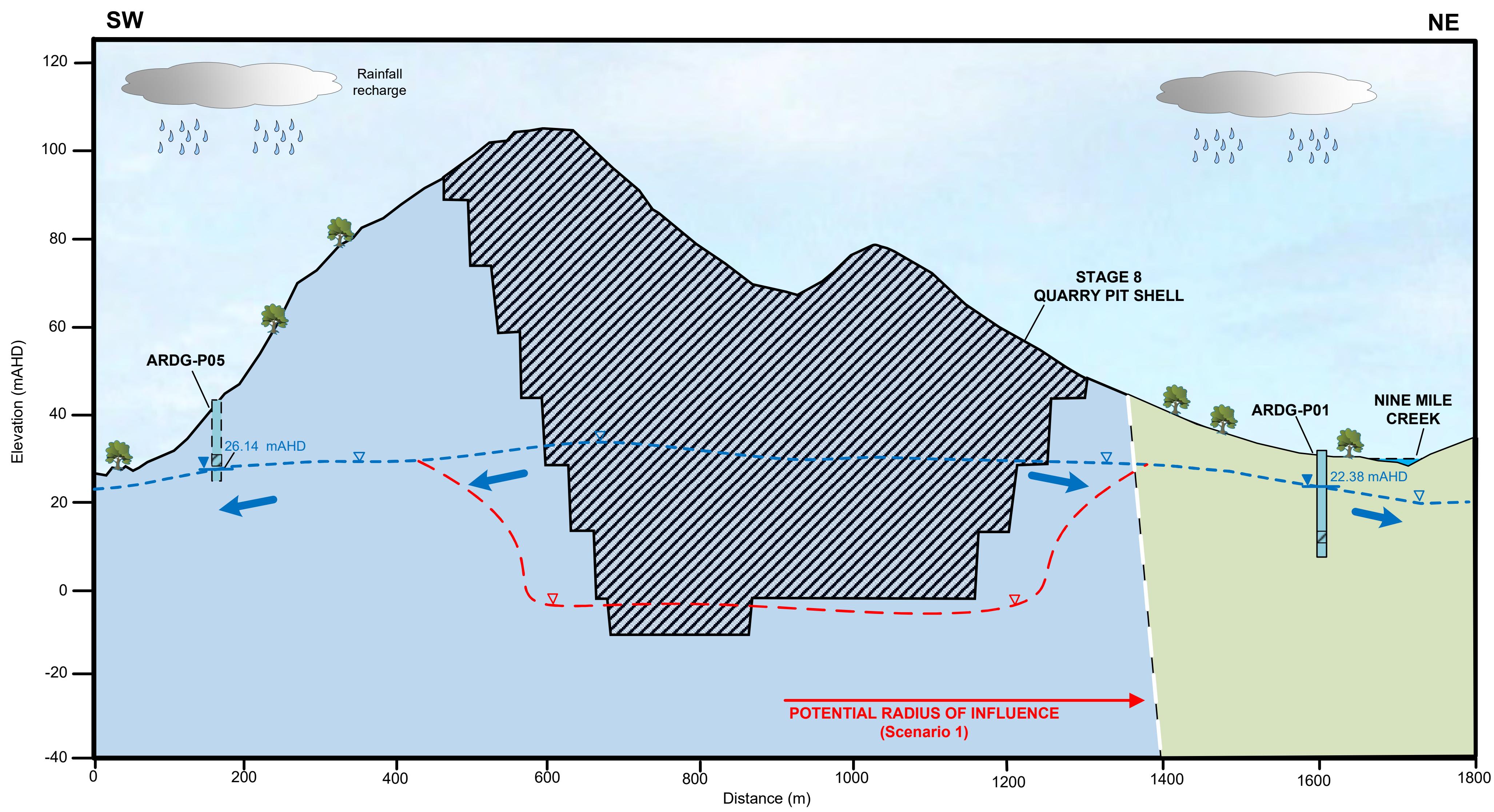
Data source: Groundwater bores, project boundary - ARDG 2022. General topography - LPI 2017. Nearmap WMS Server: . Created by: tmorton



Interpreted groundwater level

Measured groundwater level

Interpreted groundwater level (1)

Groundwater flow

Monitoring bore

Interpreted geology

Rhyodacite

Dacite

GHD

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Conceptual cross section
A - A

Project No. **22-19467**
Revision No. **2**
Date **22/02/2024**

FIGURE 5.2

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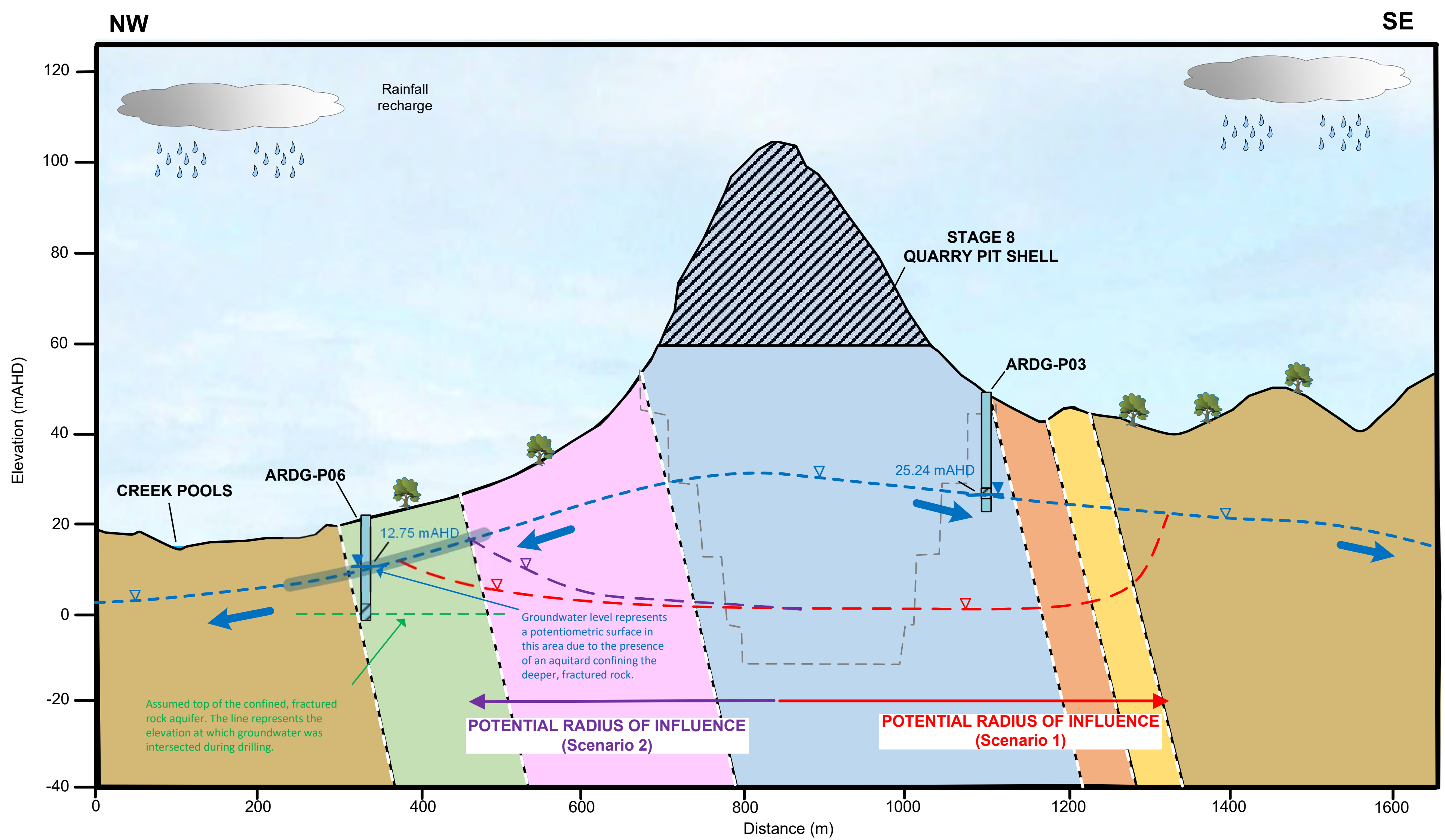

Interpreted groundwater level	Interpreted geology
Measured groundwater level	Dacite
Interpreted groundwater level (1)	Volcanic breccia
Interpreted groundwater level (2)	Rhyodacite
Maximum pit depth	Tuffaceous sediments
Groundwater flow	Rhyolitic tuff
Monitoring bore	

GHD

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Groundwater Impact Assessment

Conceptual cross section
B - B

Project No. **22-19467**
Revision No. **2**
Date **22/02/2024**

FIGURE 5.3a

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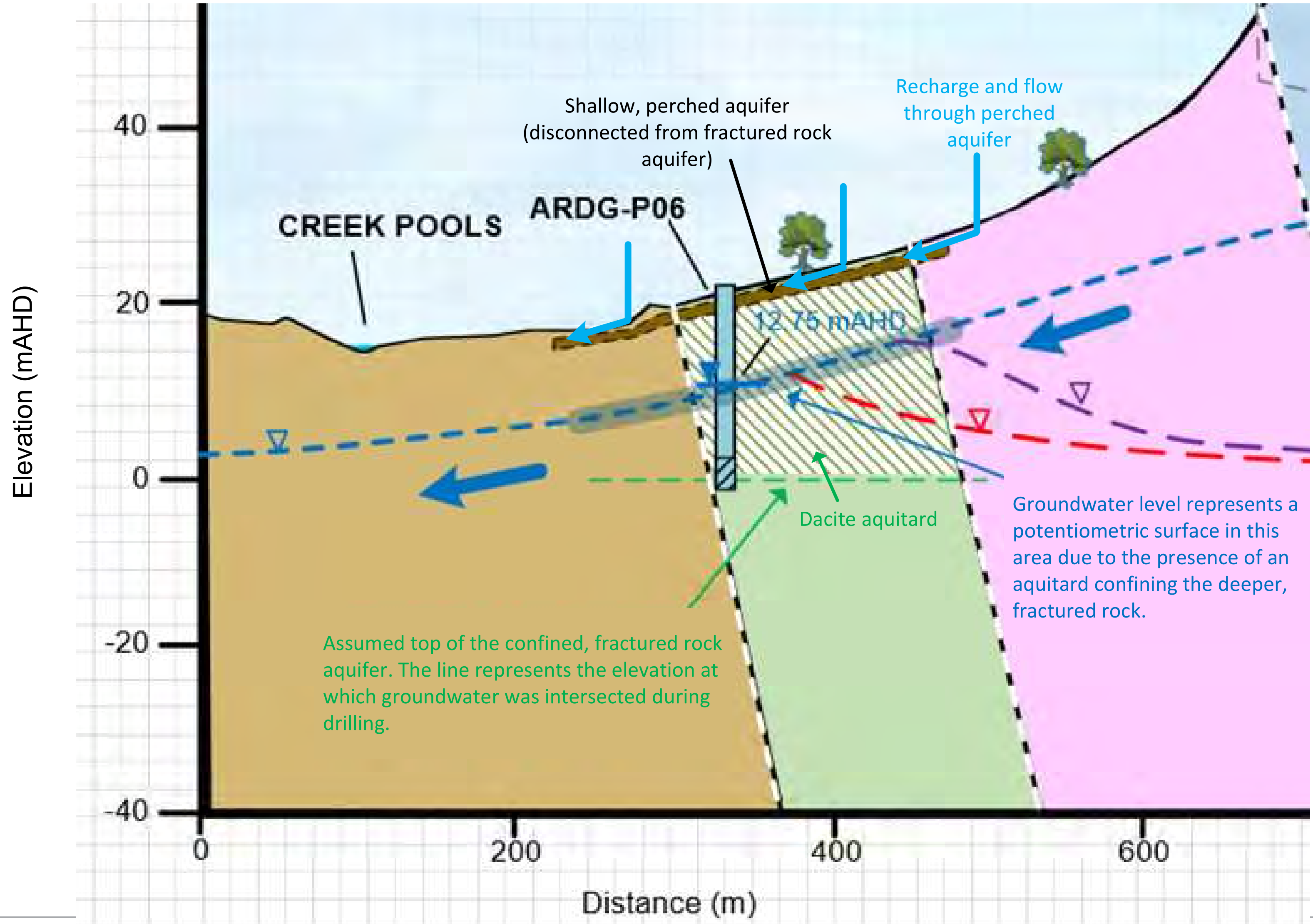

Interpreted groundwater level
Measured groundwater level
Interpreted groundwater level (1)
Interpreted groundwater level (2)

Interpreted geology
Dacite
Volcanic breccia
Rhyodacite

Tuffaceous sediments
Rhyolitic tuff
Groundwater flow
Monitoring bore

GHD

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Conceptual cross section
B – B inset

Project No. **22-19467**
Revision No. **2**
Date **22/02/2024**

FIGURE 5.3b

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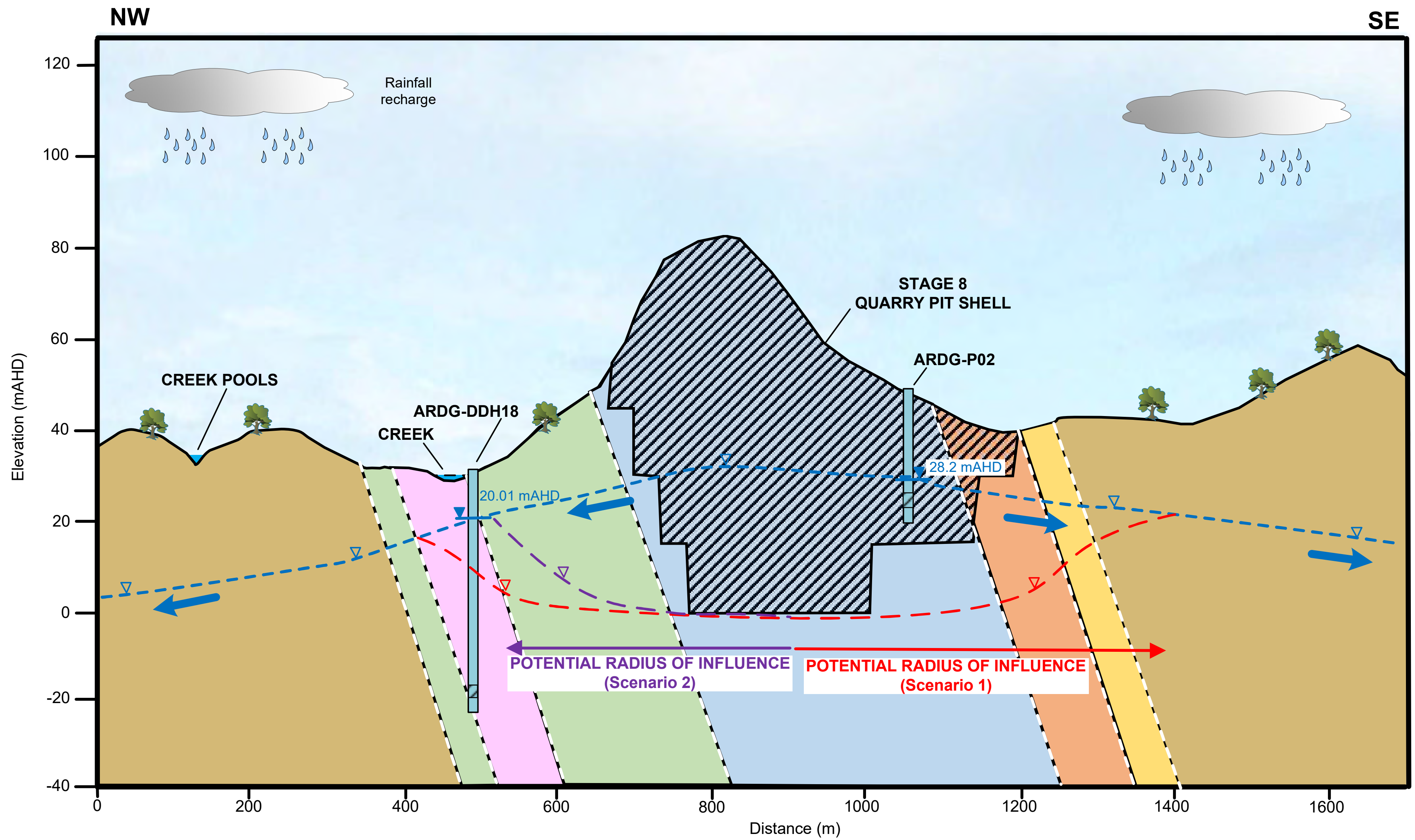

Interpreted groundwater level
Measured groundwater level
Interpreted groundwater level (1)
Interpreted groundwater level (2)

Interpreted geology
- Dacite
- Volcanic breccia
- Rhyodacite
- Tuffaceous sediments
- Rhyolitic tuff

Groundwater flow
Monitoring bore

Australia Resource Development Group Pty Ltd
Stone Ridge Quarry
Groundwater Impact Assessment

Conceptual cross section C - C

Project No. 22-19467
Revision No. 2
Date 22/02/2024

FIGURE 5.4

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6. Impact assessment

Extraction from the quarry will occur progressively over several stages (ARDG 2020). Groundwater impacts will therefore be assessed up to and including Stage 8 of the project. Groundwater impacts from a potential production bore will also be assessed.

6.1 Prediction of groundwater inflow and drawdown

6.1.1 Method

An assessment of likely groundwater inflow rates and the radius of drawdown was undertaken using the Marinelli and Niccoli (2000) steady-state analytical model. Groundwater inflows will occur from the fractured rock aquifer. Modelling is not required for the shallow, perched groundwater because the flow system is disconnected from the fractured rock aquifer in which the quarry operations will occur. Potential impacts to the perched groundwater system are discussed in Section 6.4.2.

