# Appendix R

## Hydrogeological Assessment Report

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**Project Number 19630.000.001**

# Hydrogeological Assessment

Wairakei South Development

Submitted to:
Bell Road Limited Partnership
1 Golden Sands Drive
Papamoa
Tauranga

# Executive Summary

The Wairakei South Development project involves the proposed development of 360 hectares of agricultural land on the outskirts of Tauranga into residential and commercial development. The site is low lying, in an area extensively drained and susceptible to flooding, hence, a key component of the development is earthworks to raise post development ground levels above flood levels. Engineered swales and two new stormwater attenuation ponds will replace existing agricultural drains to continue to manage groundwater levels and convey surface water to from west of the site towards the Kaituna River. This report undertakes an assessment of hydrogeologic effects as a result of the project, including the intersection between groundwater and geotechnical, civil, and stormwater design.

Subsurface conditions are comprised of Holocene alluvial deposits overlain by peat. Hydraulic conductivity of the sand and peat is generally high. The development will involve raising the landform using imported fill, proposed to be pumice dominant soils from a nearby source. The imported pumice fill is expected to be 1-2 orders of magnitude lower in permeability. Currently, groundwater is typically shallow (0.3 – 1 m below ground level). The site is susceptible to shallow ponding during high-intensity rainfall events, and sea level rise projections (up to 1.59 m by 2150 at the nearby coast and Kaituna River) present a long-term inundation risk to the broader floodplain area.

A combination of 3D numerical and 2D analytical groundwater modelling was used to assess the potential effects of the proposed development on groundwater and to provide groundwater discharge rates to the stormwater designers to account for and manage groundwater appropriately.

Modelling of the post-development condition predicts minor groundwater level changes, including drawdown of up to 0.5 m at the site boundary, with localised drawdown near the new attenuation ponds and swales. Where drawdown beyond the site is in compressible peat soils, drawdown reduces to less than 0.1 m within 160 m of the site, and the land use is agricultural farmland where we consider associated drawdown effects are less than minor for the current land use.

Groundwater level mounding is expected to occur where existing drains are infilled, however, our assessment and modelling results conclude that groundwater will remain 1 m below the finished raised ground level, even under future rainfall (1% AEP) or sensitivity scenarios. Temporary stormwater rise may cause localised saturation of embankment sides near swales during storm events but analysis indicates this dissipates quickly.

The impermeable surface area and low-permeability pumice fill will reduce percolation to groundwater by approximately 60% over the site area, resulting in a 33% reduction in groundwater discharge to the development swales compared to the existing situation in the farm drains. Net discharge to swales post development is estimated to be 3,700 $m^3$/day across the site. The balance of the reduction in infiltration is portioned to overland flow or direct discharge to stormwater systems designed to convey surface and groundwater flow safely into the on-site attenuation ponds for storage prior to controlled discharge. At the northeast attenuation pond, groundwater levels are expected to be close to the pond base in summer and to seasonally seep upward into the pond during winter. The southwest pond will experience minor but persistent upward groundwater seepage all year. Flow rates have been passed on and incorporated in the stormwater model being prepared by others. Hydraulic parameters of the compacted pumice fill contain the highest uncertainty. Infiltration and hydraulic conductivity in the pumice fill should be confirmed within a site trial setting, and this assessment updated if the tested values vary sufficiently from the values adopted in this assessment.

ENGEO's groundwater modelling and hydrogeological assessment conclude that the proposed Wairakei South development will have less than minor effects on regional groundwater levels, flow gradients, and discharges compared to existing conditions, primarily due to the proposed activity largely rearranging existing drainage network which has a controlling influence of groundwater and surface water, rather than adding or removing drainage. The proposed design manages groundwater effectively and avoids excessive groundwater mounding. In combination with the proposed civil and stormwater design, the project effectively mitigates potential groundwater impacts and supports long-term flood resilience. The hydrogeological response remains stable under current and projected climate conditions, with off-site or downstream effects considered to be less than minor.

During site formation earthworks, a groundwater take and use consent is required to supply up to 1,955 $m^3$/day of water, primarily for dust control purposes. The groundwater take is proposed to be abstracted from multiple groundwater infiltration ponds across the site (dependent on the area undergoing works at the time), sourced from the Kaituna Secondary (upper) groundwater management unit. The take being sought exceeds the indicative available aquifer allocation, however, the potential effects of groundwater abstraction on surface water features, nearby users, and ground consolidation are considered to be less than minor, provided the infiltration ponds are installed greater than 50 m from any boundary.

Additional to this assessment, ENGEO recommends field trials and testing on imported pumice fill be undertaken during detailed design to confirm the parameters adopted in this assessment are sufficiently conservative, and the development of a site-specific monitoring and mitigation plan be developed and implemented prior to any construction works to confirm groundwater changes remain within the envelope of effects outlined in this report.

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# 1 Introduction, Scope, and Purpose

ENGEO Ltd was requested by Bell Road Limited Partnership to undertake an assessment of hydrogeological effects for the proposed residential and commercial development at Bell Road, Wairakei South, (herein referred to as ‘the site’), shown in Figure 1.

This work has been undertaken in general accordance with our signed proposal (ref. P19630.000.001_14, dated 31 July 2025).

Our scope of works includes:

- Collection of factual data through the installation of piezometers and monitoring of groundwater levels and undertaking *in situ* hydraulic conductivity testing.
- Conceptual and numerical assessment of groundwater flow regime, pre and post development. In general accordance with Class 2 of the Australian Groundwater Guidelines.
- Evaluation of the potential hydrogeological effects of the development, to support Fast-Track consent application.
- Providing groundwater related inputs into stormwater design, including groundwater inflow into stormwater management areas.
- Assessment and support of application to take and use shallow groundwater for construction related activities.

# 2 Description of Activity

## 2.1 Proposed Development

The site comprises a total of 360 ha of agricultural land, comprised of eight property parcels located along Bell Road in the outskirts of Tauranga (Figure 1Figure 1). The site is legally described as:

- Lot 2 DPS 81677 – 151,685
- Lot 1 DPS 54113 – 91,825
- Lot 1 DPS 81677 – 26,974
- Lot 1 DPS 69524 – 156,134
- Lot 1 DP 537375 - 156,095.30 ha
- Section 13 SO 458365 – 600,239
- Lot 1 DP 29530 – 1,139,069
- Lot 2, DP 553506 – 992,416 ha

The proposed development at Bell Road comprises a combination of residential dwellings and commercial sites. A full project description is provided in the application AEE.

Locally sourced pumice fill is proposed to raise the Existing Ground Level (EGL) to the design elevation for the development Proposed Ground Level (PGL) to achieve finished ground levels for construction above the modelled flood levels (Maven, 2025). The pumice fill will be placed on top of a 0.5 m thick layer of pumice-dominant sand over most of the site, placed to encourage pore pressure dissipation and subsequent consolidation of the existing peat underlying the site. In the north of the site where the underlying peat is thinner or non-existent the geotechnical designs propose to excavate the existing peat and replace it with imported granular fill material.

The side slopes of the pumice fill blanket will require edge stabilisation measures, which are anticipated to consist of fill reinforced with geogrid or replacing the edge zone with cohesive fills. However, these stabilisation works were not included in the groundwater assessment as their specifics will only be determined during the detailed design phase.

The development will relocate existing agricultural surface water and groundwater drainages by infilling existing farm drains and replacing them with engineered drainage swales incorporated into the development design. Two stormwater retention ponds are proposed to attenuate stormwater flow during rainfall events, located at the north-eastern and southern extent of the site.

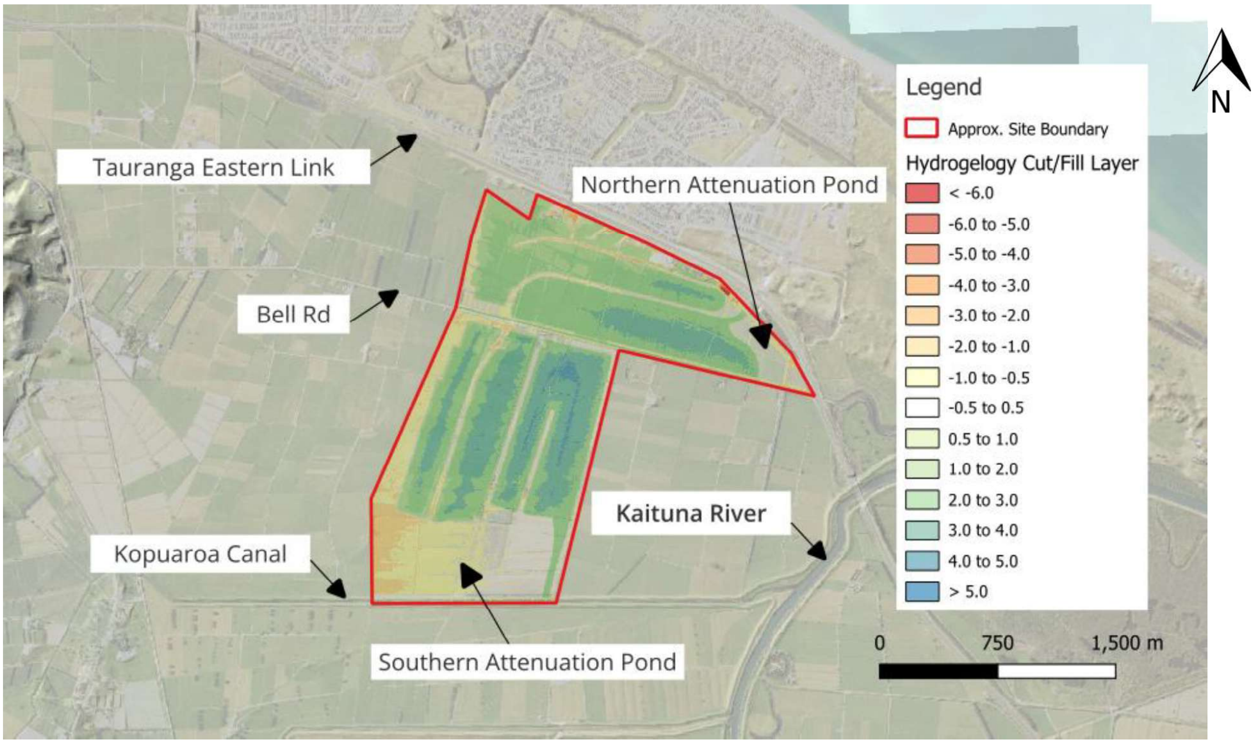


**Figure 1: Site Plan with 5 m Contours Showing Hydrogeologic Model Cut - Fill Plan for the Site**

## 2.2 Groundwater Use Requirements

### 2.2.1 Water Requirements and Effective Use of Water

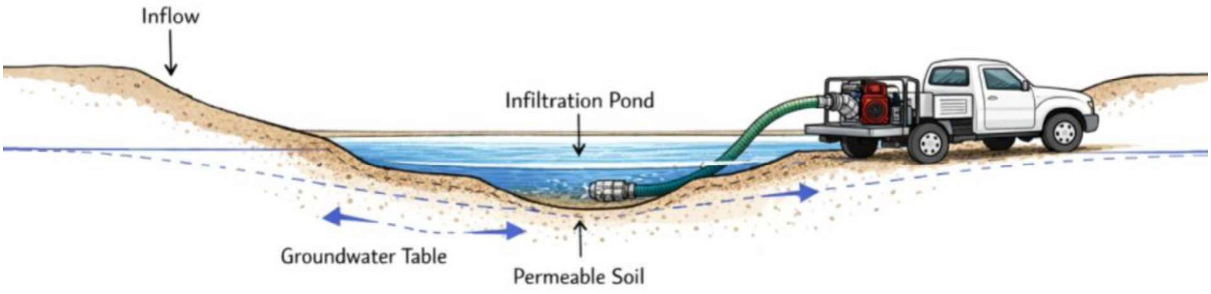
A site water supply is necessary to ensure effective dust control during the earthworks phases. Civil design engineers have calculated an estimated requirement of 1955 m³/day[1] of water, utilising the following application rates:

- 5 mm depth of dust control water for construction areas
- 7 mm depth of water for roadways

The total volume has been determined based on the largest stage of construction, providing a conservative evaluation of potential impacts from groundwater abstraction.

### 2.2.2 Method of Abstraction

Multiple groundwater infiltration ponds are the preferred method of abstraction. The assessment as adopted a design size of 10 x 10 m base x 6 m deep for the assessment. A slope of 1:5 has been modelled, however, slope stability analysis for ponds is beyond the scope of this assessment and will be considered by Geotechnical Engineers during detailed design. Dewatering spears may be considered as an alternative methodology for abstraction water, however will require more active maintenance for prolonged use at any given location. The depth and layout would be arranged in general accordance with the size and depth of the infiltration pond, and is therefore not specifically assessed in addition to the assessment herein.



## 3 Terrain and Hydrogeological Setting

The site is situated within the low-lying coastal plain of south / southeast of Papamoa, characterised by flat terrain with elevations generally between 0.5 and 2 metres above mean sea level (NZVD). Surface drainage trends southeast toward the Kaituna River and associated wetlands, supported by an extensive network of natural and artificial channels developed for agricultural purposes. The Papamoa dune sand system forms a geomorphic boundary north of site with an elevation 5 – 7 m NZVD.

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[1] Email communication from Maven. Dated 11 February 2026.

Geologically, the area is underlain by Holocene alluvial deposits of the Tauranga Basin, comprising variable sequences of sand, silt and mud, and peat at the surface. Peat up to 3 metres thick overlies the southern and eastern portions of the site, but is observed in boreholes to gradually thin at the transition with the dune sands present along the northern site margin, merging with the Papamoa dune sand beneath the Tauranga Eastern Link (TEL) (ENGEO, 2025). Beneath the peat, alluvial sand deposits approximately 6 m thick overlay thick deposits of reworked sand and silts.

Surface water and groundwater are interconnected due to the shallow water table, typically within 0.5 – 1 m of ground surface. Artificial drains and canals, including the Kopuaroa Canal and Bell Road Drain systems, exert strong control of the site hydrology. Groundwater gradients towards and across the site are small due to the flat and extensive surrounding environment. The Kaituna River has a tidal range of up to 1 m at State Highway 2, however, a tidal effect of less than 0.1 m was observed in piezometers on-site.

The shallow groundwater system is unconfined, supported by groundwater monitoring data in the peat and underlying sand units having similar levels and fluctuations during the monitoring period.

Hydraulic conductivity of alluvial sand layers beneath site is expected to be moderate to high with higher values in the dune sands to the north. Peat is of comparatively moderate hydraulic conductivity when uncompressed, reducing in conductivity when consolidated. Where present at surface, peat soil once saturated will limit groundwater recharge due to its lower hydraulic conductivity leading to higher proportions of runoff during heavy rainfall events.

Recharge is derived from rainfall and minor upgradient inflows, with discharge occurring through the drainage network toward the Kaituna River and coastal boundary.

Flood hazard mapping indicates susceptibility to shallow ponding during high-intensity rainfall (TCC, n.d.). Regional projections suggest sea level rise of 1.59 metres by year 2130, under high-emission scenarios (SSP5 8.5), increasing long-term inundation risk of the wider floodplain area (Bay of Plenty Regional Council, 2021).

The site represents an alluvial aquifer system comprising flat, low-lying land with a surficial peat layer, underlain predominantly by sandy soils. These conditions form the basis of our conceptual hydrogeologic model.

Once the landform is modified, imported pumice will overlay a sand blanket. Once consolidation has occurred, the existing peat will become a compacted thinner compressed peat layer beneath the platform areas.

# 4 Site Investigations and Available Monitoring Data

ENGEO conducted hydrogeological investigations on-site to test and refine the conceptual hydrogeological model, including recording continuous groundwater levels for flow analysis and rainfall response and to support the adoption of model parameters from hydraulic conductivity testing. Site investigation data supported calibration of numerical modelling parameters.

Additional supporting data including laboratory testing on similar geological materials, nearby piezometer, tidal heights, river stages and spot flow measurements were provided by Tauranga City Council (TCC) and Bay of Plenty Regional Council (BOPRC) to inform the hydrogeological assessment.

## 4.1 Site -Specific Installation and Instrumentation

Perry Geotech Ltd installed 21 piezometers on-site at locations presented in the Site Investigation Plan (Appendix 1:1). PZ01 to PZ09 were installed in May 2022 using wash bore techniques. PZ10 to PZ16 were installed in May 2025 using screw drilling techniques, and PZ17 to PZ21 were installed in August 2025, both using rotary core techniques. Construction details are summarised in Table 1. Piezometers were generally installed near the perimeter of paddocks to facilitate easier access for the drilling equipment across soft ground and to reduce the likelihood of damage from farm machinery and livestock.

ENGEO logged the soil core samples during drilling and installed electronic data loggers in the developed piezometers for continuous groundwater level monitoring. Investigation logs and core photographs are included in the Geotechnical Factual Report – Appendix O in the appendices to the application (ENGEO, 2025).

Piezometers were installed in a staged approach following the incorporation of additional areas into the project.

**Table 1: Piezometer Installation Details**

| Test ID | Coordinates[1] | | Elevation[2] (m NZVD) | Total Depth (m bgl) | Bottom of Peat[3] (m bgl) | Static Water Level[4] (m bgl) | Details |
|---|---|---|---|---|---|---|---|
| | mN | mE | | | | | |
| PZ01 | 802509.73 | 389733.45 | 0.63 | 4.0 | - | 0.125 | 50 mm PVC pipe, screened entire length |
| PZ02 | 802847.12 | 388446.46 | 1.29 | 4.0 | - | 0.408 | 50 mm PVC pipe, screened entire length |
| PZ03 | 802880.92 | 387700.41 | 1.43 | 4.0 | - | 0.687 | 50 mm PVC pipe, screened entire length |
| PZ04 | 803143.13 | 388269.69 | 1.59 | 4.0 | - | 0.033 | 50 mm PVC pipe, screened entire length |
| PZ05 | 802830.71 | 388912.1 | 0.81 | 4.0 | - | 0.054 | 50 mm PVC pipe, screened entire length |
| PZ06 | 802496.8 | 389237.48 | 1.46 | 4.0 | - | 1.027 | 50 mm PVC pipe, screened entire length |
| PZ07 | 803474.02 | 387814.88 | 1.59 | 4.0 | - | -0.098 | 50 mm PVC pipe, screened entire length |
| PZ08 | 802871.65 | 389390.15 | 0.94 | 4.0 | - | 0.181 | 50 mm PVC pipe, screened entire length |
| PZ09 | 803257.17 | 388792.74 | 4.0 | 4.0 | - | 1.982 | 50 mm PVC pipe, screened entire length |
| PZ10 | 801538.113 | 387140.354 | 2.8 | 4.0 | - | 1.473 | 50 mm PVC pipe, screened entire length |
| PZ11 | 801022.442 | 387617.78 | 3.0 | 4.0 | >4.0 | 2.317 | 50 mm PVC pipe, screened entire length |
| PZ12 | 801719.003 | 387847.191 | 1.49 | 4.0 | 3.2 | 0.65 | 50 mm PVC pipe, screened entire length |
| PZ13 | 802650.359 | 388076.09 | 1.38 | 4.0 | 1.8 | 0.538 | 50 mm PVC pipe, screened entire length |
| PZ14 | 801473.588 | 387432.077 | 1.62 | 4.0 | 2.8 | 0.255 | 50 mm PVC pipe, screened entire length |

| Test ID | Coordinates[1] | | Elevation[2] (m NZVD) | Total Depth (m bgl) | Bottom of Peat[3] (m bgl) | Static Water Level[4] (m bgl) | Details |
|---|---|---|---|---|---|---|---|
| | mN | mE | | | | | |
| PZ15 | 802056.503 | 387619.983 | 1.21 | 4.0 | 2.0 | 0.412 | 50 mm PVC pipe, screened entire length |
| PZ16 | 802342.83 | 387755.098 | 1.33 | 4.0 | 1.4 | 0.412 | 50 mm PVC pipe, screened entire length |
| PZ17 | 801190.738 | 388324.566 | 1.0 | 4.5 | 1.0 | 0.441 | 50 mm PVC pipe, screened between 1.2 – 4 m in sand |
| PZ18 | 801310.808 | 387976.067 | 0.85 | 4.5 | 1.5 | 0.448 | 50 mm PVC pipe, screened between 1.5 – 4 m in sand |
| PZ19 | 802273.639 | 388599.867 | 0.82 | 4.5 | 3.5 | 0.793 | 50 mm PVC pipe, screened between 0.7 – 2.2 m in peat |
| PZ20 | 801830.386 | 388380.217 | 1.06 | 4.5 | 3.0 | 0.404 | 50 mm PVC pipe, screened between 1.4 – 2.4 m in peat |
| PZ21 | 802326.558 | 388250.241 | 1.17 | 4.5 | 3.0 | 0.719 | 50 mm PVC pipe, screened between 3 – 4 m in sand |

[1] Location from handheld GPS. Coordinate system is NZGD2000 / Bay of Plenty 2000.

[2] Elevation in Meter. Vertical datum is NZVD2016.

[3] Peat depth not recorded for PZ01 to PZ09 due to wash bore installation techniques.

[4] Mean water level during July-August 2025.

## 4.2 Groundwater Level Monitoring

### Site-Specific Monitoring

ENGEO monitored groundwater levels in the north block (north of Bell Road) for an initial period of two years between May 2022 and April 2024. Observations are presented in Figure 2. Data loggers were redeployed after the incorporation of the south block (south of Bell Road) into the project across all piezometers on-site from 1 July to 1 November 2025. Monitoring results are presented in Figure 3.

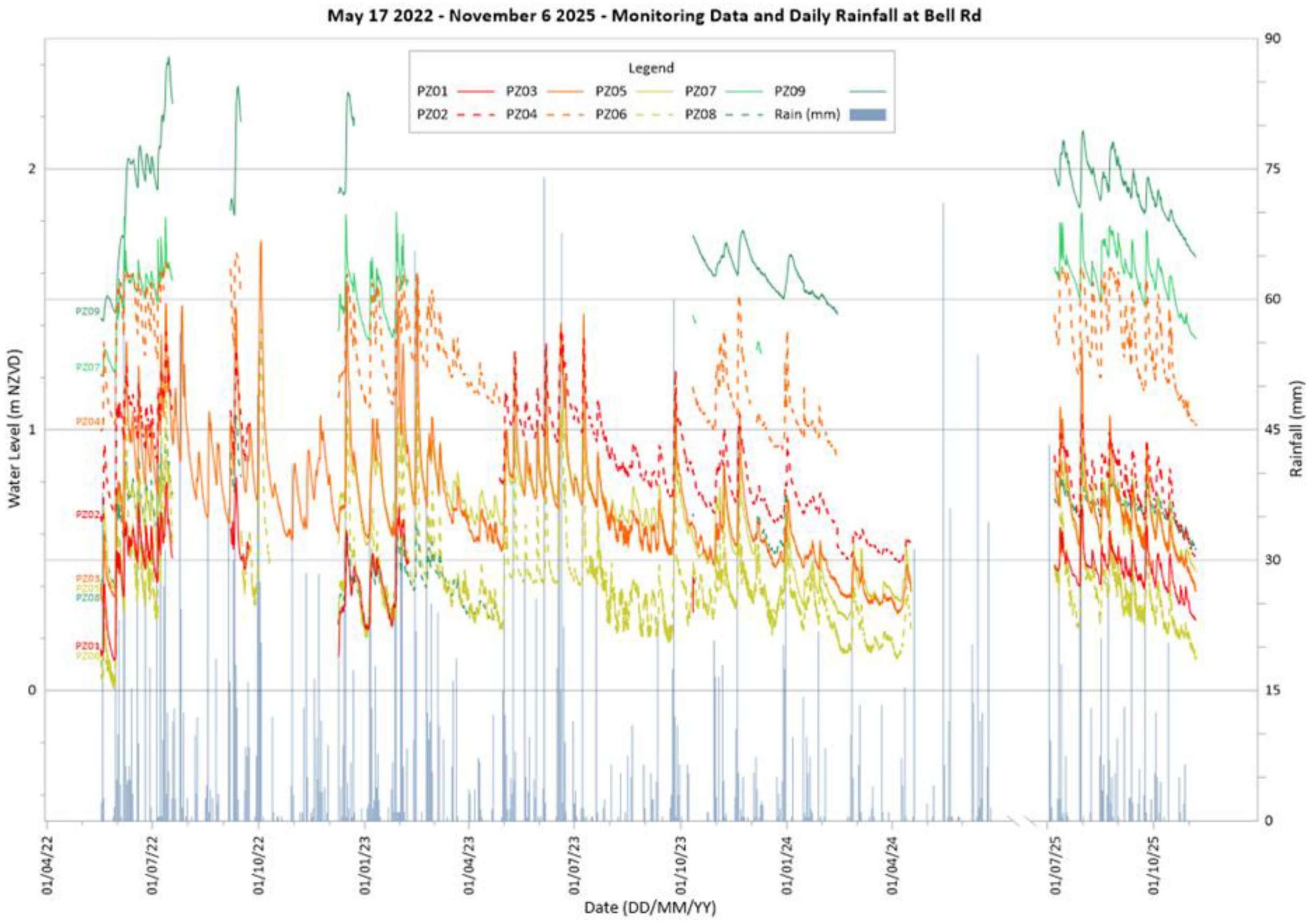

**Figure 2: Groundwater Levels - North Block Only 2022 to 2025**

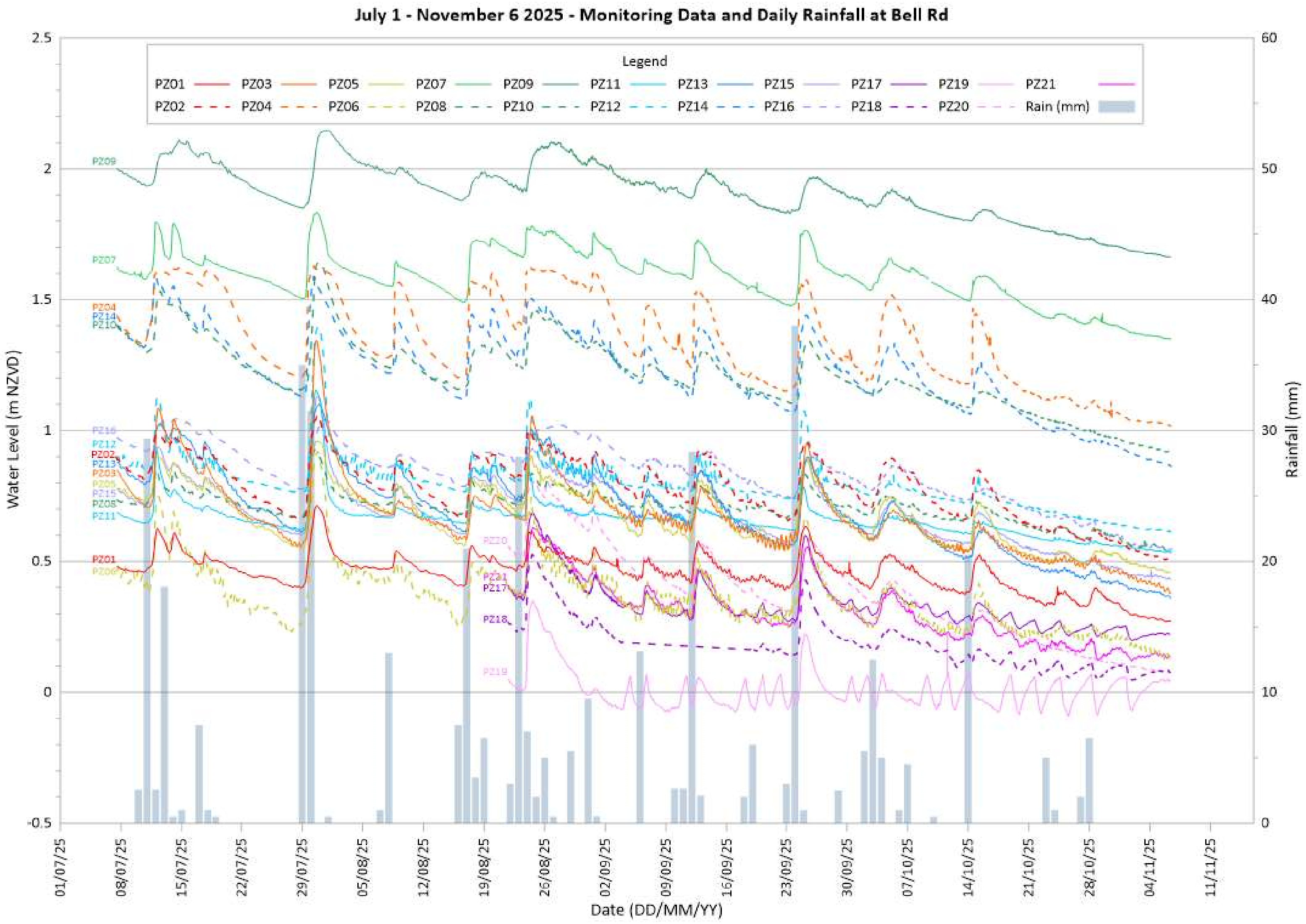

Figure 3: Groundwater Level Monitoring (North and South Blocks) - Winter 2025

A groundwater response of up to 0.25 m directly after a rainfall event was common across most of the site, with larger responses recorded during the most extreme events, confirming strong groundwater–surface water interaction. The peak groundwater response dissipates within 1 to 2 days following the rain event. A seasonal range of 0.2 m is observed between winter and summer levels in piezometers PZ01 to PZ09.

Approximately 0.1 m of tidal response was observed in piezometer PZ06, which is the closest piezometer to the Bell Road drain. Less than 0.1 to none tidal response was observed in all other piezometers. The muted tidal response provides an indication of the hydraulic connection between the site and the tidal influence of the Kaituna River and the ocean. Tide height and river stage monitoring data were obtained from the BOPRC data portal[2], and are discussed in more detail in Section 5.9.1.

Observed groundwater levels were interpolated into a piezometric surface for the site using the kriging geostatistical function in Surfer (v.27.1.299; Golden Software). Due to the response to rainfall, a 50th percentile and 95th percentile groundwater level was calculated across the three-month monitoring period at each piezometer. The 50th percentile was considered to represent a typical “winter” water level, while the 95th percentile was considered an upper bound “high water level”. Piezometric surfaces were interpolated on the observed groundwater levels only, excluding stream effects to prevent over-constraint of the mapping and support assessing the wider groundwater flow gradients. Groundwater elevation contour plans are provided in Appendix 1.2 and 1.3.

Piezometric groundwater contour plots illustrate gradients running west to east across the southern block. In the northern block, groundwater gradients are towards the southeast, indicating groundwater enters the site from the Papamoa dunes before progressing toward the Kaituna River.

## 4.3 *In Situ* Hydraulic Conductivity Testing

ENGEO attended site on 21 August and 4 September 2025 to conduct seven hydraulic conductivity slug tests. Test locations were spread across the site (Appendix 1.1). Slug tests were conducted in the piezometers with screens across the combined peat and alluvial sand beneath across site, and or within the peat or sand individually, as presented in Table 2.

Testing comprised a 2 m long physical slug and electronic data loggers recording at 5 second intervals. Testing methodology was undertaken in general accordance with BS EN ISO 22282-2:2012 as recommended in the New Zealand Ground Investigation Specification (Volume 1; Rev. 0, 2017). Data was reviewed for anomalies, then processed in AquiferTest (v14.0, Waterloo Hydrogeologic) using both the Bouwer & Rice (1976) and Hvorslev (1951) methods for unconfined conditions. Both methods returned similar estimates within each piezometer, so an average has been provided here. Results are summarised in Table 2 and full analysis outputs are included in Appendix 2.

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[2] BOPRC Data Portal: Data - Bay of Plenty Regional Council WebPortal

**Table 2: Hydraulic Conductivity Testing**

| Piezo ID | Screen Interval (m bgl) | Lithology | Static Water Level (m bgl) | Falling Head Test[1] (m/s) | Rising Head Test[1] (m/s) | Well Average (m/s) |
|---|---|---|---|---|---|---|
| | | | | Hydraulic Conductivity | | |
| PZ17 | 1.2 – 4.5 | Sand | 0.58 | 1.15 x10-5 | 1.05 x10-5 | 1.10 x10-5 |
| PZ18 | 1.5 – 4.5 | Sand | 0.575 | 3.60 x10-5 | 3.70 x10-5 | 3.65 x10-5 |
| PZ19 | 0.7 – 2.1 | Peat | 0.77 | 8.80 x10-6 | 7.60 x10-6 | 8.20 x10-6 |
| PZ20 | 1.4 – 2.4 | Peat | 0.49 | 1.39 x10-5 | 1.20 x10-5 | 1.29 x10-5 |
| PZ21 | 3 – 4 | Sand | 0.755 | 1.07 x10-5 | 9.00 x10-6 | 9.85 x10-6 |
| PZ04 | 0 – 4 | Peat & Sand | 0.265 | 4.51 x10-6 | 3.94 x10-6 | 4.23 x10-6 |
| PZ05 | 0 – 4 | Peat & Sand | 0.155 | 3.73 x10-6 | 6.10 x10-6 | 1.87 x10-6 |

1. Average of Hvorslev and Bouwer & Rice analysis

## 4.4 Additional Reference Data

### 4.4.1 Surface Flow Data

ENGEO requested available surface water flow data from TCC. Spot measurement data were provided for several locations as shown in the Site Investigation and Monitoring Plan in Appendix 1:1. However, spot data measurements were outside of the simulation period and subject to significant error given the inability to decipher between surface water and groundwater flows.

### 4.4.2 Tauranga City Council - Groundwater Monitoring Data

Additional groundwater monitoring data located on the Papamoa dunes was provided by TCC for the period from January 2020 to June 2025 at 13 locations within the model domain (Model domain discussed in Section 6.40). More up to date data was not available at the time of assessment. The Papamoa dune piezometer locations are included in Appendix 1:1.

Groundwater elevations ranged from 0.5 m NZVD in summer near the coast to 3.5 m NZVD in winter closer to the centre of the Papamoa dunes. The data was used as calibration targets for model calibration.

### 4.4.3 Laboratory Testing

Golder Associates undertook one-dimensional laboratory consolidation testing of the peat deposit samples collected at varying depths in test pits during preparation of their 2020 report (Golder Associates (NZ) Limited, 2020). Applied vertical pressures for one-dimensional consolidation tests were between 5 kPa to 800 kPa. The test method estimates a sample permeability under consolidation, which was used to inform the adopted permeability parameters for consolidated peat with the model. The results ranged from 1 x $10^{-8}$ m/s to $1 \times 10^{-11}$ m/s with increasing consolidation. The laboratory results are included within the Wairakei South - Geotechnical Factual Report – Appendix O in the appendices to the application (ENGEO, 2025).

### 4.4.4 Infiltration Testing

As the source of the pumice fill is not yet confirmed, hydraulic conductivity testing could not be undertaken. In lieu of *in situ* testing, infiltration testing from a nearby ENGEO project on Domain Road, Tauranga was reviewed. The example project pumice was a compacted fill material and is considered representative of the likely source material and placement specification that will be applied for the Wairakei South project.