Marinelli and Niccoli (2000) model calculates groundwater inflow to a mine pit excavated below the water table. The flow area is divided into two zones. Zone 1 represents flow to the pit wall and Zone 2 considers flow to the base of the pit. Groundwater inflows were calculated for Zone 1 and Zone 2 using the following equations:

$$Q_1 = W\pi(r_0^2 - r_p^2)$$

$$Q_2 = 4r_p\left(\frac{K_{h2}}{m_2}\right)(h_0 - D)$$

$$m_2 = \sqrt{\frac{K_{h2}}{K_{v2}}}$$

The radius of drawdown was determined via iteration of the following equation:

$$h_0 = \sqrt{h_p^2 + \frac{W}{K_{h1}}\left[r_0^2 ln\left(\frac{r_0}{r_p}\right) - \frac{(r_0^2 - r_p^2)}{2}\right]}$$

Where:

- $Q1$ inflow from the walls (m^3/day)
- $Q2$ inflow from the base (m^3/day)
- W distributed recharge flux
- r_0 radius of drawdown (m)
- r_p effective pit radius (m)
- K_{h1} horizontal hydraulic conductivity value for the aquifer in Zone 1
- K_{h2} horizontal hydraulic conductivity value for the aquifer in Zone 2
- K_{v2} vertical hydraulic conductivity value for the aquifer in Zone 2
- h_0 saturated thickness of the aquifer, based on the average groundwater level in the unconfined fractured rock aquifer
- h_p saturated thickness above the base of Zone 1
- D depth of water in the base of the pit

The assumptions of the Marinelli and Niccoli (2000) analytical model include:

- The aquifer is unconfined
- Lowering of the water table reduces the saturated thickness of the surrounding aquifer
- Relative to seepage through pit walls, significant inflow occurs through the pit bottom
- There is no impermeable boundary at depth

– Steady stage flow conditions exist near the pit

For Zone 1 the analytical solution considers steady-state, unconfined, horizontal radial flow with uniformly distributed recharge. The solution is also based on the following assumptions:

– Walls are approximated as a circular cylinder.
– Groundwater flow is horizontal and the Dupuit-Forchheimer approximation is used to account for changes in saturated thickness due to reduction of the water table.
– The static (pre-mining) water table is approximately horizontal.
– Uniform distributed recharge occurs across the site because of surface infiltration from rainfall and all recharge within the radius of drawdown of the pit is assumed to be captured by the excavation.
– Groundwater flow toward the pit is axially symmetric.

For Zone 2 the analytical solution is based on steady-state flow to one side of a circular disc sink of constant and uniform drawdown. The sink represents the bottom of the pit. The solution is also based on the following assumptions:

– Hydraulic head is initially uniform throughout Zone 2.
– Initial head is equal to the elevation of the initial water table in Zone 1.
– The disc sink has a constant hydraulic head equal to the elevation of the pit lake water surface. If the pit is completely dewatered the disk sink head is equal to the elevation of the pit bottom.
– Flow to the disc sink is three dimensional and axially symmetric.

The analytical model is illustrated in Figure 6.1.

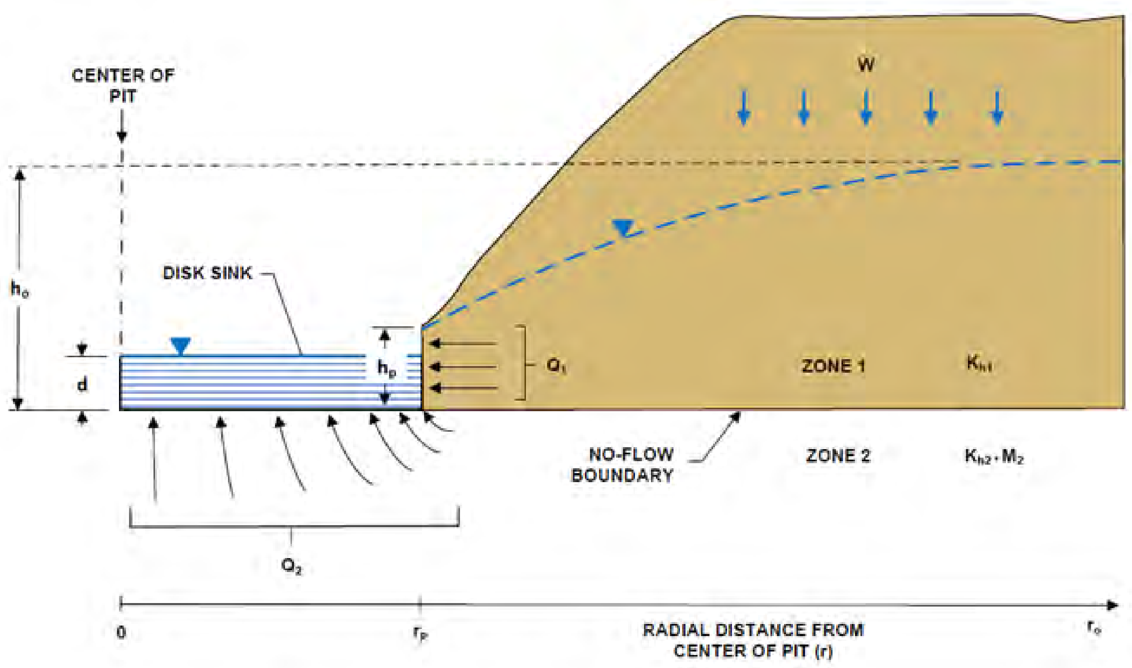


Figure 6.1 *Conceptual analytical model*

6.1.2 Justification for using an analytical model

Considering that the distance to the few registered landholder bores is greater than one kilometre, the low hydraulic conductivity of the aquifer and the lack of GDEs due to deep groundwater levels through the rhyodacite resource, it is considered that the risk to identified groundwater receptors due to the project is low. Therefore, the level of complexity of analytical equations is appropriate to assess this risk.

Additionally, Boral Quarries Seaham has operated in the vicinity of the project since 1991 with minimal groundwater encountered (based on aerial photographs) and minimal impact to the groundwater environment.

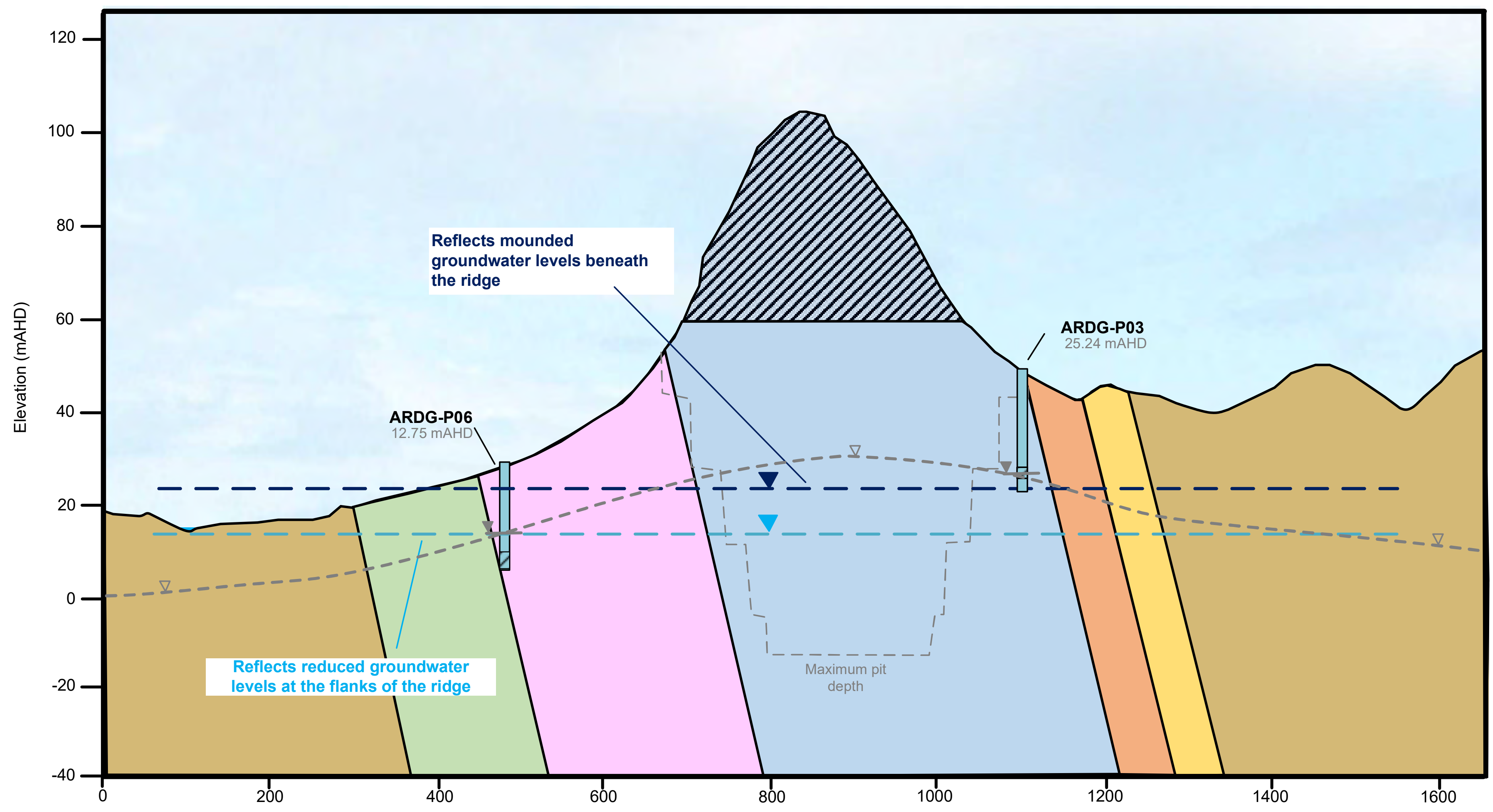
6.1.3 Scenarios

Within the quarry footprint, groundwater levels are relatively high, reflecting the elevated topography of Stone Ridge (i.e. a mounded groundwater level beneath the ridge). Outside the quarry footprint (Stage 8 quarry pit, refer to Figure 5.1), groundwater levels are reduced, reflecting the decline in elevation. The Marinelli and Niccoli (2000) method however assumes the initial groundwater level is approximately horizontal (refer Section 6.1.1) and therefore to apply this method to the project's hydrogeology, two groundwater level scenarios have been modelled.

Scenario 1 is a conservative approach. The initial groundwater level for Scenario 1 is based on the average groundwater level observed within the quarry footprint, which is elevated compared to the surrounding terrain. Scenario 1 is considered to over-estimate the magnitude of drawdown at locations outside of the quarry footprint, i.e. in the area surrounding ARDG-P06 and in the lower western flanks of Stone Ridge.

Scenario 2 is a modified approach. The initial groundwater level for Scenario 2 is based on a reduced groundwater level which reflects the lower topography in the area surrounding the quarry footprint. This scenario was modelled to refine drawdown predictions in the lower western flank areas where high probability GDEs are located. This scenario adopts the most conservative hydraulic conductivity value only (i.e. K_{max} = 9.2 x 10^{-3} m/day, refer Table 6.1).

Scenario 1 and Scenario 2 are illustrated in Figure 6.2.



Interpreted groundwater level

Scenario 1 initial groundwater level (23.40 m AHD)

Scenario 2 initial groundwater level (12.75 m AHD)

Monitoring bore

Australia Resource Development Group Pty Ltd
Stone Ridge Quarry
Groundwater Impact Assessment

Scenario 1 and Scenario 2

Project No. 22-19467
Revision No. A
Date 15/02/2024

FIGURE 6.2

Created by: MKay

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6.1.4 Analytical inputs

Analytical inputs are summarised in Table 6.1.

Table 6.1 Analytical inputs

Input	Value	Comment
Horizontal hydraulic conductivity (m/day)	K_{10} = 9.3 x 10^{-4}	Assuming 10% fractured rhyodacite resource
	K_{20} = 1.9 x 10^{-3}	Assuming 20% fractured rhyodacite resource
	K_{max} = 9.2 x 10^{-3}	ARDG-P02 rising head slug test interpretation
Vertical hydraulic conductivity	10% horizontal hydraulic conductivity	
Initial groundwater level (m AHD)	23.40 (Scenario 1)	Average groundwater level elevation excluding ARDG-P06 and ARDG-P04
	12.75 (Scenario 2)	Average groundwater level at ARDG-P06
Rainfall recharge (mm/year)	45.16 (average annual rainfall 1129 mm/year)	4% average annual rainfall (DPI 2016a)

Hydraulic conductivities estimated from slug tests are only characteristic of a small volume of aquifer material surrounding the well and it is likely that variability in hydraulic conductivity across the project is greater than the three orders of magnitude observed, particularly since only one falling head and one rising head slug test, at ARDG-P02, was performed in the rhyodacite resource.

ARDG-P02 is screened in a dolerite intrusion in the Central Fault zone. This monitoring bore, and neighbouring borehole ARDG-DDH05, were drilled into the most fractured and altered rhyodacite observed across the project. The rhyodacite resource has been extensively drilled (refer Figure 3.5) with the majority of boreholes showing competent rock with minimal fracturing. The interpreted hydraulic conductivity at ARDG-P02 from the rising head slug testing was assumed to represent the maximum expected hydraulic conductivity through the rhyodacite resource (K_{max} = 9.2 x 10^{-3} m/day). Using this value as the upper hydraulic conductivity value assumes that the entire rhyodacite resource is as fractured and altered as the rhyodacite observed at ARDG-P02 and ARDG-DDH05. This is highly conservative given that the majority of boreholes through the resource show competent rock.

Most of the fractured rock occurs along and adjacent to the Central Fault zone (e.g., ARDG-DDH04, ARDG-DDH05 and ARDG-DDH15). Given this area accounts for approximately 10% of the entire rhyodacite resource, more likely values of hydraulic conductivity were estimated. Assuming 10% and 20% fracturing of the rhyodacite resource, the fractured portion was assigned a hydraulic conductivity of 9.2 x 10^{-3} m/day (K_{max}) and the competent rhyodacite a hydraulic conductivity of 8.64 x 10^{-6} m/day (average of the unfractured metamorphic and igneous rock hydraulic conductivities presented by Domenico and Schwartz (1990)). Calculated hydraulic conductivities are shown in Table 6.1.

Vertical hydraulic conductivity was assumed to be equal to 10% of the horizontal hydraulic conductivity.

For Scenario 1, the average groundwater level elevation was adopted as the initial groundwater level. The average excluded the groundwater level elevation at monitoring bores ARDG-P06 and ARDG-P04 as these bores have much lower groundwater levels due to lower topographic elevation at these sites. These bores are also located further away from the quarry footprint. For Scenario 2, the average groundwater level at ARDG-P06 was adopted as the initial groundwater level.

Rainfall recharge was assumed to be 4% average annual rainfall, which is consistent with the WSP.

Pit floor and sump levels are summarised in Table 6.2 for each stage. For Scenario 1, the base of the pit floor for Stage 1 to Stage 4 is above the initial groundwater level. Minimal groundwater inflow is expected for these stages based on the average groundwater level. Where the initial water level is above the pit floor level, the saturated thickness of the aquifer above the pit floor level has been calculated and is shown in Table 6.2.

To apply the Marinelli and Niccoli (2000) model to the project, the quarry was represented as an open pit in the shape of a circular cylinder. The pit area in plan was calculated for each stage, based on approximate pit dimensions at RL27. The approximate pit area was then used to calculate an effective pit radius, as shown in Table 6.2. The plan dimensions of the quarry are not symmetrical and therefore representing the quarry as a circular cylinder is a significant simplification.

Table 6.2 Pit floor, sump level and aquifer thickness above the base of pit floor for each pit stage

Pit stage	Pit floor level (m RL)	Pit sump level (m RL)	Approximate pit area[1] (m^2)	Effective pit radius r_p (m)	Scenario 1 - aquifer saturated thickness above base of pit floor[2] (m)	Scenario 2 - aquifer saturated thickness above base of pit floor[3] (m)
Stage 1	43	28	-	-	0	0
Stage 2	43	28	-	-	0	0
Stage 3	43	28	-	-	0	0
Stage 4	43	28	-	-	0	0
Stage 5	13	-	36,225	107.4	10.4	0
Stage 6	13	-2	81,225	160.8	10.4	0
Stage 7	-2	-10	169,650	232.4	25.4	14.75
Stage 8	-2	-10	227,400	269.0	25.4	14.75

Notes:
1 Based on average pit dimensions at RL27
2 Based on initial groundwater level 23.4 m AHD
3 Based on initial groundwater level of 12.75 m AHD

6.1.5 Results

6.1.5.1 Scenario 1

Groundwater inflow and the radius of drawdown from the centre of the pit for Stages 5, 6, 7 and 8 are shown in Table 6.3 and Table 6.4 respectively.

Table 6.3 Groundwater inflow

Stage	Groundwater inflow (ML/year)		
	K_{max} = 9.2 x 10^{-3} m/day	K_{10} = 9.3 x 10^{-4} m/day	K_{20} = 1.9 x 10^{-3} m/day
5	8.31 (0.02 ML/day)	1.43 (0.004 ML/day)	2.35 (0.01 ML/day)
6	12.02 (0.03 ML/day)	2.13 (0.01 ML/day)	3.45 (0.01 ML/day)
7	43.74 (0.12 ML/day)	7.55 (0.02 ML/day)	12.36 (0.03 ML/day)
8	49.82 (0.14 ML/day)	8.67 (0.02 ML/day)	14.30 (0.04 ML/day)

Table 6.4 Radius of drawdown

Stage	Radius of drawdown (m)		
	K_{max} = 9.2 x 10^{-3} m/day	K_{10} = 9.3 x 10^{-4} m/day	K_{20} = 1.9 x 10^{-3} m/day
5	190	135	146
6	245	189	200
7	430	299	325
8	468	336	363

The Marinelli and Niccoli (2000) method assumes that the aquifer is laterally infinite, and drawdown will propagate until equilibrium is reached between discharged water and rainfall recharge. The method does not consider the presence of a zero-recharge aquifer boundary which would limit the radius of drawdown. If the radius of drawdown reaches a zero-recharge boundary close to the project, inflows would be significantly less. The Marinelli and Nicolli (2000) method does not consider groundwater storage.

The Marinelli and Niccoli (2000) method does not consider any increased recharge that may occur as a result of the quarry activities. During development, it has been assumed that any rainfall or runoff presenting to the pit would be removed via a stormwater collection and pumping system. Any enhanced recharge that occurs as a result of the quarry in the post closure phase would reduce the time required for groundwater levels to recover.

6.1.5.2 Scenario 2

The Marinelli and Niccoli (2000) model defined in Section 6.1.1 was used to calculate the radius of drawdown assuming an initial water level of 12.75 m AHD. Based on the most conservative hydraulic conductivity value (K_{max} = 9.2 x 10^{-3} m/day), the predicted radius of drawdown for Stage 8 of the development is approximately 389 m.

This scenario was modelled to refine drawdown predictions in the lower western flank areas where high probability GDEs are located. Predicted inflows are not reported for this scenario. The inflow rate associated with Stage 8, K_{max} however was used to determine the drawdown at a particular distance from the centre of the pit, using an analytical model developed by GHD. This model is based on the non-equilibrium Theis (1936) solution to the well equation and assumes the following:

- Pit penetrates the entire aquifer thickness.
- Discharge is instantaneous with decline in pressure.
- Flow is radial, horizontal and laminar.
- The aquifer is homogenous and isotropic.
- Aquifer thickness is uniform.
- The aquifer remains saturated during pumping.
- The aquifer is infinite (in areal extent, no areal boundaries).
- The aquifer is horizontal and bounded above and below by impermeable beds (the aquifer is confined).
- All storage of water within the aquifer comes from the cone of depression (the aquifer is isolated from overlying or underlying leaky aquifers, local recharge, precipitation, irrigation, rivers, lakes and wetlands).

Analytical inputs are summarised in Table 6.5.

Table 6.5 Analytical input

Input	Value	Comment
Transmissivity	0.8 m^2/day	Interpreted, assuming zero drawdown at 400 m from the centre of the pit.
Storativity	0.1 m/m	Unconfined aquifer, typical values 0.1 – 0.3. The aquifer is unconfined within the quarry footprint.
Pumping bore minimum radius	125 m	Width of pit floor is approximately 250 m in the direction of high probability GDE.
Aquifer thickness	30 m	True aquifer thickness is greater, however reducing the aquifer thickness allows for a more realistic drawdown since the pit only partially penetrates the aquifer.
Pumping rate	1 L/s	Based on inflow calculations for Scenario 2 (Stage 8, K_{max}).

Results are shown in Table 6.6 for the end of Stage 8, from 200 m distance from the centre of the pit.

Table 6.6 Drawdown versus distance from the centre of the pit

Distance from the centre of the pit (m)	Drawdown (m)
200	3.47
250	1.76
275	1.25
300	0.86
320	0.61
340	0.43
360	0.27
380	0.12
400	0.00

Based on the analytical modelling, drawdowns greater than one metre are not expected to occur at distances exceeding 300 m from the centre of the pit.

6.2 Prediction of drawdown from pumping bore

To determine the extent of drawdown associated with a potential production bore pumping continuously at 1.25 L/s for one year, an analytical model developed by GHD was used. This model is based on the non-equilibrium Theis (1936) solution to the well equation. Model assumptions were previously outlined in Section 6.1.5.2.

Analytical inputs are summarised in Table 6.7.

Table 6.7 Analytical input

Input	Value	Comment
Transmissivity (T)	10 m^2/day	From pump test analysis, refer Section 4.4.
Storativity	0.1 m/m 0.01 m/m	Unconfined aquifer, typical values 0.1 – 0.3. For the majority of the project site, the aquifer is considered unconfined. A storativity value one order of magnitude lower has also been modelled to consider the confined nature of the aquifer in the immediate vicinity of the potential production bore.
Pumping bore minimum radius	0.077 m	Hole diameter 154 mm.
Pumping rate	1.25 L/s	Required yield, assuming continuous pumping, refer Section 4.4.
Pump inlet depth	22.5 m	From pumping test, refer Section 4.4.
Standing water level	6.09 m	From pumping test, refer Section 4.4.

Predicted drawdowns at 365 days are summarised in Table 6.8.

Table 6.8 Distance – drawdown

Distance from pumping bore (m)	Drawdown after 365 days S = 0.1	Drawdown after 365 days S = 0.01
20	4.58	6.55
40	3.39	5.36
60	2.71	4.67
80	2.23	4.17
100	1.87	3.79
120	1.58	3.48
140	1.34	3.22

Distance from pumping bore (m)	Drawdown after 365 days S = 0.1	Drawdown after 365 days S = 0.01
160	1.14	2.99
180	0.98	2.80
200	0.84	2.62
220	0.72	2.46
240	0.61	2.32
260	0.52	2.18
280	0.44	2.06
300	0.38	1.95
320	0.32	1.85
340	0.27	1.75
360	0.23	1.66
380	0.19	1.58
400	0.16	1.50
450	0.10	1.32
500	0.06	1.16
550	0.03	1.03
600	0.01	0.91
700	0.00	0.71
800	0.00	0.55
900	0.00	0.43
1,000	0.00	0.33
1,300	0.00	0.15
1,900	0.00	0.01

For a storativity of 0.1 m/m, the predicted radius of influence is approximately 600 m. Drawdowns greater than one metre are not expected to occur at distances exceeding 180 m from the production bore.

For a storativity of 0.01 m/m, the predicted radius of influence is approximately 1,900 m, however drawdowns greater than one metre are not expected to occur at distances exceeding 600 m.

Note this method does not consider discontinuous pumping and groundwater level recovery. If the production bore is only pumped for 12 hours at 2.5 L/s, and the groundwater level completely recovers in the 12-hour non pumping period, then even for a storativity of 0.01 m/m, the radius of influence does not extend beyond 300 m. For this scenario, drawdowns greater than one metre are not expected to occur at distances exceeding 50 m.

6.3 Water sharing plan licensing requirements

The project is located within the New England Fold Belt Coast Groundwater Source which is managed by the WSP for the North Coast Fractured and Porous Rock Groundwater Sources. Any take of groundwater associated with the project (through passive inflow or direct take through extraction for operational purposes) will require a WAL under the WM Act.

Based on water balance modelling results as part of the Surface Water Impact Assessment, ARDG requires 39 ML/year in the early stages of the project (Stage 1), assuming a very dry year (90th percentile bore import volume). A WAL will therefore be required for direct take prior to the commencement of the project. Based on recent (2021/2022) trades within the New England Fold Belt Coast Groundwater Source there is sufficient market depth for ARDG to obtain a licence for 39 ML/year.

The requirement for a WAL for passive take will not arise until the pit floor of the quarry progresses below the pre-quarry groundwater level. Based on the more likely inflow predictions (assuming 10% and 20% fracturing of the rhyodacite resource) for Stage 8 of the development, a WAL for approximately 9 – 15 ML/year would be required. However, the 39 ML/year take is only required to meet operational demands in the early stages of the project, well before the quarry floor intercepts groundwater. Therefore, no additional licencing would be required for the pit inflows, noting that updated predictions will be obtained based on groundwater monitoring and observed inflows into the pit.

Groundwater inflows are expected to continue for a period of time post closure until water levels within the pit have recovered above the pre-quarry groundwater levels. A WAL will therefore still be required in the post closure phase of the project. Impacts post closure are discussed further in Section 6.4.5.

6.4 Impact assessment criteria

The potential impacts have been assessed in accordance with the NSW AIP. The AIP requires that potential impacts on groundwater sources, including their users and GDEs, be assessed against minimal impact considerations, outlined in Table 1 of the policy. If the predicted impacts meet the Level 1 Minimal Impact Considerations, then these impacts will be considered as acceptable.

The NSW AIP divides groundwater into “highly productive” and “less productive” groundwater sources. Highly productive groundwater is defined in this policy as having:

– Total dissolved solids of less than 1,500 mg/L
– Contains water supply works that can yield water at a rate greater than 5 L/sec

Based on the reported yields for the registered bores outlined in Section 3.6.2, groundwater yields within the fractured rock aquifer are less than 5 L/s, varying between 0.63 L/s and 0.9 L/s. Groundwater at the project is therefore defined as “less productive” as per the NSW AIP.

Level 1 minimal impact considerations for Less Productive Groundwater Sources – Porous and Fractured Rock Water Sources have therefore been adopted for the GIA and are defined as follows:

– Water table:
  - Less than or equal to 10% cumulative variation in the water table, allowing for typical climatic ‘post-water sharing plan’ variations, at a distance of 40 m from any high priority GDE or high priority culturally significant site listed in the schedule of the relevant WSP.
  - A maximum of a 2 m water table decline cumulatively at any water supply work.
  - If more than 10% cumulative variation in the water table, allowing for typical climatic ‘post-water sharing plan’ variations, 40 m from any high priority GDE; or high priority culturally significant site; listed in the schedule of the relevant WSP then appropriate studies 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 2 m decline cumulatively at any water supply work, then make good provisions should apply.
– Water pressure:
  - A cumulative pressure head decline of not more than a 2 m decline at any water supply work.
  - If the predicted pressure head decline is greater than the requirement 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.
– Water quality:
  - Any change in groundwater quality should not lower the beneficial use category of the groundwater source, beyond 40 m from the activity.
  - If the above condition 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 work.

6.4.1 Impact to existing groundwater users

As the quarry pit floor deepens below the pre-quarry groundwater level and is dewatered, the pit will create a hydraulic gradient towards the project. This will cause a decline in groundwater levels surrounding the project. The decline in groundwater levels is referred to as a cone of depression, and the extent to which the cone of depression extends is referred to as the radius of drawdown. Drawdown will only occur once the pit floor is below the pre-quarry groundwater level. The lowest point of the cone of depression will be the ultimate floor level of the quarry or the level of water within the quarry (whichever is higher).

The radius of drawdown depends primarily on the nature of the aquifer and the floor level of the quarry or the level of water within the quarry (if the pit lake is not dewatered). For a fractured rock aquifer, it is likely that the drawdown will propagate in certain preferential directions, rather than in a circular shape.

The magnitude and extent of drawdown also depends on factors such as leakage between aquifers and aquitards, interactions with connected surface water systems and discharge features. Where the hydrogeological systems become more complex, the accuracy of the drawdown predictions reduces.

The nearest stock and domestic bores to the project are:

– GW060834 located approximately 1,720 m to the north-west of the project
– GW060853 located approximately 2,120 m to the north-west of the project
– GW066683 located approximately 4,550 m to the south-west of the project

The nearest water supply bore is:

– Works Approval 20WA214724 located approximately 1.5 km to the south-west of the project.

Based on the most conservative hydraulic conductivity value (K_{max} = 9.2 x 10^{-3} m/day), the most conservative predicted radius of drawdown for Stage 8 of the development is 468 m (Scenario 1). The registered bores are well outside the influence of the project's radius of drawdown and therefore no drawdown is expected to occur at any of the bores. The impact of the project therefore meets the NSW AIP Level 1 Minimal Impact Considerations for Landholder Bores.

Based on the conservative predicted radius of drawdown for the potential production bore (continuous pumping at 1.25 L/s, S = 0.01 m/m), drawdowns exceeding one metre are not expected to occur at distances beyond 600 m. Given that the nearest water supply bores (GW060834 and GW060853) are located more than one kilometre to the north-west of ARDG-P06, it is unlikely that a production bore located near ARDG-P06 will cause more than a 2 m water table decline cumulatively at any water supply work. The impact of the potential production bore therefore meets the NSW AIP Level 1 Minimal Impact Considerations for Landholder Bores.

6.4.2 Impact to shallow, perched groundwater system

The shallow, perched groundwater system is disconnected from the fractured rock aquifer in which the quarry pit is located and therefore groundwater level drawdown associated with quarry operations will not have an impact on the perched system.

The potential production bore would target the fractured rock aquifer and therefore would also be disconnected from the shallow perched groundwater system. Therefore, groundwater level drawdown associated with a potential production bore would not have an impact on the perched system.

Potential impacts to the perched groundwater system are associated with the reduction of up-slope catchment, which is discussed in the Surface Water Impact Assessment for the project.

6.4.3 Impact to GDEs

The nearest high priority GDEs are located near the Williams River, approximately eight and five kilometres from the project, to the west and north-west. These GDEs are well outside the most conservative predicted radius of drawdown of the project (Scenario 1, refer Figure 6.3) and therefore will not be impacted by drawdown associated with the project. These GDEs are also well outside the conservative radius of influence (refer Section 6.2) of a potential production bore located near ARDG-P06. No drawdown is therefore expected to occur at any of the high priority GDEs as a result of the project or potential production bore.

Areas of probable vegetation GDEs within the most conservative predicted radius of drawdown of the project (Scenario 1, refer Figure 6.3) have been mapped as part of the *Probable Vegetation Groundwater Dependent Ecosystems – Hunter/Central Rivers* dataset (DPE Water 2022). Within this area of predicted drawdown, this vegetation aligns with locations where the shallow perched aquifer system is disconnected from the deeper, fractured rock aquifer system. Field inspections around bore ARDG-P06 and ARDG-DDH18 indicate the presence of a very shallow perched groundwater system. The groundwater level intersected in ARDG-P06 during drilling (i.e., at 20 m below surface) is well below the shallow perched system. The presence of the low permeability dacite aquitard disconnects the perched system from the deeper fractured rock aquifer. Therefore, the mapped high probability vegetation GDEs are associated with the perched aquifer system rather than the deeper, fractured rock aquifer.

The maximum drawdown in the fractured rock aquifer will occur immediately adjacent to the pit where drawdown of up to 25 m is predicted (based on Scenario 1). This drawdown is relative to the current depth to groundwater. The magnitude of drawdown decreases away from the pit in a cone shape, with areas at the maximum extent of the radius of drawdown experiencing zero drawdown. Drawdown impacts associated with the project are only predicted to commence in Stage 5, where drawdown impacts will be constrained to areas immediately adjacent to the pit. As quarry operations progress deeper, the radius of drawdown will expand out and reach the maximum extent in Stage 8. After closure, the radius of drawdown will decline, as a pit lake forms in the final void.

The areas within the centre of the radius of drawdown, where the greatest drawdown is predicted, are within the development footprint. As a result, there will be no vegetation present within this area to be impacted by groundwater drawdown. In the areas outside the development footprint, only the northern extent of drawdown has potential to impact areas of vegetation mapped as having a high probability of being a GDE. This vegetation is associated with the unnamed first and second order tributary of Caswells Creek. This tributary and associated riparian vegetation have not been mapped as a high ecological value aquatic ecosystem (HEVAE) by DPE-Water.

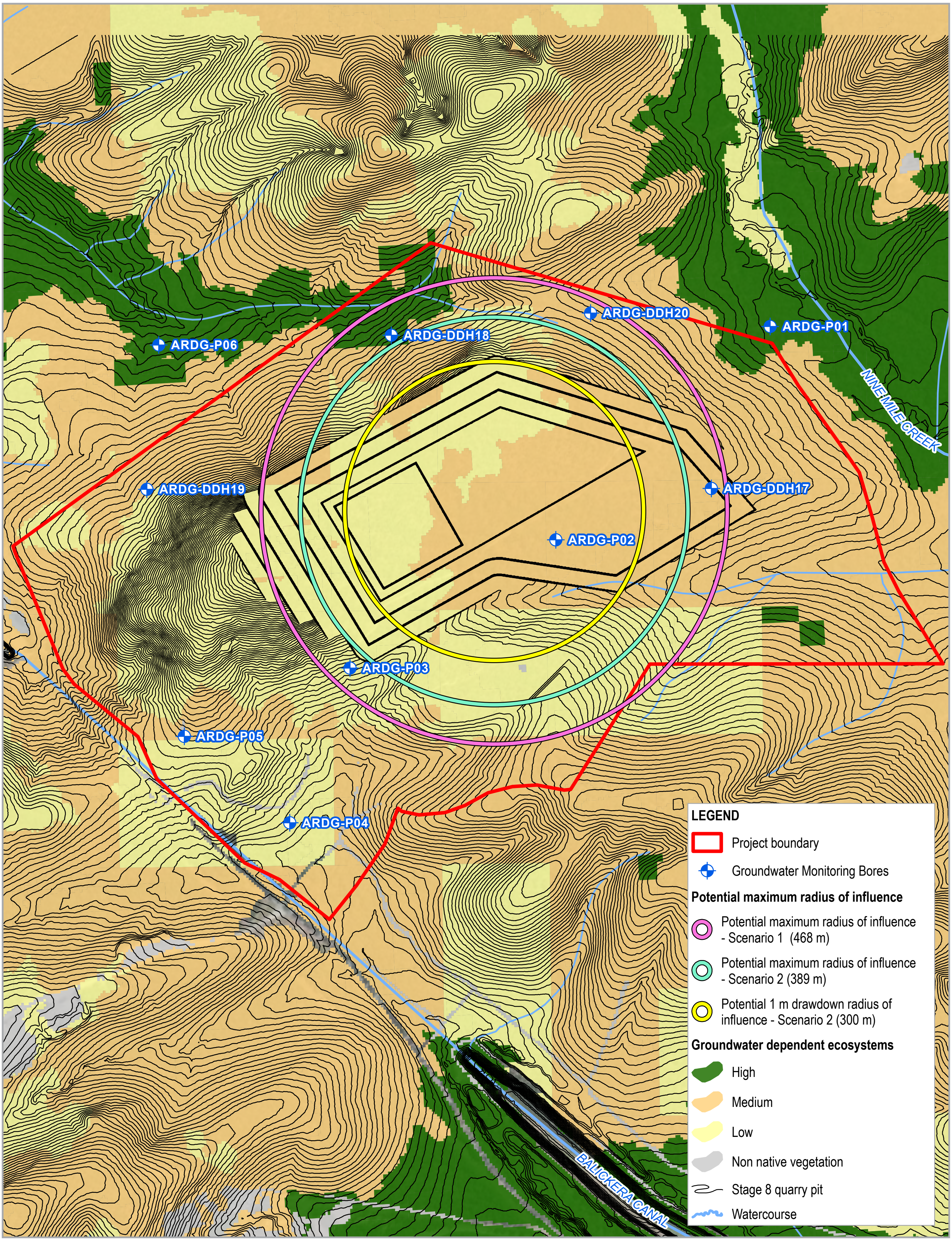
Drawdowns, assuming the most conservative refined drawdown prediction (Scenario 2, refer Figure 6.3) in the area of the high probability GDEs are not expected to be greater than one metre (refer Section 6.1.5.2 and Figure 6.3). The potential production bore could also be site so that drawdown in the area of the high probability GDEs would not exceed one metre (refer Section 6.2).

Notwithstanding, the terrestrial vegetation in these areas that have groundwater dependence would therefore be highly unlikely to be impacted by any drawdown induced in the much deeper bedrock layers. Even if drawdowns in the fractured rock aquifer of up to five metres occurred, given that groundwater is at 20 m below the surface, it would be unlikely to have a material impact on vegetation associated with the perched systems, which are primarily influenced by rainfall and surface flow recharge. Therefore, the modelled drawdown of groundwater in the deeper, fractured rock system is not predicted to adversely impact these high probability GDEs. The potential production bore would also target the deeper, fractured rock aquifer and therefore drawdown associated with pumping is not predicted to adversely impact these high probability GDEs.

Based on standing water levels measured in groundwater monitoring bores, the average depth to groundwater in the fractured rock aquifer at the project varies between 7.31 and 23.3 m below ground level. Average groundwater levels in the area where the high probability GDEs are present (within the maximum extent of predicted drawdown) are between approximately 7 and 13 metres below ground level. However, as discussed in Section 4.2.1, groundwater in the area around the ephemeral tributaries of Caswells Creek (underlain by impermeable dacite) is significantly lower due to the confining nature of the overlying Dacite. As noted above, the location and nature of the terrestrial GDE vegetation mapped in these areas is consistent with these communities being associated with a shallow perched aquifer system which is hydraulically disconnected to the regional water table in these areas. Given the depth to groundwater in this area (i.e., 20 m below surface), it is apparent that the presence of high probability GDEs is due to shallow groundwater in the overlying alluvial/colluvial material (recharged from creeks and rainfall), rather than the deeper regional groundwater table from which it is disconnected in this area.