Two infiltration tests were conducted in 0.95 m deep, 100 mm diameter unscreened hand auger locations. Results returned very low infiltration rates at 1.0 and 1.3 Litres/m$^2$/hour for the compacted pumice. The tests are considered more representative of horizontal infiltration, and vertical infiltration is likely to be even less.

Given the hydraulic properties of the proposed pumice fill are not yet confirmed, ENGEO recommend infiltration and permeability testing be undertaken once the source material is confirmed. If the tested parameters are sufficiently different to those adopted in this assessment, the assessment should be revised and additional mitigation measures may be required to keep impacts on the groundwater regime within the assessed effects envelope.

## 4.5 Additional Data Sources

The following data was downloaded from BOPRC data portal[3]. The data was incorporated into modelled boundary conditions, discussed in detail in Section 5.9.

- Rainfall Data at Marshals farm (2015 – 2025)
- Kaituna Stage at SH2 (2015 – 2025)
- Tidal Height Maketu (2015 – 2025)

## 4.6 Active Bores

A review of the BOPRC mapping software, accessed February 2026, identified three active bores within the site, which are detailed Table 3. All other bores are located > 200 m from the site.

**Table 3: Bay of Plenty Consented Bore Information**

| Bore No. | Property Address | Drill Date | Bore Type | Bore Depth | Water Level (before pumping) | Use |
|---|---|---|---|---|---|---|
| BN17-0062 | Section 26 SO 427562 | 22/1/2018 | Cold water | 11 m | 6.1 m | Production |
| BN-3296 | 339 Bell Road | 22/3/1995 | Cold water | 9 m | 4 m | - |
| BN-4925 | 252 Bell Road | - | Cold Water | - | - | - |

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[3] BOPRC Data Portal: Data - Bay of Plenty Regional Council WebPortal

# 5 Model Conceptualisation

## 5.1 Topography

The Wairakei South site lies within the low-lying coastal plains south / southeast of Tauranga, with elevations generally ranging from 0.5 to 2 m NZVD. The terrain is approximately level for greater than 2 km on three sides of the site (east, south and west), with subtle gradients trending southeast toward the Kaituna River. North of the site, topography rises over the Papamoa sand dunes to between 5 – 7 m NZVD. The LINZ 1 m LiDAR Digital Elevation Model (DEM) has been used for the basis of elevation (LINZ, n.d.).

## 5.2 Regional Geology and Geomorphology

GNS regional maps indicate the site is underlain by Holocene alluvial river deposits, primarily composed of gravel, sand, silt, mud and clay with local peat (GNS, 2020, n.d.). These sediments are typical of the low-lying Tauranga Basin and have been formed by fluvial processes from historical meandering of streams and rivers. North of the site is the Holocene beach and dune deposits forming the Papamoa dunes (Figure 4).

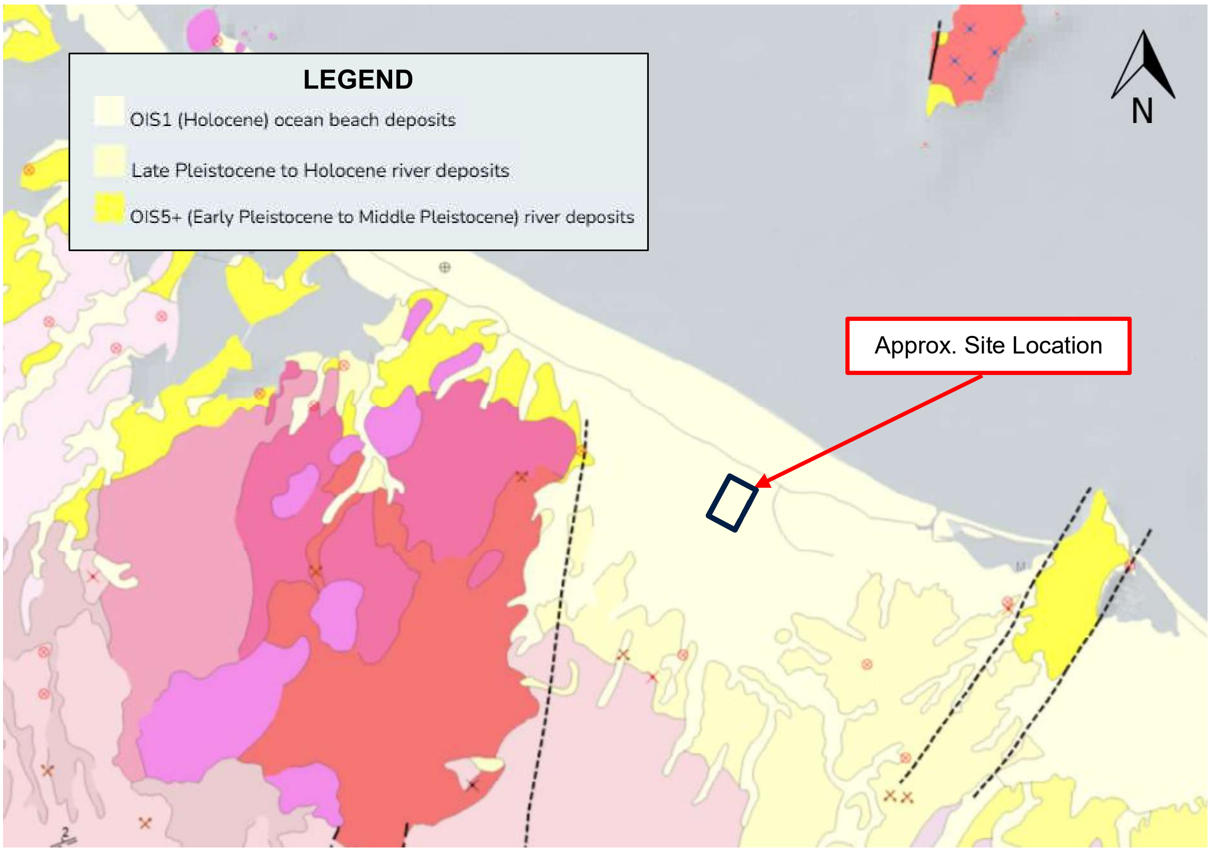

Figure 4: Extract from GNS Science Presenting 1:250,000 Geological Map

## 5.3 Hydrostratigraphy

The engineering geological model for the site was formulated using comprehensive subsurface investigation data and subsequently developed into a Leapfrog geological model (v2025, Seequent). Further details regarding the model's construction are provided in the Wairakei South Geotechnical Interpretive Report – Attachment P in the appendices to the application (ENGEO, 2025). This geological model forms the basis for defining the lateral extents of hydrogeological units as part of the groundwater assessment, within the constraints of the geological project boundary. For the purposes of the groundwater model, geological units were extrapolated further to the nearest hydrogeological boundary with reference to publicly available datasets from New Zealand Geotechnical Data Base and inferred geomorphological features.

The site is predominantly underlain by moderately permeable, soft, compressible peat, 2–3 metres thick, which becomes thicker toward the south and east near Bell Road and the Kaituna River. Towards the northern boundary of the site, surficial peat pinches out, and the surface material becomes sand, merging with the high permeability Papamoa dune sand that extends to the coast.

Moderate to high permeability Holocene alluvial sand underlie the peat across at site, comprising a laterally continuous unit measuring 5–8 m thick, which in turn are underlain by fine grained alluvial sand and silt. The silt composition of the underlying alluvium suggests a degree of anisotropy, inhibiting vertical groundwater flow. The base of this unit is -14 m NZVD (approximately 7 m thick), which has been adopted as the base of the shallow groundwater system for the purpose of this assessment.

Due to the shallow groundwater at the project site, two units of new fill are part of the hydrogeological conceptual model. A sand drainage blanket is proposed to be placed over the peat before placing and compacting the imported pumice. For this assessment, the drainage sand has been parameterised with the same hydraulic properties as on-site alluvial sand.

Engineered fill is proposed to be imported from local pumice quarry sources to form the project landform. The pumice will be compacted in lifts using heavy equipment, largely crushing the pumice structure into fine grained material forming a silt like material.

A summary table of the hydraulic characteristics for each unit is provided in Table 4.

**Table 4: Hydrogeological Units**

| Unit ID | Unit | Description | Occurrence[1] | Hydraulic Characteristics | Hydraulic Conductivity (m/s) |
|---|---|---|---|---|---|
| 1a | Engineered Fill | Compacted Pumice | Imported and placed for development subgrade | Low permeability under compaction, likely to have vertical anisotropy from layered compaction | [2]$1\times10^{-7}$ to $1\times10^{-8}$ |
| 1b | Sand Drainage Blanket | Alluvial Sands | Proposed site-won | Moderately clean sand with high hydraulic conductivity | [3,4]$1\times10^{-5}$ |
| 2 | Holocene Peat | Fibrous silty PEAT, BLACK, spongy, saturated | Present at surface in typically 2–3 m thick, pinching out against the northern boundary of the Papamoa sand dunes | Moderate hydraulic conductivity<br>Susceptible to primary consolidation<br>Hydraulic conductivity to significantly decrease with consolidation | [3]Uncompacted $5\times10^{-6}$ to $1\times10^{-5}$<br>Compacted $1\times10^{-8}$ to $1\times10^{-10}$ |
| 3 | [3]Alluvial Sands | Fine to medium, SAND, loose, poorly graded, trace of silt | Encountered across the site within the pre-historic alluvial flood plain<br>5-8 m thick underlying the entire site | Moderately clean sand with high hydraulic conductivity | [3,4]$1\times10^{-5}$ |
| 4 | Holocene Fluvial Deposits | Fine to medium SAND and SILT, interbedded and spatially variable | Underlying the alluvial sand at all drill locations, approximately 10 m thick | Moderate horizontal hydraulic conductivity, low vertical hydraulic conductivity. Relatively strong anisotropy | ~$1\times10^{-5}$ |
| 5 | Holocene Dune Sand | Fine to coarse, light brown SAND, loose, poorly graded | Exposed to the north of the site | Clean sand with high hydraulic conductivity | [2]$1\times10^{-4}$ |

[1] Lateral extent inferred from Geotechnical Assessment

[2] Testing results referenced from other sites in comparable geological material

[3] Site-specific hydraulic conductivity testing

[4] Assumed for sand drainage blanket

## 5.4 Surface Water and Groundwater Interaction

Surface water and groundwater are directly connected at the site due to the low-lying land surface elevation and extensive existing agricultural drains that interconnect surface water and groundwater in the area. The site and surrounding area have had extensive drainage systems constructed to support agricultural use since the early 1900s (TeMaruoKaituna, 2018).

The Kopuaroa Canal and the Bell Road drain convey surface water from further upgradient to Kaituna River. Both features have embankments to retain flows during high flow events. To avoid backflow onto the farms, several of the agricultural drains have one-way outlet valves that allow discharge to Bell Road drain yet avoid reverse flow (Figure 5).

Capturing this surface water / groundwater interconnection at the scale of the site during a storm event in 3D modelling is complex and numerically demanding due to the large number of drains across the site. As an alternative, two representative 2D cross sections with smaller timesteps and the ability to capture dynamic drain stage response at specific drain locations has been adopted as the preferred approach to evaluate storm response. The hydrograph for drain stage was extracted from the 1 in 100-year stormwater model and provided as a groundwater model input (Data provided by Maven[4]).

**Figure 5: Typical Farm Drain Connecting to Bell Road Drain, with One-Way Valve**

## 5.5 Groundwater Levels

Groundwater level monitoring comprised 21 on-site piezometers and 13 piezometers on the neighbouring Papamoa dunes as discussed in Section 4.2 and Section 4.4.2. Piezometers were installed on the perimeter of farm paddocks to avoid damage and increases drill rig access on soft ground which resulted in many locations being close to farm drains. The resulting effect of this is discussed further in Section 7.9.

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[4] Email from Maven, Monday 10/20/2025. Points for swale head vs time results from Stormwater model

Groundwater at the site is typically within 1 metre of land surface on the west of site, reducing to 0.3 m to the east of site, and higher during heavy rainfall events. A direct response to rainfall was observed in the groundwater data. A 10 cm tidal effect was observed in piezometer 06, located near the Bell Road drain.

Groundwater at the site is primarily regulated by the network of agricultural drains discharging groundwater. During a rainfall event, drains are expected to fill up faster than adjacent groundwater levels, potentially providing short-term groundwater recharge. Modelling such complexity in 3D is numerically challenging and can require either over-parameterisation or over-simplification of inputs in order make a convergent (i.e. running) model. Due to the challenges encountered, and the degree to which inputs were being ‘optimised’ for model convergence, an alternative 2D model approach was adopted to assess the influence of water storage in the swales under storm events. 2D modelling using transient swale boundaries allowed for minimal optimization of input parameters for assessment of effects during significant rainfall events. Flooding and Future Rainfall Events

LIDAR-based flood hazard mapping presented on the TCC government website indicates potential for shallow ponding in low-lying areas in the vicinity of the site, and flood inundation during intense rainfall events (TCC, n.d.).

Future rainfall events were incorporated in the groundwater assessment, including a 1% and 10% AEP event predicted in year 2130. The rainfall hydrograph for these events is presented in Figure 6.

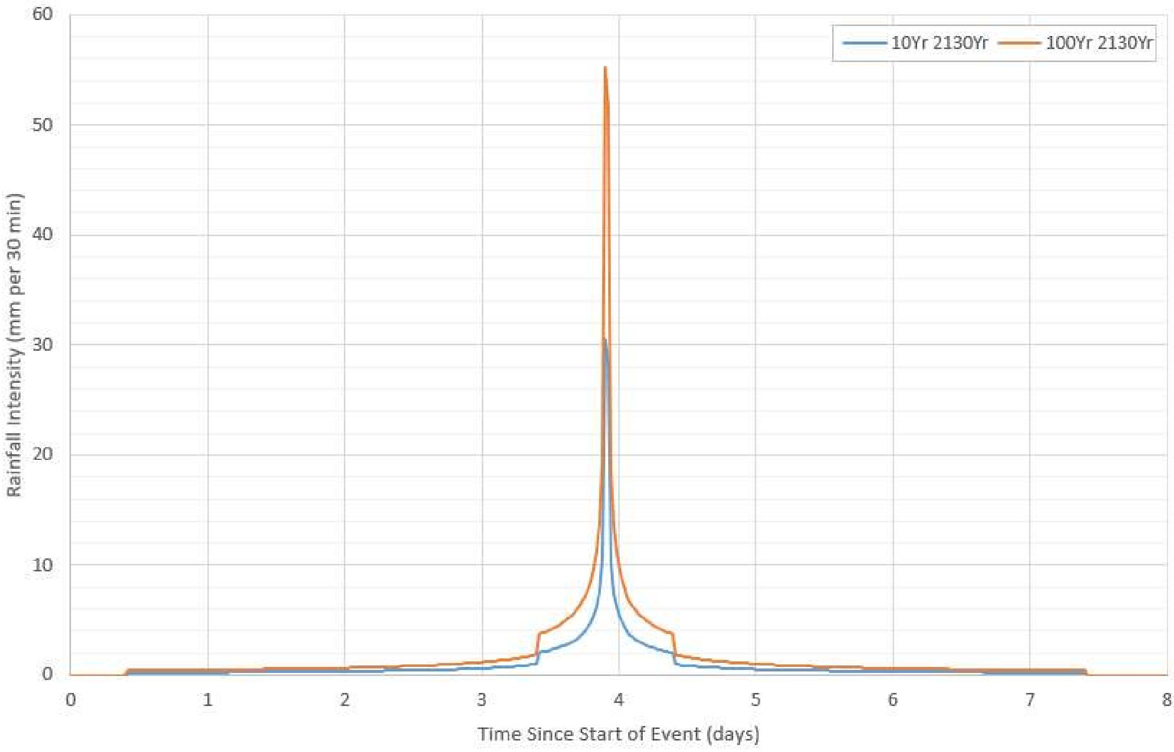
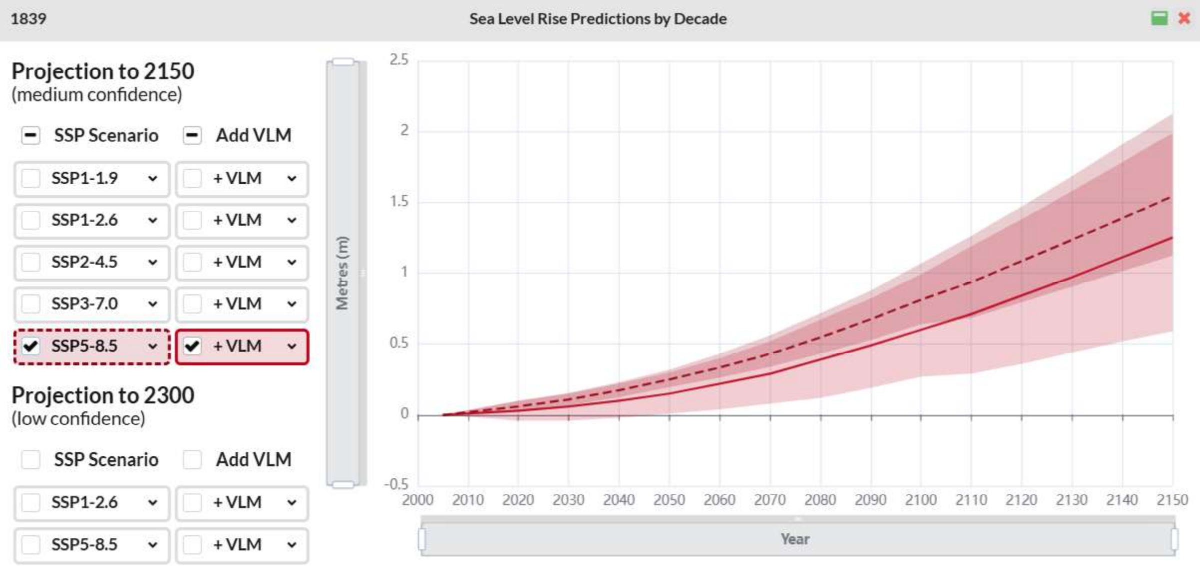


**Figure 6: Rainfall Intensity under 1% and 10% AEP, Predicted in Year 2130**

## 5.6 Predicted Sea Level Rise

The BOPRC Sea Level rise tool maps the site as susceptible to in-direct inundation due to sea level rise[5].

Sea level rise predictions in the coastal area are available under various SSP5 scenarios. The SSP5 8.5 estimates 1.59 m of sea level rise. In the Papamoa region, ground uplift is predicted to offset the predicted relative sea level rise to a net increase of 1.25 m (P50) by 2150 as shown in Figure 7 (NZ SeaRise, n.d.). For this assessment, a conservative approach has been taken by adopting 1.59 m of sea level rise by 2130 in line with BOPRC recommendations (Bay of Plenty Regional Council, 2021).



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Sea Level Rise Predictions by Decade

Projection to 2150
(medium confidence)

☐ SSP1-1.9 ☐ +VLM
☐ SSP1-2.6 ☐ +VLM
☐ SSP2-4.5 ☐ +VLM
☐ SSP3-7.0 ☐ +VLM
☑ SSP5-8.5 ☑ +VLM

Projection to 2300
(low confidence)

☐ SSP1-2.6 ☐ +VLM
☐ SSP5-8.5 ☐ +VLM

**Figure 7: Sea Level Rise Prediction for Papamoa (source: Searise.takiwa.co)**

## 5.7 Groundwater Inflow and Outflow

The water mass balance for the site can be described in terms of primary inflows and outflows, individually discussed in the following sections.

Inflows are driven by direct precipitation with additional groundwater recharge from the Kaituna River leakage during times of water levels higher than groundwater level. Regional groundwater inflows to the model domain are small due to the flat topography in the area surrounding the model domain.

Darcy flow calculations were used to calculate regional flow from west to east across the site, and from north to south from the Papamoa dunes. The calculated flow from the west is 17 m$^3$/day, while flow from the north is estimated to be 200 m$^3$/day. The low flow rates are due to the low hydraulic gradients.

Outflows from the site occur via the network of agricultural drains and canals, which provide conveyance of both surface and near-surface groundwater to the Bell Road drain and onward to the Kaituna River, via pumps when necessary. Evapotranspiration represents a significant loss in the mass balance equation. NIWA (National Institute of Water and Atmospheric Research; Earth Sciences New Zealand) estimates potential annual evapotranspiration at 1,052 mm/year or 75% of annual rainfall.

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[5] Bay of Plenty Regional Council. (n.d.). BayExplorer – Regional GIS Map Viewer. Retrieved August 13, 2025, from https://maps.boprc.govt.nz/apps/8581209df0a34211bc56bcf3ec2afff0/explore

Outflows via groundwater abstraction wells were not incorporated into the model due to uncertainties regarding abstraction rates and pumping schedules. This exclusion was made due to the estimated low volume of groundwater pumping overall.

Groundwater storage acts as a buffer between inflows and outflows to regulate and slow the rate at which flows pass through the groundwater system.

## 5.8 Rainfall Recharge and Groundwater Percolation

NIWA rainfall statistics (NIWA, n.d) state the site receives an average annual rainfall of 1,400 mm/year (3.83 mm/day). Site-specific rainfall observations align with regional data statistics, with a mean daily rainfall of 3.85 mm/day recorded at BOPRC rainfall station FO490726-Kaituna at Marshalls Farm. Cumulative annual rainfall is presented in Figure 8.

NIWA and Ministry for the Environment (MfE) climate change projections anticipate a reduction in total annual average rainfall for the region by 7% ( (MfE, 2024) Prediction for 2080, SSP3-7). However, individual rainfall events are expected to become more intense  (NIWA, 2013).

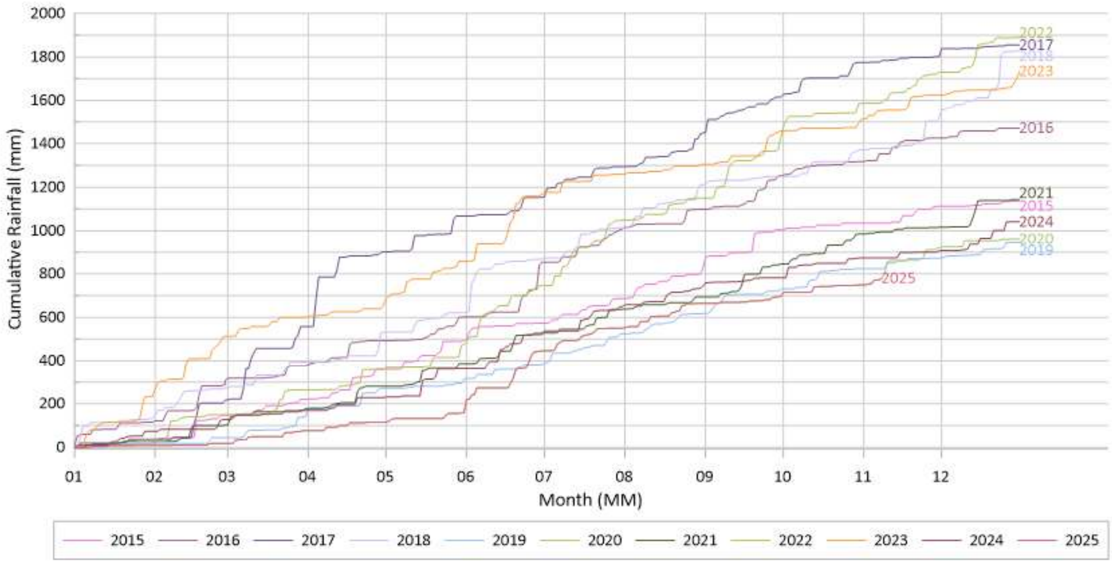


**Figure 8: Rainfall Data Observed at Marshalls Farm**

### 5.8.1 Percolation to Groundwater

The proportion of rainfall that infiltrates the soil and ultimately recharges the groundwater system was assessed for the three main geology and landform types across the surface of the model.

- Papamoa Dune Sands
- Peat
- Compacted Pumice Fill

Climate data obtained from the *FO490726-Kaituna at Marshalls Farm* and *Tauranga Aero EWS* weather stations was processed through a Soil Moisture Water Balance Model (WWLA SMWBM 4.5.0), to generate the percolation to groundwater data set.

The model uses daily rainfall and potential evaporation to calculate soil moisture conditions, including accounting for components such as soil moisture storage, saturated infiltration rates, surface ponding and runoff due to impermeable surface areas.

The model simulates a corresponding groundwater level which allows for calibration to groundwater monitoring data. Calibration was achieved by adjusting input parameters to replicate observed groundwater level response to recharge within in the dune sand and peat recharge zones. Further details on the process of generating the groundwater recharge data set for use in the model are provided in Appendix 2. The resulting groundwater recharge data sets are presented in Figure 9.

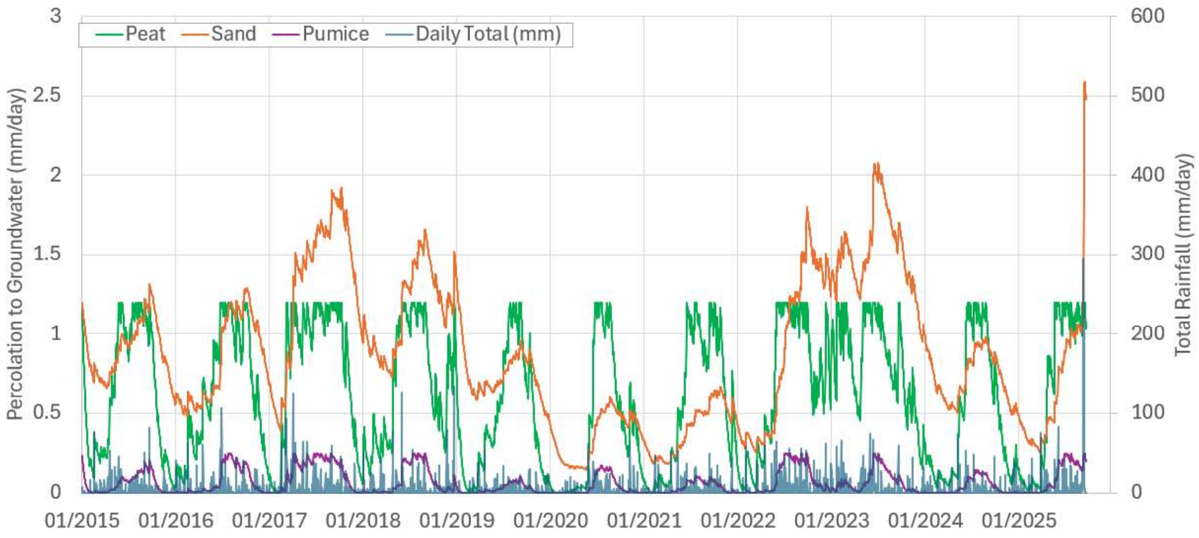


**Figure 9: Daily Recharge to Groundwater**

Dune sand has the highest potential saturated infiltration rate at 4 mm/day, (percolating to groundwater after near surface storage) consistent with rapid water absorption, while peat was lower at 1.2 mm/day based on calibration to observed groundwater response. Infiltration for compacted pumice fill was estimated based on soakage testing undertaken at another site (ENGEO, 2024). The maximum value adopted for pumice was 0.25 mm/day, consistent with low infiltration capacity.

The impervious area, which contributes to direct runoff from roofs / roads is highest for pumice fill, reflecting the development master plan and assumed impervious area of 60%, and lowest for peat (5%) reflecting the flat nature of the existing site. An impervious proportion of 25% was adopted for the Papamoa dune sands to account for residential land-use zoning comprising roofs, roads and built infrastructure channeling rainfall to reticulated stormwater systems or drains.

During periods of heavy rainfall, dune sand shows the highest groundwater recharge ratio at 80%, indicating strong infiltration. Peat was assigned a 20% value to represent standing surface water, while pumice fill was given 0% due to sloped design, which prevents water from pooling on the site.

For daily groundwater recharge, peat soils typically reach their maximum infiltration rates each winter, suggesting that these soils are often fully saturated due to frequent rain and a shallow groundwater table. In contrast, sandy soils have a greater unsaturated zone and higher infiltration ability, allowing both greater absorption during larger rainfall events and quicker drainage times. However, an assumed 25% impermeable area for the sand reduces its net infiltration at times compared to peat.

Pumice soils have the lowest estimated infiltration as compacted pumice fill and impermeable surfaces after development limit how much water can infiltrate to groundwater. In a real-word application the sloped design also lessens any chance of surface ponding in these areas, however modelling such interaction requires advanced surface / groundwater coupled numerical models. As such, the adopted model conservatively introduces marginally more rainfall recharge than is likely to occur due to the 'flat' nature of the recharge model.

The mean daily recharge results are compared to regional long-term average rainfall data in Table 5.

**Table 5: Summary of Recharge Rates**

| Soil | Peat | Dune Sand | Pumice Fill |
|---|---|---|---|
| Saturated groundwater percolation rate (mm/day) | 1.2 | 4 | 0.25 |
| Impervious area (direct runoff) % | 5 | 25 | 60 |
| Surface pondage to recharge to groundwater % | 20 | 80 | 0 |
| Mean Daily Percolation to Groundwater (mm/day) | 0.61 | 0.87 | 0.06 |
| % of Total Rainfall[1] | 16.1 % | 22.5% | 1.7% |
| % of Rainfall after Evapotranspiration1 | 64.5% | 90.3% | 6.6% |

1. Based on mean daily rainfall of 3.835 mm/day and mean daily evapotranspiration of 2.876 mm/day (Niwa, n.d)

Estimated recharge into the peat is 220 mm/year, aligning closely with the NZ rainfall recharge model (Westerhoff, 2018) which estimates 200 mm/year for this area. This model uses a 1 km × 1 km grid and monthly data from 2000–2014, incorporating satellite-based evapotranspiration and vegetation data with national datasets on rainfall, elevation, soil, and geology. Given the scale of the project site, we consider the larger grid model a reasonable and satisfactory reference check.

## 5.9 Drainage Features

Boundary conditions were applied to the model using GMS interface and mapped to the MODFLOW package files. Figure 10 presents the post development grid and applied boundary conditions in plan view. The boundary conditions are discussed in detail below.

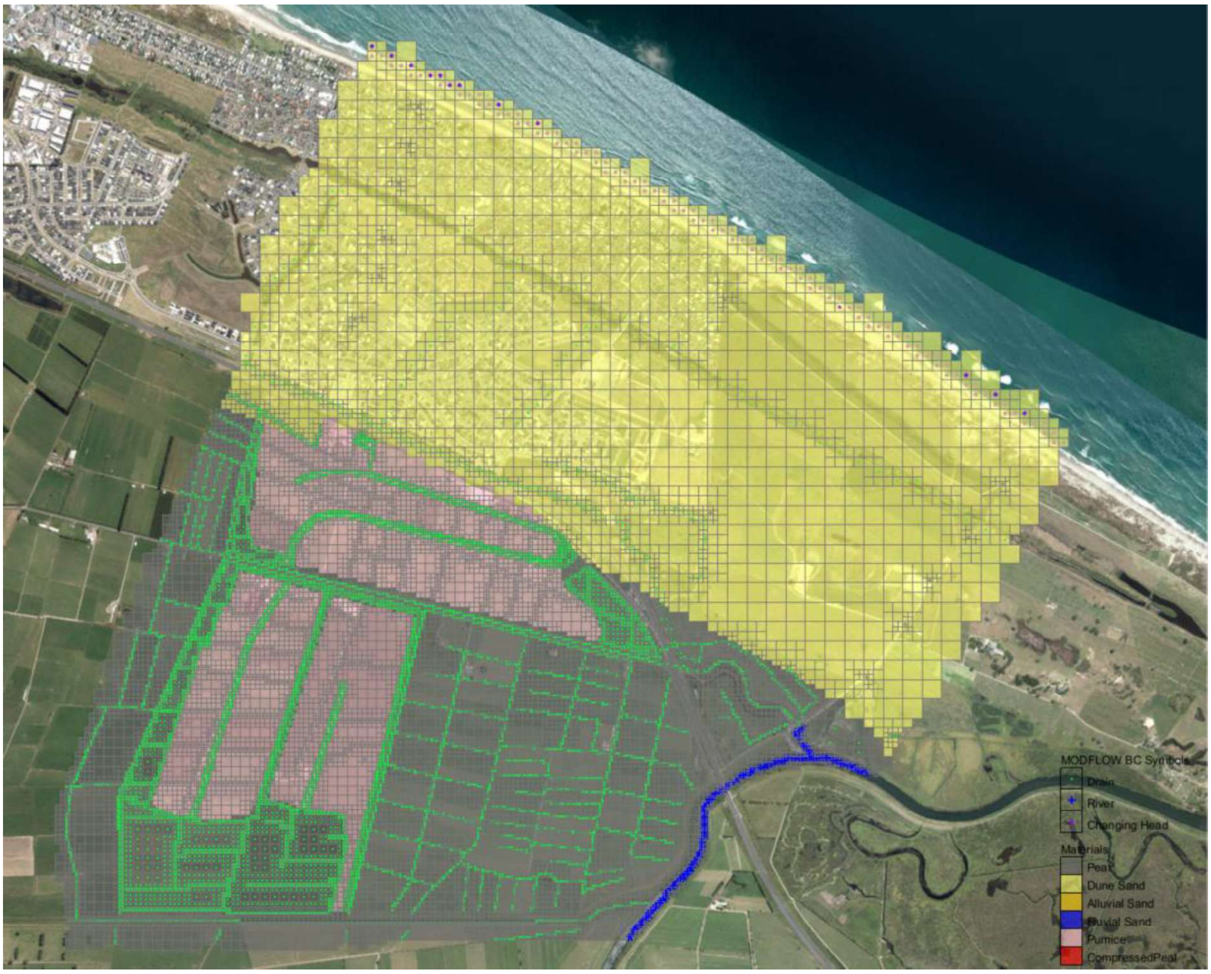


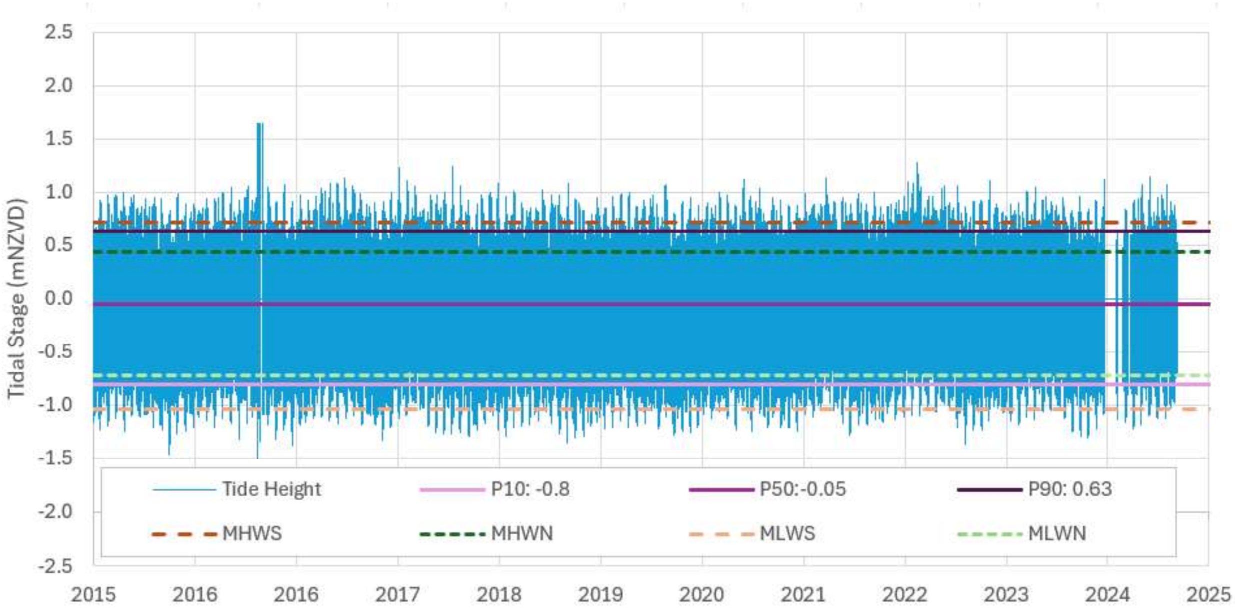
**Figure 10: Applied Boundary Conditions**

### 5.9.1 Papamoa Coast - Northern Boundary

Papamoa Beach is located approximately 1,750 metres north of the site and functions as a natural discharge point for groundwater traversing the area from the central part of the dune field. The coastal environment is subject to regular tidal fluctuations, resulting in variations in water levels. Measurements indicate that tidal water levels at the coast can range from -1 to 1.5 m NZVD (Council, n.d.). Data collected at the Maketu station (CQ947053) further detail the average tidal range, with the P10 (low tide) at -0.8 m, the P50 (median tide) at -0.05 m, and the P90 (high tide) at 0.63 m NZVD. The overall difference between the high and low tide peaks is approximately 2.25 metres (Figure 11).

Apart from the direct tidal influences on the Kaituna River, groundwater movement at the site was also evaluated. The most significant fluctuation recorded in the groundwater monitoring data at location PZ06 was only 0.1 m, indicating that tidal effects have a minimal impact on groundwater levels at the site.

Given the relatively minor influence of tidal changes on groundwater levels, the coastline boundary has been represented as a steady state condition in the groundwater model. The model uses the P90 tide height observed at Maketu as the boundary value. This approach is conservative, ensuring that scenarios where groundwater levels may be elevated are adequately accounted for in the assessment.



Notes

1. Tide Height sourced from BOPRC data portal
2. MHWS, MHWN, MLWS and MlWN sourced from LINZ Standard port tidal levels for Port of Tauranga

**Figure 11: Observed Tide Height (Source: BOPRC data portal)**

### 5.9.2 Kaituna River - Eastern Boundary

The Kaituna River is an important hydrological feature in the region, situated approximately 700 metres east of the site. Both the Bell Road Drain and the Kopuaroa Canal flow into the Kaituna, making it an important boundary condition for the site.

The lower Kaituna River is tidal, with fluctuations up to 0.8 metres recorded at the SH2 station (FO562709), as shown in Figure 12. Rainfall can maintain elevated river levels closer to those resembling upper tidal influence during periods of heavy or prolonged rainfall. Generally, river peaks align with high tides however the recorded river stage heights at SH2 rarely fall below 0.1 m NZVD, even at low tide.

Stage heights recorded at the SH2 station are presented in Figure 13, with P10, P50, and P90 values calculated at 0.28 m, 0.71 m, and 1.1 m NZVD, respectively. Like the coastal boundary, the Kaituna River was modelled as a steady state boundary at the P90 level.

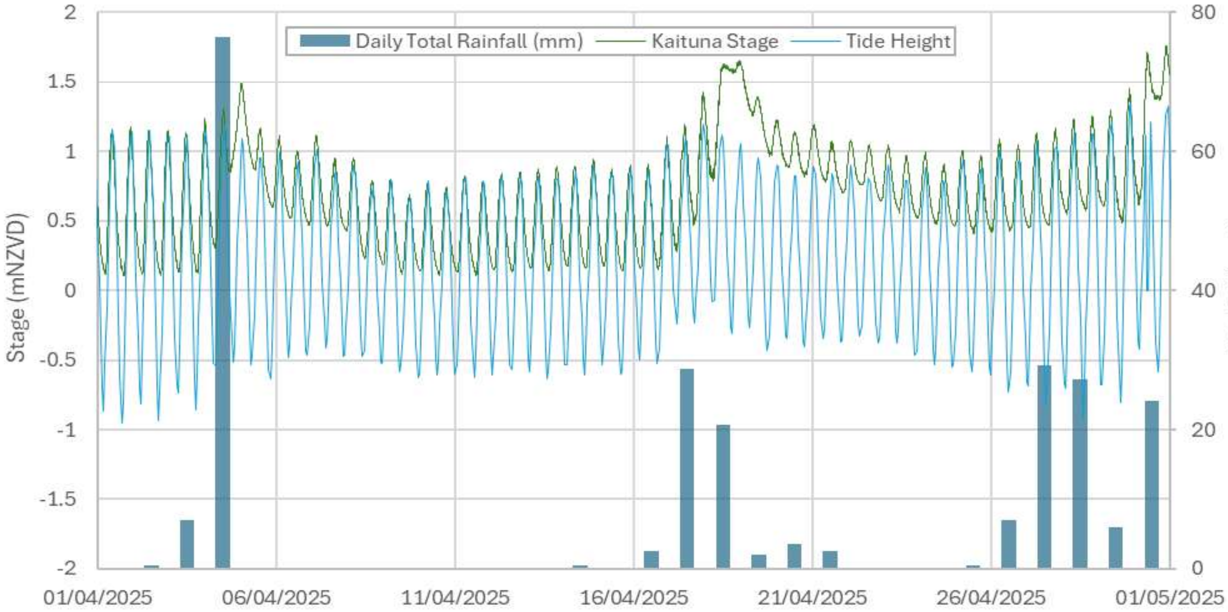


**Figure 12: Comparison of Maketu Tidal Cycles and Kaituna Stage Height at SH2 during April 2025**

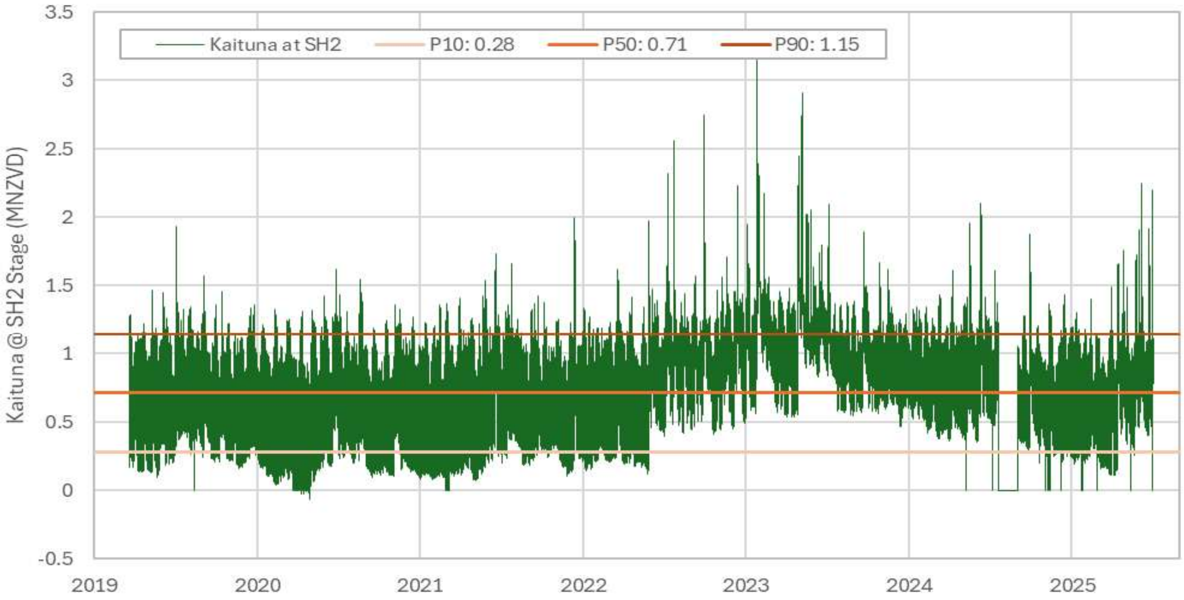


**Figure 13: Observed Kaituna Stage Height at SH2**

### 5.9.3 Kopuaroa Canal – Southern Boundary

The Kopuaroa Canal forms the southern boundary of the site and comprises a manmade drainage channel conveying water from the Kopuaroa catchment to the Kaituna. The canal is bordered by embankments to allow for high flow rates and discharges to the Kaituna River by gravity and an existing pump station. Channel invert elevations were extracted from the 1m LiDAR DEM.

Spot flow measurements between 1970 and 2010 were provided by BOPRC and ranged from 0.03 $m^3/s$ to 1 $m^3/s$. Although not exhaustive, the upper values are considered to represent a higher proportion of rainfall runoff, while the lower values are considered more representative of groundwater baseflows.

ENGEO understands plans for a future pump station aim to support continued drainage and stormwater management.

### 5.9.4 Bell Road Drain

The Bell Road Drain serves as a linear drainage feature, running west to east through the site. The engineered channel is equipped with tidal gates and a pump station to optimize water outflow to the Kaituna River.

Historical flow measurements by BOPRC from 2012 indicate notable variability, with observed rates of 4.3 and 2.7 $m^3/s$[6] recorded just one hour apart. This rapid fluctuation suggests the drain is highly responsive to rainfall events. Several farm drains are connected into the Bell Road Drain as a drainage point towards the Kaituna River.

Drain elevations were extracted from 1 m LiDAR DEM.

### 5.9.5 Farm Drains

Numerous farm drains traverse the site to manage surface water and support agriculture in this low-gradient, low-lying area. The farm drains channel excess water via one-way valves into the Bell Road Drain and Kopuaroa Canal. Drain elevations were extracted from 1 m LiDAR DEM.

### 5.9.6 Existing Surface Water Management Ponds in the Vicinity

In discussions with Bay of Plenty Regional Council, two stormwater management ponds have been identified in the vicinity of the site.

Pond levels were estimated based on 1m LiDAR as no pond level monitoring data was available for for either pond at the time of modelling.

## 5.10 Development Swales

Under the post-development scenario, the land surface is to be modified by raising the ground level with imported fill to create stable building platforms. This elevation change is complemented by the excavation of stormwater attenuation ponds and swales distributed across the site, which are designed to effectively manage both conveyance of stormwater runoff, and potential groundwater mounding beneath the platforms.

Two major stormwater attenuation ponds are planned for the site. The northeast pond has a design invert level 0.4 m NZVD. The south-western pond has a design invert level of 0.7 m NZVD and discharges under passive flow to the Kopuaroa Canal, with potential for pumped discharge during higher water levels (Maven, 2025).

The integration of these features to the existing drainage network surrounding the site minimises the disruption to how water is controlled, reducing flood risk and promoting efficient drainage. The post-development ground levels, as depicted in Figure 14, were provided to ENGEO by the stormwater design engineers, illustrating how the new landform will manage the site's overall drainage capacity and maintain the hydrological balance.

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[6] Data received from BOPRC via email on September 12, 2025

Swale drainage generally follows existing farm drain gradients from east to west, with some swales encouraging flow to the southern stormwater attenuation pond. The design swales have invert levels at elevations similar to the existing farm drains with the following exceptions.

- The design invert of the swale drain in the northwest corner of the northern block is 0.5 m lower than existing farm drains.
- The southern attenuation pond is lower than the existing drain to the west of the site.

The potential magnitude and extent of drawdown at these locations to propagate beyond the boundary is included in the assessment undertaken.

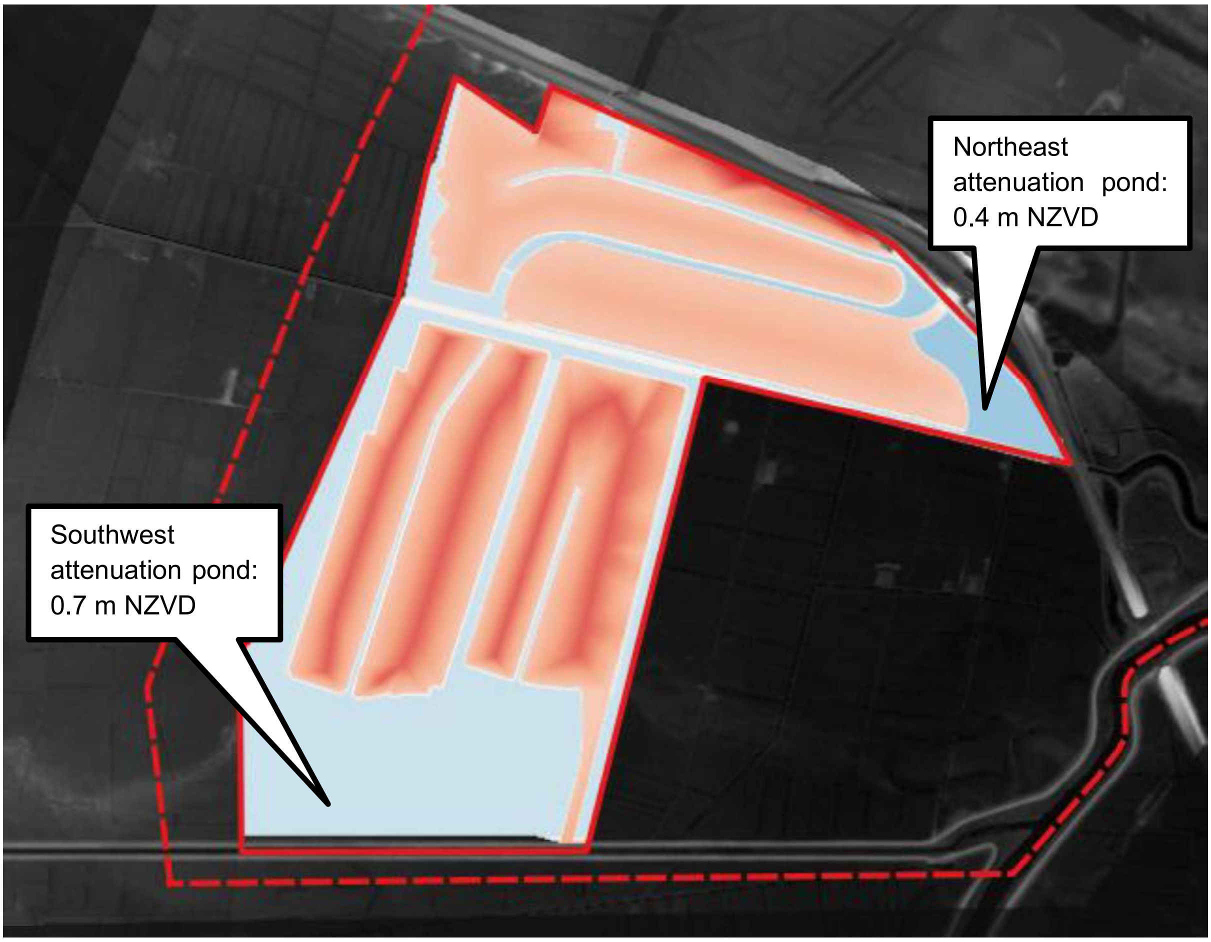

**Figure 14: Post-Development Ground Level, Showing Location of Attenuation Ponds and Swales Relative to the Built Ground Level**

## 5.11 Developed Hydrogeological Model

The site sits within a low lying coastal alluvial aquifer system, characterised by:

- Flat, low-lying land, sloping gently slightly towards the east and Kaituna River. Groundwater flows into the site from the west and the north, then east towards the surface water drainage features, locally, and towards the east more regionally. Groundwater flux into the site is low due to the extensively drained area surrounding the site with small groundwater gradients.

- Hydraulic boundaries influencing the site include the Papamoa coast to the north, Kopuaroa Canal to the south, Kaituna River to the east, and continuation of low-lying drained farmland to the west.

- Except for the northernmost boundary area of the site, the site is underlain by 2–3 m of moderately permeability peat when uncompressed ($7x10^{-5}$ m/s). The peat is in turn underlain by 4–8 m of moderate to high permeability alluvial sands ($1x10^{-5}$ m/s) which is underlain by older fluvial sediments comprising lenses of silt (anisotropy of 10). High permeability dune sands are present north of the site below Papamoa.

- Rainfall recharge is estimated to be 220 mm/year. Groundwater is typically shallow and responsive to rainfall but is predominantly controlled by extensive drainage pathways.

- The areas for residential and commercial development are proposed on top of a 3 m thick layer of low permeability compacted pumice. The impermeable area associated with the development (assumed to be 60%) and the low infiltration potential for compacted pumice are anticipated to reduce groundwater recharge in the post development scenario. A sand blanket below the pumice is proposed to assist in relieving the temporary buildup of hydrostatic pressure in the peat under consolidation pressure.

  Areas beyond the extent of proposed pumice fill are to be cut, creating stormwater swales and attenuation ponds. Smaller drains within the swale areas are proposed to manage groundwater discharge under baseflow conditions.

# 6 Numerical Model Configuration

## 6.1 Analysis Approach (3D and 2D)

The assessment has adopted both 3D and 2D modelling to undertake the groundwater assessment as summarised in Table 6.

3D numerical modelling was selected as the primary approach to replicate the site hydrogeological conditions focusing on capturing and predicting long-term changes to the groundwater flow regime. 3D modelling is used to estimate groundwater discharge rates to the network of drains and canals under existing conditions and estimating changes to discharge under the proposed landform. As well as assessing changes in flow, the model is a useful tool in quantifying flow rates to account for groundwater contributions to stormwater management assessment at site.

Due to the size of the site and the complexity of its interconnected drainage system, 3D modelling in this context is restricted for estimating short-term responses to rainfall. This limitation is attributed to difficulties in applying transient drain boundaries to numerous drainage features within the model domain, as well as increased computational effort and memory requirements associated with shorter time steps.

2D cross-sectional analytical modelling is used for assessing short-term groundwater response to storm events (1% AEP), with the inclusion of transient stage height within the swales. 2D modelling allowed shorter timestep intervals and accurate representation of transient drain boundaries to assess potential for groundwater mounding beneath the site platform. Two sections were cut to align with regional flow and transect zones with larger spacing between swales to evaluate the most conservative mounding scenario.

The initial groundwater model build and calibration focused on the 3D model, with calibrated parameters applied in the 2D model.

An additional 2D radial model (axisymmetric analysis) was created to separately assess the possible impacts of shallow groundwater extraction from a groundwater infiltration pond. The parameters for the model were taken from the earlier SEEP/W model. The simulation ran for 90 days, during which dewatering was found to approximate steady state conditions.

**Table 6: Model Summary**

| Type | Software | Purpose |
|---|---|---|
| 3D | GMS v.10.4 (Aquaveo) | Groundwater levels and flow regime<br>Drain, canal and swale discharges<br>Sea Level Rise |
| 2D Cros-Section | Geostudio SEEP/W v.2024 (Seequent) | Groundwater mounding between swales in response to 1 and 10% AEP events |
| 2D Radial | Geostudio SEEP/W v.2024 (Seequent) | Groundwater abstraction potential and potential groundwater effects |

## 6.2 Model Build Level for Project and Stage

Modelling has been developed in general accordance with Class 2 confidence as described in the Australian Groundwater Modelling Guidelines (Sinclair Knight Merz, 2012). The conceptual model reflects a sound understanding of the local hydrogeology, supported by available site data and regional boundary constraints. The numerical model has been calibrated against observed groundwater levels across the site for feasibility and impact assessment level decision-making.

Spatial and temporal resolution were chosen to capture major factors affecting groundwater flow, and sensitivity analysis was performed using conservative estimates for uncertain input parameters.

While some uncertainty remains, assumptions and limitations have been documented, and the model provides a robust basis for assessing project impacts at feasibility stage of design.

## 6.3 Analysis Software

3D modelled was developed in GMS v.10.4, a user interface platform to construct and run the simulations The underlying code was MODFLOW-USG (Unstructured Grid) developed by the United States Geological Survey (USGS).

2D analysis was completed using SEEP/W 2024.2.1. SEEP/W is a two-dimensional finite element software program that is widely used in soil and other material seepage evaluations.

Both software are industry standard packages.

## 6.4 Model Domain and Geometry

The 3D numerical model was constructed based on six layers, with a total of 146,442 cells (or polygons) and covers an area of 14.8 km$^2$. The model boundary was created based on key hydrogeological boundary conditions, discussed in Section 6.60.

The model is built in EPSG:2106 Bay of Plenty 2000 using New Zealand Vertical Datum.

The model uses MODFLOW-USG (Unstructured Grid) to allow refinement at locations of interest. Model grid spacing ranges from as small as 15 x 15 metres aligned along the model boundary conditions such as the drains, and swales rivers and piezometer observation wells, out to 100 x 100 metres in areas where no boundary conditions are present (Figure 15).

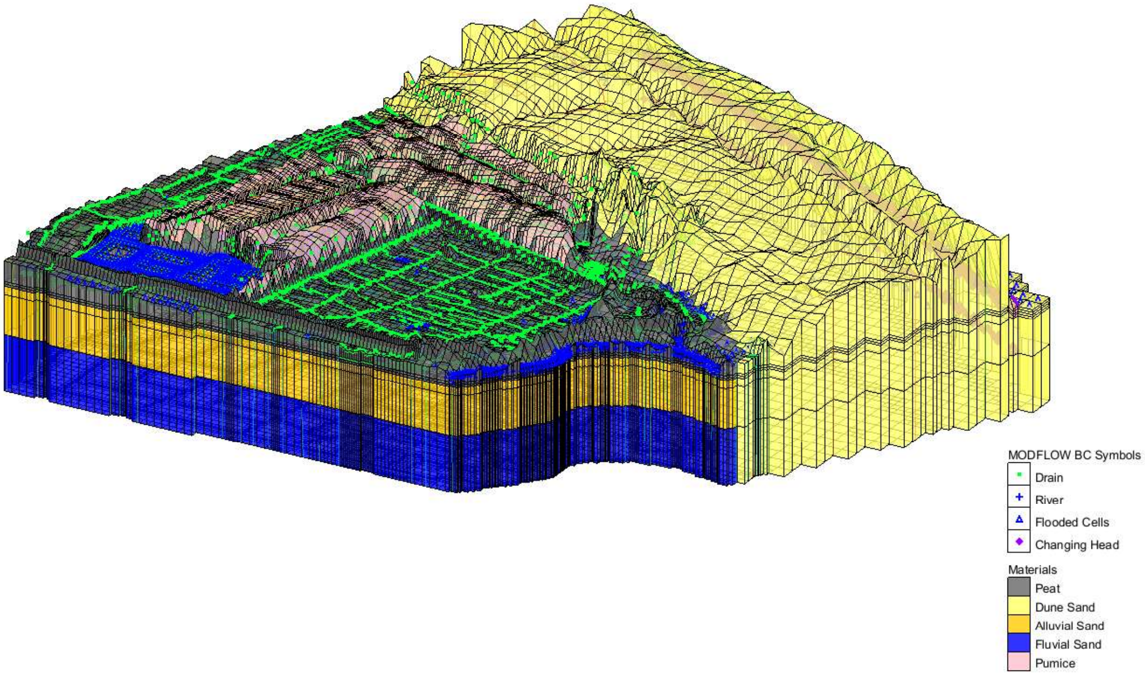

**Figure 15: Model Domain**

## 6.5 Layer Elevations

Top surface of Layer 1 was set equal to the existing ground surface based on 1m LiDAR DEM, then updated for the predictive scenarios to represent the post development landform provided by Maven[7].

Geological layers are modelled as horizontal stratigraphy, vertically positioned in accordance with the contact elevation extracted from the Leapfrog geological model – detailed in the Wairakei South Geotechnical Interpretive Report - Appendix P to the appendices to the application (ENGEO, 2025). The simplified approach is representative of the horizontal geology at the site. Discontinuous layers were accounted for by mapping material zone boundaries within each layer.

The surficial peat was divided into three layers during the model calibration of current conditions. In the post development model these layers were re-classified as compacted pumice, drainage sand and compacted peat respectively.

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[7] Email from Maven. Monday 20 October.

One area of the model where this approach diverges from the proposed construction methodology is where the peat 'pinches out' against the northern boundary. When converting layer one into pumice for the post development scenarios, the pumice against the northern boundary extends as deep as - 0.4 m NZVD. This approach over represents the pumice fill thickness. The proposed construction methodology for the northern area of the site is remove and replace any existing peat, acknowledging that it pinches out towards the north of the site, and then place pumice on top of the sand. Therefore, modelling the pumice block as thicker is considered to provide a conservative assessment of mounding effects due to the lesser incorporation of the natural dune sand and sand drainage blanket in this area of the model.

## 6.6 Model Domain Boundary Conditions

The model domain was extended out from the site boundary to the nearest significant natural hydrogeological feature to constrain the model within the wider hydrogeological setting.

Additional boundary conditions were included within the site domain to represent groundwater recharge, existing drains, etc. The boundary conditions are summarised in Table 7.

**Table 7: Model Domain Boundary Conditions**

| Boundary Condition | Description | Reference Data |
|---|---|---|
| Pacific Ocean - Northern Boundary | • The northern site boundary is defined at Papamoa beach.<br>• Modelled as a constant head boundary set to 0.63 m NZVD, (the 90th percentile tide height recorded at Maketu) to provide a conservative assessment of groundwater level.<br>• Future Sea Level Rise was simulated by adding an additional 1.59 m predicted under upper bound SSP8.5 scenario at 2130. | Section 5.9.1<br>Section 00 |
| Kopuaroa Canal - Southern Boundary | • The southern site boundary is delineated by the Kopuaroa Canal.<br>• Modelled as a head-dependent drain boundary, with canal stage set to elevation abstracted from 1 m LiDAR DEM. | Section 5.9.3 |
| Kaituna River - Eastern Boundary | • The eastern boundary is defined at the Kaituna River.<br>• Modelled as a river boundary condition, with stage set to 90th percentile river stage at SH2. | Section 5.9.2 |
| Western Boundary | • West of the site is ongoing low-lying agricultural land, The western boundary comprises no significant hydrogeological boundary.<br>• The model boundary was defined at the next farm drain approximately 250 m west, discussed below. | Section 5.9.5 |
| Swales, drains and canals | • Swales and farm drains were identified on the 1m lidar and replicated in the model using the Drain function.<br>• Elevation was set based on 1 m Lidar DEM.<br>• The conductance value of the drains was set relatively high to reflect limited impedance to water removal (or drain functionality). | |
| Groundwater Recharge | • Recharge was applied to model layer 1 using recharge zones.<br>• Recharge rate was specific to the hydrogeological and topographical characteristics of the surface material. | Section 5.8 |

## 6.7 Zone Budgets

Four zone budgets were mapped over the model domain to allow post processing of the model results. Zone budgets were mapped to areas of interest within the model domain, specifically the north (Zone 2 - Teal) and south (Zone 3 - Green) blocks which represent the north and south blocks of the project (Figure 16).

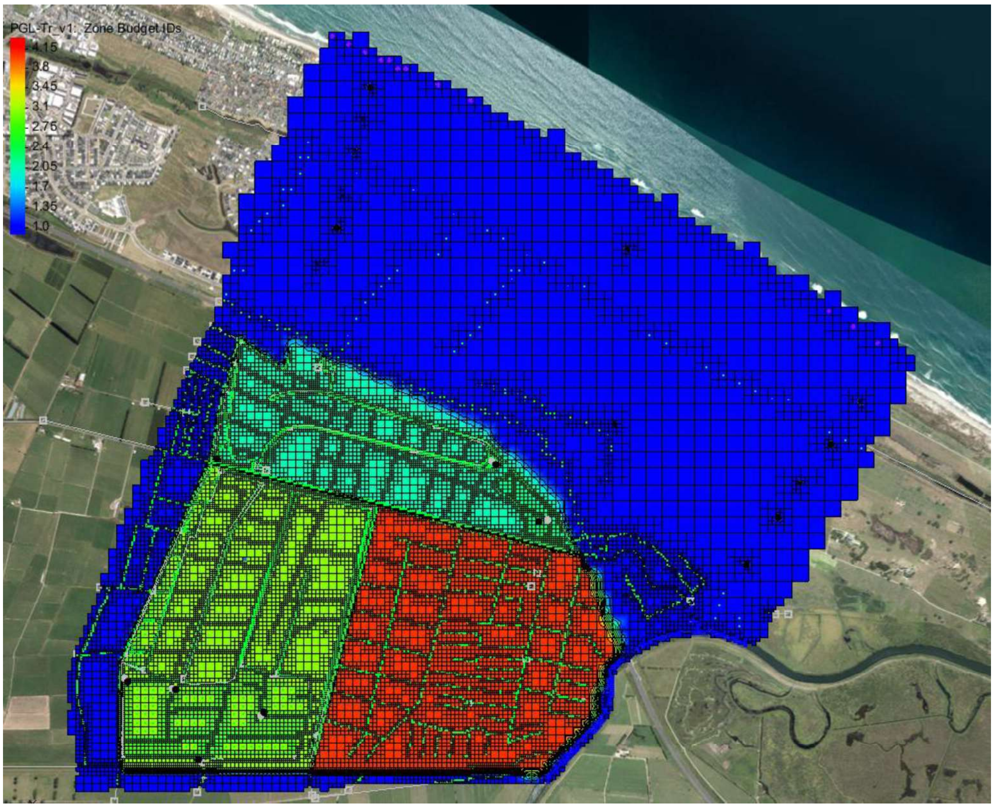

**Figure 16: Zone Budgets Mapped to the Model to Extract Flow Budgets in Specific Areas**

## 6.8 2D Modelling - Transient Swale Boundary

2D seep modelling was adopted for a more refined assessment of groundwater mounding by directly incorporating the changing surface water levels in post development swales during major rainfall events. This approach reflects the true variability in water movement, which constant drain elevations models might miss.

Surface water head data extracted from the stormwater model was provided to ENGEO from the civil engineering model, at specific locations along representative 2D seep transects, as shown in Figure 16. The stage versus time data simulated during the rainfall event in the stormwater model were then applied to the SEEP/W model as a transient boundary condition to the swales, allowing for the modelling of groundwater mounding in response to real-world hydrological inputs (Figure 18).

All additional modelling parameters were kept consistent with those calibrated in the 3D model, ensuring that the results were grounded in previously validated hydrogeological conditions.

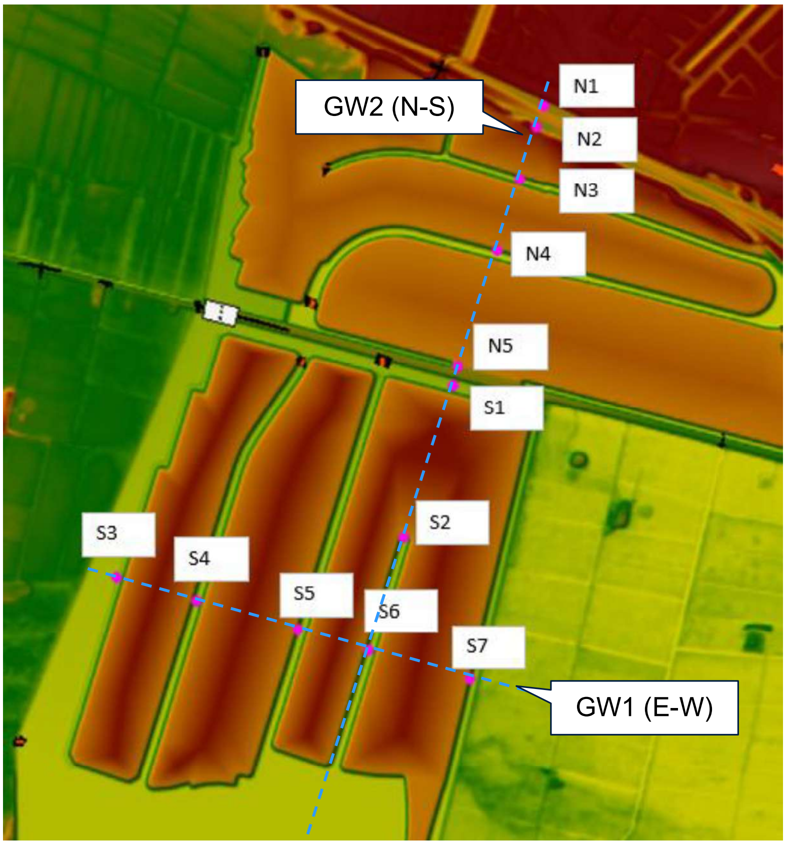

Figure 17: Location of Data Nodes Extracted from Stormwater Modelling

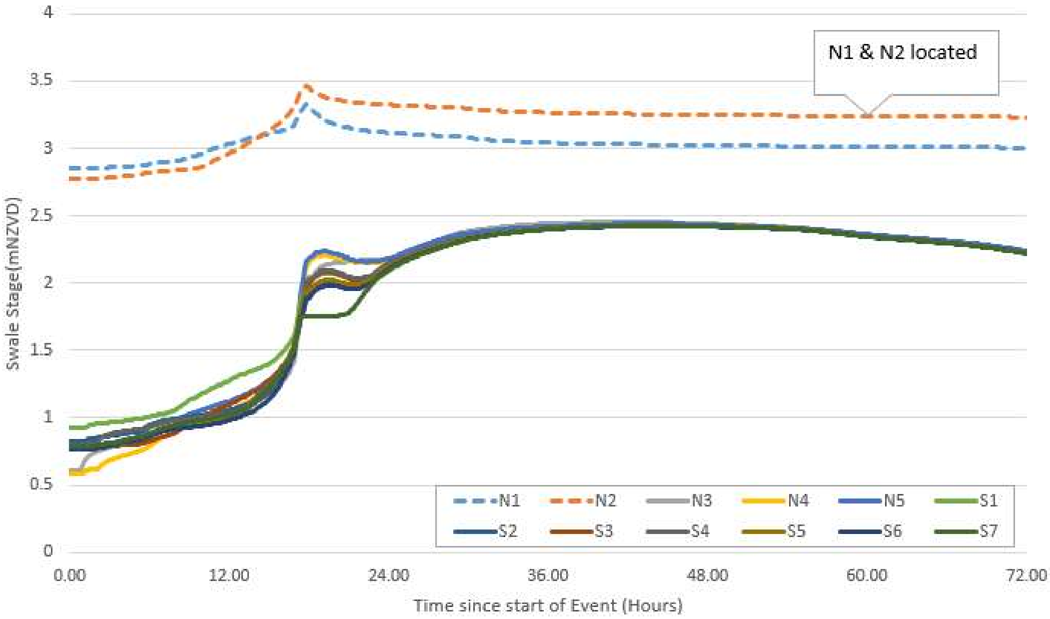

Figure 18: Transient Head vs Time Data for Selected Swale Locations

## 6.9 2D Radial Modelling – Groundwater Abstraction

Given the scope of the development and the necessity to relocate groundwater infiltration ponds during construction planning and phased earthworks, it is not feasible to determine the exact pond locations at this stage of the project. Instead, the assessment comprises an independent evaluation of potential impacts that may arise from an infiltration pond based on typical site conditions, to determine the extent of effects. This provides an effects envelope that can be considered as a template across the site.