Nine Mile Creek, Balickera Channel and the tributary of Caswell's Creek located to the north-west of the project are likely too high in elevation relative to the groundwater levels to be points of discharge for groundwater from within the project area. The project is therefore unlikely to have an impact on aquatic GDEs or baseflow, as the groundwater is well below the creeks in the area (approximately 20 m). The EC of groundwater in bores adjacent to Nine Mile Creek is much higher than in the creek itself, which also suggests the systems are unlikely to be connected.

The impact of the project therefore meets the NSW AIP Level 1 Minimal Impact Considerations for GDEs.



Australian Resource Development Group Pty Ltd
Stone Ridge Quarry
Groundwater Impact Assessment

Project No. 22-19467
Revision No. 1
Date 16/02/2024

Potential maximum radius of influence

FIGURE 6.3

N:\AU\Newcastle\Projects\22\19467\GIS\Maps\Deliverables\2219467_GWIA_1.aprx

Data source: Groundwater dependent ecosystems - BoM 2022. Project boundary - ARDG 2022. General topography - LPI 2017. Sixmaps/DCS - Imagery (date extracted: 16/02/2024) World Imagery: Maxar. Created by: tmorton

6.4.4 Impact to groundwater quality

The project is not expected to cause any significant change in groundwater quality or in the beneficial use of the groundwater. Quarrying activities may increase groundwater recharge in the post closure phase which may result in a localised improvement in groundwater quality.

Due to the relatively small, predicted radius of drawdown, it is not expected that the project will result in the interaction between fresh and saline groundwater sources. In addition, due to the high chloride to sulfate ratios in groundwater (generally greater than 10), there is no evidence of pyrite oxidation, and it is not expected that the project will result in the generation of acid in groundwater.

It is recommended that groundwater quality be monitored regularly at ARDG-P01 and ARDG-P06 prior to the project commencing, and during the project to minimise the project's potential for impacting groundwater quality. Groundwater monitoring is further discussed in Section 7.

The impact of the project therefore meets the NSW AIP Level 1 Minimal Impact Considerations for Groundwater Quality.

6.4.5 Cumulative impacts

The potential production bore would only be operating during Stage 1 of the project. Groundwater inflow is only expected from Stage 5 onwards. An assessment of cumulative impacts is therefore not required.

6.4.6 Impacts post closure

The project is expected be completed after 30 years. At the end of development, the groundwater table will be locally depressed to -2 m AHD. With time, groundwater levels in the aquifer surrounding the project will recover until equilibrium within the system occurs, and a pit lake forms within the final void. Once the system is in equilibrium, the flux of water within the pit lake will only be from rainfall and evaporation. During the recovery stage however, groundwater inflows will occur, and a WAL will still be required in the initial post closure phase of the project.

Recovery of groundwater levels post closure has not been modelled in this assessment. Water level recovery in the final voids however has been modelled in the Surface Water Impact Assessment for the project.

Any enhanced recharge that occurs as a result of the quarry in the post closure phase would reduce the time required for groundwater levels to recover. The increased groundwater recharge in the post closure phase may also result in a localised improvement in groundwater quality.

It is recommended that groundwater monitoring continues in the post closure period however, so that groundwater level recovery can be monitored, and predictions made regarding how long a WAL may be required after the project is completed. Groundwater monitoring is further discussed in Section 7.

7. Mitigation measures

7.1 Groundwater monitoring

It is recommended that the existing groundwater monitoring program be continued. It is recommended that groundwater be monitored to:

– Measure dewatering performance.
– Assess potential impacts to groundwater levels and quality on other groundwater users in the vicinity.
– Identify groundwater issues such as potential large drawdowns at receptors as early as possible.
– Provide data which can be used to calibrate the analytical model and update the groundwater inflow predictions.
– Measure groundwater level recovery post closure and provide data which can be used to predict how long a WAL may be required after the project is completed.

It is recommended that the existing monitoring program be extended to include an additional monitoring bore installed approximately one kilometre from the project, to the north-west. Groundwater level data from this bore can be used to determine whether or not the radius of drawdown is extending further than was predicted. Monitoring groundwater quality at this bore can be used to identify any potential impacts of the project prior to these impacts extending further towards landholder bores or GDEs.

7.2 Monitoring program

The monitoring program should include regular monitoring of water levels and water quality. The monitoring program should be established prior to Stage 5 development commencing.

The existing monitoring bores which are not affected by the quarry operations should continue to be monitored during operations. It is recommended that groundwater levels initially be monitored quarterly at all existing monitoring bores and in the proposed monitoring bore until the quarry reaches Stage 5. Once the quarry development reaches Stage 5, it is recommended that the monitoring frequency increase to monthly at selected bores. Given that the site can become inaccessible in very wet weather, it is recommended that data loggers be installed in two monitoring bores (the proposed bore, ARDG-P06 or ARDG-DDH19) to provide a continuous record.

It is recommended that water quality be monitored quarterly in all monitoring bores for the first two years after the project commences. Water quality samples should be analysed for pH, EC, nutrients, major ions and dissolved metals (aluminium, arsenic, cadmium, chromium, copper, iron, manganese, nickel, lead, zinc and mercury). Due to the low level of risk, after two years it is recommended that only EC and pH be monitored quarterly, with selected metals monitored annually. A TARP should be developed to monitor the full suite of parameters in the event of a significant departure from pH and EC triggers.

It is recommended that the monitoring program be reviewed every two years to determine if monitoring results indicate that less frequent monitoring would still provide a reasonable level of data to enable the impacts to be reliably detected.

The monitoring program should also include monitoring of groundwater inflow into the quarry. Measuring groundwater take is a requirement from a licensing perspective and the measured inflows can also be used to calibrate the analytical model and provide updated predictions. Groundwater inflow rates should therefore be accurately recorded.

Groundwater quality monitoring requirements post closure should be reviewed as part of closure planning with a focus on understanding the impacts of groundwater recharge from a recovering pit lake on the local groundwater system. Groundwater levels should continue to be monitored in the post closure phase until groundwater levels stabilise and/or regulation requirements are met. Monitoring locations and frequency in the post closure period should be identified as part of the quarry closure planning process and be informed by monitoring undertaken during the life of project, updated predictions of pit lake recovery and likely water quality and risks presented from pit lake recovery.

The groundwater monitoring program will provide a safeguard against any impacts that have not been identified in this assessment. If unforeseen impacts are identified during monitoring ARDG will be able to amend the dewatering operation and/or the monitoring program to prevent further reductions in groundwater levels and/or quality.

8. Conclusions

For the project, an assessment of likely groundwater inflow rates and the radius of drawdown was undertaken using a steady-state analytical model. Considering that the distance to the few registered landholder bores is greater than one kilometre, the low hydraulic conductivity of the aquifer and the lack of GDEs due to deep groundwater levels through the rhyodacite resource, it is considered that the risk to identified groundwater receptors due to the project is low. The level of complexity of analytical equations is therefore appropriate to assess this risk.

The interpreted hydraulic conductivity at monitoring bore ARDG-P02 from rising slug testing was assumed to represent the maximum expected hydraulic conductivity in the rhyodacite resource (K_{max} = 9.2 x 10^{-3} m/day). More likely values of hydraulic conductivity were estimated assuming 10% and 20% fracturing of the rhyodacite resource and averaged literature value for unfractured igneous and metamorphic rock (K_{10} = 9.3 x 10^{-4} m/day and K_{20} = 1.9 x 10^{-3} m/day). Using this more likely range of hydraulic conductivity estimates, Stage 8 groundwater inflows into the pit were predicted to range from 8.7 ML/year to 14.3 ML/year, and the radius of drawdown was predicted to be between 336 m and 363 m from the centre of the pit.

The project area is located within the New England Fold Belt Coast Groundwater Source which is managed by the Water Sharing Plan for the North Coast Fractured and Porous Rock Groundwater Sources. Any take of groundwater associated with the project (through passive inflow or direct take through extraction for operational purposes) will require a WAL under the WM Act. Based on water balance modelling results as part of the Surface Water Impact Assessment, ARDG requires 39 ML/year in the early stages of the project. A WAL will therefore be required for direct take prior to the commencement of the project. Based on recent (2021/2022) trades within the New England Fold Belt Coast Groundwater Source there is sufficient market depth for ARDG to obtain a licence for up to 39 ML/year.

The requirement for a WAL for passive take will not arise until the pit floor of the quarry progresses below the pre-quarry groundwater level. Based on the more likely inflow predictions for Stage 8 (the final and deepest stage of quarry operations), a WAL for approximately 9 – 15 ML/year would be required. However, the 39 ML/year take is only required to meet operational demands in the early stages of the project, well before the quarry floor intercepts groundwater. Therefore, no additional licencing would be required for the pit inflows, noting that updated predictions will be obtained based on groundwater monitoring and observed inflows into the pit.

The most conservative predicted radius of drawdown for Stage 8 was used to assess the impact of the project on existing groundwater users. Landholder bores are well outside the project's radius of drawdown. No drawdown is therefore expected to occur at any of the landholder bores. Based on the most conservative predicted radius of influence for the potential production bore, drawdowns exceeding one metre are not expected to occur at distances beyond 600 m. Given that the nearest water supply bores are located more than one kilometre to the north-west of ARDG-P06, it is unlikely that a production bore located near ARDG-P06 would cause more than a 2 m water table decline cumulatively at any water supply work. Therefore, the impacts of the project and potential production bore therefore meet the NSW Aquifer Interference Policy (AIP) Level 1 Minimal Impact Considerations for landholder bores.

The most conservative predicted radius of drawdown for Stage 8 was used to assess the impact of the project on GDEs. High priority GDEs are well outside the project's radius of drawdown. These GDEs are also well outside the conservative radius of influence of a potential production bore located near ARDG-P06. No drawdown is therefore expected to occur at any of the high priority GDEs as a result of the project or potential production bore.

Based on standing water levels measured in groundwater monitoring bores, the average depth to groundwater in the fractured rock aquifer at the project varies between 7.31 and 23.3 metres below ground level. Average groundwater levels in the area where the high probability GDEs are present, within the maximum extent of predicted drawdown (468 m; Scenario 1 based on Kmax), are between approximately seven and 13 metres below ground level. However, groundwater in these areas is actually significantly lower (i.e., 20 m below the surface) due to the presence of a zone of low permeability dacite which acts as an aquitard, confining groundwater in the deeper, more permeable units at a depth.

A refined, conservative predicted radius of drawdown was used to assess the potential impact of the project on high probability GDEs located in the western flank area. This scenario is based on a reduced groundwater level which more closely represents impacts to the terrain surrounding the quarry footprint. Drawdowns greater than one metre are not expected to occur at distances exceeding 300 m from the centre of the pit. Therefore, drawdowns in the area of the high probability GDEs are not expected to be greater than one metre. The potential production bore could also be positioned so that drawdown in the area of the high probability GDEs would not exceed one metre.

Notwithstanding, given the depth to groundwater in this area (i.e., 20 m below surface), it is apparent that the presence of high probability GDEs is due to shallow groundwater in the overlying alluvial/colluvial material (recharged from creeks and rainfall), rather than the deeper regional groundwater table from which it is disconnected in this area. Therefore, the modelled drawdown of groundwater in the deeper, fractured rock system (including from a potential production bore) is not predicted to adversely impact these high probability GDEs.

The impacts of the project and potential production bore therefore meet the NSW Aquifer Interference Policy (AIP) Level 1 Minimal Impact Considerations for GDEs.

The project or a potential production bore are not expected to cause any significant change in groundwater quality or in the beneficial use of the groundwater. The increased groundwater recharge in the post closure phase may also result in a localised improvement in groundwater quality. The impacts of the project and potential production bore therefore meet the NSW Aquifer Interference Policy (AIP) Level 1 Minimal Impact Considerations for groundwater quality.

The potential production bore would only be operating during Stage 1 of the project. Groundwater inflow is only expected from Stage 5 onwards. An assessment of cumulative impacts is therefore not required.

The project is expected be completed after 30 years. With time, groundwater levels in the aquifer surrounding the project will recover until equilibrium within the system occurs, and a pit lake forms within the final voids. Once the system is in equilibrium, the flux of water within the pit lake will only be from rainfall and evaporation. During the recovery stage however, groundwater inflows will occur, and a WAL will still be required in the initial post closure phase of the project. Any enhanced recharge that occurs as a result of the quarry in the post closure phase would reduce the time required for groundwater levels to recover.

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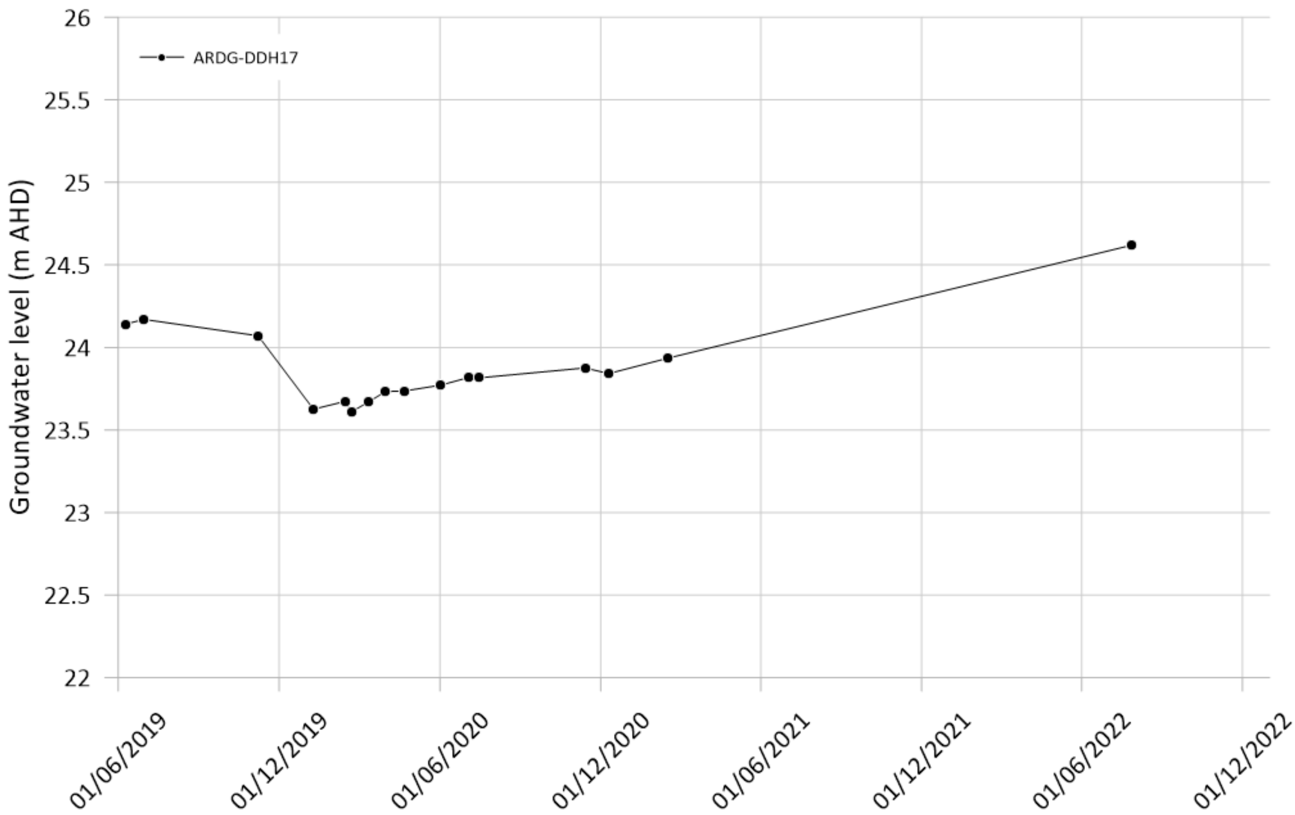
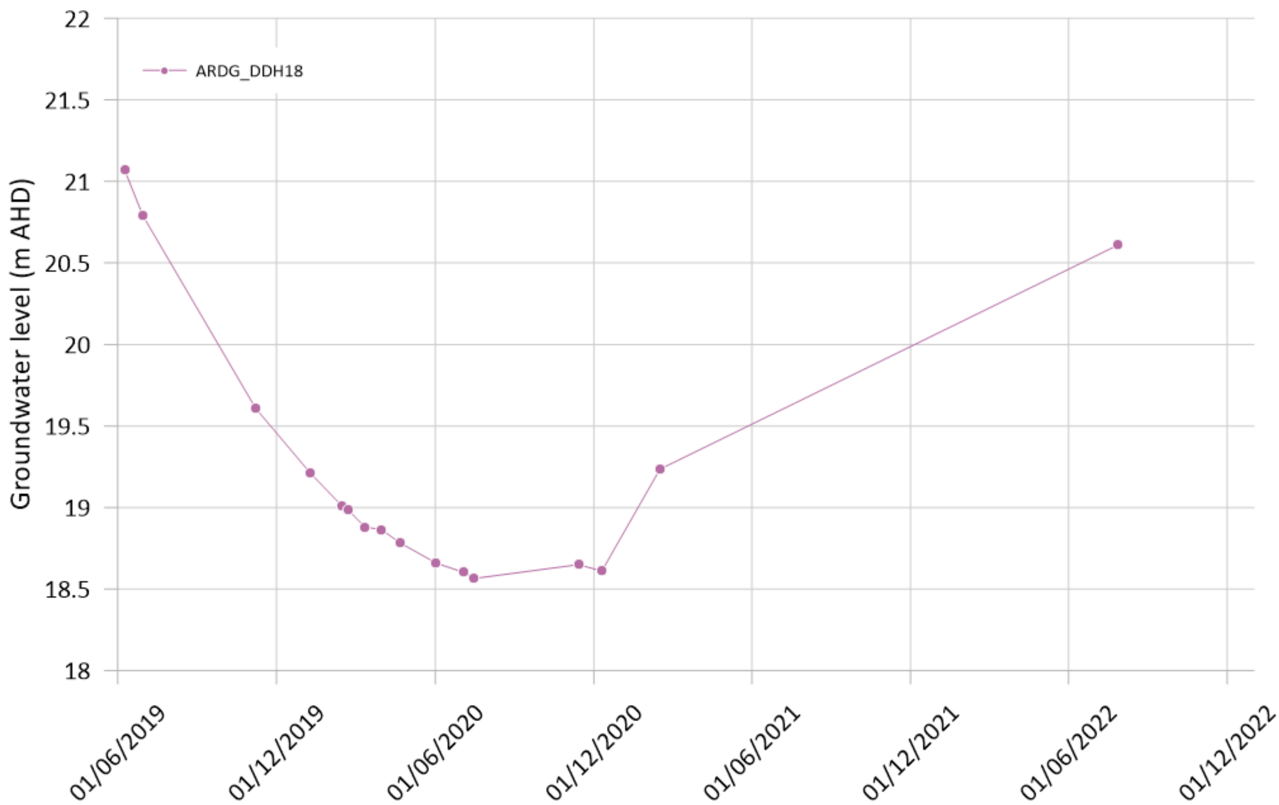
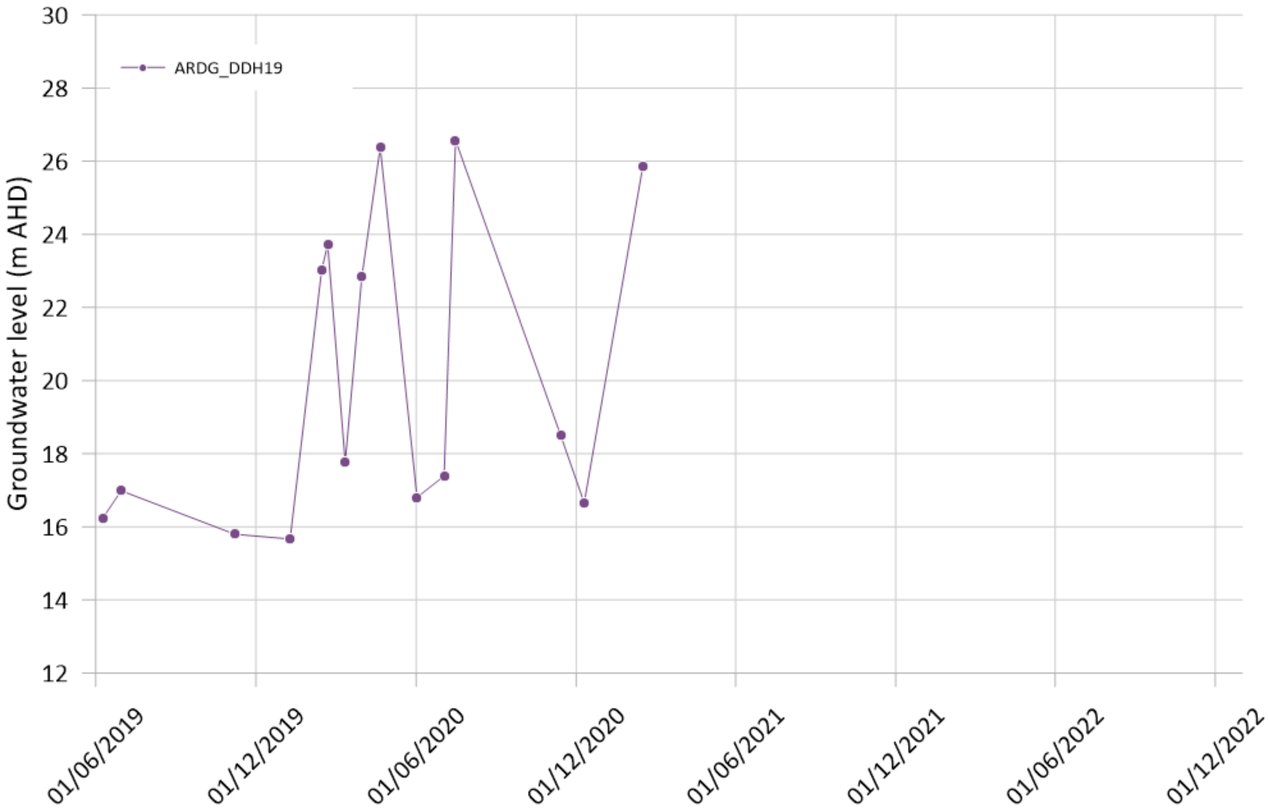
Appendix A