An axisymmetric SEEP/W analytical model was used to evaluate the potential yield and groundwater related effects from the prosed infiltration ponds (Figure 19: 2D Axisymmetric Model Presenting Proposed Groundwater Infiltration PondFigure 19). A radial model is a quasi-3D model, using the simplicity and refinement of a 2D SEEP/W model, with the shape of the infiltration pond extrapolated around a central axis. This stylised approach is considered suitable given the generally flat topography, consistent geology and low groundwater gradients across the site. It allows a greater degree of resolution for the infiltration pond than can be afforded in the larger 3D model, in this case, 0.25 m cell size compared to 15 m cell size respectively.

Site materials, hydraulic parameters, and model boundary conditions were adopted from previous 2D models, ensuring results align with validated hydrogeological data. The static groundwater level was assumed to be 0.5 m bgl.

Additionally, drain cells were spaced every 200 m across the model section to account for the network of surface drains on-site.

A drain boundary was located in the base of the pond, with seepage review faces along the battered sides to calculate the groundwater inflows during the simulation. A review of groundwater heads was used to calculate the radial groundwater drawdown away from the pond.

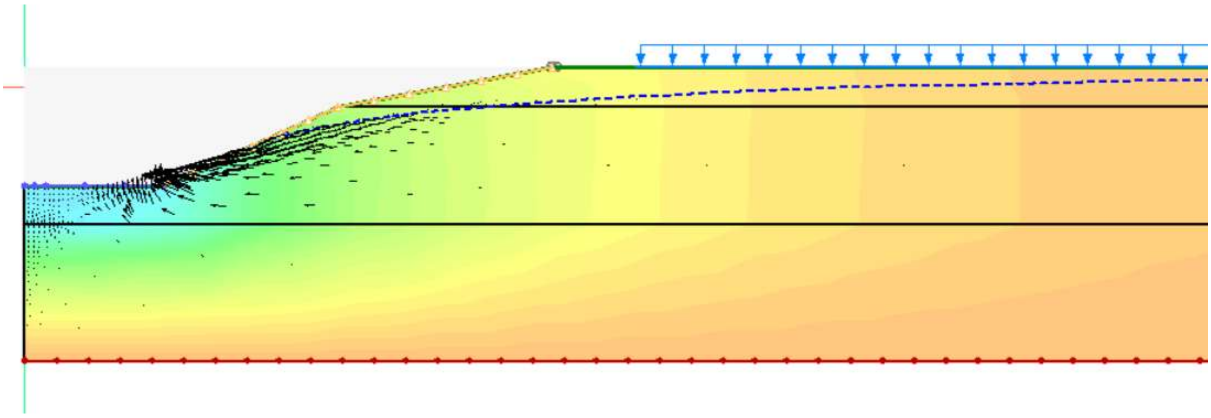

Figure 19: 2D Axisymmetric Model Presenting Proposed Groundwater Infiltration Pond

# 7 Model Calibration

3D model calibration was conducted by adjusting the model hydraulic parameters to achieve an acceptable fit to measured groundwater levels, which was undertaken using both automated procedures (PEST) and manual refinement.

## 7.1 Steady State Calibration

A steady state model was developed and calibrated to validate the conceptualisation of the groundwater flow model. The objective of the calibration was to obtain approximate values of the model parameters. An automated parameter estimation tool, PEST, was used to calibrate rainfall recharge into the sand and peat units, with upper and lower bounds based on literature values (Freeze & Cherry, 1979). Using this method, calibration of hydraulic parameters was obtained while maintaining calibrated parameters within reasonable ranges for the specified material types.

Steady state model outputs were used as a starting point for the transient model calibration process.

The water levels from 21 on-site piezometers and 13 TCC monitoring piezometers were used as the calibration targets. The groundwater monitoring records were converted into a representative steady state value as follows:

- On-site piezometer record: adopt 75th percentile water level across August – September 2025 (winter).
- TCC Papamoa Piezometers: adopt 90th percentile water level across 5-year data record available to target an upper bound value, consistent with winter water levels monitored on site.

The simulated head values are plotted against observed head values in Figure 20.

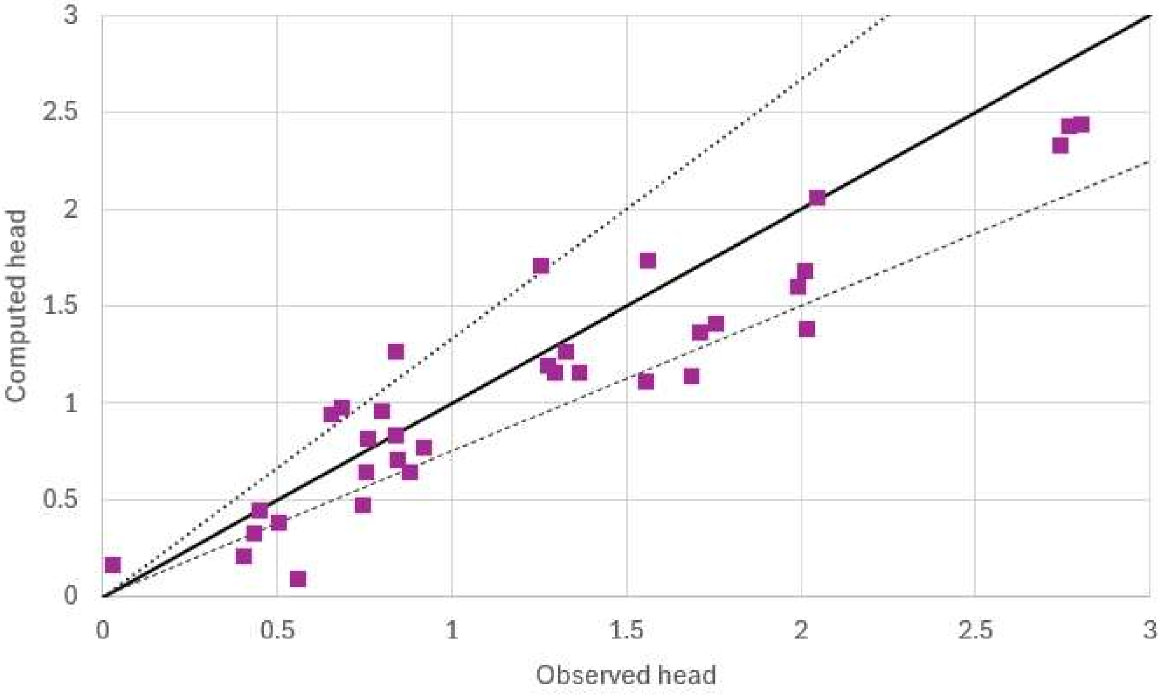

**Figure 20: Computed vs Observed Groundwater Head Values**

Simulated groundwater heads correlate to observed heads with a mean residual head of 0.13 m, and an RMS error of 0.30 m. Simulated values were slightly lower than observed reflective of the observed targets being winter measurements (seasonally higher) while steady state inputs were annual averages.

An additional challenge to model calibration is the piezometers proximity to drains. As noted above, drain boundary conditions are constant within the drain cells, and it is not readily achievable, within the aforementioned computational confines of the model, to raise simulated heads near the drains without introducing unrealistic flooding of groundwater cells elsewhere. Acknowledging these challenges, we consider the slightly lower calibration values acceptable based on the site constraints and consider the calibrated model is fit for purpose.

The simulated piezometric surface is consistent with the conceptual model showing gradients towards drains and discharge points locally, and more generally gradients from Papamoa towards the coastal boundary to the north, and gradients south towards the Tauranga Eastern Link. Similarly, the piezometric surface simulated on-site is consistent with site observations showing higher groundwater levels in the west of the domain, reducing consistent with subtle gradients from west to east towards the Kaituna River.

The simulated steady state piezometric head is presented in Figure 21.

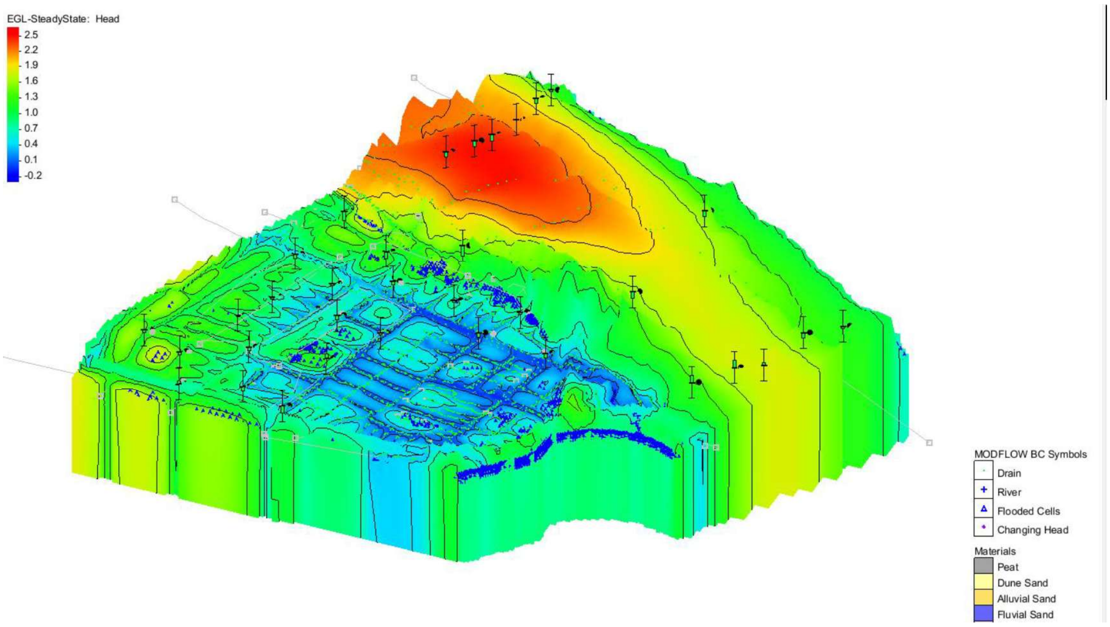

**Figure 21: Simulated Groundwater Levels (3D) – Steady State Calibration**

## 7.2 Transient Calibration

Transient calibration provides a more powerful and representative method of model parameterisation and was undertaken after the initial steady state calibration was accepted. Transient calibration involved dividing the simulation into individual stress periods and applying transient recharge data for each period. The model calculates the resulting groundwater level through iterations of time which is compared to piezometer monitoring data.

Transient calibration focused on replicating observed groundwater head trends within the 10-year period between 2015 and 2025. As noted above, simulating response to rainfall within the piezometers was challenging due to proximity to the drain cells within the model. The calibration focused on matching seasonal variation as a priority above the short-term spikes in response to rainfall events.

## 7.3 Stress Periods

The model comprised 223-time steps, comprising monthly time steps from 1 January 2015 to 30 June 2025, followed by daily timesteps from 1 July to 14 September 2025.

At the end of the simulation, in winter of September 2025, a 1% AEP storm event based on a 2130 climate change scenario SSP5 8.5 was applied to the model.

## 7.4 Recharge

Transient recharge data sets were applied to model layer one in accordance with the soil type, as discussed in Section 5.9.

## 7.5 Initial Conditions

The transient model initially used the steady state model heads as the starting condition. During the transient calibration process, the starting heads were re-set periodically as parameters were updated. This enabled the starting condition to better reflect the dynamic head distribution within the model under the imposed set of stresses and resulted in i) minimisation of rapid fluctuations in simulated levels and flows at the start of the simulation (i.e., increased stability), and ii) reduced initial gains or losses in groundwater mass within the model domain over the simulation period.

## 7.6 Hydrogeological Parameters

Table 8 Table 8 presents the parameters adopted to represent the hydrogeological units in the model. All layers were modelled as convertible within GMS. Layers

**Table 8: Adopted Parameters for Modelled Hydrogeological Units**

| Unit ID | Geological Unit | Hydraulic Conductivity (m/s) | Vertical Anisotropy ($k_v/k_h$) | Porosity | Specific Yield |
|---|---|---|---|---|---|
| 1 | Engineered Pumice Fill | 1 x 10-7 | 2 | 0.15 | 0.1 |
| 2 | Uncompressed Peat | 7x10-6 | 2 | 0.3 | 0.2 |
| 3 | Alluvial Sand (*in situ*)<br>Drainage Blanket (assumed) | 1x10-5 | 1 | 0.15 | 0.1 |
| 4 | Lower Alluvial Sands / muds | 5x10-6 | 10 | 0.15 | 0.1 |
| 5 | Dune Sand | 1 x 10-4 | 1 | 0.3 | 0.2 |
| 6 | Compacted Peat | 1x10-8 | 5 | 0.2 | 0.1 |

## 7.7 Calibration to Observed Heads

Site observation data, along with observation data provided by TCC, was used to calibrate the simulated groundwater levels. The data sets range from five years to two months. Twenty-one wells are monitored as part of groundwater monitoring on-site, while the remainder are monitored as part of routine State of Environment (SOE) reporting by TCC.

As above, stress periods started as monthly intervals and reduced to daily where greater coverage of monitoring is available. Groundwater monitoring was block averaged into daily timesteps to align with the later stress periods of the simulation.

The Mean Residual (head) for the transient model is 0.05 m (water levels are on average 5 cm above observed. The Mean Absolute Residual (head) is 0.22 m and the Root Mean Squared (RMS) Residual (head) is 0.32 m.

An example of calibration plots for PZ01 (Peat) and PZ4A (Dune Sand) is provided below in Figure 22 and Figure 23 respectively. All calibration plots are provided in Appendix 3.

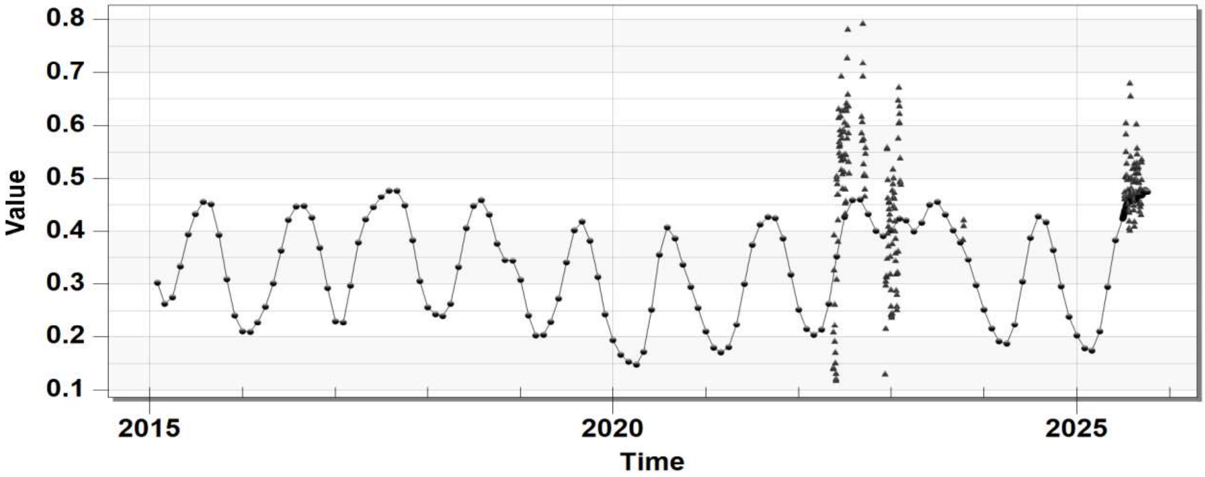

Figure 22: Simulated vs Observed Groundwater Levels - PZ01 (Site)

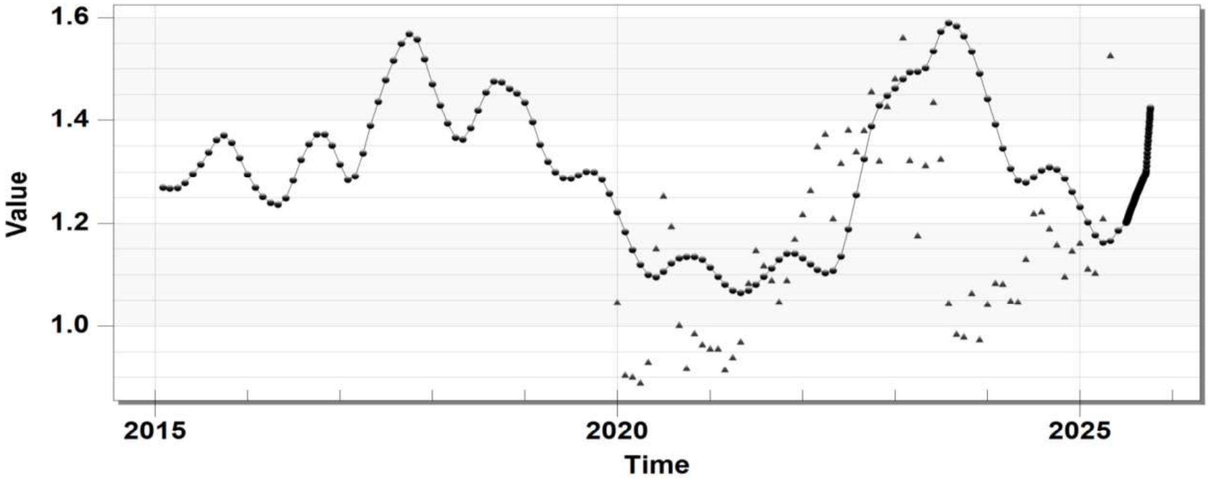

Figure 23: Simulated vs Observed Groundwater Levels – 4A (Papamoa Dune Sand)

## 7.8 Mass Balance

The main inflow to the model is recharge generated from rainfall, representing 77% of total inflows. Additional inflows comprise aquifer storage returning to the model and river leakance from the river package (Kaituna).

The main outflow is through drain cells (agricultural drains, canals, swales) which account for 79% of total outflows. Additional outflows comprise water leaving the model to storage, river leakance and discharge to the constant head boundary representing the coast.

The mass balance water budget is presented in Table 9. The mass balance was calculated for the ten-year simulation period from 1 Jan 2015 to 1 Jan 2025, where all stress periods are equal length, providing a balanced representation of the model. At simulation end, only a 0.02% mass balance error remains.

**Table 9: Mass Balance Existing Conditions**

| Mass Balance | Components | Flow (m3/d) | Percentage of Flow (%) |
|---|---|---|---|
| Inflow | Storage[1] | 1,009 | 8 |
| | River Leakage | 8 | 0.1 |
| | Recharge | 11,212 | 92 |
| | **Total inflow** | **12,229** | **100** |
| Outflow | Storage[1] | 901 | 7 |
| | Constant Head | 2,220 | 18 |
| | Drains | 9,025 | 1 |
| | River Leakage | 84 | 0.7 |
| | **Total out** | **12,231** | **100** |
| Percent discrepancy | | 0.02% (2m$^3$/day) | - |

1. Storage mass balance biased towards inflow as simulation begins in summer (low) and ends after winter, with a 1 in 100-year rainfall event (high). Mass balance in vs out from January 2015 to January 2025 has less than 10% difference

The groundwater flow budget for each simulation period step is shown below. Figure 24 displays the flow budget for the entire model domain, while Figure 25 shows the flow budget within the site boundaries only, represented by Zones 2 and 3 (north and south blocks of the site).

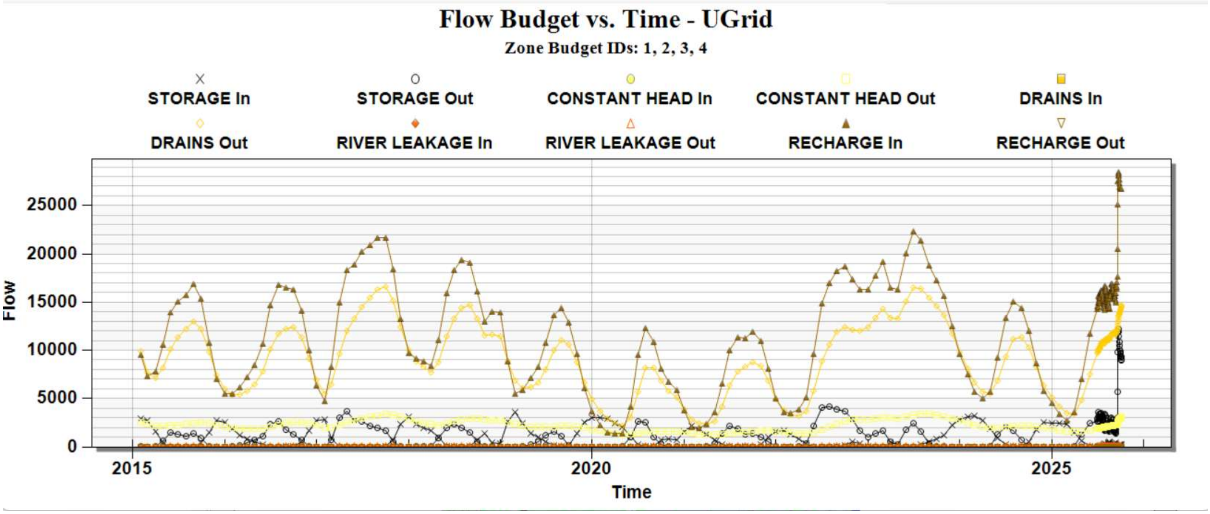

Figure 24: Flow Budget – Existing Conditions

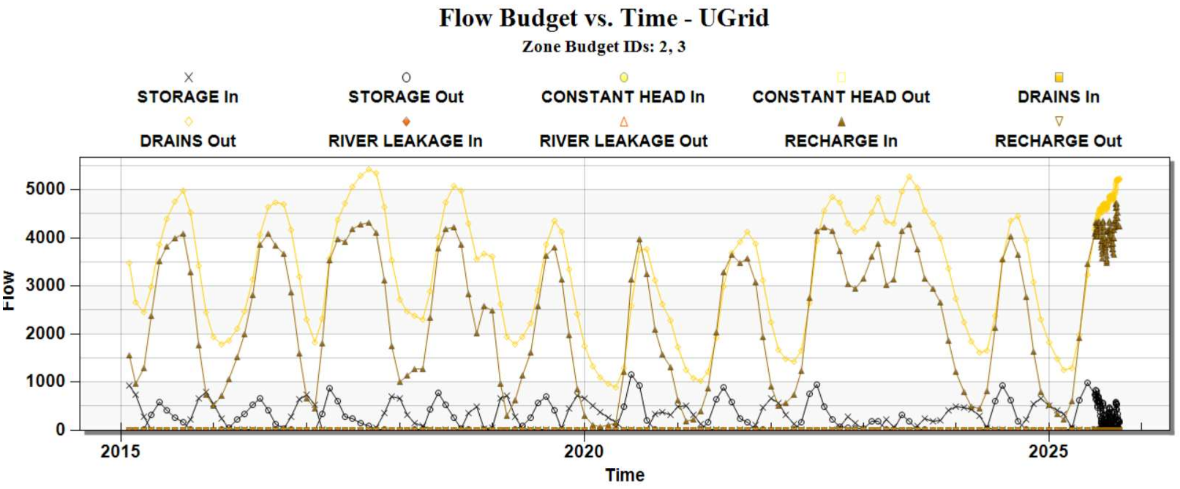

Figure 25: Flow Budget – Existing Conditions within the Boundaries of the Site only

## 7.9 Model Uncertainty

Uncertainty of assigned hydraulic parameters is a potential issue for any numerical groundwater model. This is irrespective of the amount of field testing, groundwater or surface water data that is available to calibrate the model against and to reduce non-uniqueness within the model parameters.

The vertical depths of each layer are obtained from borehole data across the site and are considered well constrained. Hydraulic conductivity of the geological units were consistent across all site testing and is in general agreement with reference values (Freeze and Cherry, 1979).

Discharge via drains provides a strong control on groundwater levels, and shallow groundwater left only a small margin for groundwater simulations. Rainfall recharge to groundwater was a key consideration due surface flooding, which is known to occur at areas within the model domain. Calibration of soil moisture to site piezometers was used to quantify recharge, however, this could not be done for the pumice. Parameters for the pumice were informed by field test on similar soils, however, there remains uncertainty in this regard. We recommend this be investigated and confirmed during the earthworks trial.

It was not considered feasible within the 3D model to replicate short duration groundwater response to rainfall within the piezometers due to the proximity of piezometers to drain cells. A minimum of daily timesteps also restricted the ability of the piezometric surface to fluctuate rapidly. This included when reducing the specific storage values to the lower bound of realistic ranges, and when converting all but layer one to 'confined'.

Reducing the hydraulic conductivity or increasing recharge resulted in the unrealistic flooding of paddocks, not consistent with observed conditions on-site. This provided practical calibration constraints. The recharge data set was adopted when the range of seasonal fluctuations provided a representative response to site observation data sets.

Transient stream flow data during the monitoring period was not available for model calibration during the model development. Therefore, the analysis used in this assessment also considers the relative change in stream flows under different scenarios as well as absolute values.

# 8 Numerical Analysis – Predictive Scenarios and Results

## 8.1 Simulation Scenarios

Predictive simulations were conducted to assess the potential hydrological impacts resulting from the proposed development at Wairakei South. Numerical analyses compared the existing conditions (pre-development) model to the post-development model under various scenarios, as outlined in Table 10.

The simulations evaluated anticipated changes in long-term groundwater levels and hydraulic gradients (Section 8.2), and groundwater flow budgets within the site (Section 8.2.2).

The outcomes of these analyses provide a quantitative basis for understanding how the project may alter site hydrogeology to quantify the potential for groundwater effects and inform design decisions moving forward.

**Table 10: Summary of Model Scenarios**

| Simulation Period | Scenario | Output Objective | Report Section |
|---|---|---|---|
| **3D Transient Simulations**<br>**Period:** 2015-2025<br>Plus 1% AEP Rainfall event | Existing Conditions | Transient Calibration<br>Simulated groundwater levels<br>Flow directions and discharges across the site | Section 7 |
| | Scenario 1: Post Development – Base case conditions | Simulated groundwater levels<br>Flow directions and discharges across the site | Section 8.2<br>Section 8.2.2 |
| | Scenario 2: Post Development + Sea Level Rise | Simulated groundwater levels<br>Flow directions and discharges across the site | Section 8.2.3 |
| | Sensitivity A: Low permeability Pumice | Sensitivity Analysis | Section 8.3.1 |
| | Sensitivity B: High recharge | Sensitivity Analysis | Section 8.3.1 |
| **2D Transient**<br>**Period:** 2015-2025 + 1 in 10-year AEP event | Scenario 3: Post development response to 1% AEP storm event | Estimate groundwater response under a 1 in 100 yr storm event. | Section 8.4 |
| **2D Radial Transient**<br>**Period: 90 days** | Abstraction Potential: 90 day constant abstraction | Localised Groundwater effects from groundwater abstraction | Section 8.5 |

## 8.2 Groundwater Levels and Gradients Pre and Post Development

### 8.2.1 Changes Under Existing and Post Development Scenario

The assessment of long-term groundwater levels and gradients was undertaken by comparing simulated conditions prior to and following development. Winter 2025 groundwater head data were used as a benchmark to compare changes in groundwater gradients resulting from the proposed development. The findings are presented in side-by-side piezometric surface maps in Appendix 1:4, which illustrate the differences in groundwater levels and gradients between pre- and post-development scenarios. Simulated groundwater surface levels show an increase in head beneath the development, but remain below design surface level 3.5–4 m RL, with the highest simulated heads of 1.5 m NZVD and 1.3 m NZVD in the northern and southern block respectively.

The groundwater gradient at the macro scale remains unchanged over the model domain, including the Papamoa dune area. A general trend of flows towards the Bell Road drain and east towards the Kaituna River remains. Groundwater gradients within the pumice fill areas now flow toward the post development swales as opposed to the farm drains that have been removed, representing a minor shift in gradients on the micro scale. 3D modelling simulates higher heads (mounding) against the northern boundary, reaching 2.1 m, which still equates to 1 m below the design surface level, however, we consider this is an overly conservative estimate given the simplified geology within the model space discussed in Section 6.5, which extends the pumice down to -0.4 m NZVD when there is mapped to be higher permeability dune sands and granular backfill where peat removal and replacement is undertaken.

Groundwater levels within the pumice material have a longer response time to seasonal rainfall variation, given the slower infiltration rate compared to the peat. Noticeably, groundwater levels in the pumice are more responsive to consistent moderate rainfall, rather than high intensity rainfall.

Appendix 1:5 shows a groundwater drawdown / mounding plan based on 2025 winter water levels, highlighting pre- and post-development changes specifically for the assessment of groundwater head and flow changes. Mounding up to 0.8 m occurs where existing drains are filled for platform construction and drawdown at new swale locations, up to 1.6 m at the location of the southern attenuation pond.

A supplemental groundwater drawdown plan was developed to assess potential consolidation risk beyond the site. The drawdown calculations use summer groundwater levels, as any consolidation within the summer-winter groundwater range is assumed to have already occurred. Results compared to summer levels are shown in Appendix 1:6 and discussed in Section 9.1

Beyond the site, drawdown is modelled to occur at three locations as a result project works. Described as:

- West of the southern attenuation pond, a 0.5 m drawdown is expected at the site boundary, and the 0.1 m contour extends 160 metres further west over similar farmland.
- In the northwest corner of the site, groundwater drawdown is modelled to occur because of the new swale being deeper than the existing farm drain in this area by approximately 0.2 m, and 0.5 m near the underpass. The 0.1 m contour, albeit minor, extends under the TEL at this location.
- In the northeast, drawdown is caused by excavating the stormwater pond to the invert level of 0.4 m NZVD. Groundwater was observed at this depth during monitoring, so no drawdown relative to summer low water is expected. However, a 0.5 m drop is calculated from winter levels. The 0.1 m drawdown contour extends 180 m.

Groundwater mounding is illustrated on the drawdown plans in Appendix 1.5. As discussed in Section 6.5, this results from a modelling simplification of the geological layers. The 2D SEEP/W modelling (Section 6.7) examines this area directly, with geology reflecting the pinching out of peat which was excluded from the macro-view 3D model. In the 2D simulation, mounding is not predicted to occur due to development of preferential flow paths through the underlying *in situ* dune sand and granular backfill, which will replace the shallow depth of peat that may be encountered in this area.

Given the potential for drawdown to extend beyond the site. A groundwater management plan is recommended to be developed prior to works commencing, in accordance with industry standard practice and to a level of monitoring compliant with avoidance of intolerable deformation of structures and infrastructure. The management plan will stipulate a range of measures to manage groundwater levels within and beyond the site to maintain groundwater levels within the envelope of effects assessed here. Preliminary monitoring and Contingency recommendations are discussed in Section 10.

### 8.2.2 Groundwater Flow

Groundwater flow budgets are a fundamental part of understanding how water moves beneath the ground surface, especially when assessing the effects of land development and environmental changes. A key consideration for is quantifying the rate of groundwater discharge to swales in the post development case so this can be sufficiently accounted for in the stormwater design.

As discussed in Section 6.7, Zone Budgets 1 and 2 represent the site area within the wider model and these are used to quantify the movement of groundwater in and out of the site. The flow budget results from the site pre- and post-development are presented as timeseries in Figure 26. Calculated average and peak flows during the simulation period are presented in Table 11.

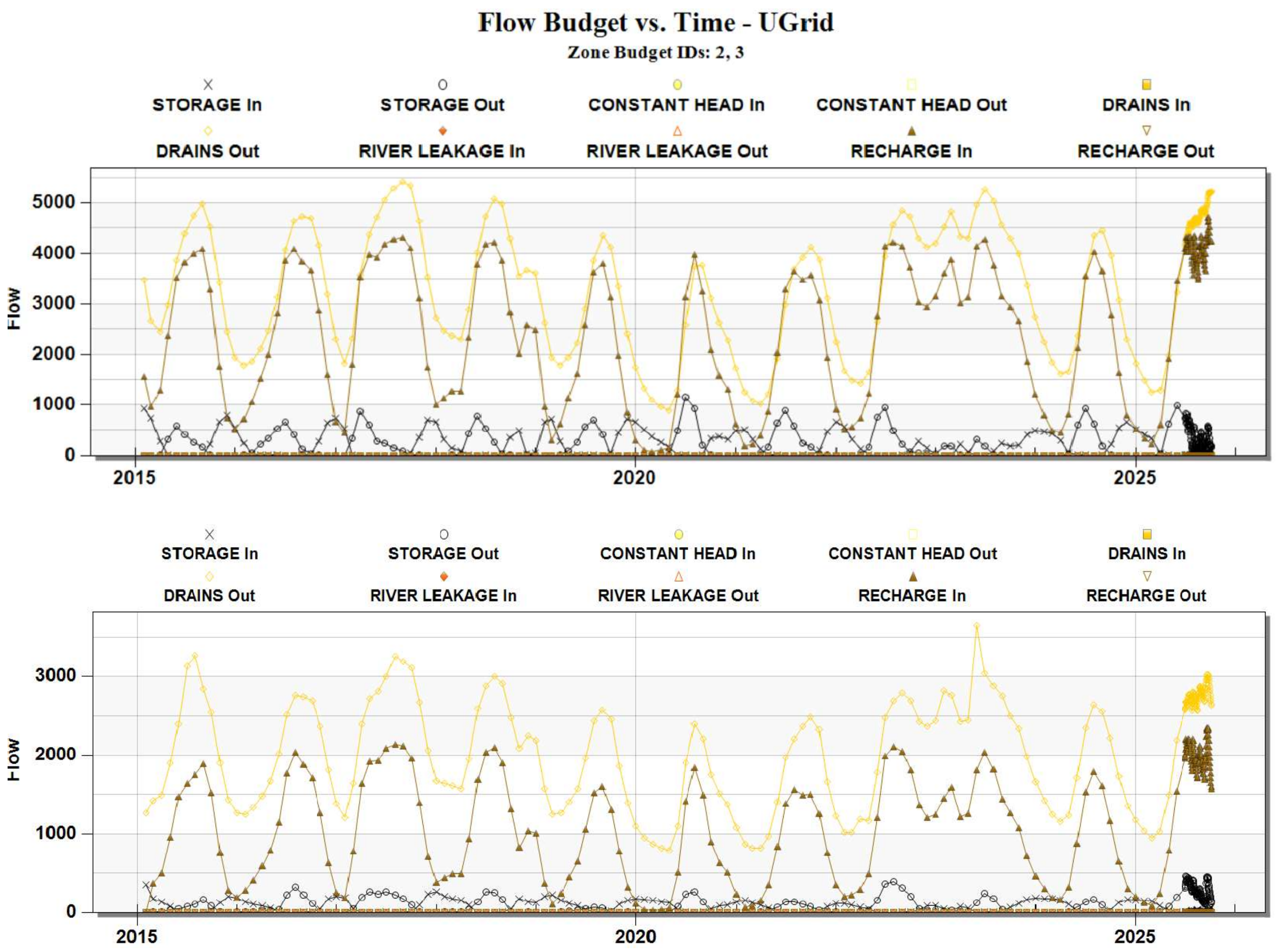

**Figure 26: Pre-Development (upper)– and Post-Development (lower) Groundwater Flow Budgets PGL**

Table 11: Summary of Groundwater Flow Budgets within the Site Boundary

| Average Mass Balance | Components | EGL – Average Flow (m$^3$/d) | PGL – Average Flow (m$^3$/d) | Change in Flow (m$^3$/day) | Change in Flow % |
|---|---|---|---|---|---|
| Inflow | Storage | 208 | 77 | -131 | -5% |
| | Recharge | 2,312 | 1,009 | -1,303 | -52% |
| | **Total inflow** | **2,520** | **1,086** | **-1,434** | **-57%** |
| Outflow | Storage | 184.10 | 66 | -118 | -3% |
| | Drains | 3,233 | 1,989 | -1,244 | -36% |
| | **Total out** | **3,417** | **2,054** | **-1,362** | **-40%** |
| **Discharge Comparison** | | **EGL - Pead Discharge** | **PGL – Peak Discharge** | **Change in Flow (m$^3$/day)** | **Change in Flow %** |
| **Peak Drain Discharge** | | **5,428** | **3,648** | **-1,780** | **-33** |

In the post-development scenario, the flow budget decreases due to a 57% reduction in net infiltration. Both pre- and post-development simulations show that increased rainfall leads to higher drainage outflow. Storage buffers these flows, releasing water during dry months and absorbing it during wet months.

Consequently, as less flow enters groundwater, less groundwater discharges the model via drains and the balance of flow is captured within stormwater modelling by way of increased runoff. Peak discharge via drains reduced by 40% in the post development scenario. The amount of water flowing in and out of storage is muted by the lower permeability and lower porosity of the compacted pumice compared to the peat.

The effect of sea level rise on flow budget was assessed. The results found that within the site boundary, change in flow budget was less than 1% compared to base case models, consistent with indiscernible changes in groundwater table on-site under sea level rise conditions. This aligns with the conceptual model for the site with groundwater primarily controlled by drains and the elevation of the drains which are assumed to remain largely unchanged.

### 8.2.3 Results Under Sea Level Rise Scenario

To assess the impact of sea level rise on groundwater conditions, simulations were conducted by increasing the groundwater head by 1.59 m at both the coastal constant head boundary and the Kaituna River boundary. The configuration of drain cells within the modelling domain remained unchanged from previous simulations, based on the assumption that the existing pump stations at Bell Road and Kopuaroa Canal will continue to operate and maintain water levels at their current elevations.

The results show groundwater levels rise along the coastal margin, mainly on the northern side of the Papamoa dunes, as well as along the Kaituna River. In these areas, higher groundwater heads led to steeper gradients towards and higher discharge to the nearest discharge boundary conditions being the coastal boundary and the drains nearest to the Kaituna River. Pre- and post- development heads are presented in Appendix 1:7.

Despite these changes in groundwater elevation and gradient near the coastal boundaries, there was no significant change observed in groundwater levels within the vicinity of the site itself. This constancy is attributed to the presence of multiple layers of drainage features between the coastal boundaries and the site, which effectively buffer the site from impacts associated with elevated sea levels.

Overall, the model simulation supports our conceptual model that, under the assumed operational conditions of ongoing maintenance of the surrounding drainage network including local pump stations, the effects of the seal level rise on the site are likely to be less than minor.

To view the overall effect of existing conditions versus the 2130 Climate Change prediction scenario, a drawdown comparison of sea level rise compares a 1% AEP rainfall event including sea level rise against the existing conditions under a 1% AEP event. The results are presented in Appendix 1:8.

The simulation shows the areas expected to undergo the greatest change in groundwater levels.

## 8.3 Sensitivity Assessment

As discussed above, the existing control of groundwater by the extensive network of drains surrounding the site limit the off-site effects on the project. Therefore, the uncertainty assessment focused on the infiltration of groundwater recharge through the pumice, and the hydraulic parameters of the compacted pumice.

### 8.3.1 Sensitivity A - Low Permeability Pumice Fill

A sensitivity analysis was conducted to evaluate the groundwater response under a scenario where the pumice fill exhibits significantly lower permeability, specifically, with hydraulic conductivity values an order of magnitude lower than those used in the base case. In this scenario, both horizontal and vertical conductivities were reduced to $1\times10^{-8}$ m/s and $5\times10^{-9}$ m/s, respectively.

The results demonstrated that, although the model simulated increased groundwater mounding (due to the lower permeability pumice) relative to the base case, the groundwater levels remained below the development platform surface. The highest simulated heads were 1.55 m NZVD in the northern block and 1.65 m NZVD in the southern block, indicating that groundwater did not rise within 1 metre of the FGL. The simulated heads from the highest time step are presented in Appendix 1:9

A review of resulting mass balance showed a reduction in overall groundwater flow compared to the base case scenario, with lower drain discharges. This reduction is attributable to the decreased permeability of the pumice, which limits the rate at which groundwater can move through the system.

This scenario provides confidence that under conservative assumptions of low pumice permeability, the risk of groundwater daylighting the surface remains low.

### 8.3.2 Sensitivity B - High Groundwater Recharge

Recharge to the surficial pumice unit plays a key role in the groundwater model by controlling how much water enters the system. This impacts both baseflow discharges into nearby streams and the formation of groundwater mounding beneath the site. ENGEO selected recharge values informed by infiltration tests from similar compacted pumice soils at a nearby site. However, it is important to note that these recharge estimates carry greater uncertainty since no direct field infiltration testing was performed on-site for this project.

To address this uncertainty and assess the robustness of the model predictions, a sensitivity analysis was undertaken to evaluate the potential impact of higher recharge rates on groundwater behaviour. Specifically, the model was stress tested by increasing the recharge rate applied to the pumice unit by 200%. This sensitivity check allows for the evaluation of a potential upper bound scenario, providing insight into how elevated infiltration could influence groundwater levels, baseflow contributions to surface water, and the extent of groundwater mounding beneath the site. The outcomes of this analysis inform the understanding of risk and help guide management decisions in the context of limited field data.

Groundwater surface levels presented in Appendix 1:9 show an increase in head beneath the development, but remain below design surface level, with the highest simulated heads of 1.75 m NZVD and 1.85 m NZVD in the northern and southern block respectively. 3D modelling simulates higher heads against the northern boundary, reaching 2.25 m NZVD. We consider this is an over conservative estimate given the simplified geology within the model space discussed in Section 6.5, which extends the pumice down to -0.4 m NZVD. Regardless, simulated levels remain 1m below the design surface level.

Groundwater discharges via drains under a high rainfall recharge scenario within the site (Zones 1 and 2) increased from an average of 1,999 m3/day to 2,089 m3/day compared to the base case scenario, with a peak discharge of 3,500 m3/day during the simulation period.

## 8.4 Groundwater Level During Future Storm Events

To further evaluate the interaction between stormwater management features and the underlying pumice fill, a two-dimensional SEEP/W analytical model was developed. The primary objective of this modelling exercise is to assess whether elevated water levels (hydraulic heads) within the stormwater swales—particularly during extreme rainfall events such as a 1 in 100-year storm—have any appreciable effect on groundwater levels within the low permeability pumice, particularly at locations spatially removed from the swale features.

This approach is grounded in the understanding that the pumice fill underlying the development is characterised by low hydraulic conductivity, resulting in limited infiltration capacity. As discussed previously, the groundwater system in this setting is more responsive to sustained, moderate rainfall over extended periods than to short-term, high-intensity storm events. This is due to a combination of low permeability, a high proportion of impermeable surfaces (such as buildings and roads), the absence of soakage features, and minimal surface ponding. These factors collectively promote greater surface runoff and restrict the direct infiltration of stormwater into the pumice layer.

The SEEP/W model simulates the transient response of groundwater levels to temporary increases in surface water heads within the swales, by applying a time-dependent total head boundary representative of the highest water levels observed during the modelled 10-year period. The results, presented in Appendix 1:10, demonstrate a limited groundwater response to stormwater events, with only minor infiltration occurring in the immediate vicinity of the swales. The low hydraulic conductivity of the pumice acts as a barrier, preventing significant recharge to groundwater even under high surface water conditions in the relatively short time frame of a 10-day rainfall event. Following storm events, both swale water levels and adjacent groundwater levels are simulated to rapidly return to baseline conditions, indicating that the risk of widespread groundwater daylighting due to one off rainfall infiltration is low.

In summary, the SEEP/W analysis provides robust evidence that high water levels in stormwater swales during extreme rainfall events have a minimal influence on groundwater levels within the pumice fill, except for small, localised increases near the swales. This supports the conclusion that the current stormwater management design is effective in mitigating potential impacts on the groundwater regime.

## 8.5 Groundwater Abstraction Analysis

Given the scope of the development and the necessity to relocate ponds during construction planning and phased earthworks, it is not feasible to determine the exact pond locations at this stage of the project. Instead, the assessment comprises an independent evaluation of potential impacts that may arise from an infiltration pond based on typical site conditions to determine the extent of potential effects, which can then be used to restrict the locating of ponds in areas where adverse affects may arise.

Model scenarios included a 2D axisymmetric assessment of groundwater inflows, with an analytical sensitivity check using a pit inflow calculator.

### 8.5.1 Available Aquifer Allocation

Groundwater availability is derived from the Kaituna Plains Secondary (upper) groundwater management unit, which specifies an allocation limit of 5.65 Mm³/year (Bay of Plenty, n.d.). Of this, approximately 5.4 mm³/year has been allocated, equating to roughly 95.5% of the total. This results in an estimated 253,787 m³/year remaining available for allocation.

A supplementary groundwater allocation assessment conducted by PDP on behalf of BOPRC in January 2025 (PDP, 2025) identified new allocation boundaries and sustainable allocation limits. The PDP estimated an allocation of 4.997 mm³/year for shallow groundwater in this area.

The current proposal seeks to abstract 1,955 m³/day from shallow groundwater, amounting to 713,575 m³/year. The challenge remains of estimating allocated groundwater in the previous allocation system, against the proposed allocation boundaries and units of the proposed system. An assessment will be undertaken in collaboration with BOPRC.

We note that the water will be distributed across the site, with a portion of the dust control returning to the shallow groundwater aquifer.

### 8.5.2 Reasonable and Efficient Use

The volume of water requested is proportional to the scale of earthworks being undertaken. Dust suppression will be managed using water carts and established industry-standard methods. Where feasible, stabilisation techniques will be implemented to reduce exposed areas and associated dust potential. In light of the costs related to dust control, there is an inherent need to balance minimising water usage with fulfilling dust control obligations. Accordingly, we believe that the application of water for dust control purposes aligns with the principles of reasonable and efficient use.

### 8.5.3 Consultation

No consultation has been undertaken at the time of writing.

### 8.5.4 Groundwater Inflow Potential

Variable depths were assessed and sensitivity analyses were undertaken to provide a range of estimated inflow rates. Specifically, the sensitivity focused on the depth of the pit extending from only 0.5 m below the top of sand layer, up to 4 m below, and included a higher permeability sensitivity check with the sand hydraulic conductivity set to $5 \times 10^{-5}$ m/s.

The modelled inflow rate ranged between 1 L/s and 4 L/s per infiltration pond. Modelling suggests inflows will stabilise quickly, as presented in the water inflow rate vs time plot shown below.

A cross-check was undertaken using analytical calculations for inflows into a pit (ECan, 2024)[8], which incorporates Thiem-Dupuit-Forcheimer & Bear equations. The analytical method estimated inflow rates of 0.6 to 2.2 L/s.

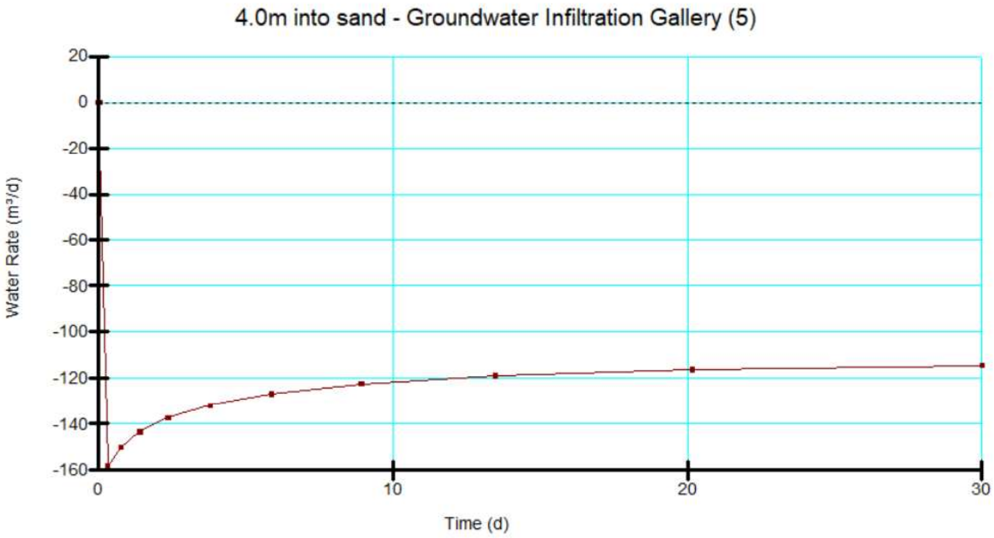

**Figure 27: Estimated Groundwater Inflow**

### 8.5.5 Groundwater Drawdown

Groundwater level drawdown as a result of continuous abstraction is presented in Figure 28, presented as static water level (time 0) and heads after 30 days of pumping (t=30 days). The change in groundwater level between pre and post groundwater abstraction shows drawdown within the centre of the pond reaches 5.5 m. Groundwater seepage through the battered sides reducing the magnitude of drawdown to 2.5 m at the edge of the pond.

Drawdown reduces with distance, reaching 0.5 m at approximately 50 m from the centre of the excavation.

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[8] Environment Canterbury Regional Council, 2024 – Dewatering Well or Pit – Unconfined Groundwater

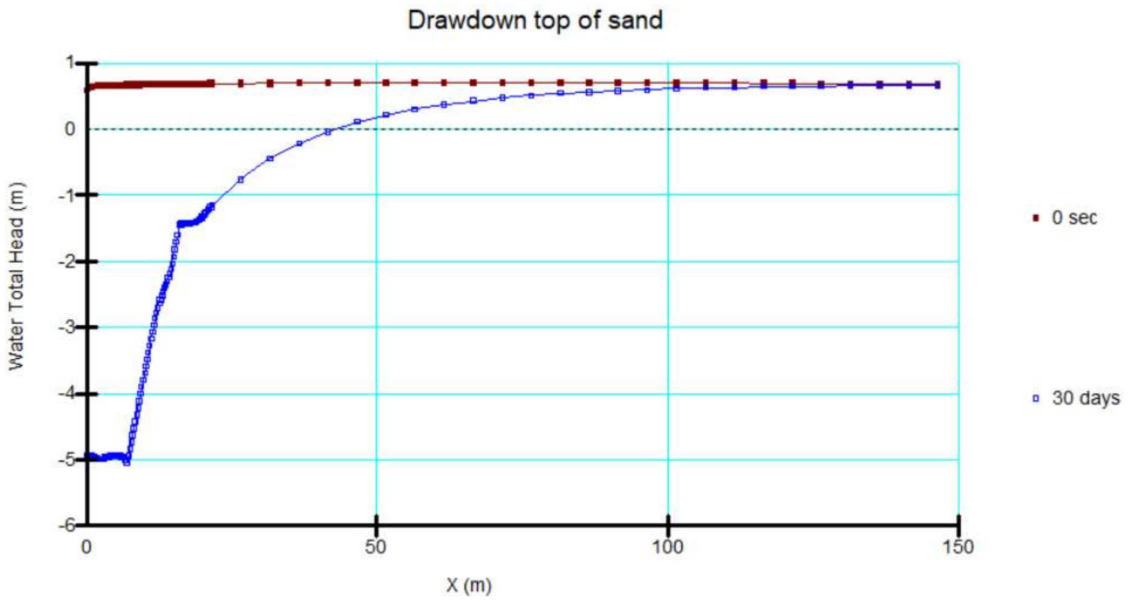

Figure 28: Groundwater Level Pre and Post Abstraction from Groundwater Infiltration Pond

### 8.5.6 Surface Water Effects

The network of drains across the site is represented by drain nodes spaced every 200 m. Given the radial symmetry of the model, these drains represent a radial boundary. It is recognised that this differs from the typical grid pattern observed on-site, however the resulting effect is considered adequately captured. Recharge from the nearest drain was simulated in the model, and the change in recharge to groundwater presented in Figure 29. The plot shows discharge of groundwater to the drain of 43.5 $m^3$/day under steady state conditions, reducing to 41.25 $m^3$/day when under the effect of groundwater drawdown from the infiltration pond (i.e. the infiltration pond reduces groundwater pressure expressing at surface drains). This equates to an estimated reduction in drain discharge at the nearest surface drain by only 5%.

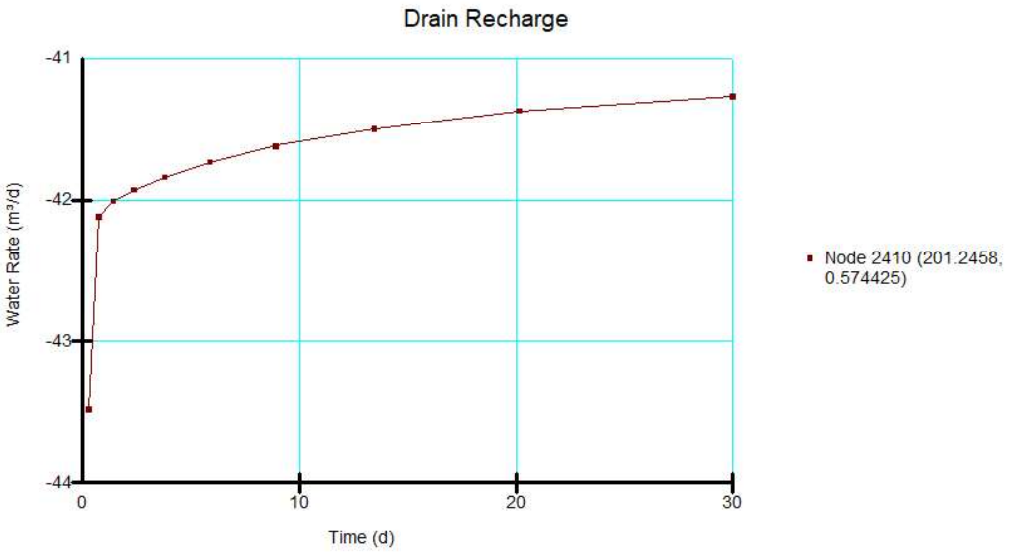

Figure 29: Change in Discharge to Surface Drains

# 9 Hydrogeological Effects Assessment and Key Findings

## 9.1 Effects on Groundwater Levels and Gradients

Changes to long-term groundwater levels are assessed to be less than 0.4 m beyond the site boundary. Our review of the modelling results confirm minor drawdowns are anticipated near the southern and north-eastern attenuation ponds, with localized drawdown also anticipated in the north-eastern corner of the site. Across the wider area surrounding the site, groundwater levels and gradients are not expected to be appreciably impacted by the development.

Within the site, groundwater heads will experience localised changes, with drawdown occurring where new drains are introduced and slight mounding where existing drains are backfilled. Post-development modelling indicates that groundwater levels will be one metre below the FGL of the development filled surface, supported by the installation of sand drainage blanket beneath the fill. Groundwater is modelled to remain greater than 1 m below the designed surface level even under sensitivity scenarios including 200% rainfall infiltration to pumice, reduced pumice permeability and a 1% AEP rainfall event. Assessment of the groundwater impact from a 1% AEP rainfall event indicates that, even when groundwater attenuation ponds and swales reach capacity during extreme rain, they have little effect on groundwater levels within the pumice fill. The only exception is minor, local increases near the swales, which subside once the rainfall event ends.

The post development modelling suggests groundwater levels tend to rise more significantly after long durations of moderate rainfall, than after specific intense ‘design storm’ rainfall events, due to the low infiltration capacity and associated high proportion of runoff. Future climates in the Bay of Plenty are expected to be drier yet accompanied by more intense rainfall events (NIWA, n.d.). Although this does not eliminate the possibility of increasing groundwater levels, it implies that the risk under future climate conditions may be lower than in regions expecting future climate conditions to have consistently moderate rainfall.

Groundwater drawdown, while not an effect on its own, can be a pre-curser for consolidation settlement. For consolidation settlement potential, the summer of 2024 was selected as a typical summer to calculate groundwater drawdown against previous seasonal-low groundwater levels (above which consolidation effects would have occurred in the geological past). The assessment calculated 0.3 m of drawdown extends over the site boundary on the southwest corner of the model. This reduces to 0.1 m with 160 m of the site. The land use in this area is agricultural farmland, and the mapped geology is peat. While the consolidation potential in peat is high, we consider the potential for adverse consolidation settlement effects on the farmland are less than minor due to the absence of infrastructure.

Drawdown of 0.2 m is also predicted on the northwest corner of the site and 0.5 m near the location of the existing underpass structure and 0.3 adjacent the north-eastern attenuation pond. Land use is primarily agricultural farmland with the TEL further north. Drawdown in the north of the site extends further as the geology in this area is mapped as dune sand of high hydraulic conductivity. 0.2 m of drawdown is mapped under the TEL. The potential for consolidation settlement in sand is low due to this material exhibiting elastic settlement characteristics and therefore the potential for adverse effects to be less than minor.

## 9.2 Effects on Groundwater Mass Balance

The overall mass balance shows a 60% reduction in rainfall infiltration to groundwater (recharge) on-site due to the increase of impermeable surfaces associated with converting the flat farmland into an elevated, graded development and the site development platform being constructed from low hydraulic conductivity pumice fill. The balance is partitioned as overland flow or direct runoff to stormwater which is not simulated within the groundwater flow model and is considered and mitigated through the design of stormwater management features such as swales and attenuation ponds as part of the civil works stormwater design.

The reduced infiltration to groundwater results in a 33% reduction in groundwater discharge to drains and swales under average winter conditions. This equates to 3,700 $m^3$/day in the post development scenario. This water will be conveyed through the site swales and attenuation ponds before discharging to the Bell Road drain or the Kopuaroa Canal through an outflow to algin with pre-development conditions. The rate of groundwater discharge is small compared to the stormwater during a storm event. As a conservative assumption, the groundwater discharge component of the total flow was multiplied by an order of magnitude when incorporated into the stormwater modelling simulations to provide a conservative assessment and account for uncertainty in the model outputs.

## 9.3 Groundwater - Surface Water Effects

The northeast and southwest stormwater attenuation ponds proposed invert levels are 0.4 m NZVD and 0.7 m NZVD, respectively.

For the northeast pond, this level is comparable to existing summer conditions, while a up to 0.5 m below winter water levels. The design water level in the pond will ultimately be controlled by the outlet elevation; however, these elevations suggest groundwater level will be near the invert of the attenuation pond in summer and seasonally seeping into the base of the pond in winter. For the southwest pond, this invert level equates to a drawdown of 0.6 m compared to summer levels, and up to 1.5 m of drawdown compared to the simulated winter water levels. Groundwater will persistently seep into the base of the pond. This water, along with rainfall run-off will be retained in the attenuation ponds before discharging to the Bell Road drain or the Kopuaroa Canal.

As noted above, overall estimated baseflows into the drains and ultimately flowing into the ponds is 3,700 $m^3$/day. This flow has been accounted for in the stormwater design.

The reduction in infiltration to groundwater and subsequent increase in surface run-off is mitigated on off-site surface water receiving environments by the implementation of stormwater attenuation ponds, which regulate the discharge of water to the downstream environment, designed to align with pre-development water flows. Therefore, the conceptual flow balance to downgradient surface water features remains in generally alignment pre and post development. Overall, the off-site effects to downgradient environments from the project are considered to be less than minor.

## 9.4 Groundwater Abstraction Effects

Shallow groundwater abstraction is proposed for dust suppression during construction. The Civil Engineers have estimated a daily water rate required up to 1955 $m^3$/day[9].

Abstraction method is proposed via infiltration ponds or a series of groundwater spears. The water will be abstracted for dust control on an as-needed basis, and distributed across the open earthworks areas of the development. Areas open for extended periods will be stabilised to minimise water use. Where water is abstracted for construction dewatering, this water will be reused as dust control.

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[9] Email communication from Maven. Dated 11 February 2026.

Multiple ponds are likely to be required per stage to make up the required yield, with ponds moving with progressive development of the respective stages. The assessment assumed a 10 x 10 m pond base, dug to 6 m below ground level, with static groundwater level 0.5 m below ground surface.

Modelling described in Section 8.5 shows groundwater abstraction between 1 and 4 L/s can be abstracted from a single infiltration pond.

The modelling shows a drawdown of 5.5 m at the location of the pond, extending radially. Modelling shows the 0.5 m contour is predicted to extend 50 m from the centre of the excavation after 30 days of constant dewatering. The model results estimated a reduction in drain discharge at the nearest surface drain by 5%. We note this is a conservative assessment as it assumes constant dewatering, not accounting for recovery during times where dust control is not required. Additionally, given the network of drains across the site and the exact location of pits to be determined, recharge from local drains could reduce the extent of drawdown when compared to the estimated cone of depression presented above.

Based on the assessment findings, we recommend that the ponds be installed at least 50 m from any existing structures which could be prone to ground settlement in the peat soils, thereby reducing the maximum drawdown to 0.5 m. Provided the ponds are located at least 50 m from the boundary, drawdown and potential consolidation settlement beyond the site where peat is present is conservatively calculated to be less than 300 mm. We consider the estimated drawdown and potential settlement effects of the proposed abstraction to be less than minor, and within the magnitude of effects associated with the project.

Drawdown of less than 0.5 m is considered sufficient to avoid adverse effects on other registered bore owners. Additionally, the abstracted water will be re-applied to the site and a portion will likely re-enter the groundwater system.

# 10 Recommendations for Monitoring and Contingency

## 10.1 Earthworks Trial

As discussed throughout this report, there is no on-site testing of the proposed pumice fill. Additional confidence in the model outcomes can be obtained if *in situ* testing of infiltration and hydraulic conductivity can be completed.

We recommend utilising any embankment trials as an opportunity to perform site-specific testing on the pumice to obtain measured hydraulic parameters of the compacted pumice *in situ* material.

Should results from site-specific testing yield parameters that differ significantly from those applied in this assessment, the groundwater analysis should be revised accordingly to evaluate any potential changes in groundwater effects.

Recommended testing procedures include:

- Construction of a graded test pad to replicate the equipment, materials, and compactive effort planned for the project. A minimum basal pad size of 10 x 20 m is recommended from a hydrogeological perspective.
- Vertical infiltration soakage tests in pumice fill (recompacted samples when source materials confirmed and during site trial).

- Groundwater monitoring within and below the pumice layer for groundwater mounding potential.
- Hydraulic conductivity testing in pumice.

## 10.2 Earthworks Drainage Recommendations

Where cut and replacement of the peat is proposed, excavations may extend below the groundwater table. During excavation, groundwater inflows are anticipated. However, based on our experience with similar earthworks projects and the proposed site-specific monitoring and management measures, these inflows can be effectively controlled.

Should groundwater inflows exceed manageable levels, targeted dewatering solutions or wet excavation techniques shall be implemented to maintain stability and ensure construction proceeds safely and efficiently. These methods are well-established in the industry and, when combined with ongoing groundwater monitoring and adaptive management, will ensure that any potential groundwater-related impacts adjacent to the site are minimised and remain within the parameters assessed.

An assessment will be required under Rule 42 and / or Rule 43 of the BOPRC Natural Resources Plan for the take and use of water for dewatering purposes

## 10.3 Groundwater Monitoring

Prior to the commencement of works, ENGEO recommend a groundwater monitoring and contingency programme be prepared, including installation of piezometers to monitor the locations where drawdown is expected to occur specified in section 9.1.

Baseline groundwater monitoring should begin at least three months before earthworks to ensure that sufficient data is collected to represent pre-construction groundwater conditions accurately. This monitoring will enable the identification of any deviations from the expected groundwater behaviour during construction.

To ensure that groundwater effects remain within those for the parameters assessed, trigger levels must be set. Trigger levels provide an early actionable indicator if site groundwater responses exceed those anticipated in the assessment. Should these trigger levels be reached or exceeded, the contingency plan should be enacted. This may include the implementation of additional groundwater control measures, such as targeted dewatering, wet excavation techniques, or adaptive management strategies, to mitigate any potential adverse effects on neighbouring properties.

Should trial embankment monitoring indicate groundwater response in the pumice is greater than simulated in this assessment, mitigation measures, such as preferential subsurface drainage pathways, should be incorporated into the detailed design.

# 11 Limitations

i. We have prepared this report in accordance with the brief as provided. This report has been prepared for the use of our client, Bell Road Limited Partnership, their professional advisers, the relevant Territorial Authorities and the appointed Fast Track Panel in relation to the specified project brief described in this report. No liability is accepted for the use of any part of the report for any other purpose or by any other person or entity.

ii. The recommendations in this report are based on the ground conditions indicated from published sources, site assessments and subsurface investigations described in this report based on accepted normal methods of site investigations. Only a limited amount of information has been collected to meet the specific financial and technical requirements of the client's brief and this report does not purport to completely describe all the site characteristics and properties. The nature and continuity of the ground between test locations has been inferred using experience and judgement and it should be appreciated that actual conditions could vary from the assumed model.

iii. Subsurface conditions relevant to construction works should be assessed by contractors who can make their own interpretation of the factual data provided. They should perform any additional tests as necessary for their own purposes.

iv. This Limitation should be read in conjunction with the Engineering NZ / ACENZ Standard Terms of Engagement.

v. This report is not to be reproduced either wholly or in part without our prior written permission.

We trust that this information meets your current requirements. Please do not hesitate to contact the undersigned on (07) 777 0209 if you require any further information.

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# 12 References

Bay of Plenty Regional Council. (2021). *MfE Coastal Hazards and Climate Change Guidance Report.* Operations Publication.

Bay of Plenty Regional Council (n.d.) *Indicative Groundwater Availability and Consented Allocation https://experience.arcgis.com/experience/1e275c87a19d4a038d9dd192d1c45c9a/page/Western-BOP---Upper#data_s=id%3AdataSource_4-19742adb731-layer-6%3A38. Accessed February 2026.*

Boffa Miskell. (2025). *Wairakei South Development.* Appendix C in the appendicies to the application..

Council, B. o. (n.d.). *Environmental Data Portal*. Retrieved from https://envdata.boprc.govt.nz/Data/Map

ENGEO. (2023). *57 Pitau Road Assessment of Effects.*

ENGEO. (2024). *Falling Head Soakge Test - 105 Domain Road.*

ENGEO. (2025). *Wairakei South - Geotechnical Factual Report.* Appendix O in the appendicies to the application.

ENGEO. (2025). *Wairakei South - Geotechnical Interpretive Report.* Appendix P in the appendicies to the application.

Freeze, R. A., & Cherry, J. A. (1979). *Groundwater.* Englewood Cliffs.

GNS, 2020. (n.d.). *1:250 000 Geological Map of New Zealand dataset (Heron 2020)*. Retrieved from https://data.gns.cri.nz/geology/

Golder Associates (NZ) Limited. (2020). *Golden Sands Development, Papamoa, Geophysical and Geotechnical Investigations Factual Report (Nos 19135525-001-R-Rev0).*

LINZ. (n.d.). *Land Information New Zealand*. Retrieved from Bay of Plenty LIDAR 1m DEM (2024).

Maven. (2025). *Bell Road Stormwater Assessment.*

Maven. (2025). *Bell Road Stormwater Design.*

MfE. (2024). *Climate Projections Map*. Retrieved from https://map.climatedata.environment.govt.nz/

NIWA. (2013). *The Climate and Weather of Bay of Plenty 3rd edition.*

NIWA. (n.d.). *Climate change scenarios for New Zealand*. Retrieved from https://niwa.co.nz/climate-and-weather/climate-change-scenarios-new-zealand

*NZ SeaRise*. (n.d.). Retrieved from https://searise.takiwa.co/map

PDP. (2025). *Groundwater Allocation Limits for the Bay of Plenty Region. Prepared for Bay of Plenty Regional Council.*

Sinclair Knight Merz. (2012). *Australian Groundwater Modelling Guidelines.*

TaurangaCityCouncil. (n.d.). *Mapi – Council GIS Web Map Viewer*. Retrieved from https://gis.tauranga.govt.nz/vertigisstudio/web/?app=8d1e800bf4314d8b89bd41a84e5daeb5

TCC. (n.d.). Data provided by TTC.