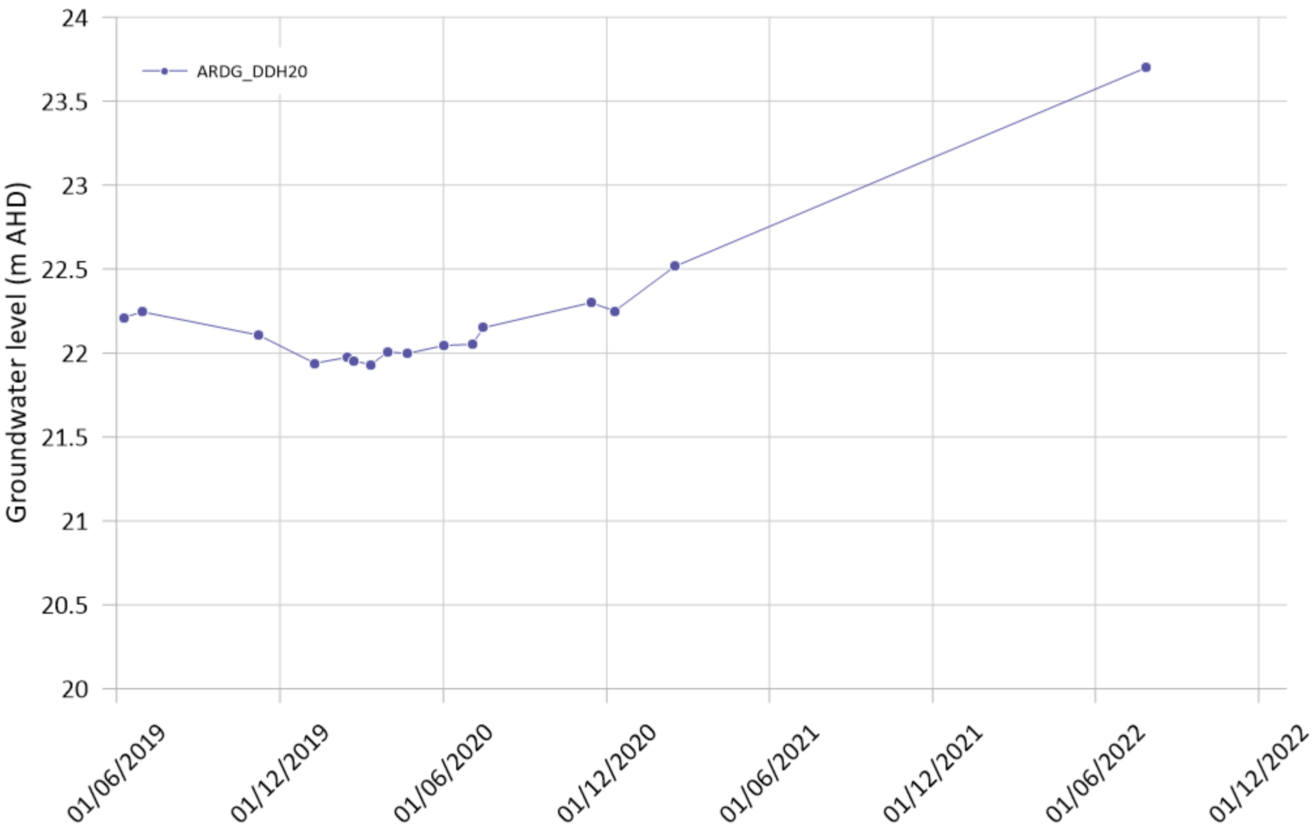
Groundwater level hydrographs

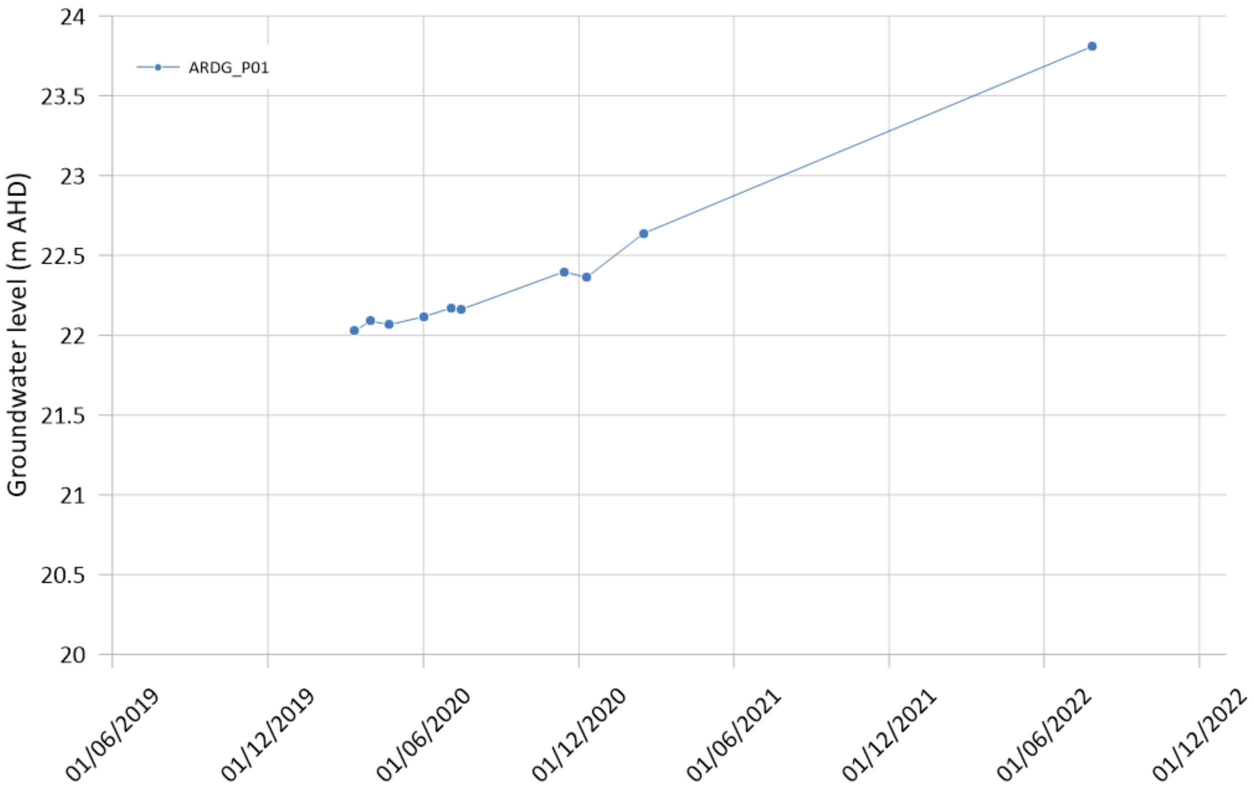
A-1 Groundwater level hydrographs






30
28
26
24
22
20
18
16
14
12
ARDG_DDH19
Groundwater level (m AHD)
01/06/2019
01/12/2019
01/06/2020
01/12/2020
01/06/2021
01/12/2021
01/06/2022
01/12/2022


24
23.5
23
22.5
22
21.5
21
20.5
20
ARDG_DDH20
Groundwater level (m AHD)
01/06/2019
01/12/2019
01/06/2020
01/12/2020
01/06/2021
01/12/2021
01/06/2022
01/12/2022


ARDG_P01
Groundwater level (m AHD)
24
23.5
23
22.5
22
21.5
21
20.5
20
01/06/2019
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01/06/2020
01/12/2020
01/06/2021
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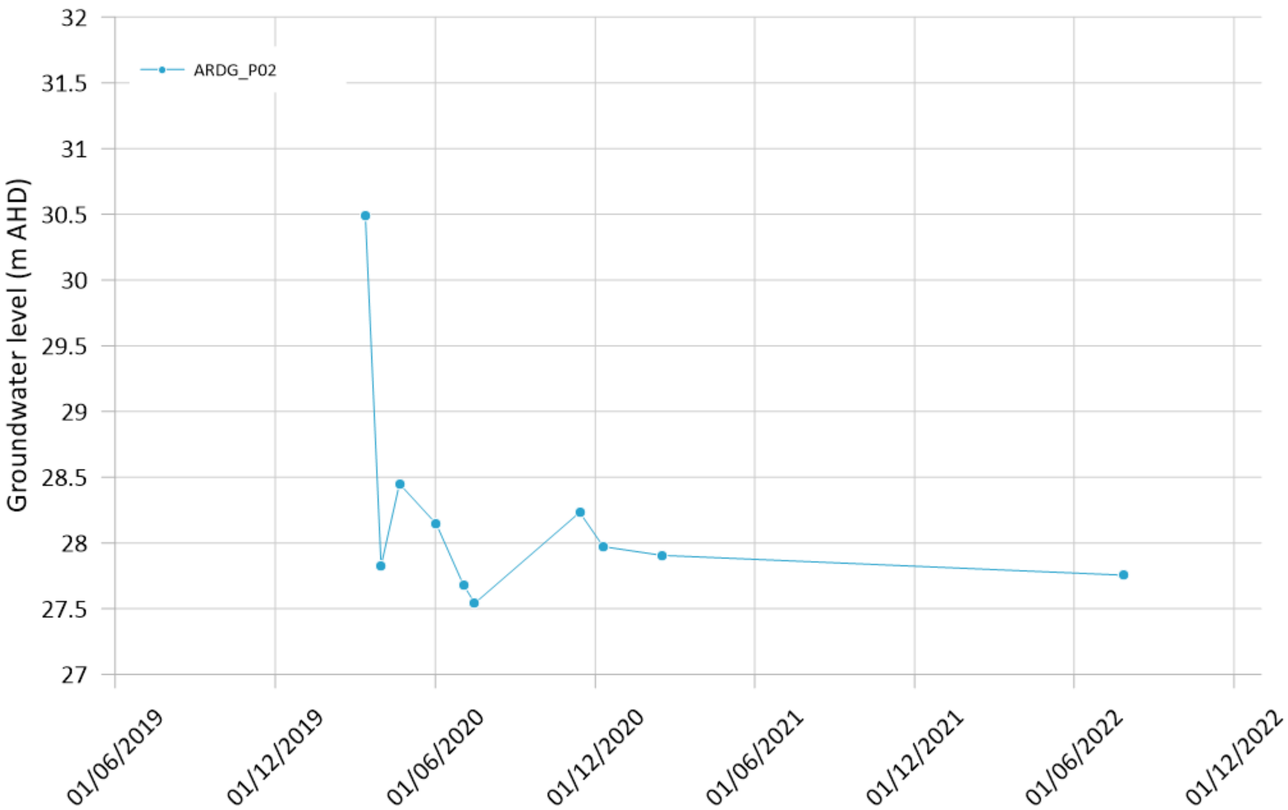
01/06/2022
01/12/2022


ARDG_P02
Groundwater level (m AHD)
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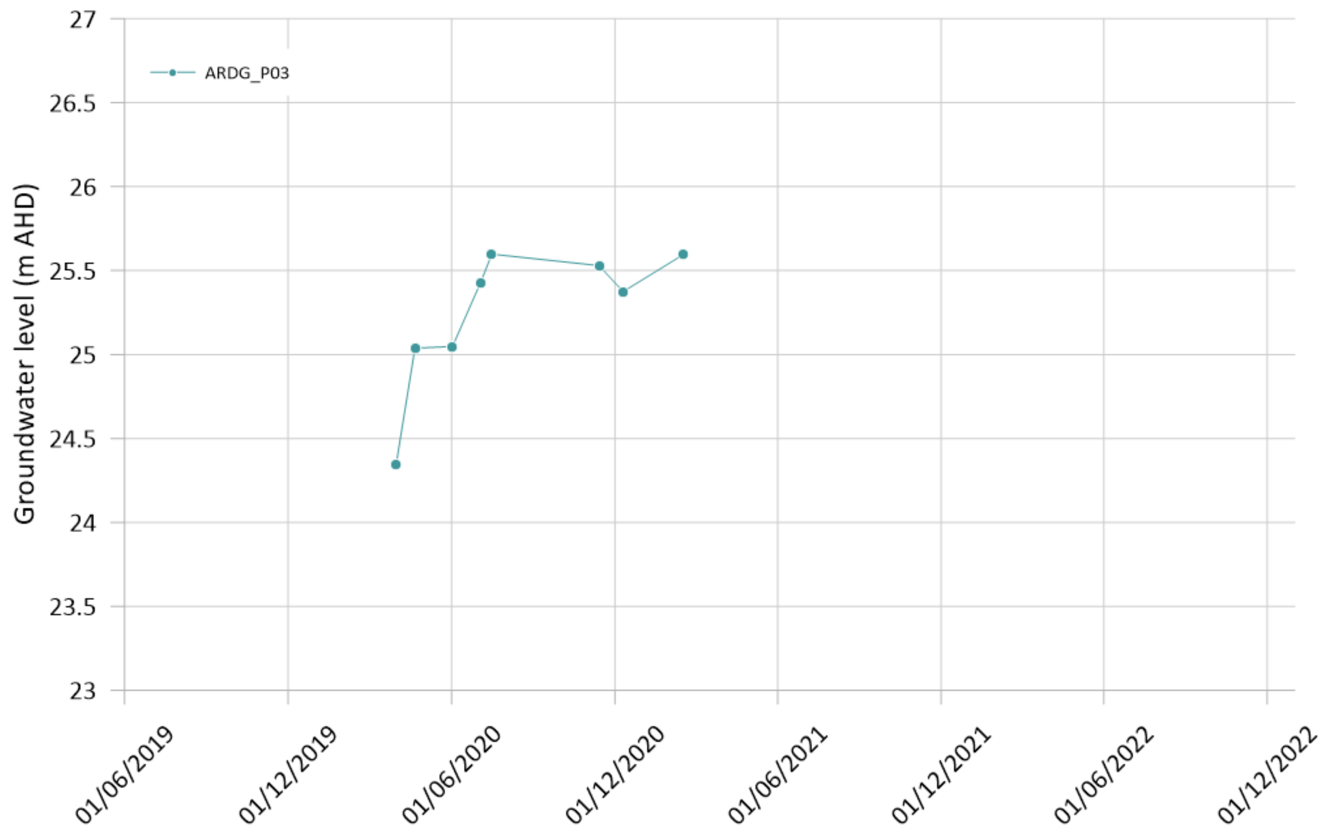
31.5
31
30.5
30
29.5
29
28.5
28
27.5
27
01/06/2019
01/12/2019
01/06/2020
01/12/2020
01/06/2021
01/12/2021
01/06/2022
01/12/2022