TCC. (n.d.). *GIS Portal - Groundwater 2022*. Retrieved from https://gis.tauranga.govt.nz/portal/apps/experiencebuilder/experience/?id=eaf07d993750457ea733b263ce5ecc3e#data_s=id%3AdataSource_1-18e01c5f2f1-layer-13%3A215

TCC. (n.d.). *Groundwater 2022*. Retrieved from Flooding From Rising Groundwater: https://gis.tauranga.govt.nz/portal/apps/experiencebuilder/experience/?id=eaf07d993750457ea733b263ce5ecc3e

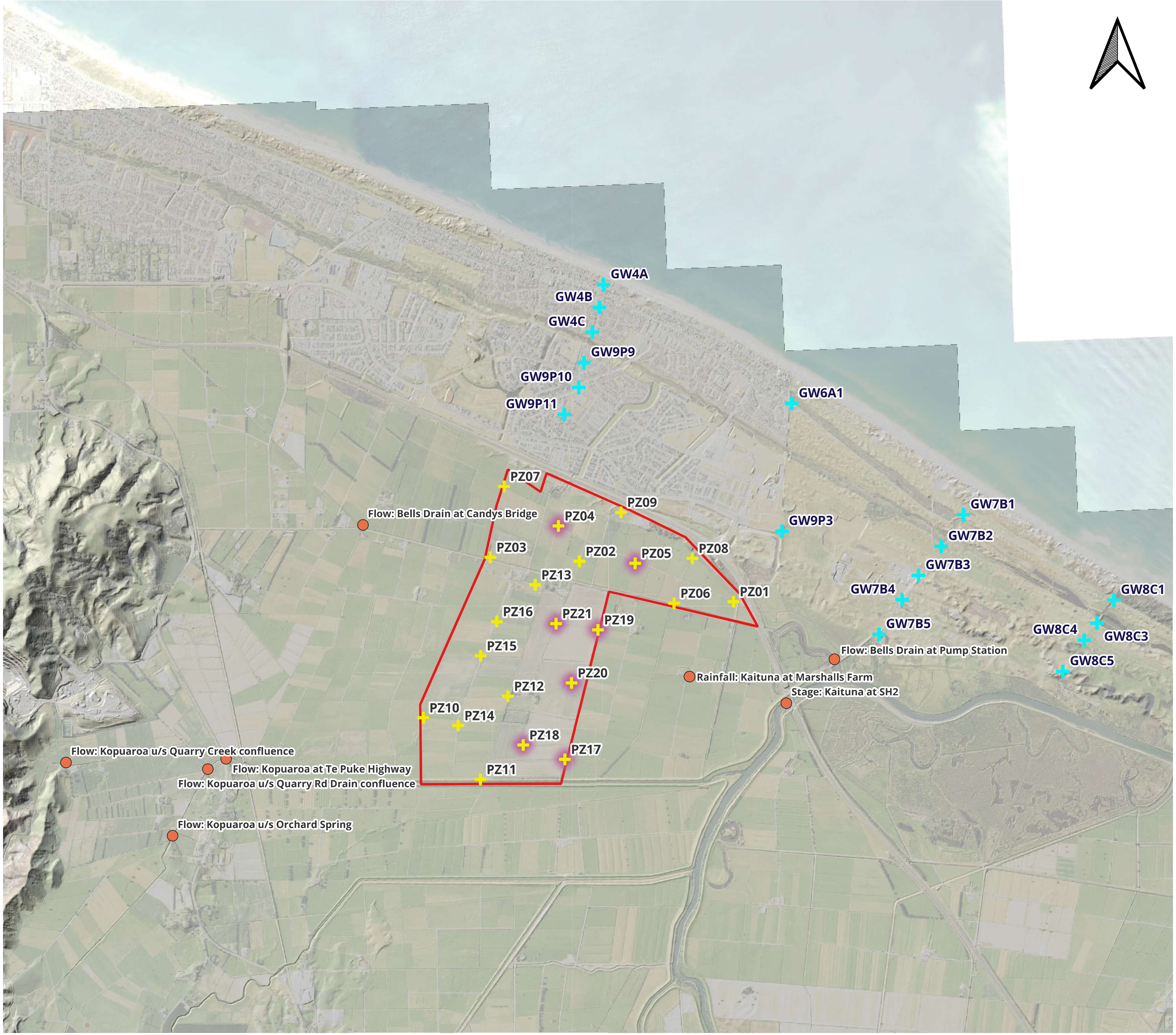
TCC. (n.d.). *Infrastructure development code - DS5.*

TeMaruoKaituna. (2018). *The Kaituna River Document: Kaituna, he taonga tuku iho – A treasure handed down.* Bay of Plenty Regional Council.

Westerhoff, R. (2018). *nzrainfallrecharge*. Retrieved from https://rogierwesterhoff.users.earthengine.app/view/nzrainfallrecharge

# Appendix 1:

## A3 Plan Drawings



Legend

- ✚ Bell Rd Piezometers (Slug Tested)
- ✚ Bell Rd Piezometers
- ✚ Papamoa Pizometers TCC
- ● BOPRC Data Measurement Stations
- ▭ Approx. Site Boundary

Notes:
1. ENGEO site investigations surveyed by Maven, received May 29 2025
2. Monitoring Station locations provided by BOPRC
3. Aerial Imagery from Bay of Plenty 0.2m Rural Aerial Photos (2023-2024)

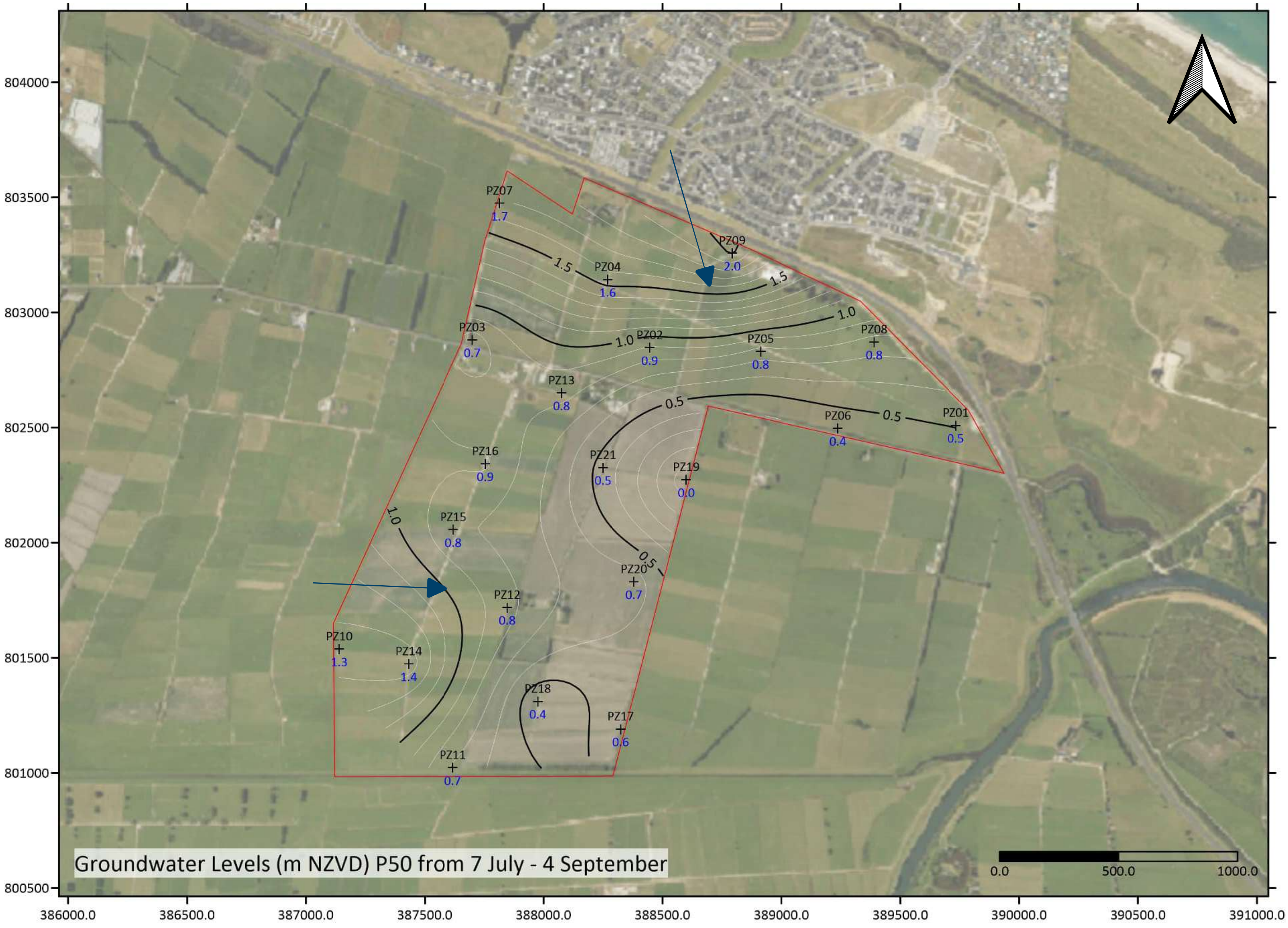
0 750 1,500 m

ENGEO

Projection: NZTM 2000NZGD2000 / Bay of Plenty 2000

Client: Bell Road Limited Partners

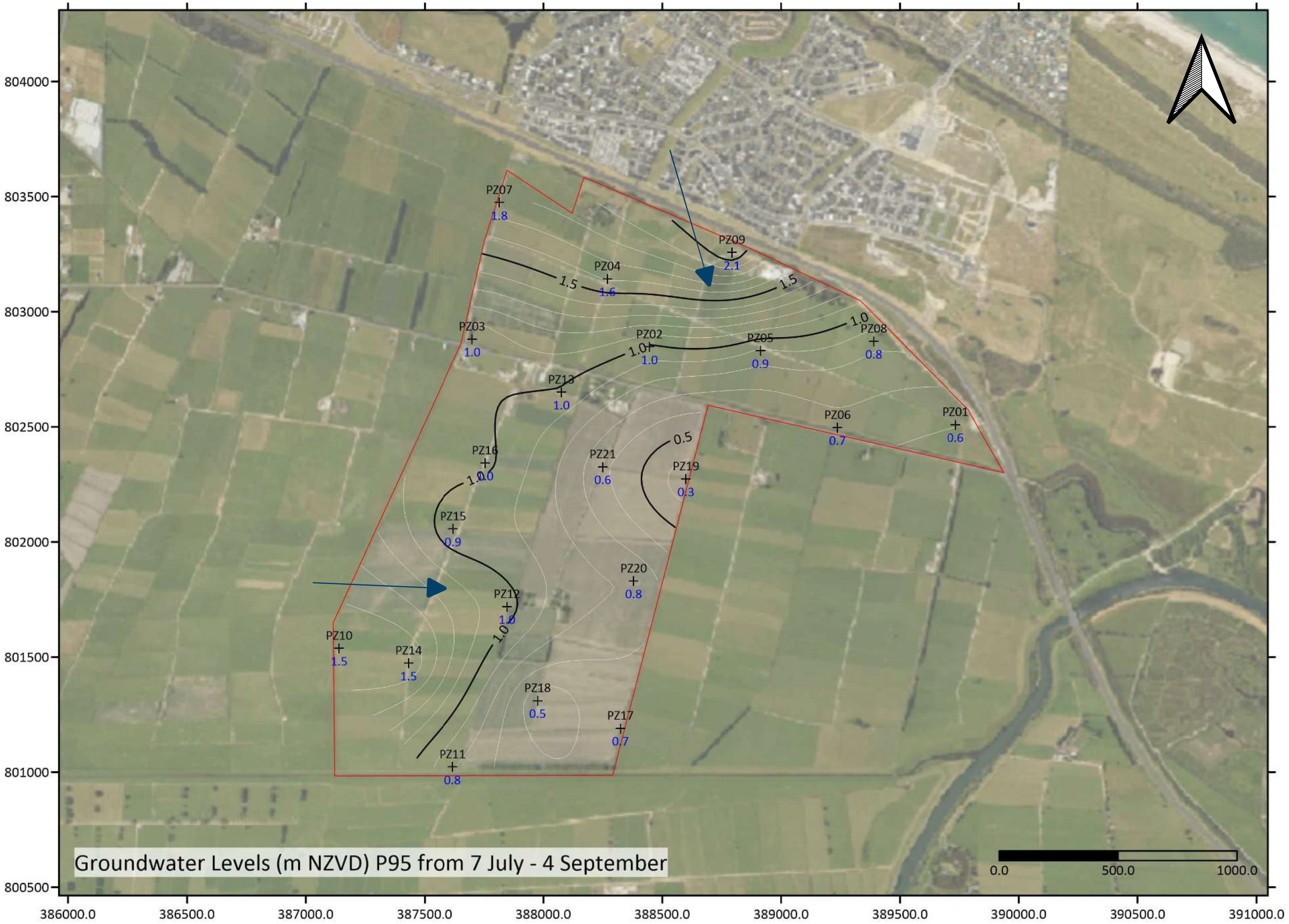
| | | |
|---|---|---|
| Title: Site Investigation and Monitoring Plan | | Appendix No. 1<br>Size: A3<br>Sheet 01 |
| Project: Bell Road | Drawn: ZM | |
| Date: 14/11/2025 | Checked: SB | |
| Proj No: 196300.000.001 | Scale: 30000 @ A3 | Version: 1.0 |


Groundwater Levels (m NZVD) P50 from 7 July - 4 September

Inferred Groundwater Flow Direction

ENGEO

| | |
|---|---|
| Client: Bell Road Limited Partners | |
| Title: Piezometric surface based on P50 Groundwater levels | Appendix No. 1<br>Size: A3<br>Sheet 02 |
| Project: Bell Road | Drawn: ZM |
| Date: 17/11/2025 | Checked: SB |
| Proj No: 196300.000.001 | Version: 1.0 |



Groundwater Levels (m NZVD) P95 from 7 July - 4 September

→ Inferred Groundwater Flow Direction

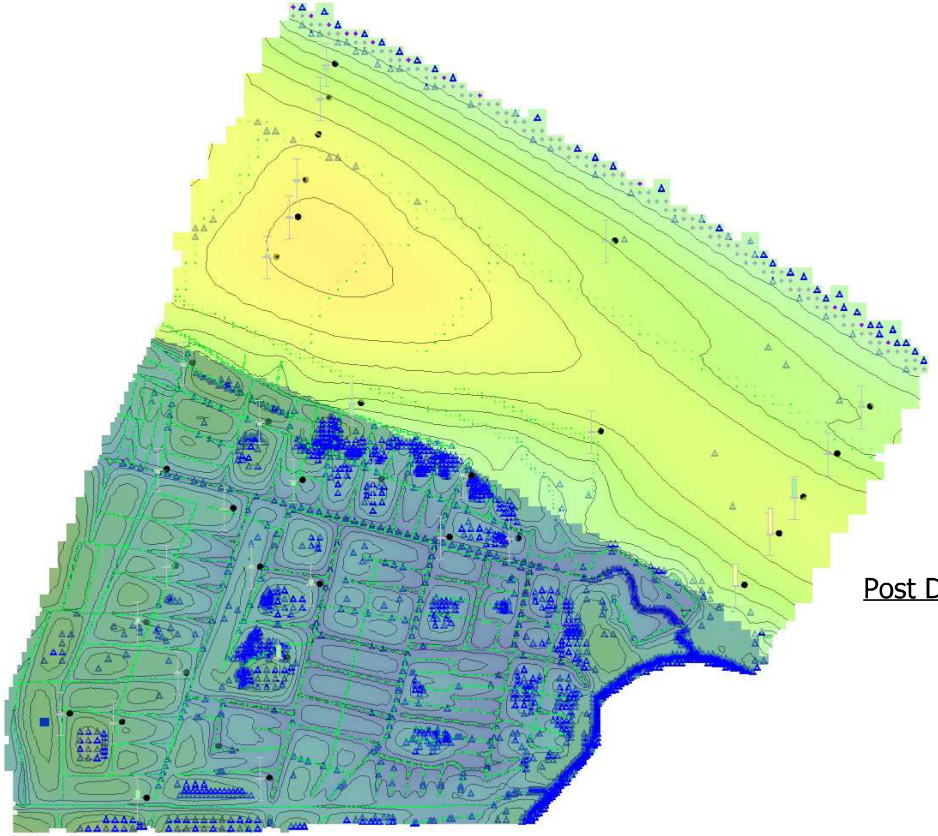
ENGEO

Client: Bell Road Limited Partners

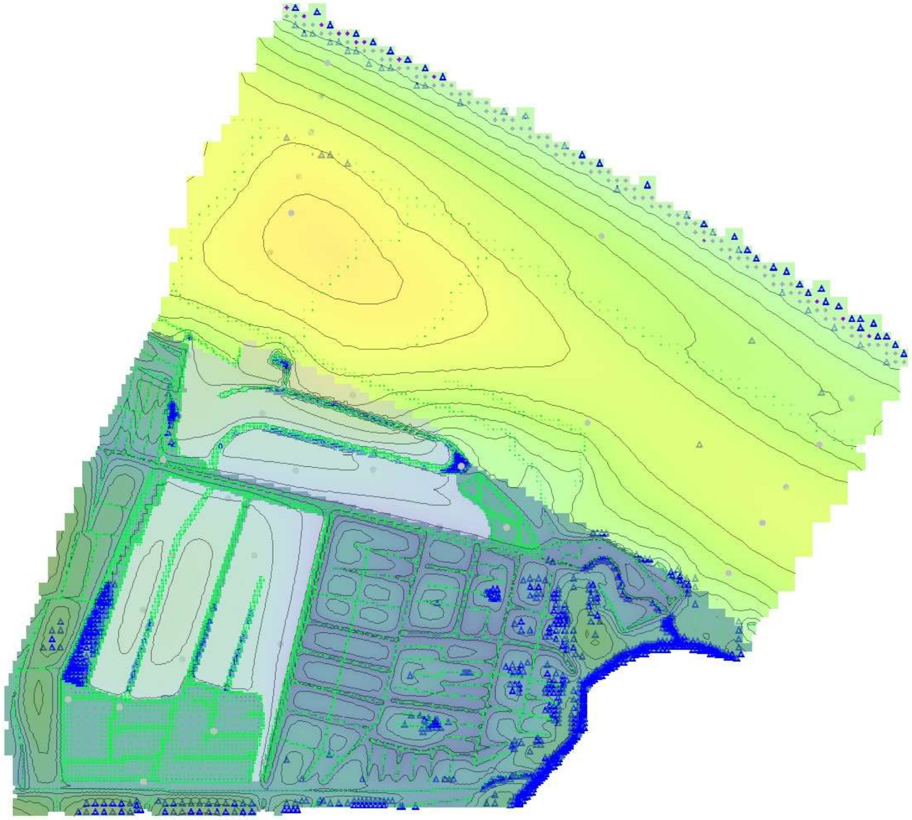
Title: Piezometric surface based on P95 Groundwater levels

| | | |
|---|---|---|
| Project: Bell Road | Drawn: ZM | Appendix No. 1<br>Size: A3<br>Sheet 03 |
| Date: 17/11/2025 | Checked: SB | |
| Proj No: 196300.000.001 | | Version: 1.0 |

## Pre Development - Simulated Groundwater Levels (Winter 2025)



## Post Development - Simulated Groundwater levels (Winter 2025)



Legend:

MODFLOW BC Symbols
- · Drain
- + River
- ▵ Flooded Cells
- • Changing Head

Materials
- ▨ Peat
- 🟨 Dune Sand
- 🟨 Alluvial Sand
- 🟦 Fluvial Sand
- ▨ Pumice
- 🟥 CompressedPeat

Groundwater Head:
3.3
3.05
2.8
2.55
2.3
2.05
1.8
1.55
1.3
1.05
0.8
0.55
0.3
0.05
-0.2

Notes:
1. Exported GW Heads from GMS

ENGEO

| | | |
|---|---|---|
| Client: Bell Road Limited Partners | | |
| Title: Comparison of Simulated Groundwater Heads (Winter 2025) | | Appendix No. 1<br>Size: A3<br>Sheet 04 |
| Project: Bell Road | Drawn: ZM | |
| Date: 14/11/2025 | Checked: SB | |
| Proj No: 196300.000.001 | | Version: 1.0 |

## Legend

Groundwater Drawdown (m):
1.6
-1.6

- ± 0.1 m Groundwater Drawdown
- ± 0.5 m Groundwater Drawdown
- Farm Drains
- External Drains
- Model Boundary
- Site Boundary

Notes:
1. ENGEO site investigations surveyed by Maven, received May 29 2025
2. Aerial Imagery from Bay of Plenty 0.2m Rural Aerial Photos (2023-2024)

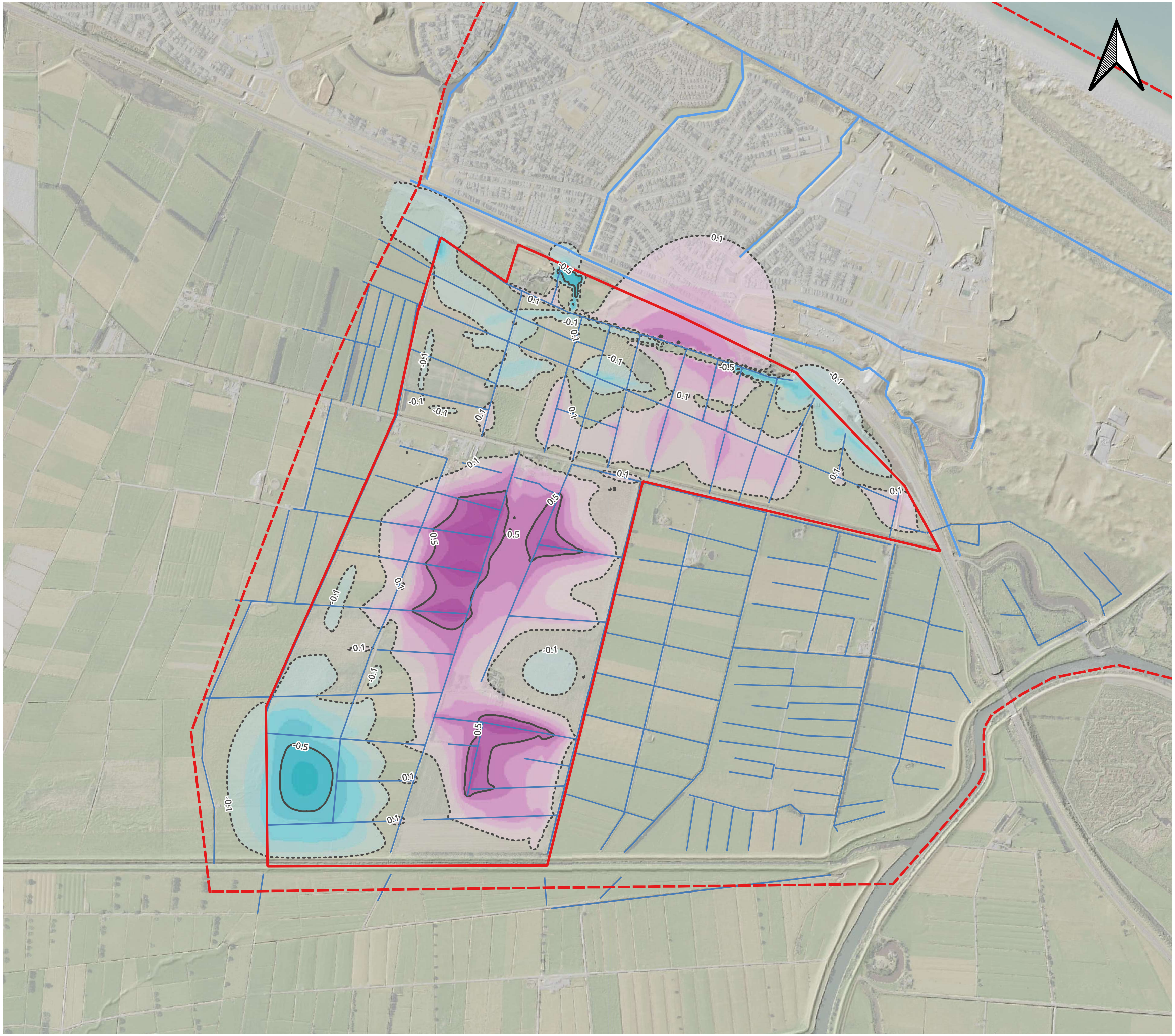
0 250 500 m

ENGEO

Projection: NZTM 2000NZGD2000 / Bay of Plenty 2000

Client: Bell Road Limited Partners

| | | |
|---|---|---|
| Title: Change in Groundwater - (Winter 2025) | | Appendix No. 1<br>Size: A3<br>Sheet 05 |
| Project: Bell Road | Drawn: ZM | |
| Date: 14/11/2025 | Checked: SB | |
| Proj No: 196300.000.001 | Scale: 15000 @ A3 | Version: 1.0 |



## Legend

Groundwater Drawdown (m):
1.6
-1.6

- ± 0.1 m Groundwater Drawdown
- ± 0.5 m Groundwater Drawdown
- Farm Drains
- External Drains
- Model Boundary
- Site Boundary

Notes:
1. ENGEO site investigations surveyed by Maven, received May 29 2025
2. Aerial Imagery from Bay of Plenty 0.2m Rural Aerial Photos (2023-2024)

0 250 500 m

ENGEO

Projection: NZTM 2000NZGD2000 / Bay of Plenty 2000

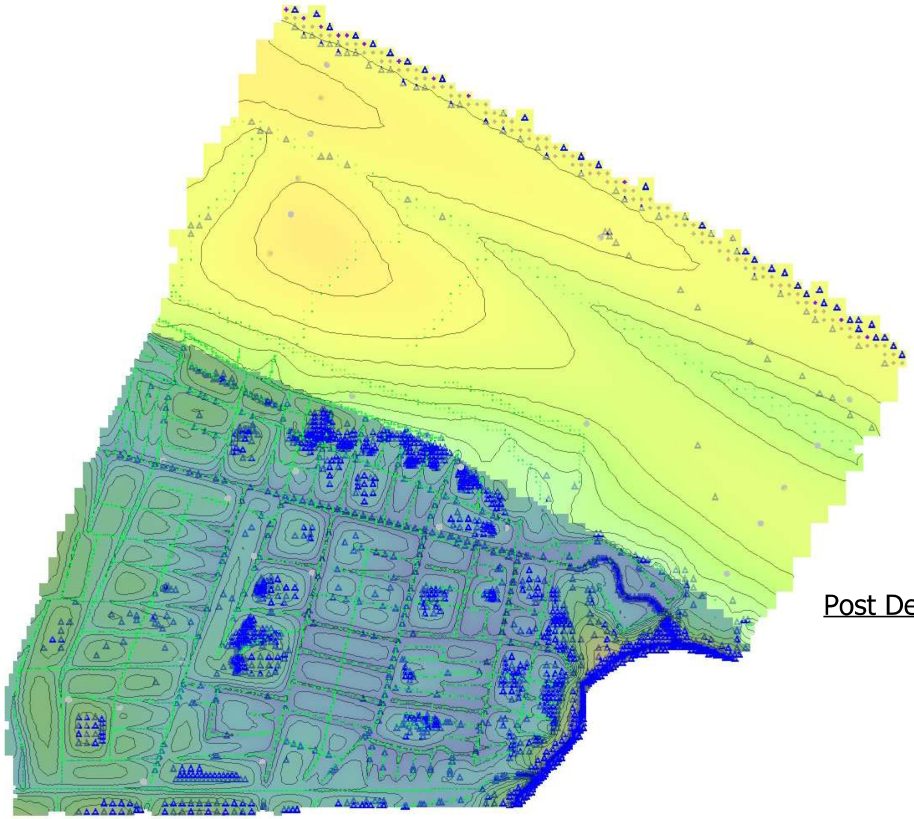
Client: Bell Road Limited Partners

Title: Change in Groundwater - (Summer 2024)

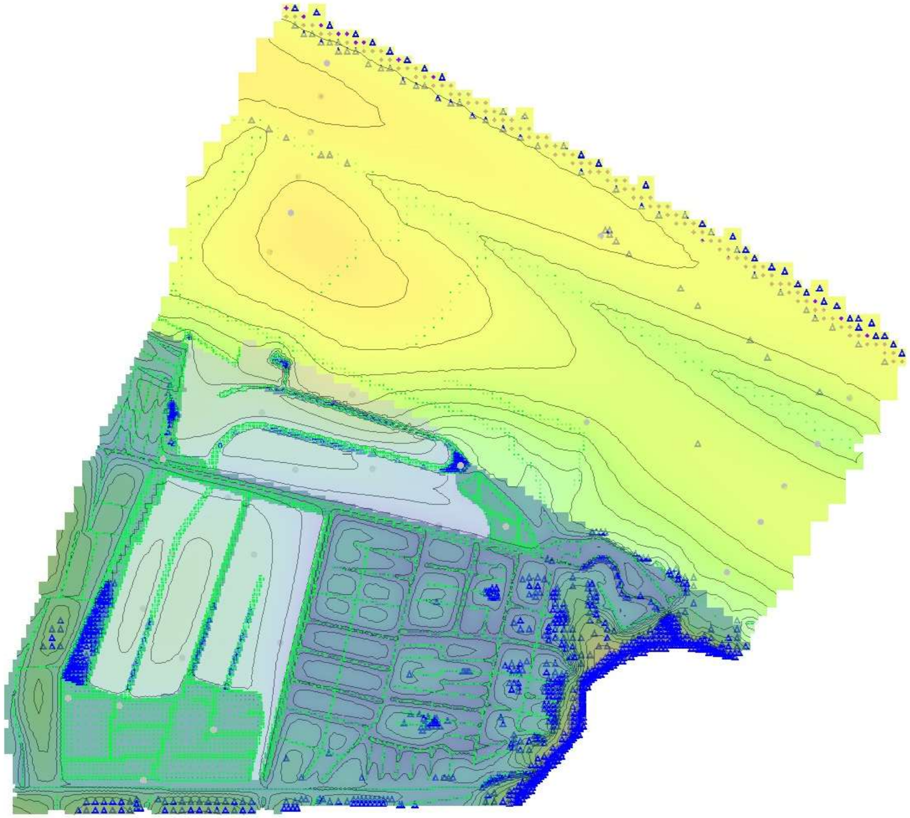
Appendix No. 1
Size: A3
Sheet 06

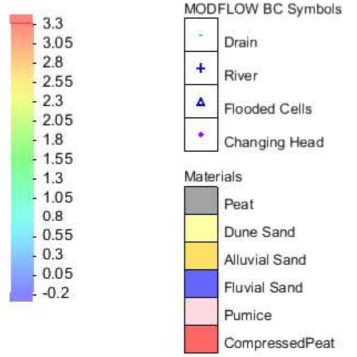
Project: Bell Road
Drawn: ZM
Date: 14/11/2025
Checked: SB
Proj No: 196300.000.001
Scale: 15000 @ A3
Version: 1.0

## Pre Development - Simulated Groundwater Levels (Winter 2025)



## Post Development - Simulated Groundwater levels (Winter 2025)





Notes:
1. Exported GW Heads from GMS
2. 1% AEP in year 2130 Sea Level Rise Scenario assumed

ENGEO

| | | |
|---|---|---|
| Client: Bell Road Limited Partners | | |
| Title: Comparison of Simulated Groundwater Heads under SSSP8.5 2130 Sea level Rise Scenario | | Appendix No. 1<br>Size: A3<br>Sheet 07 |
| Project: Bell Road | Drawn: ZM | |
| Date: 14/11/2025 | Checked: SB | |
| Proj No: 196300.000.001 | | Version: 1.0 |

## Legend

Groundwater Drawdown (m):

1.6

-1.6

- ± 0.1 m Groundwater Drawdown
- ± 0.5 m Groundwater Drawdown
- Farm Drains
- External Drains
- Model Boundary
- Site Boundary

Notes:
1. ENGEO site investigations surveyed by Maven, received May 29 2025
2. Aerial Imagery from Bay of Plenty 0.2m Rural Aerial Photos (2023-2024)

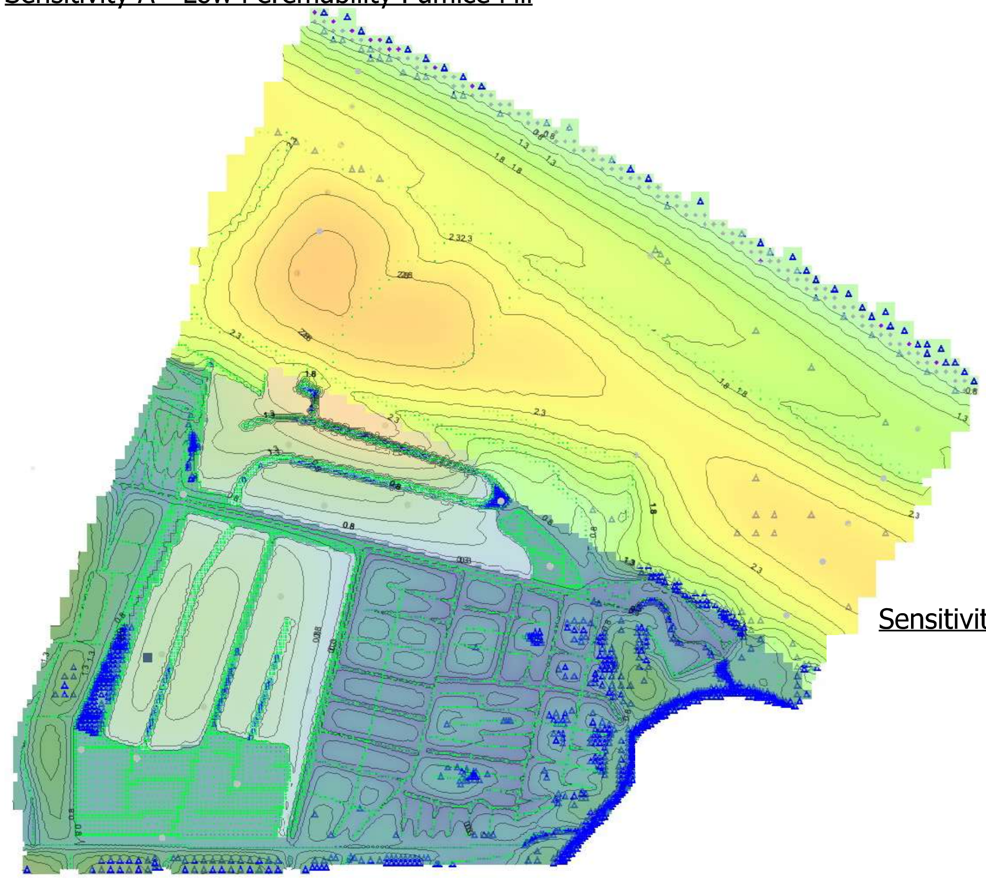
0 250 500 m

ENGEO

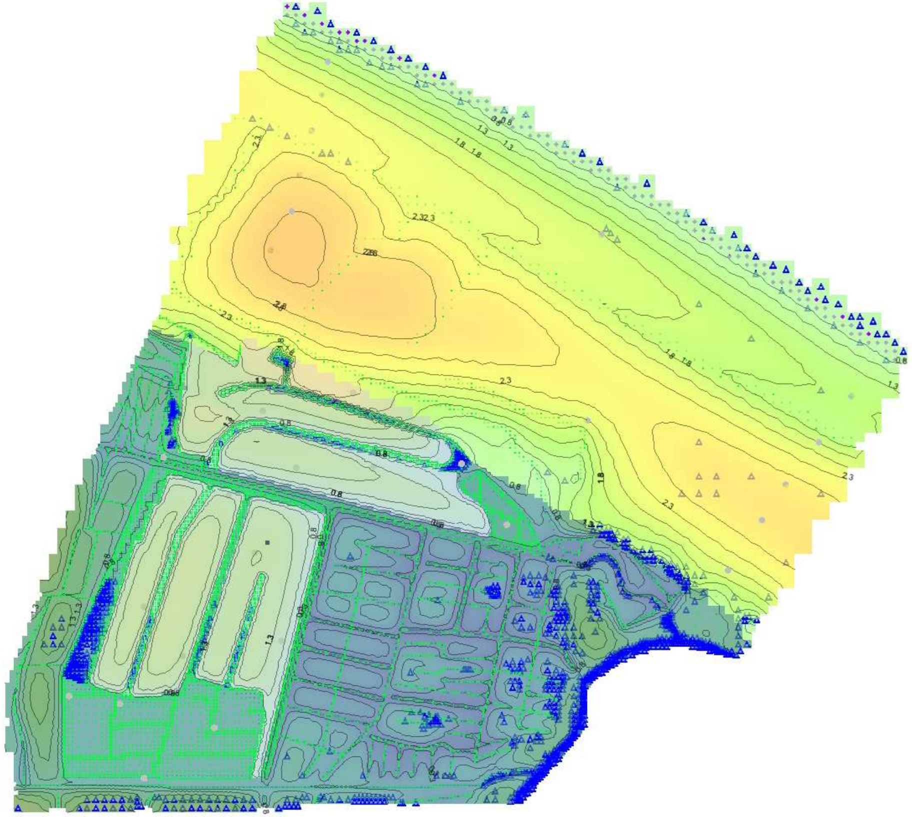
Projection: NZTM 2000NZGD2000 / Bay of Plenty 2000

| | |
|---|---|
| Client: Bell Road Limited Partners | |
| Title: Change in Groundwater - Exisitng Pre Development Conditions vs. Post Development 2130 under Sea Level Rise and 1% AEP | Appendix No. 1<br>Size: A3<br>Sheet 08 |
| Project: Bell Road | Drawn: ZM |
| Date: 17/11/2025 | Checked: SB |
| Proj No: 196300.000.001 | Scale: 15000 @ A3 |
| | Version: 1.0 |