ARDG_P03
Groundwater level (m AHD)
27
26.5
26
25.5
25
24.5
24
23.5
23
01/06/2019
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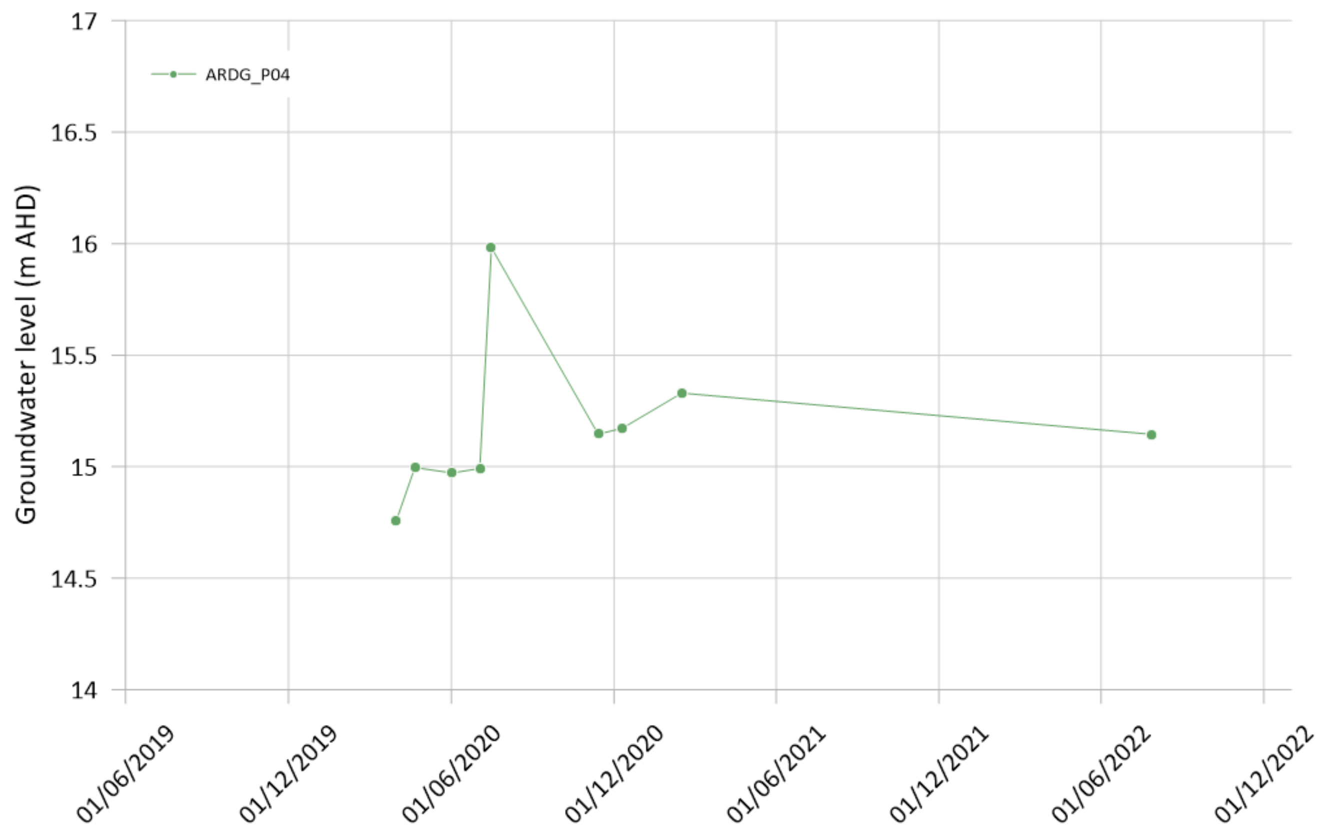
01/06/2022
01/12/2022


ARDG_P04
Groundwater level (m AHD)
17
16.5
16
15.5
15
14.5
14
01/06/2019
01/12/2019
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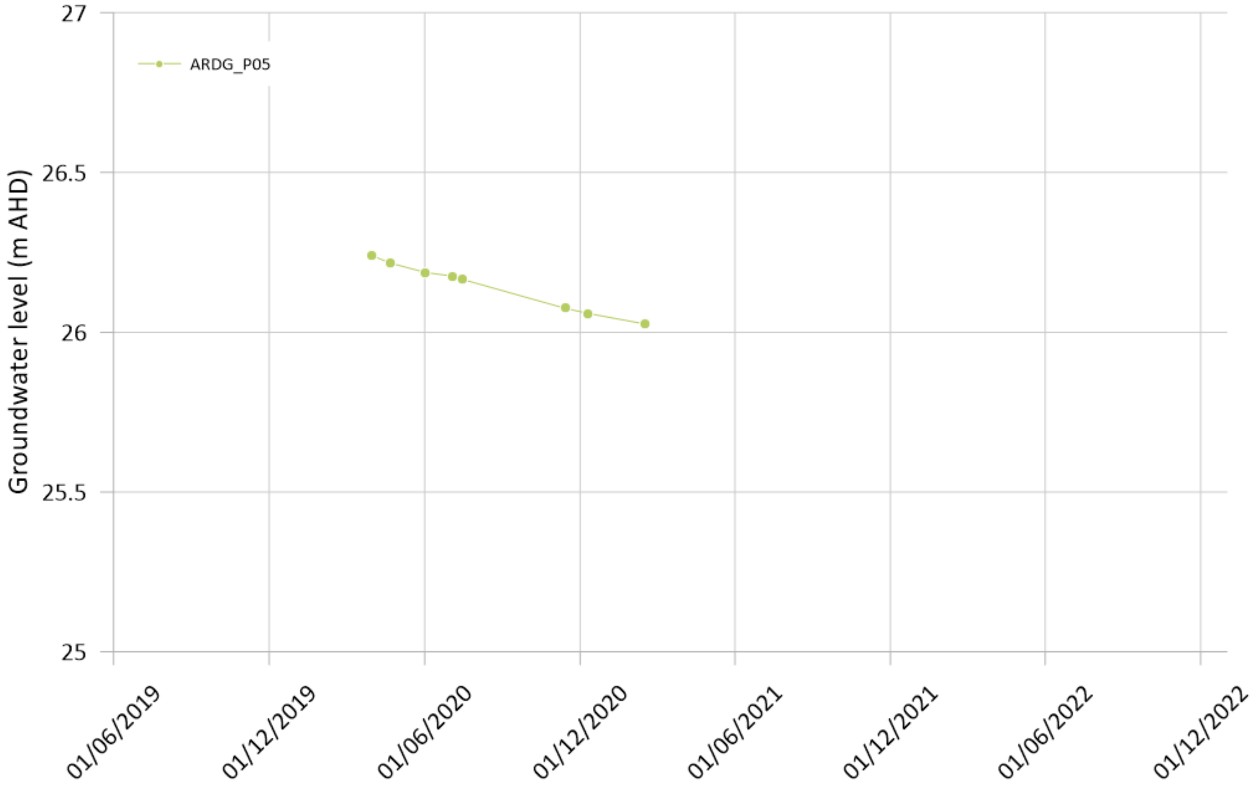
01/06/2022
01/12/2022


27
26.5
26
25.5
25
Groundwater level (m AHD)
ARDG_P05
01/06/2019
01/12/2019
01/06/2020
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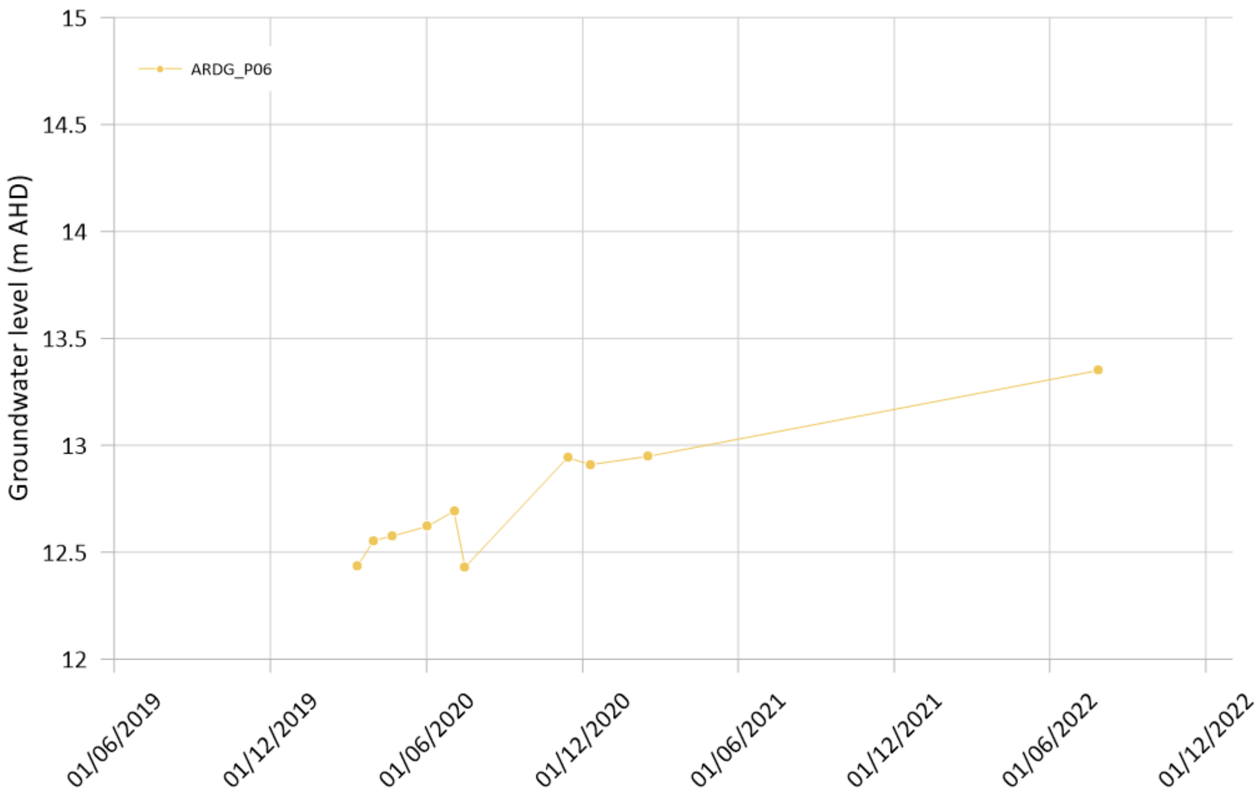
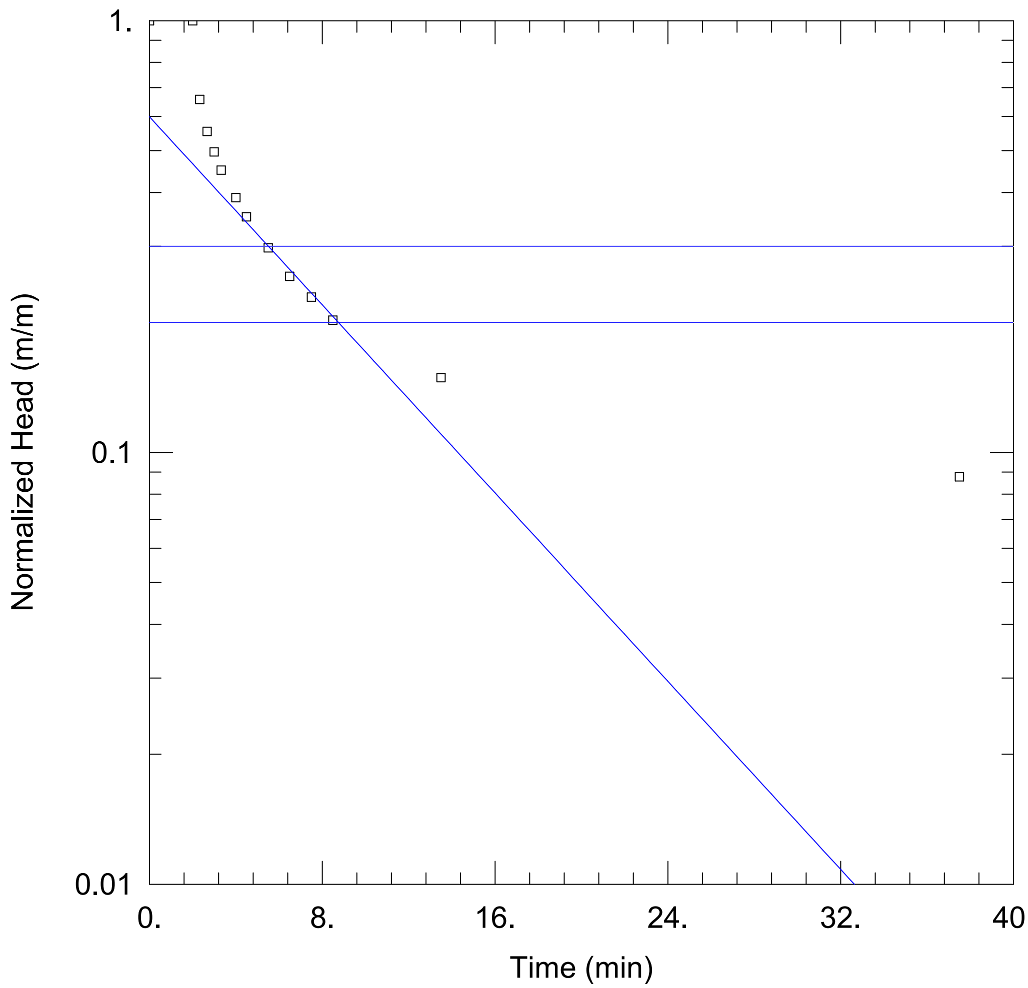
01/06/2022
01/12/2022


15
14.5
14
13.5
13
12.5
12
Groundwater level (m AHD)
ARDG_P06
01/06/2019
01/12/2019
01/06/2020
01/12/2020
01/06/2021
01/12/2021
01/06/2022
01/12/2022

Appendix B

Slug test interpretation plots

B-1 Slug test interpretation plots



FALLING HEAD SLUG TEST

Data Set: \...\DDH17.aqt
Date: 08/26/22 Time: 08:16:33

PROJECT INFORMATION

Company: GHD Pty Ltd
Client: ARDG Pty Ltd
Project: 2219467
Location: Stone Ridge Quarry
Test Well: ARDG-DDH17
Test Date: 23/08/2022

AQUIFER DATA

Saturated Thickness: 43.34 m Anisotropy Ratio (Kz/Kr): 1.

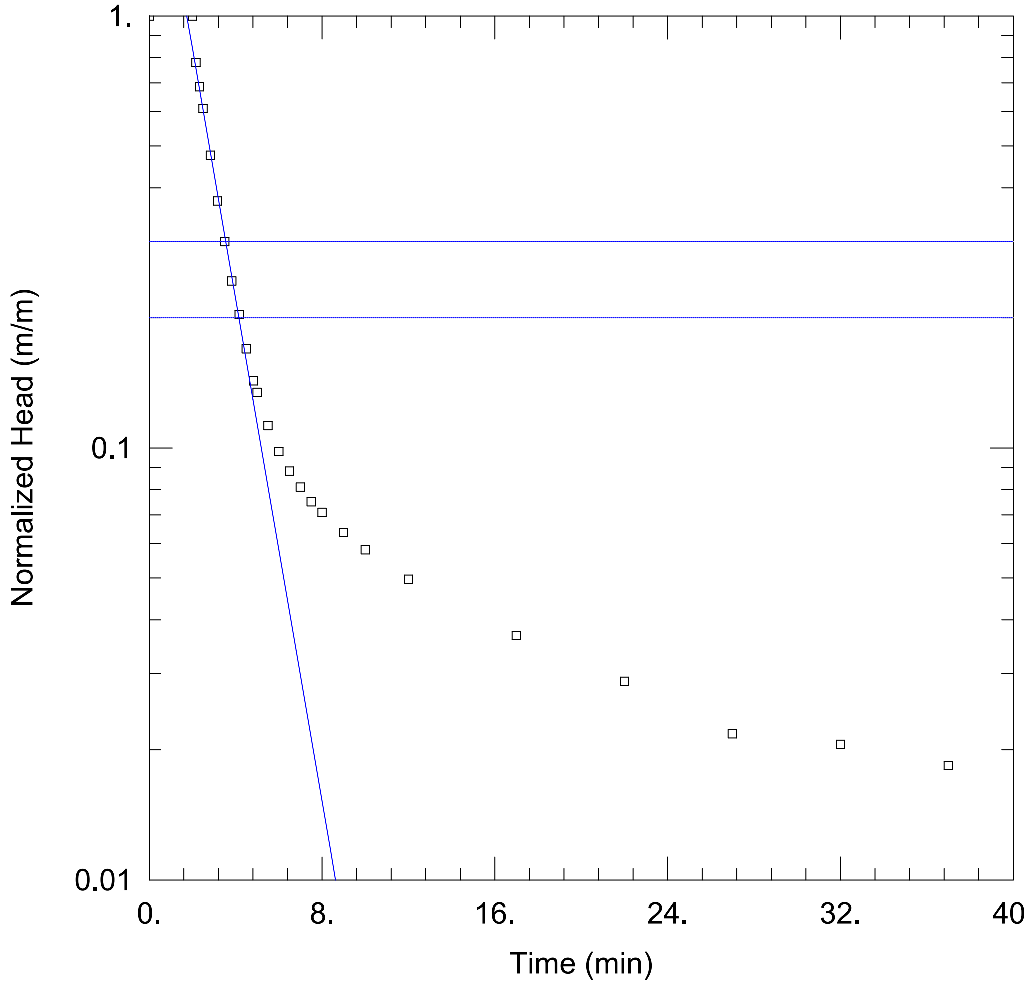
WELL DATA (ARDG-DDH17)

Initial Displacement: 0.262 m
Total Well Penetration Depth: 19.84 m
Casing Radius: 0.025 m
Static Water Column Height: 19.84 m
Screen Length: 3. m
Well Radius: 0.048 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice

K = 0.0647 m/day y0 = 0.1568 m



WELL TEST ANALYSIS

Data Set: \...\DDH18.aqt
Date: 08/26/22 Time: 08:17:34

PROJECT INFORMATION

Company: GHD Pty Ltd
Client: ARDG Pty Ltd
Project: 2219467
Location: Stone Ridge Quarry
Test Well: ARDG-DDH18
Test Date: 23/08/2022

AQUIFER DATA

Saturated Thickness: 48.2 m Anisotropy Ratio (Kz/Kr): 1.

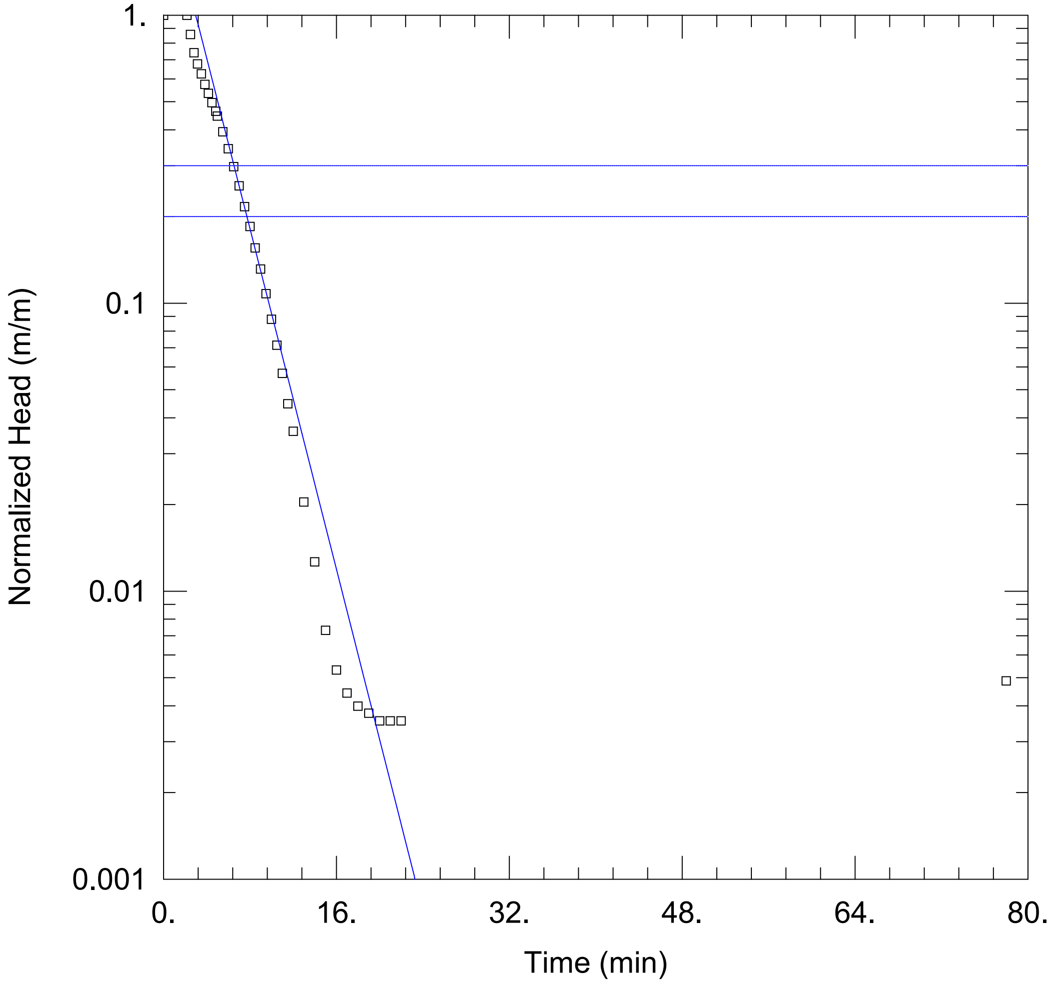
WELL DATA (ARDG-DDH18)

Initial Displacement: 4.13 m Static Water Column Height: 39.85 m
Total Well Penetration Depth: 39.85 m Screen Length: 3. m
Casing Radius: 0.025 m Well Radius: 0.048 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice

K = 0.3795 m/day y0 = 13.25 m



WELL TEST ANALYSIS

Data Set: \...\DDH20.aqt
Date: 08/26/22
Time: 08:20:09

PROJECT INFORMATION

Company: GHD Pty Ltd
Client: ARDG Pty Ltd
Project: 2219467
Location: Stone Ridge Quarry
Test Well: ARDG-DDH20
Test Date: 23/08/2022

AQUIFER DATA

Saturated Thickness: 43.09 m
Anisotropy Ratio (Kz/Kr): 1.

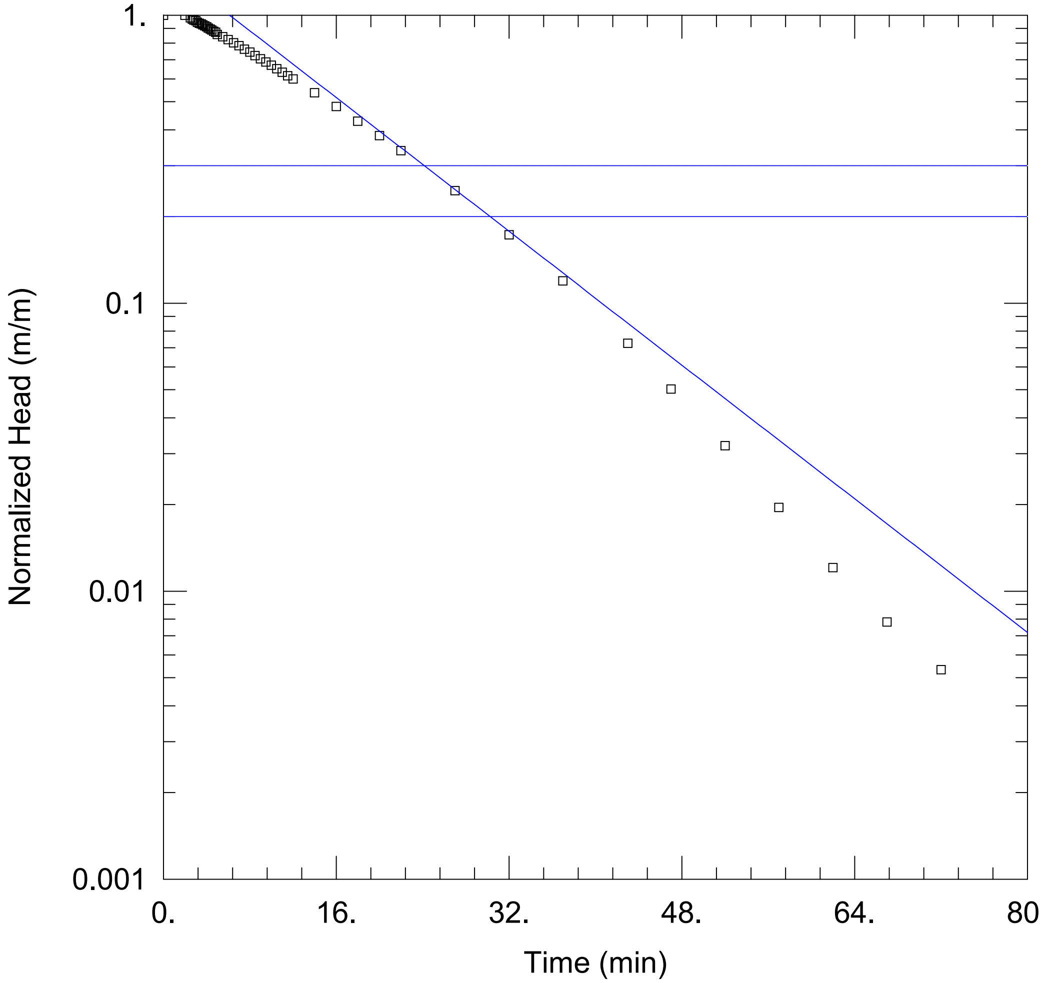
WELL DATA (ARDG-DDH20)

Initial Displacement: 4.512 m
Total Well Penetration Depth: 16.59 m
Casing Radius: 0.025 m
Static Water Column Height: 16.59 m
Screen Length: 3. m
Well Radius: 0.048 m

SOLUTION

Aquifer Model: Unconfined
Solution Method: Bouwer-Rice

K = 0.1723 m/day
y0 = 12.45 m



WELL TEST ANALYSIS

Data Set: \...\ARDG-P01.aqt
Date: 08/26/22
Time: 08:23:57

PROJECT INFORMATION

Company: GHD Pty Ltd
Client: ARDG Pty Ltd
Project: 2219467
Location: Stone Ridge Quarry
Test Well: ARDG-P01
Test Date: 23/08/2022

AQUIFER DATA

Saturated Thickness: 53.99 m
Anisotropy Ratio (Kz/Kr): 1.

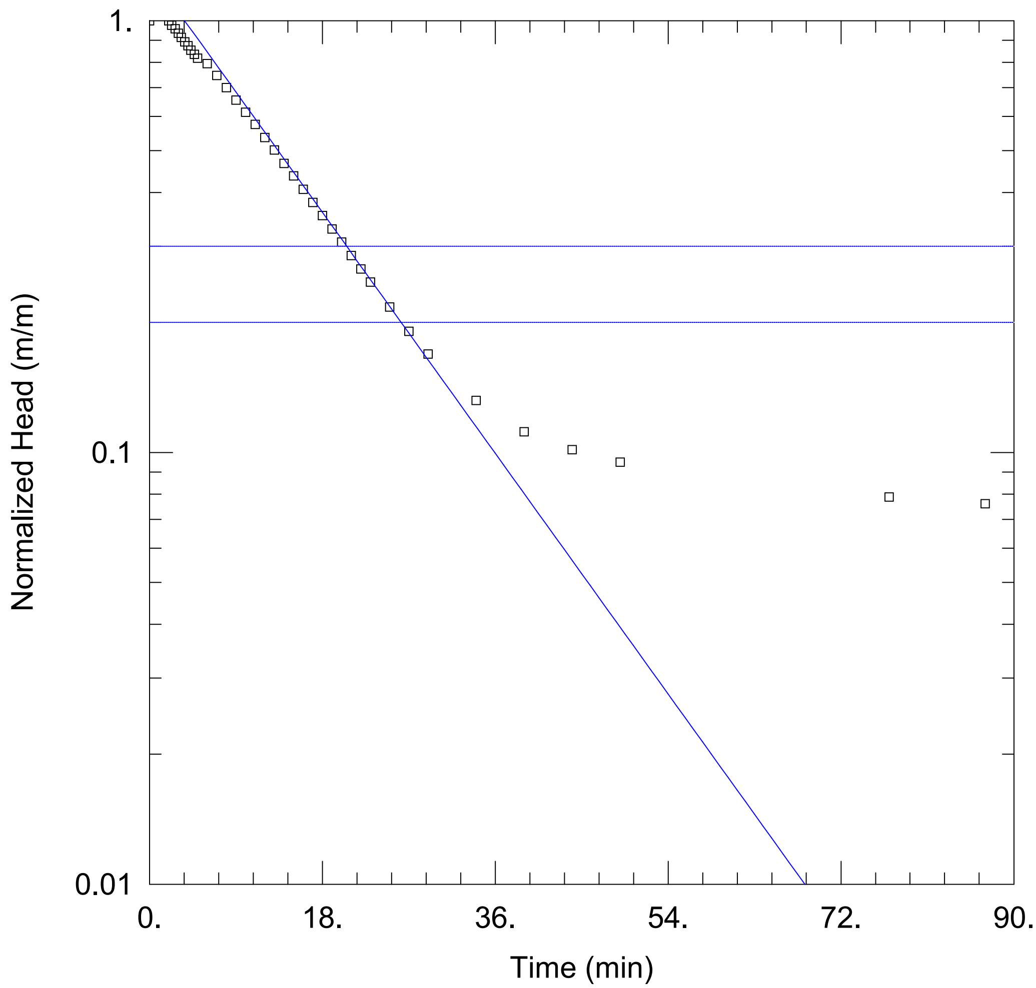
WELL DATA (ARDG-P01)

Initial Displacement: 5.628 m
Total Well Penetration Depth: 11.99 m
Casing Radius: 0.025 m
Static Water Column Height: 11.99 m
Screen Length: 3. m
Well Radius: 0.051 m

SOLUTION

Aquifer Model: Unconfined
Solution Method: Bouwer-Rice

K = 0.03203 m/day
y0 = 8.47 m



WELL TEST ANALYSIS

Data Set: \...\ARDG-P02.aqt
Date: 08/26/22 Time: 08:25:26

PROJECT INFORMATION

Company: GHD Pty Ltd
Client: ARDG Pty Ltd
Project: 2219467
Location: Stone Ridge Quarry
Test Well: ARDG-P02
Test Date: 23/08/2022

AQUIFER DATA

Saturated Thickness: 42.38 m Anisotropy Ratio (Kz/Kr): 1.

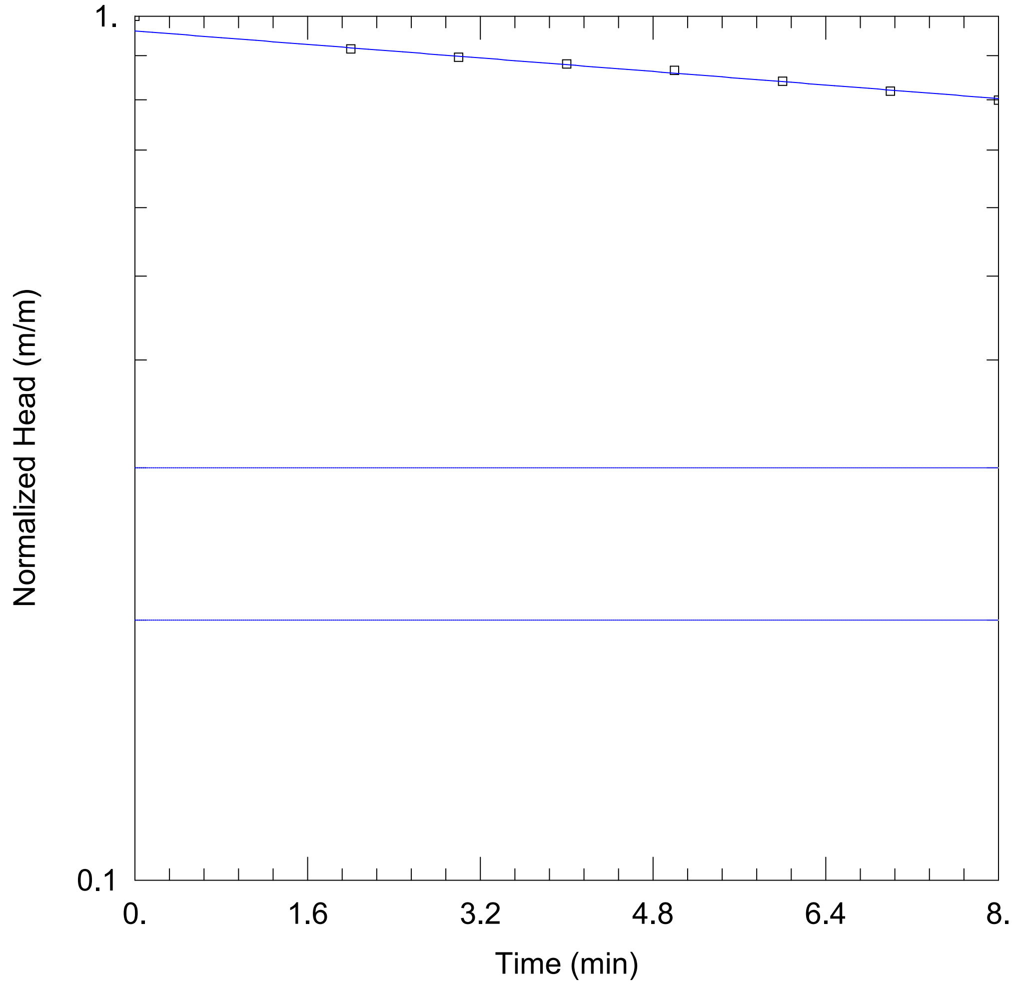
WELL DATA (ARDG-P02)

Initial Displacement: 9.17 m Static Water Column Height: 3.78 m
Total Well Penetration Depth: 3.78 m Screen Length: 3. m
Casing Radius: 0.025 m Well Radius: 0.051 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice

K = 0.02916 m/day y0 = 11.9 m



WELL TEST ANALYSIS

Data Set: \...\ARDG-P02_rising.aqt
Date: 12/08/23 Time: 10:15:35

PROJECT INFORMATION

Company: GHD Pty Ltd
Client: ARDG Pty Ltd
Project: 2219467
Location: Stone Ridge Quarry
Test Well: ARDG-P02
Test Date: 1/12/2023

AQUIFER DATA

Saturated Thickness: 42.47 m
Anisotropy Ratio (Kz/Kr): 1.