## Sensitivity A - Low Peremability Pumice Fill



## Sensitivity B - High Groundwater Recharge



Legend:

MODFLOW BC Symbols

- · Drain
- + River
- △ Flooded Cells
- • Changing Head

Materials

- ▨ Peat
- ▨ Dune Sand
- ▨ Alluvial Sand
- ▨ Fluvial Sand
- ▨ Pumice
- ▨ CompressedPeat

Groundwater Head:

- 3.3
- 3.05
- 2.8
- 2.55
- 2.3
- 2.05
- 1.8
- 1.55
- 1.3
- 1.05
- 0.8
- 0.55
- 0.3
- 0.05
- -0.2

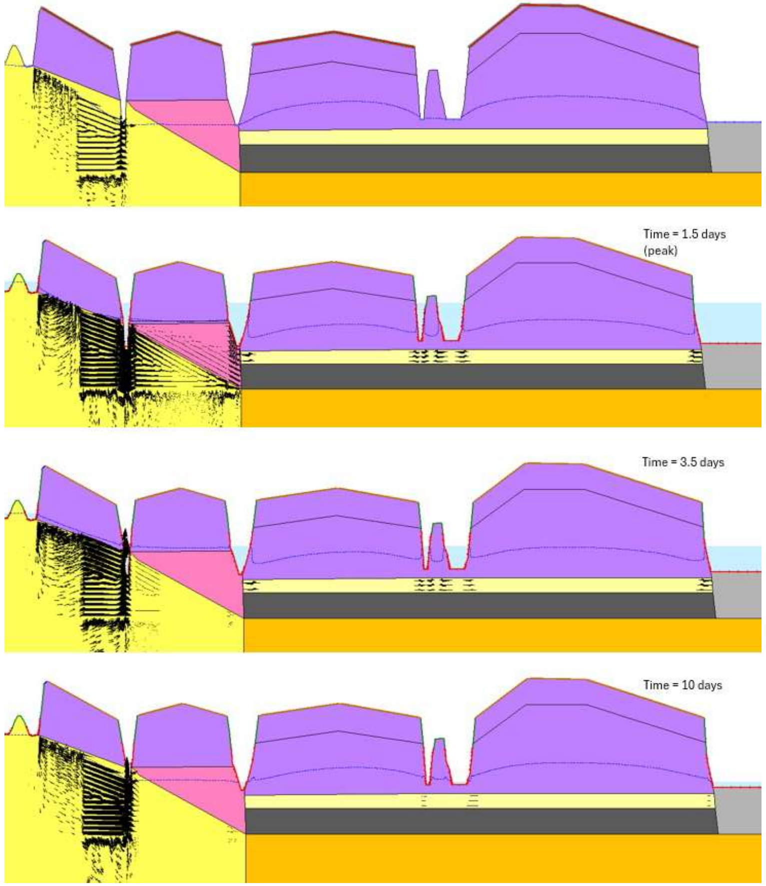
Notes:
1. Exported GW Heads from GMS

Client: Bell Road Limited Partners

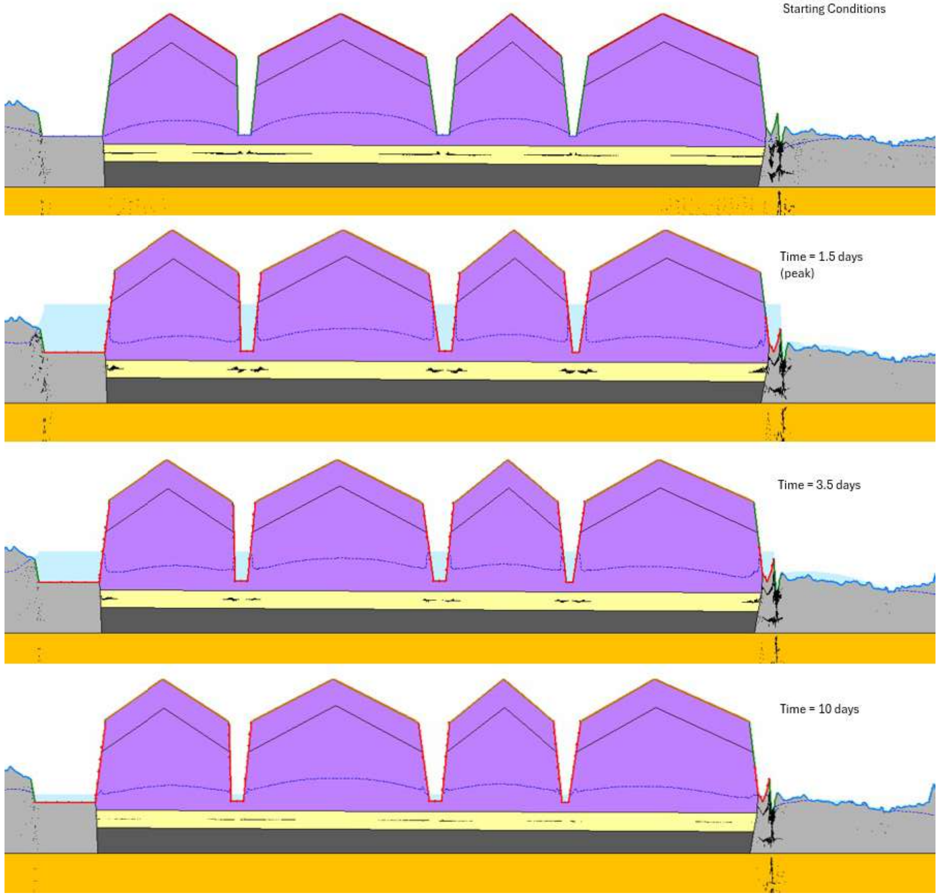
Title: Simulated Groundwater Heads Under Sensitivity Scenarios

Project: Bell Road
Drawn: ZM
Date: 17/11/2025
Checked: SB
Appendix No. 1
Size: A3
Sheet 09
Proj No: 196300.000.001
Version: 1.0

## North South Cross Section - Southern Block



## West to East Cross Section - Southern Block



Client: Bell Road Limited Partners

Title: Groundwater Response in 1% AEP rainfall event

Appendix
No. 1
Size: A3
Sheet 10

Project: Bell Road
Drawn: ZM

Date: 17/11/2025
Checked: SB

Proj No: 196300.000.001

Version: 1.0

# Appendix 2:

## Hydraulic Conductivity Testing

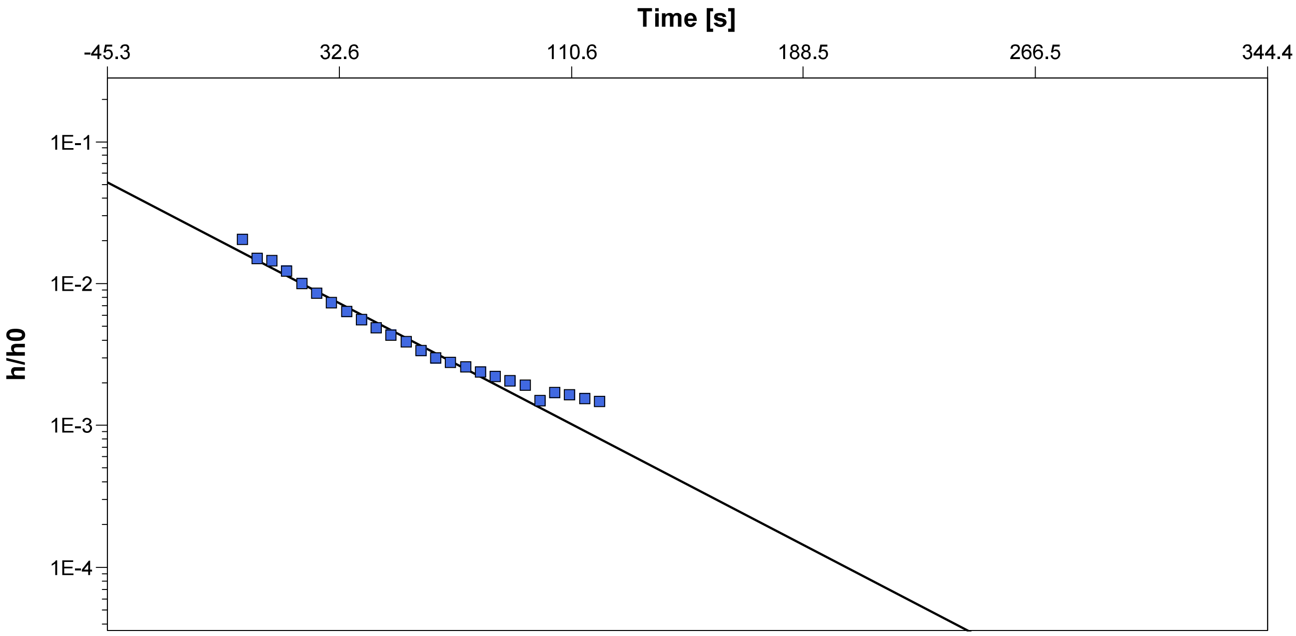
**Slug Test Analysis Report**

Project: Bell Road, Papamoa

Number: 19630.000.001

Client: Bell Road Partnership

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ17 (SAND) (FHT) | Test Well: PZ17 (SAND) |
| Test Conducted by: DRT | | Test Date: 21/08/2025 |
| Analysis Performed by: DRT | Hvorslev | Analysis Date: 29/08/2025 |
| Aquifer Thickness: 7.00 m | | |

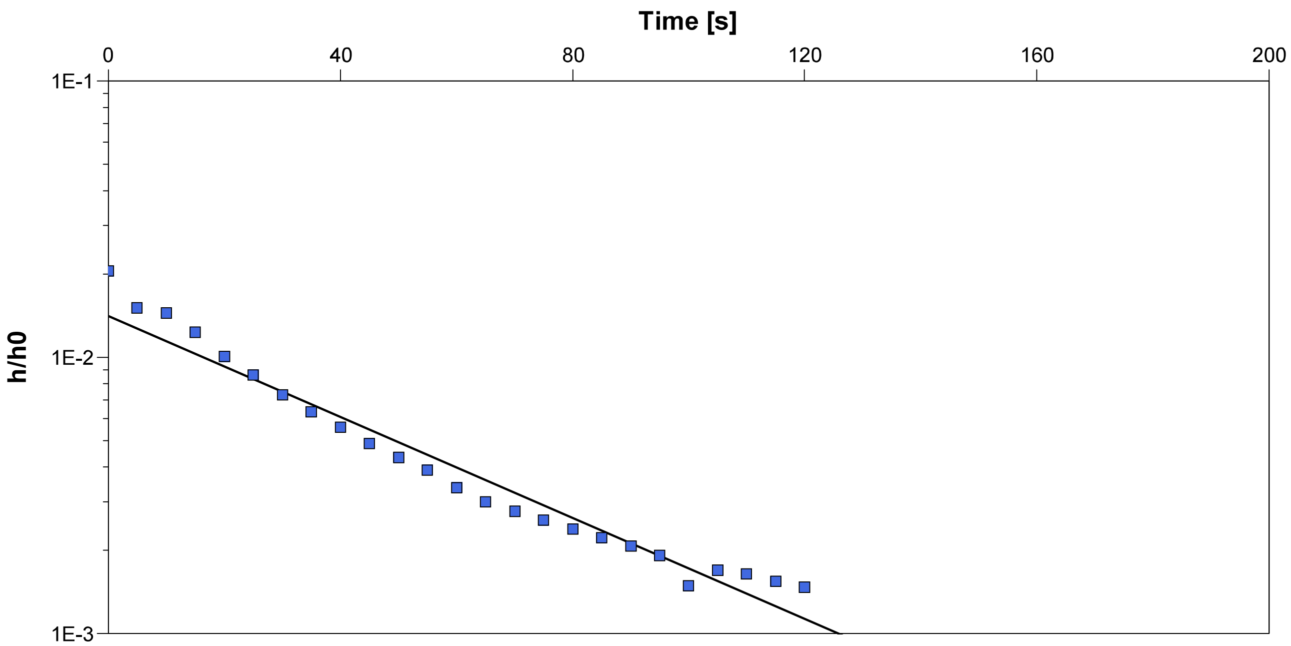


Calculation using Hvorslev

| Observation Well | Hydraulic Conductivity [m/s] |
|---|---|
| PZ17 (SAND) | $1.25 \times 10^{-5}$ |

| | |
|---|---|
| | **Slug Test Analysis Report** |
| | Project: Bell Road, Papamoa |
| | Number: 19630.000.001 |
| | Client: Bell Road Partnership |

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ17 (SAND) (FHT) | Test Well: PZ17 (SAND) |
| Test Conducted by: DRT | | Test Date: 21/08/2025 |
| Analysis Performed by: DRT | Bouwer & Rice | Analysis Date: 29/08/2025 |
| Aquifer Thickness: 7.00 m | | |



Calculation using Hvorslev

| Observation Well | Hydraulic Conductivity [m/s] | |
|---|---|---|
| PZ17 (SAND) | $1.05 \times 10^{-5}$ | |

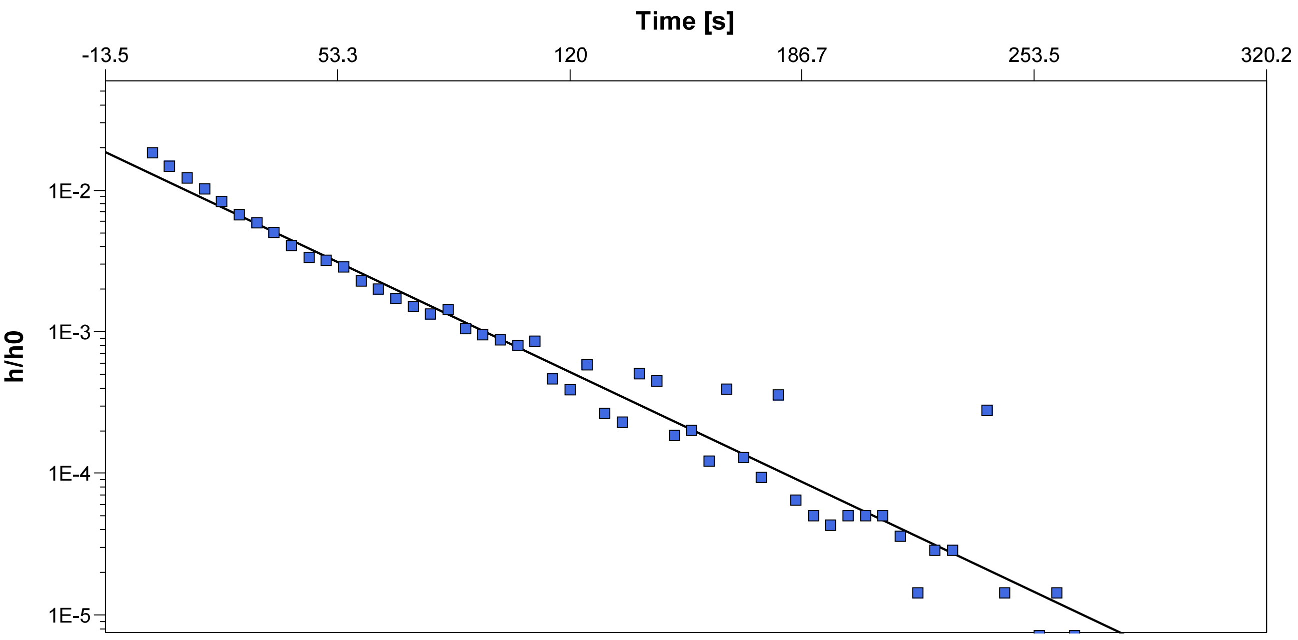
| | |
|---|---|
| | **Slug Test - Analyses Report** |
| | Project: Bell Road, Papamoa |
| | Number: 19630.000.001 |
| | Client: Bell Road Partnership |

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ17 (SAND) (FHT) | Test Well: PZ17 (SAND) |
| Test Conducted by: DRT | | Test Date: 21/08/2025 |
| Aquifer Thickness: 7.00 m | | |

| | Analysis Name | Analysis Performed b | Analysis Date | Method name | Well | T [m²/s] | K [m/s] | S |
|---|---|---|---|---|---|---|---|---|
| 1 | Hvorslev | DRT | 29/08/2025 | Hvorslev | PZ17 (SAND) | | $1.25 \times 10^{-5}$ | |
| 2 | Bouwer & Rice | DRT | 29/08/2025 | Hvorslev | PZ17 (SAND) | | $1.05 \times 10^{-5}$ | |

|  |  |
|---|---|
|  | **Slug Test Analysis Report** |
|  | Project: Bell Road, Papamoa |
|  | Number: 19630.000.001 |
|  | Client: Bell Road Partnership |

|  |  |  |
|---|---|---|
| Location: Bell Road | Slug Test: PZ17 (SAND) (RHT) | Test Well: PZ17 (SAND) |
| Test Conducted by: DRT |  | Test Date: 21/08/2025 |
| Analysis Performed by: DRT | Hvorslev | Analysis Date: 29/08/2025 |
| Aquifer Thickness: 7.00 m |  |  |

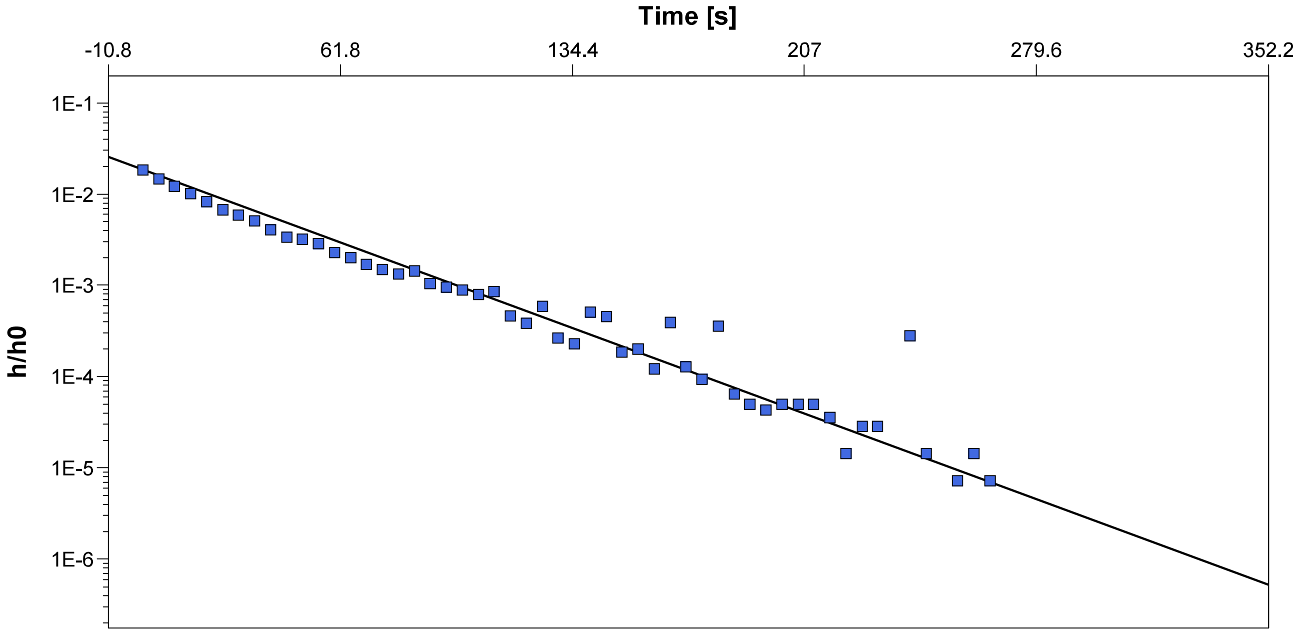


Calculation using Hvorslev

| Observation Well | Hydraulic Conductivity [m/s] |  |
|---|---|---|
| PZ17 (SAND) | $1.33 \times 10^{-5}$ |  |

| | |
|---|---|
| | **Slug Test Analysis Report** |
| | Project: Bell Road, Papamoa |
| | Number: 19630.000.001 |
| | Client: Bell Road Partnership |

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ17 (SAND) (RHT) | Test Well: PZ17 (SAND) |
| Test Conducted by: DRT | | Test Date: 21/08/2025 |
| Analysis Performed by: DRT | Bouwer & Rice | Analysis Date: 29/08/2025 |
| Aquifer Thickness: 7.00 m | | |



Calculation using Bouwer & Rice

| Observation Well | Hydraulic Conductivity [m/s] | |
|---|---|---|
| PZ17 (SAND) | $1.00 \times 10^{-5}$ | |

**Slug Test - Analyses Report**

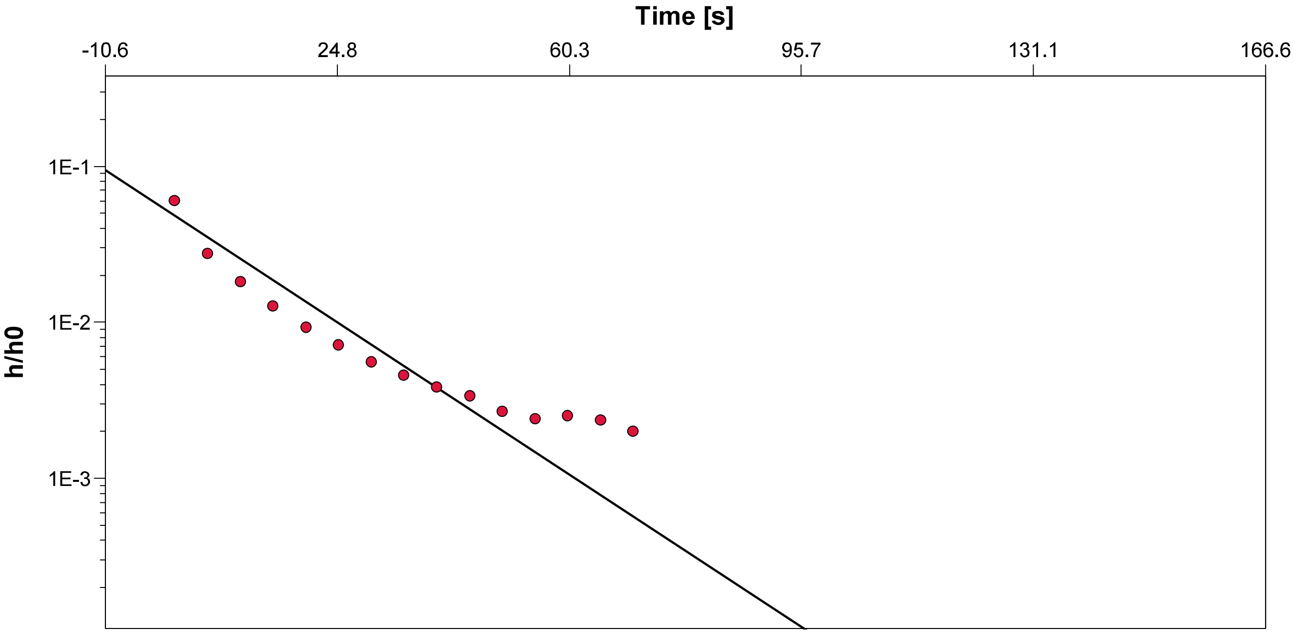
Project: Bell Road, Papamoa
Number: 19630.000.001
Client: Bell Road Partnership

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ17 (SAND) (RHT) | Test Well: PZ17 (SAND) |
| Test Conducted by: DRT | | Test Date: 21/08/2025 |
| Aquifer Thickness: 7.00 m | | |

| | Analysis Name | Analysis Performed b | Analysis Date | Method name | Well | T [m²/s] | K [m/s] | S |
|---|---|---|---|---|---|---|---|---|
| 1 | Hvorslev | DRT | 29/08/2025 | Hvorslev | PZ17 (SAND) | | $1.33 \times 10^{-5}$ | |
| 2 | Bouwer & Rice | DRT | 29/08/2025 | Bouwer & Rice | PZ17 (SAND) | | $1.00 \times 10^{-5}$ | |

| | |
|---|---|
| | **Slug Test Analysis Report** |
| | Project: Bell Road, Papamoa |
| | Number: 19630.000.001 |
| | Client: Bell Road Partnership |

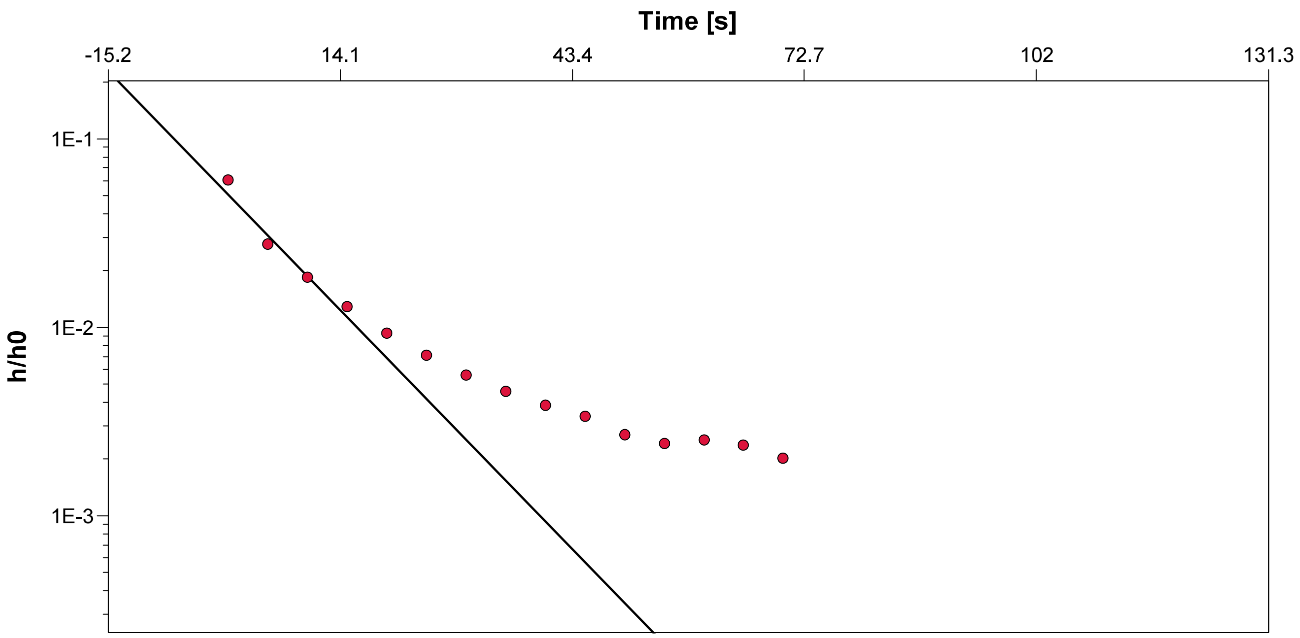
| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ18 (SAND) (FHT) | Test Well: PZ18 (SAND) |
| Test Conducted by: DRT | | Test Date: 21/08/2025 |
| Analysis Performed by: DRT | Hvorslev | Analysis Date: 29/08/2025 |
| Aquifer Thickness: 7.00 m | | |



Calculation using Hvorslev

| Observation Well | Hydraulic Conductivity [m/s] | |
|---|---|---|
| PZ18 (SAND) | $3.50 \times 10^{-5}$ | |

|  |  |  |
|---|---|---|
|  | **Slug Test Analysis Report** |  |
|  | Project: Bell Road, Papamoa |  |
|  | Number: 19630.000.001 |  |
|  | Client: Bell Road Partnership |  |
| Location: Bell Road | Slug Test: PZ18 (SAND) (FHT) | Test Well: PZ18 (SAND) |
| Test Conducted by: DRT |  | Test Date: 21/08/2025 |
| Analysis Performed by: DRT | Bouwer & Rice | Analysis Date: 29/08/2025 |
| Aquifer Thickness: 7.00 m |  |  |



Calculation using Bouwer & Rice

| Observation Well | Hydraulic Conductivity [m/s] |
|---|---|
| PZ18 (SAND) | $3.70 \times 10^{-5}$ |

**Slug Test - Analyses Report**

Project: Bell Road, Papamoa

Number: 19630.000.001

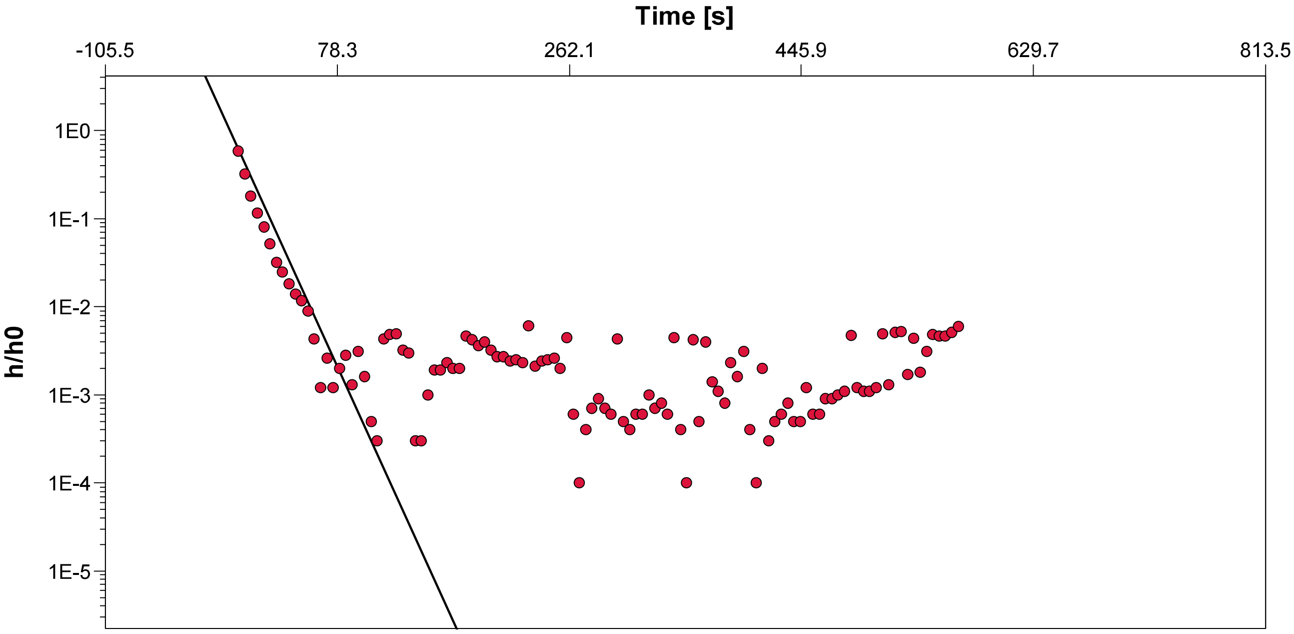
Client: Bell Road Partnership

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ18 (SAND) (FHT) | Test Well: PZ18 (SAND) |
| Test Conducted by: DRT | | Test Date: 21/08/2025 |
| Aquifer Thickness: 7.00 m | | |

| | Analysis Name | Analysis Performed b | Analysis Date | Method name | Well | T [m²/s] | K [m/s] | S |
|---|---|---|---|---|---|---|---|---|
| 1 | Hvorslev | DRT | 29/08/2025 | Hvorslev | PZ18 (SAND) | | $3.50 \times 10^{-5}$ | |
| 2 | Bouwer & Rice | DRT | 29/08/2025 | Bouwer & Rice | PZ18 (SAND) | | $3.70 \times 10^{-5}$ | |

| | |
|---|---|
| | **Slug Test Analysis Report** |
| | Project: Bell Road, Papamoa |
| | Number: 19630.000.001 |
| | Client: Bell Road Partnership |

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ18 (SAND) (RHT) | Test Well: PZ18 (SAND) |
| Test Conducted by: DRT | | Test Date: 21/08/2025 |
| Analysis Performed by: DRT | Hvorslev | Analysis Date: 29/08/2025 |
| Aquifer Thickness: 7.00 m | | |

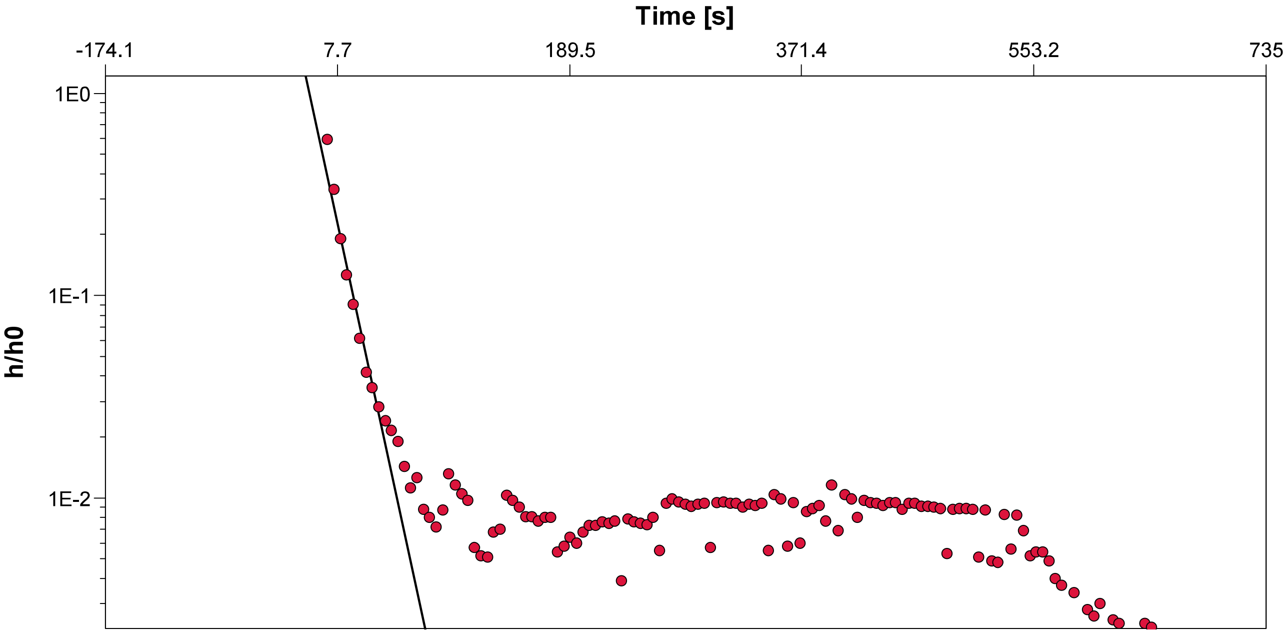


Calculation using Hvorslev

| Observation Well | Hydraulic Conductivity [m/s] | |
|---|---|---|
| PZ18 (SAND) | $4.00 \times 10^{-5}$ | |

| | |
|---|---|
| | **Slug Test Analysis Report** |
| | Project: Bell Road, Papamoa |
| | Number: 19630.000.001 |
| | Client: Bell Road Partnership |