WELL DATA (ARDG-P02)

Initial Displacement: 6.87 m
Total Well Penetration Depth: 3.873 m
Casing Radius: 0.025 m
Static Water Column Height: 6.87 m
Screen Length: 3. m
Well Radius: 0.051 m

SOLUTION

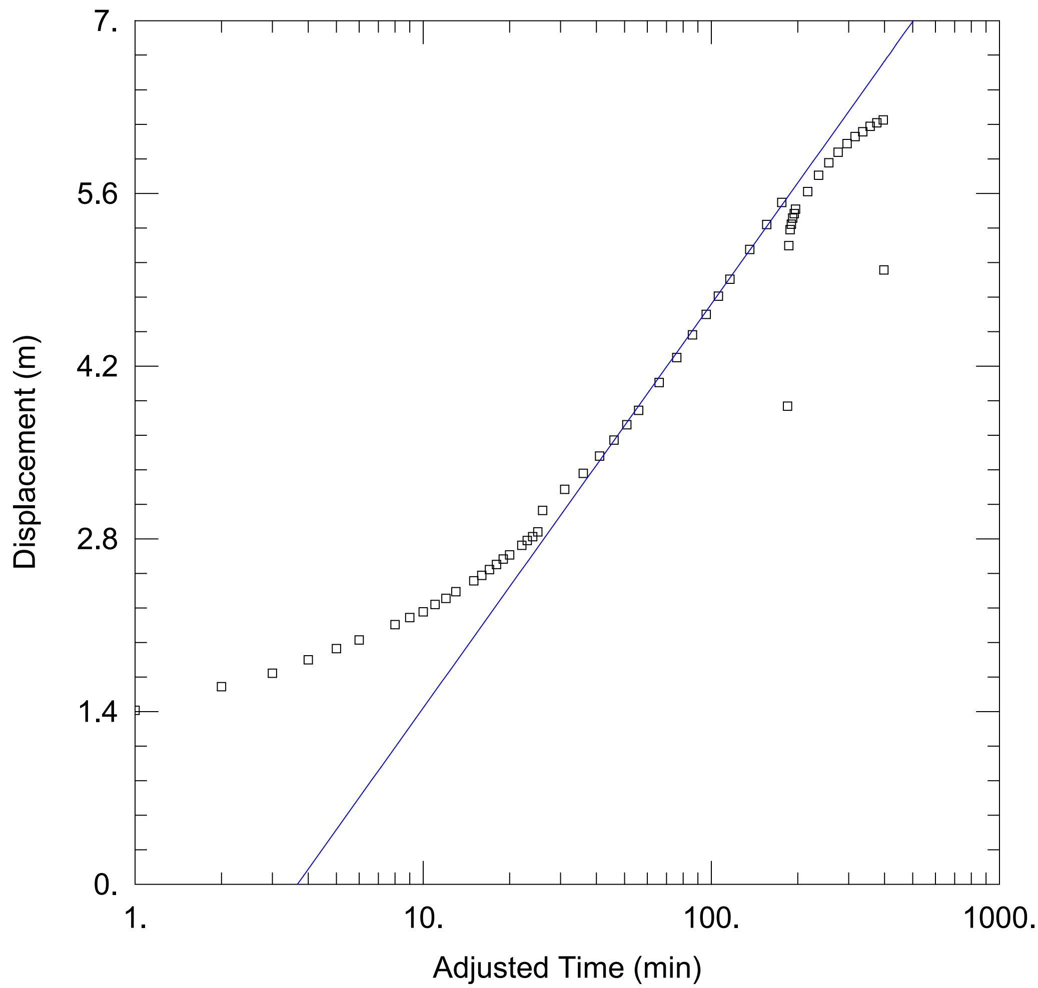
Aquifer Model: Unconfined
Solution Method: Bouwer-Rice

K = 0.009244 m/day
y0 = 6.604 m

Appendix C

Pump test plots

C-1 Pump test plots



WELL TEST ANALYSIS

Data Set: \...\ARDG-P06_PW_Average.aqt
Date: 02/19/24 Time: 09:56:07

PROJECT INFORMATION

Company: GHD Pty Ltd
Client: ARDG Pty Ltd
Project: 2219467
Location: Stone Ridge Quarry
Test Well: ARDG-P06
Test Date: 17/02/2024

AQUIFER DATA

Saturated Thickness: 20. m Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA

Pumping Wells

Well Name	X (m)	Y (m)
ARDG-P06	0	0

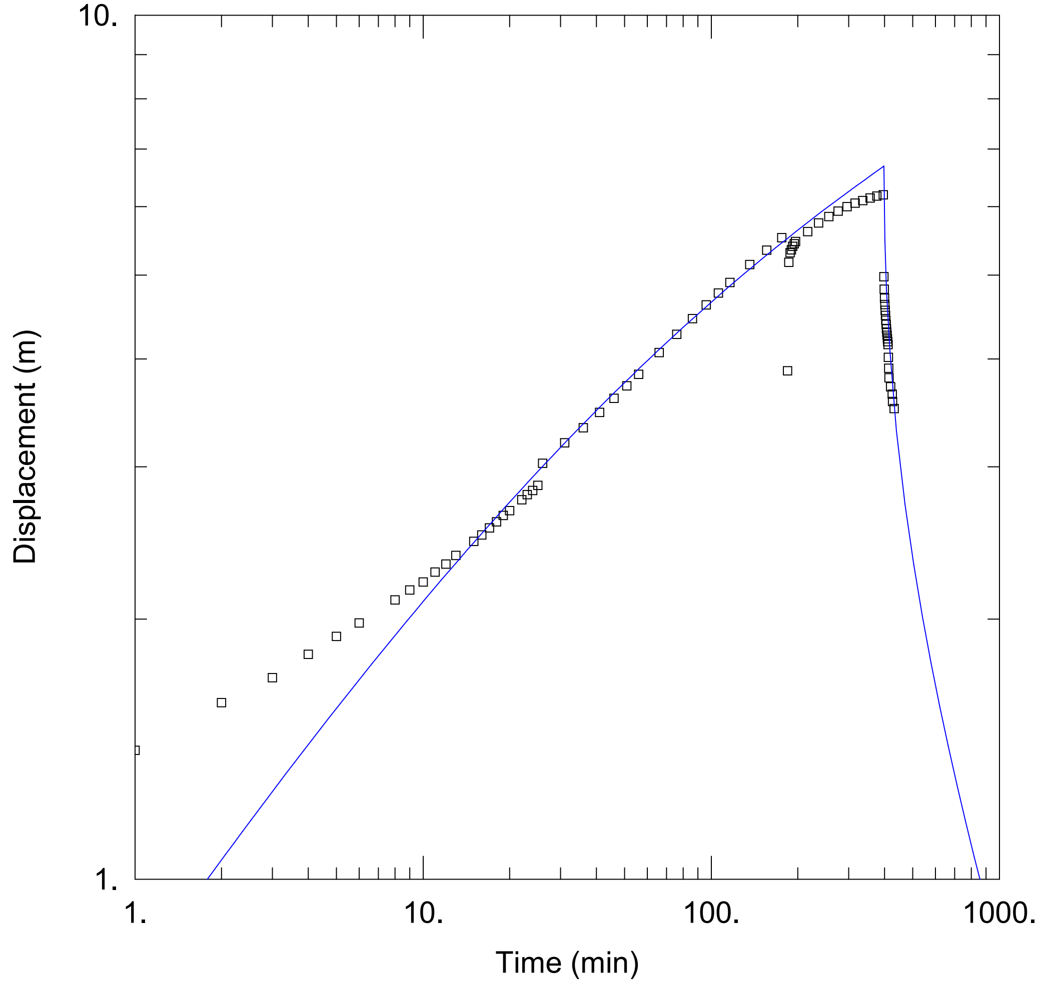
Observation Wells

Well Name	X (m)	Y (m)
□ ARDG-P06	0	0

SOLUTION

Aquifer Model: Confined Solution Method: Cooper-Jacob

T = 11.61 m^2/day S = 11.18



WELL TEST ANALYSIS

Data Set: \...\ARDG-P06_PW_Average.aqt
Date: 02/19/24 Time: 09:57:37

PROJECT INFORMATION

Company: GHD Pty Ltd
Client: ARDG Pty Ltd
Project: 2219467
Location: Stone Ridge Quarry
Test Well: ARDG-P06
Test Date: 17/02/2024

AQUIFER DATA

Saturated Thickness: 20. m Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA

Pumping Wells

Well Name	X (m)	Y (m)
ARDG-P06	0	0

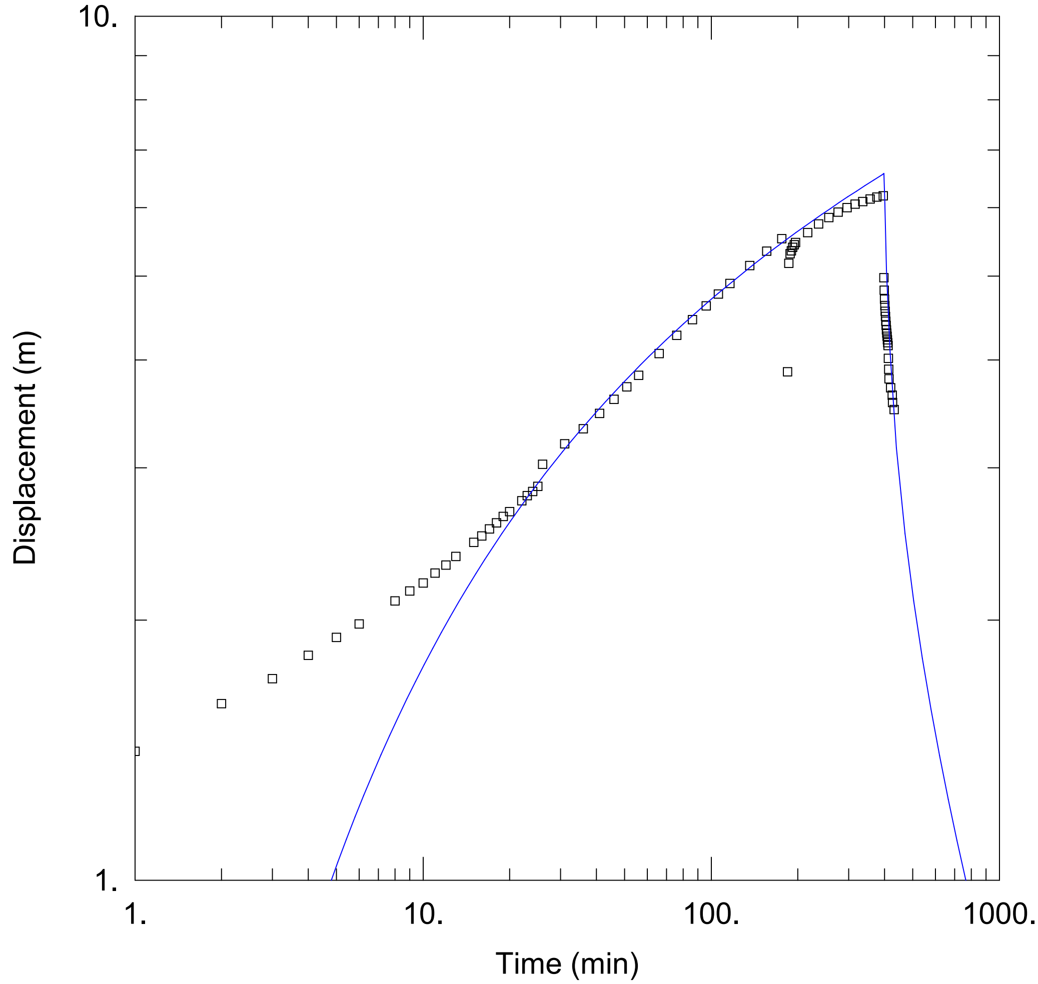
Observation Wells

Well Name	X (m)	Y (m)
□ ARDG-P06	0	0

SOLUTION

Aquifer Model: Confined Solution Method: Papadopulos-Cooper

T = 9.672 m^2/day S = 22.46
r(w) = 0.077 m r(c) = 0.05 m



WELL TEST ANALYSIS

Data Set: \...\ARDG-P06_PW_Average.aqt
Date: 02/19/24
Time: 09:55:27

PROJECT INFORMATION

Company: GHD Pty Ltd
Client: ARDG Pty Ltd
Project: 2219467
Location: Stone Ridge Quarry
Test Well: ARDG-P06
Test Date: 17/02/2024

WELL DATA

Pumping Wells

Well Name	X (m)	Y (m)
ARDG-P06	0	0

Observation Wells

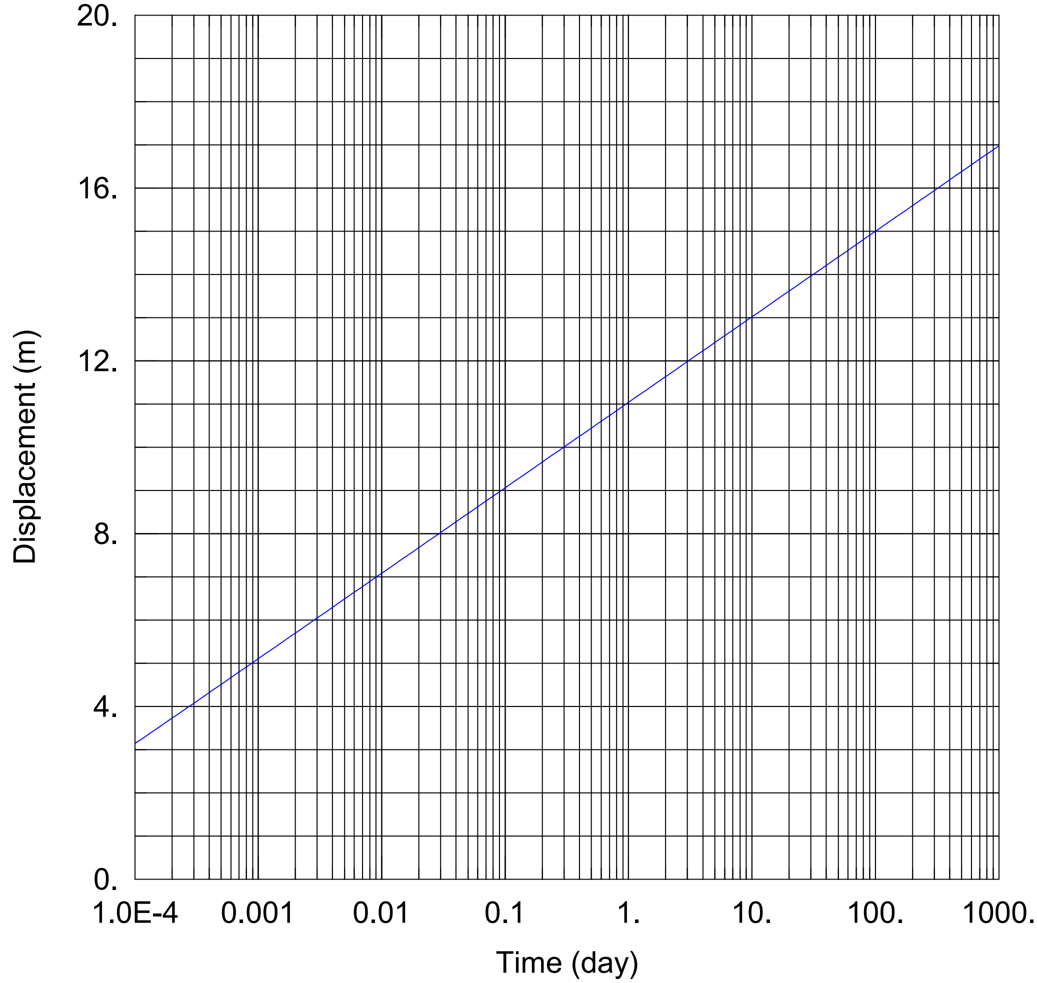
Well Name	X (m)	Y (m)
□ ARDG-P06	0	0

SOLUTION

Aquifer Model: Confined
Solution Method: Theis/Hantush

T = 12.06 m^2/day
Kz/Kr = 0.1
S = 10.4
b = 20. m

C-2 Forward analysis



WELL TEST ANALYSIS

Data Set: \...\FWD.aqt
Date: 02/22/24 Time: 11:05:55

PROJECT INFORMATION

Company: GHD Pty Ltd
Client: ARDG Pty Ltd
Project: 2219467
Location: Stone Ridge Quarry
Test Well: ARDG-P06 - FW
Test Date: 17/02/2024

WELL DATA

Pumping Wells

Well Name	X (m)	Y (m)
ARDG-P06-FW	0	0

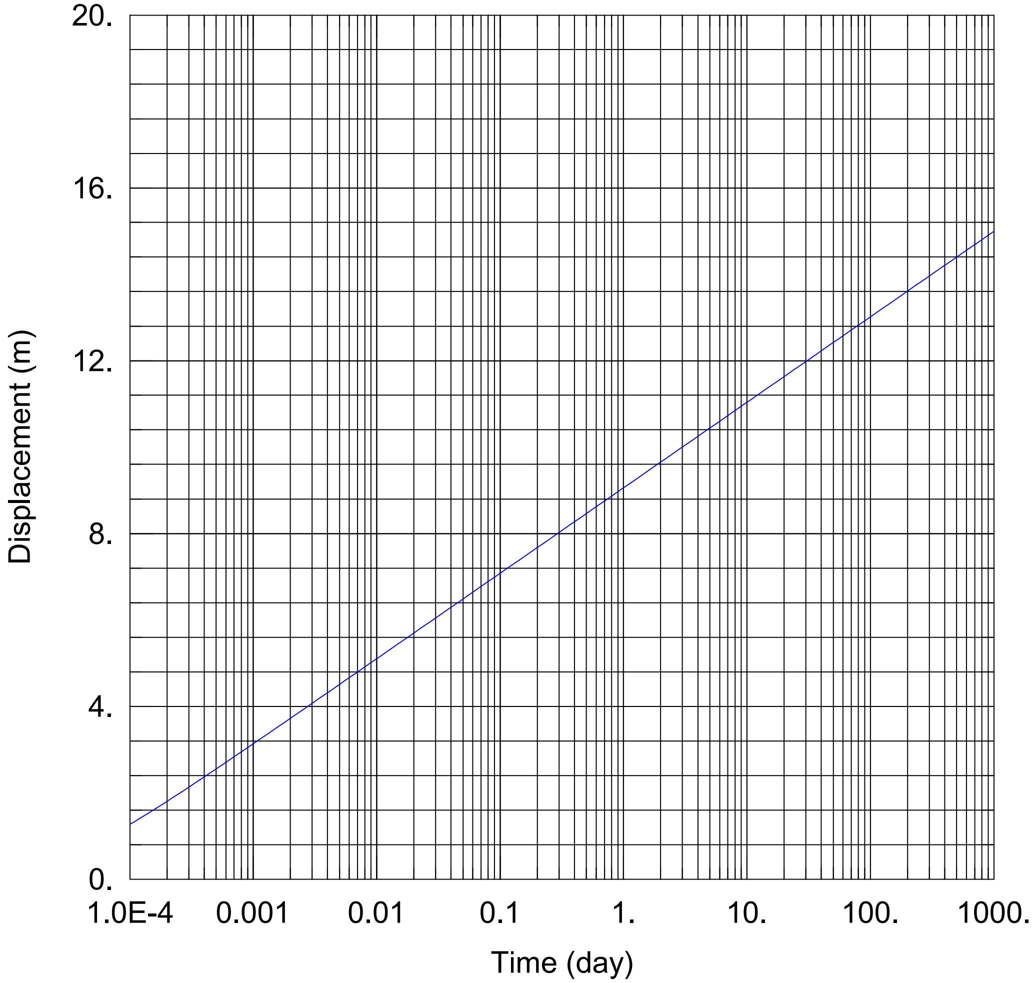
Observation Wells

Well Name	X (m)	Y (m)
□ ARDG-P06-FW	0	0

SOLUTION

Aquifer Model: Confined
Solution Method: Theis/Hantush

T = 10. m^2/day
S = 0.01
Kz/Kr = 0.1
b = 20. m



WELL TEST ANALYSIS

Data Set: \...\FWD.aqt
Date: 02/22/24 Time: 11:03:24

PROJECT INFORMATION

Company: GHD Pty Ltd
Client: ARDG Pty Ltd
Project: 2219467
Location: Stone Ridge Quarry
Test Well: ARDG-P06 - FW
Test Date: 17/02/2024

WELL DATA

Pumping Wells

Well Name	X (m)	Y (m)
ARDG-P06-FW	0	0

Observation Wells

Well Name	X (m)	Y (m)
▫ ARDG-P06-FW	0	0

SOLUTION

Aquifer Model: Confined
Solution Method: Theis/Hantush

T = 10. m^2/day
Kz/Kr = 0.1
S = 0.1
b = 20. m

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