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ18 (SAND) (RHT) | Test Well: PZ18 (SAND) |
| Test Conducted by: DRT | | Test Date: 21/08/2025 |
| Analysis Performed by: DRT | Bouwer & Rice | Analysis Date: 29/08/2025 |
| Aquifer Thickness: 7.00 m | | |



Calculation using Bouwer & Rice

| Observation Well | Hydraulic Conductivity [m/s] | |
|---|---|---|
| PZ18 (SAND) | $2.49 \times 10^{-5}$ | |

**Slug Test - Analyses Report**

Project: Bell Road, Papamoa

Number: 19630.000.001

Client: Bell Road Partnership

Location: Bell Road

Slug Test: PZ18 (SAND) (RHT)

Test Well: PZ18 (SAND)

Test Conducted by: DRT

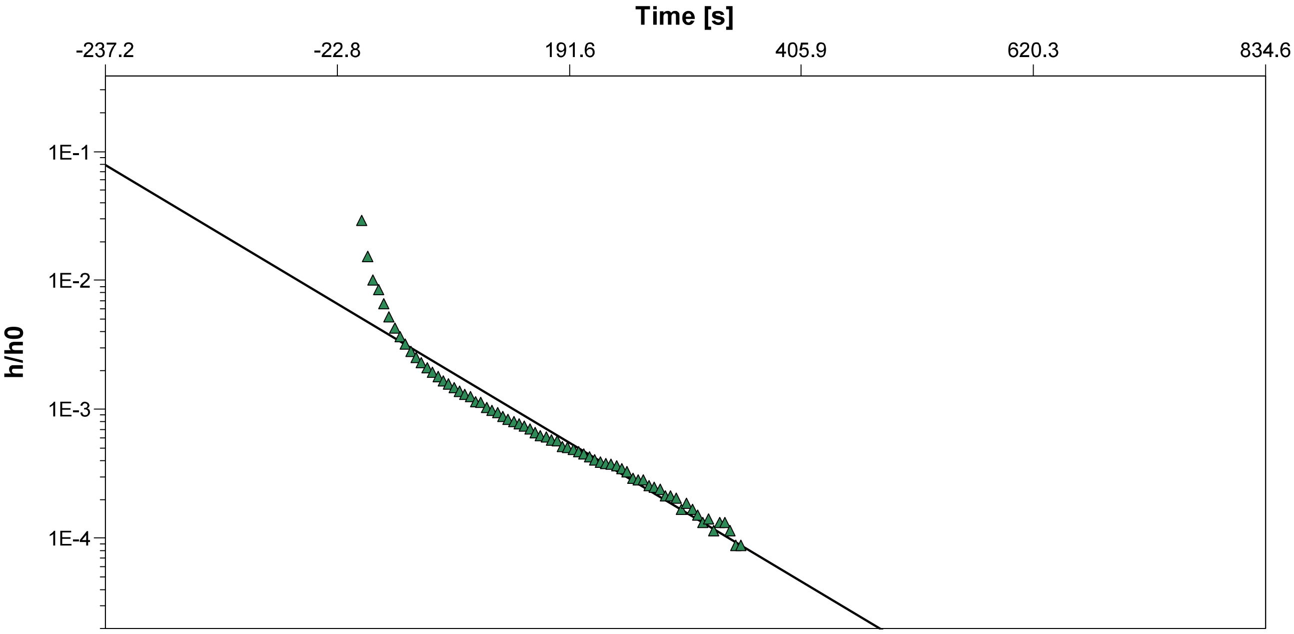
Test Date: 21/08/2025

Aquifer Thickness: 7.00 m

| | Analysis Name | Analysis Performed b | Analysis Date | Method name | Well | T [m$^2$/s] | K [m/s] | S |
|---|---|---|---|---|---|---|---|---|
| 1 | Hvorslev | DRT | 29/08/2025 | Hvorslev | PZ18 (SAND) | | $4.00 \times 10^{-5}$ | |
| 2 | Bouwer & Rice | DRT | 29/08/2025 | Bouwer & Rice | PZ18 (SAND) | | $2.49 \times 10^{-5}$ | |

| | |
|---|---|
| | **Slug Test Analysis Report** |
| | Project: Bell Road, Papamoa |
| | Number: 19630.000.001 |
| | Client: Bell Road Partnership |

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ19 (PEAT) (FHT) | Test Well: P19 (PEAT) |
| Test Conducted by: DRT | | Test Date: 21/08/2025 |
| Analysis Performed by: DRT | Hvorslev | Analysis Date: 29/08/2025 |
| Aquifer Thickness: 3.20 m | | |



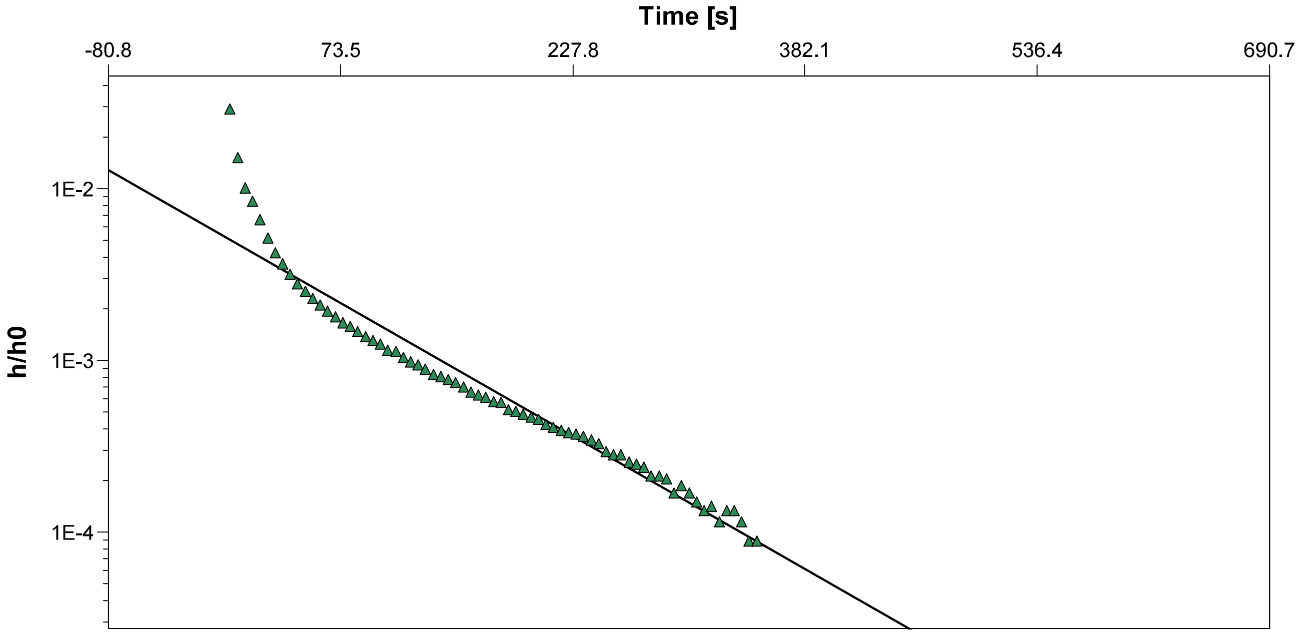
Calculation using Hvorslev

| Observation Well | Hydraulic Conductivity [m/s] | |
|---|---|---|
| P19 (PEAT) | $9.97 \times 10^{-6}$ | |

|  |  |
|---|---|
|  | **Slug Test Analysis Report** |
|  | Project: Bell Road, Papamoa |
|  | Number: 19630.000.001 |
|  | Client: Bell Road Partnership |

| Location: Bell Road | Slug Test: PZ19 (PEAT) (FHT) | Test Well: P19 (PEAT) |
|---|---|---|
| Test Conducted by: DRT |  | Test Date: 21/08/2025 |
| Analysis Performed by: DRT | Bouwer & Rice | Analysis Date: 29/08/2025 |

Aquifer Thickness: 3.20 m



Calculation using Bouwer & Rice

| Observation Well | Hydraulic Conductivity [m/s] |  |
|---|---|---|
| P19 (PEAT) | $7.65 \times 10^{-6}$ |  |

**Slug Test - Analyses Report**

Project: Bell Road, Papamoa

Number: 19630.000.001

Client: Bell Road Partnership

Location: Bell Road

Slug Test: PZ19 (PEAT) (FHT)

Test Well: P19 (PEAT)

Test Conducted by: DRT

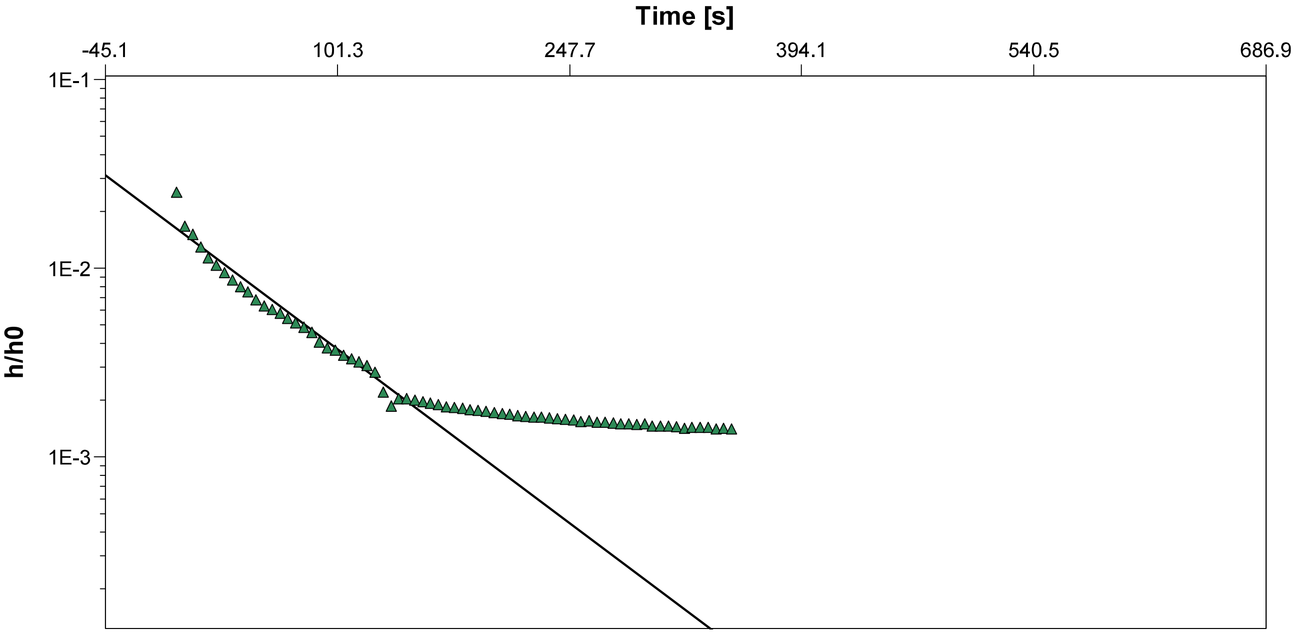
Test Date: 21/08/2025

Aquifer Thickness: 3.20 m

| | Analysis Name | Analysis Performed b | Analysis Date | Method name | Well | T [m²/s] | K [m/s] | S |
|---|---|---|---|---|---|---|---|---|
| 1 | Hvorslev | DRT | 29/08/2025 | Hvorslev | P19 (PEAT) | | $9.97 \times 10^{-6}$ | |
| 2 | Bouwer & Rice | DRT | 29/08/2025 | Bouwer & Rice | P19 (PEAT) | | $7.65 \times 10^{-6}$ | |

| | |
|---|---|
| | **Slug Test Analysis Report** |
| | Project: Bell Road, Papamoa |
| | Number: 19630.000.001 |
| | Client: Bell Road Partnership |

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ19 (PEAT (RHT) | Test Well: P19 (PEAT) |
| Test Conducted by: DRT | | Test Date: 21/08/2025 |
| Analysis Performed by: DRT | Hvorslev | Analysis Date: 29/08/2025 |
| Aquifer Thickness: 3.20 m | | |

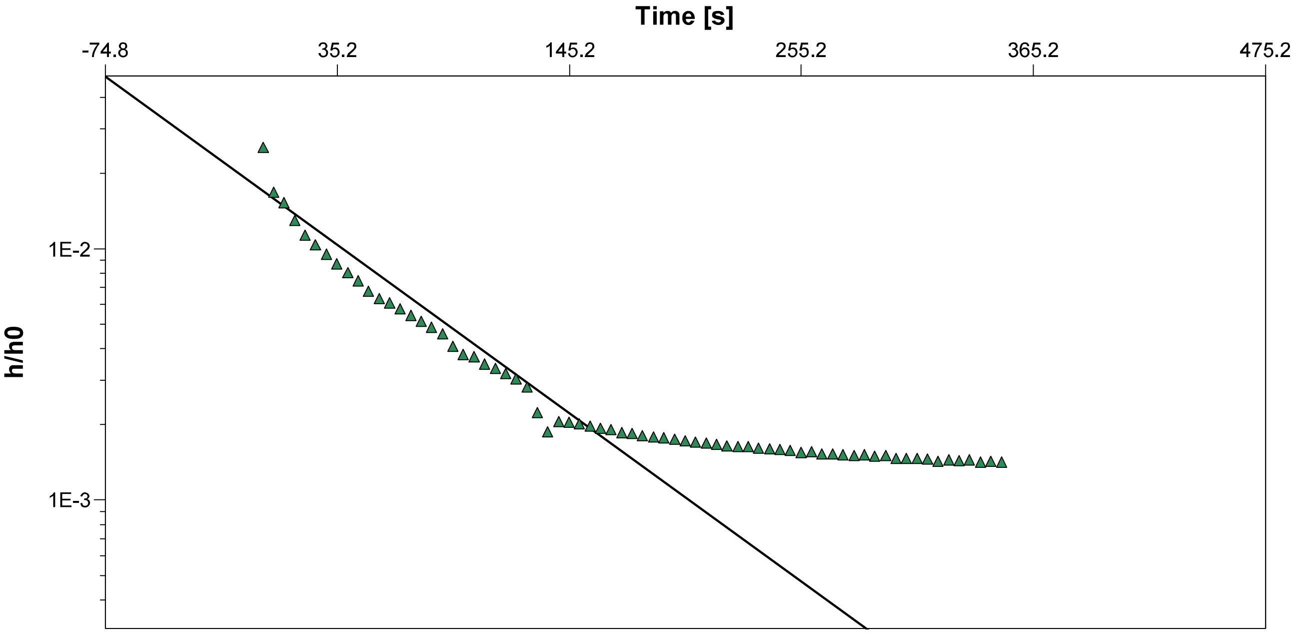


Calculation using Hvorslev

| Observation Well | Hydraulic Conductivity [m/s] | |
|---|---|---|
| P19 (PEAT) | $1.25 \times 10^{-5}$ | |

| | |
|---|---|
| | **Slug Test Analysis Report** |
| | Project: Bell Road, Papamoa |
| | Number: 19630.000.001 |
| | Client: Bell Road Partnership |

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ19 (PEAT (RHT) | Test Well: P19 (PEAT) |
| Test Conducted by: DRT | | Test Date: 21/08/2025 |
| Analysis Performed by: DRT | Bouwer & Rice | Analysis Date: 29/08/2025 |
| Aquifer Thickness: 3.20 m | | |



Calculation using Bouwer & Rice

| Observation Well | Hydraulic Conductivity [m/s] | |
|---|---|---|
| P19 (PEAT) | $9.27 \times 10^{-6}$ | |

**Slug Test - Analyses Report**

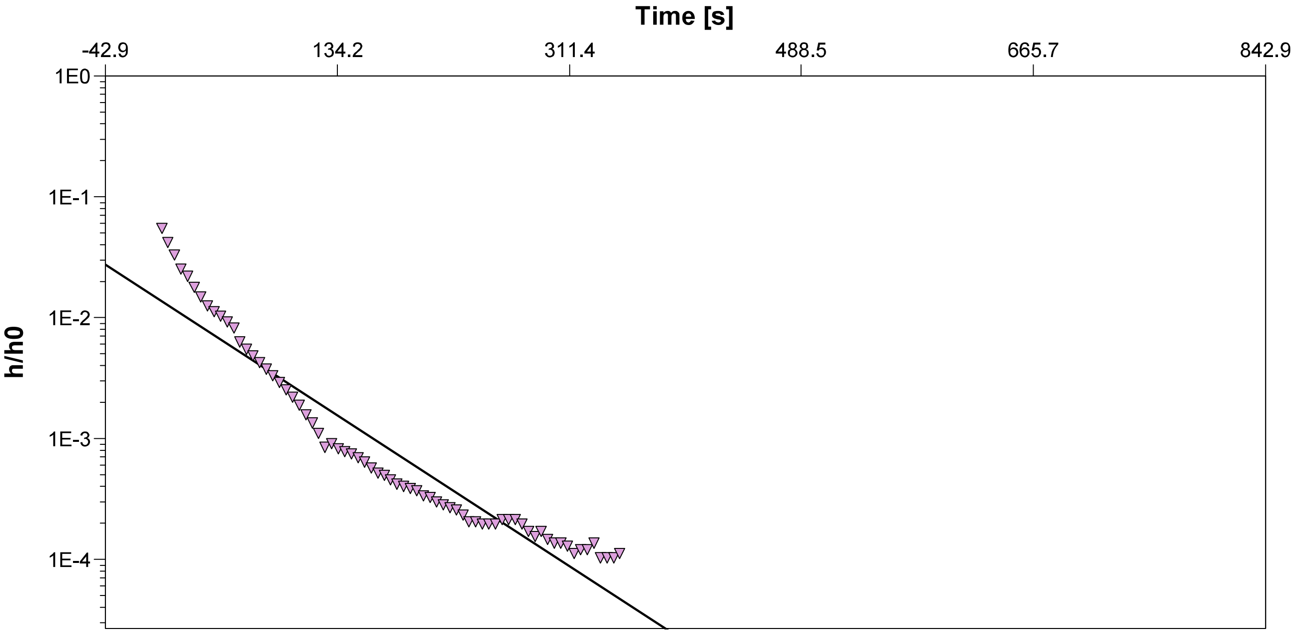
Project: Bell Road, Papamoa
Number: 19630.000.001
Client: Bell Road Partnership

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ19 (PEAT (RHT) | Test Well: P19 (PEAT) |
| Test Conducted by: DRT | | Test Date: 21/08/2025 |
| Aquifer Thickness: 3.20 m | | |

| | Analysis Name | Analysis Performed b | Analysis Date | Method name | Well | T [m²/s] | K [m/s] | S |
|---|---|---|---|---|---|---|---|---|
| 1 | Hvorslev | DRT | 29/08/2025 | Hvorslev | P19 (PEAT) | | $1.25 \times 10^{-5}$ | |
| 2 | Bouwer & Rice | DRT | 29/08/2025 | Bouwer & Rice | P19 (PEAT) | | $9.27 \times 10^{-6}$ | |

|  |  |  |
|---|---|---|
|  | **Slug Test Analysis Report** |  |
|  | Project: Bell Road, Papamoa |  |
|  | Number: 19630.000.001 |  |
|  | Client: Bell Road Partnership |  |

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ20 (PEAT) (FHT) | Test Well: PZ20 (PEAT) |
| Test Conducted by: DRT | | Test Date: 21/08/2025 |
| Analysis Performed by: DRT | Hvorslev | Analysis Date: 29/08/2025 |
| Aquifer Thickness: 1.60 m | | |

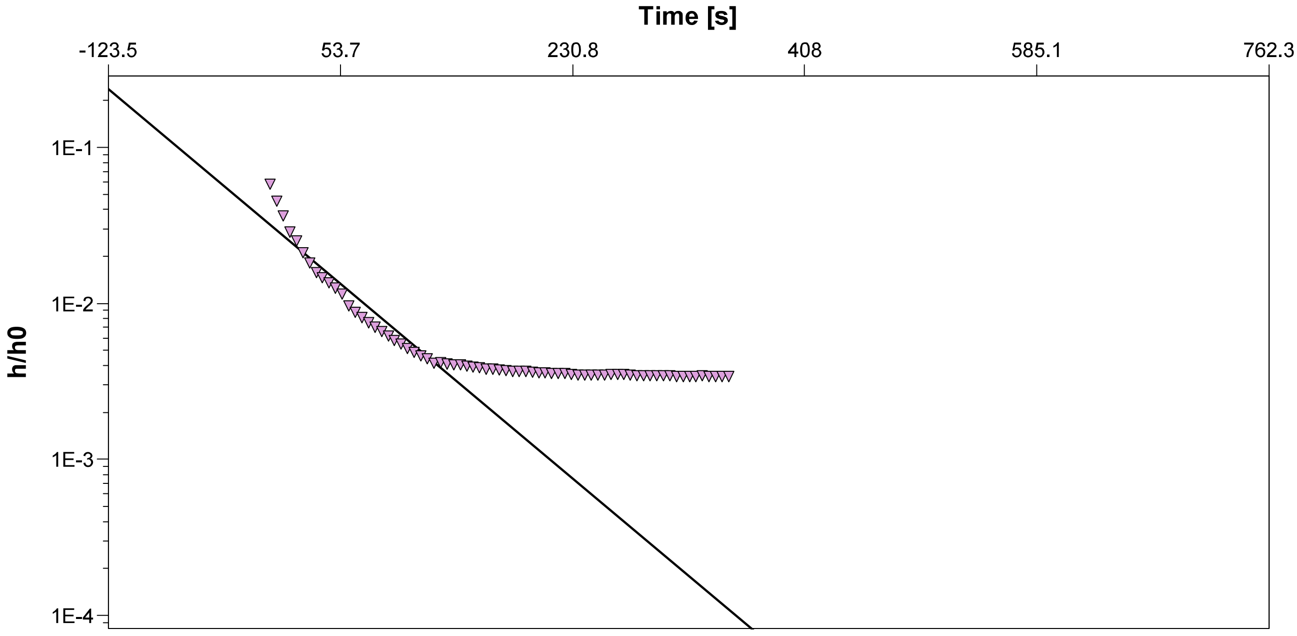


Calculation using Bouwer & Rice

| Observation Well | Hydraulic Conductivity [m/s] |  |
|---|---|---|
| PZ20 (PEAT) | $1.20 \times 10^{-5}$ |  |

| | |
|---|---|
| | **Slug Test Analysis Report** |
| | Project: Bell Road, Papamoa |
| | Number: 19630.000.001 |
| | Client: Bell Road Partnership |

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ20 (PEAT) (FHT) | Test Well: PZ20 (PEAT) |
| Test Conducted by: DRT | | Test Date: 21/08/2025 |
| Analysis Performed by: DRT | Bouwer & Rice | Analysis Date: 29/08/2025 |
| Aquifer Thickness: 1.60 m | | |



Calculation using Bouwer & Rice

| Observation Well | Hydraulic Conductivity [m/s] | |
|---|---|---|
| PZ20 (PEAT) | $1.20 \times 10^{-5}$ | |

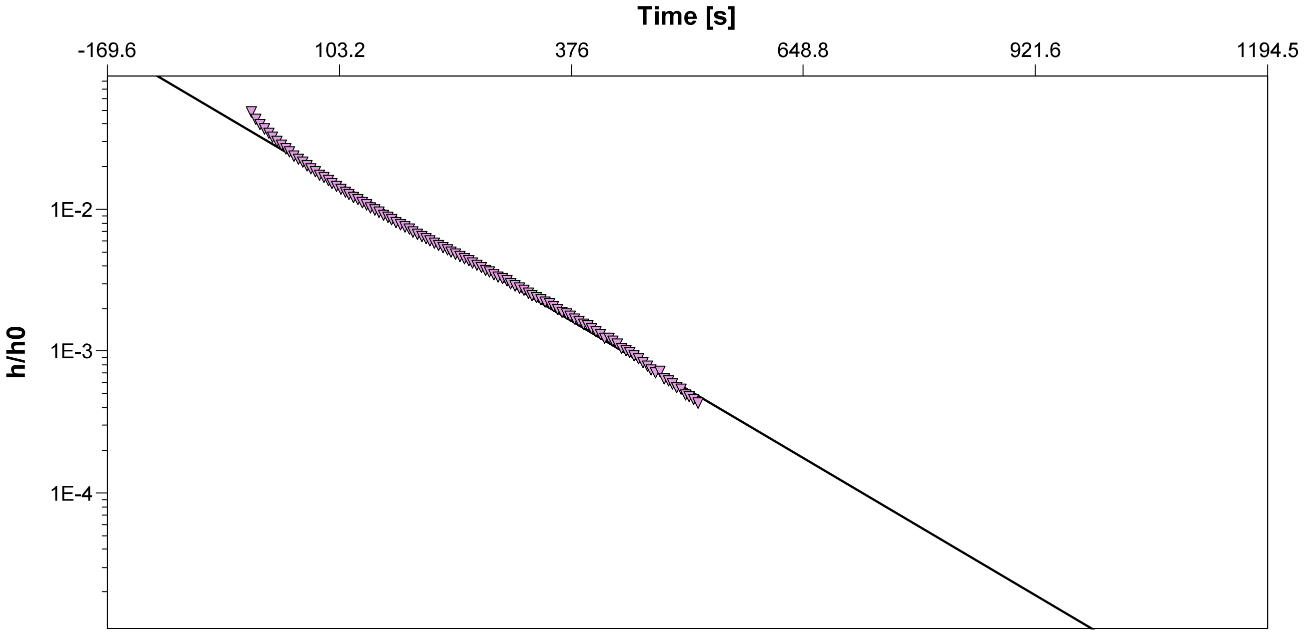
| | |
|---|---|
| | **Slug Test - Analyses Report** |
| | Project: Bell Road, Papamoa |
| | Number: 19630.000.001 |
| | Client: Bell Road Partnership |

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ20 (PEAT) (FHT) | Test Well: PZ20 (PEAT) |
| Test Conducted by: DRT | | Test Date: 21/08/2025 |
| Aquifer Thickness: 1.60 m | | |

| | Analysis Name | Analysis Performed b | Analysis Date | Method name | Well | T [m²/s] | K [m/s] | S |
|---|---|---|---|---|---|---|---|---|
| 1 | Hvorslev | DRT | 29/08/2025 | Bouwer & Rice | PZ20 (PEAT) | | $1.20 \times 10^{-5}$ | |
| 2 | Bouwer & Rice | DRT | 29/08/2025 | Bouwer & Rice | PZ20 (PEAT) | | $1.20 \times 10^{-5}$ | |

| | |
|---|---|
| | **Slug Test Analysis Report** |
| | Project: Bell Road, Papamoa |
| | Number: 19630.000.001 |
| | Client: Bell Road Partnership |

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ20 (PEAT) (RHT) | Test Well: PZ20 (PEAT) |
| Test Conducted by: DRT | | Test Date: 21/08/2025 |
| Analysis Performed by: DRT | Hvorslev | Analysis Date: 29/08/2025 |
| Aquifer Thickness: 1.60 m | | |

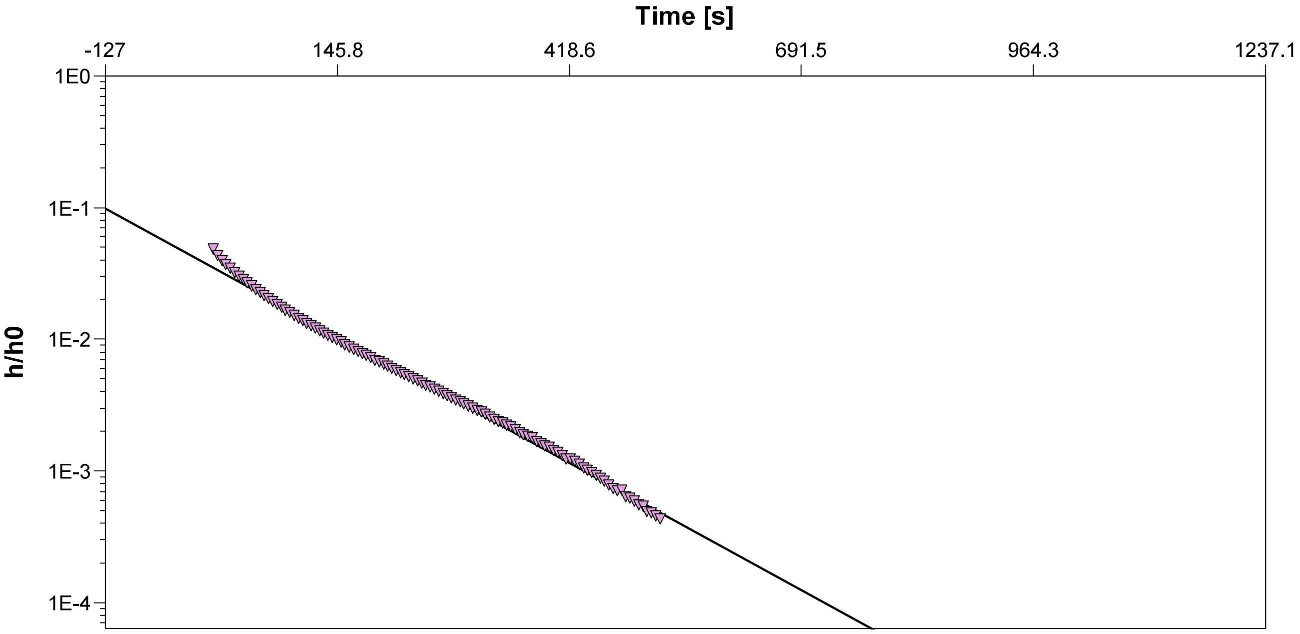


Calculation using Hvorslev

| Observation Well | Hydraulic Conductivity [m/s] | |
|---|---|---|
| PZ20 (PEAT) | $7.89 \times 10^{-6}$ | |

| | |
|---|---|
| | **Slug Test Analysis Report** |
| | Project: Bell Road, Papamoa |
| | Number: 19630.000.001 |
| | Client: Bell Road Partnership |

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ20 (PEAT) (RHT) | Test Well: PZ20 (PEAT) |
| Test Conducted by: DRT | | Test Date: 21/08/2025 |
| Analysis Performed by: DRT | Bouwer & Rice | Analysis Date: 29/08/2025 |
| Aquifer Thickness: 1.60 m | | |



Calculation using Bouwer & Rice

| Observation Well | Hydraulic Conductivity [m/s] | |
|---|---|---|
| PZ20 (PEAT) | $6.04 \times 10^{-6}$ | |

**Slug Test - Analyses Report**

Project: Bell Road, Papamoa

Number: 19630.000.001

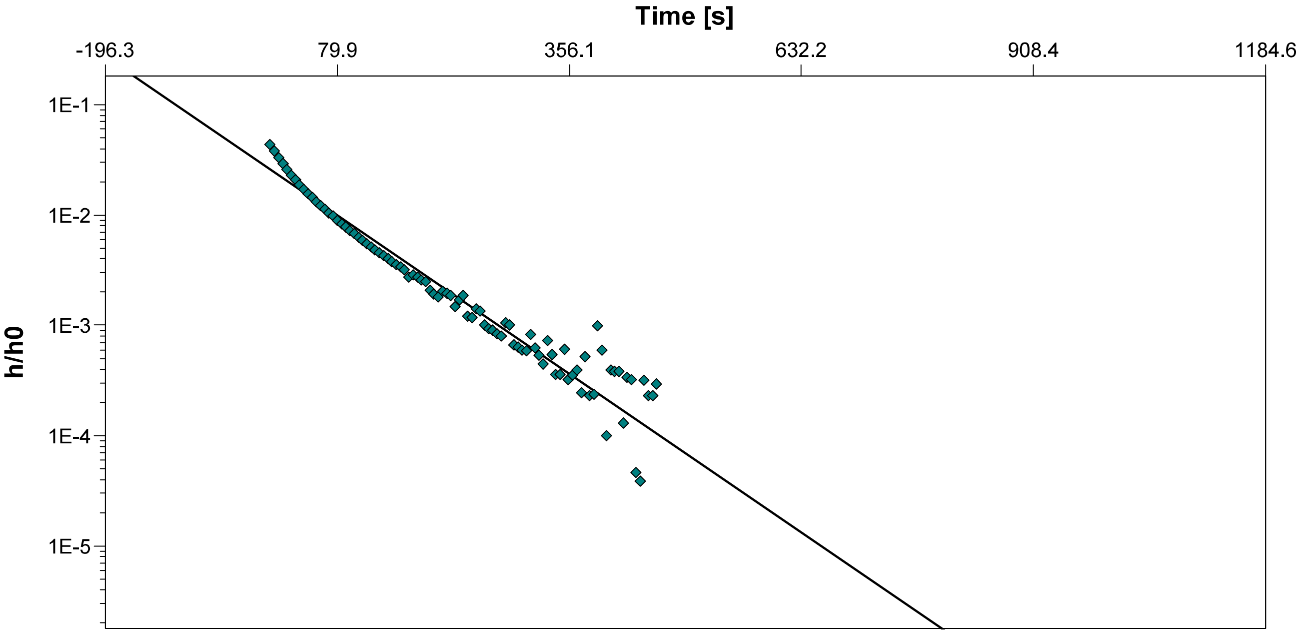
Client: Bell Road Partnership

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ20 (PEAT) (RHT) | Test Well: PZ20 (PEAT) |
| Test Conducted by: DRT | | Test Date: 21/08/2025 |
| Aquifer Thickness: 1.60 m | | |

| | Analysis Name | Analysis Performed b | Analysis Date | Method name | Well | T [m²/s] | K [m/s] | S |
|---|---|---|---|---|---|---|---|---|
| 1 | Hvorslev | DRT | 29/08/2025 | Hvorslev | PZ20 (PEAT) | | $7.89 \times 10^{-6}$ | |
| 2 | Bouwer & Rice | DRT | 29/08/2025 | Bouwer & Rice | PZ20 (PEAT) | | $6.04 \times 10^{-6}$ | |

| | |
|---|---|
| | **Slug Test Analysis Report** |
| | Project: Bell Road, Papamoa |
| | Number: 19630.000.001 |
| | Client: Bell Road Partnership |

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ21 (SAND) (FHT) | Test Well: PZ21 (SAND) |
| Test Conducted by: DRT | | Test Date: 21/08/2025 |
| Analysis Performed by: DRT | Hvorslev | Analysis Date: 29/08/2025 |
| Aquifer Thickness: 7.00 m | | |

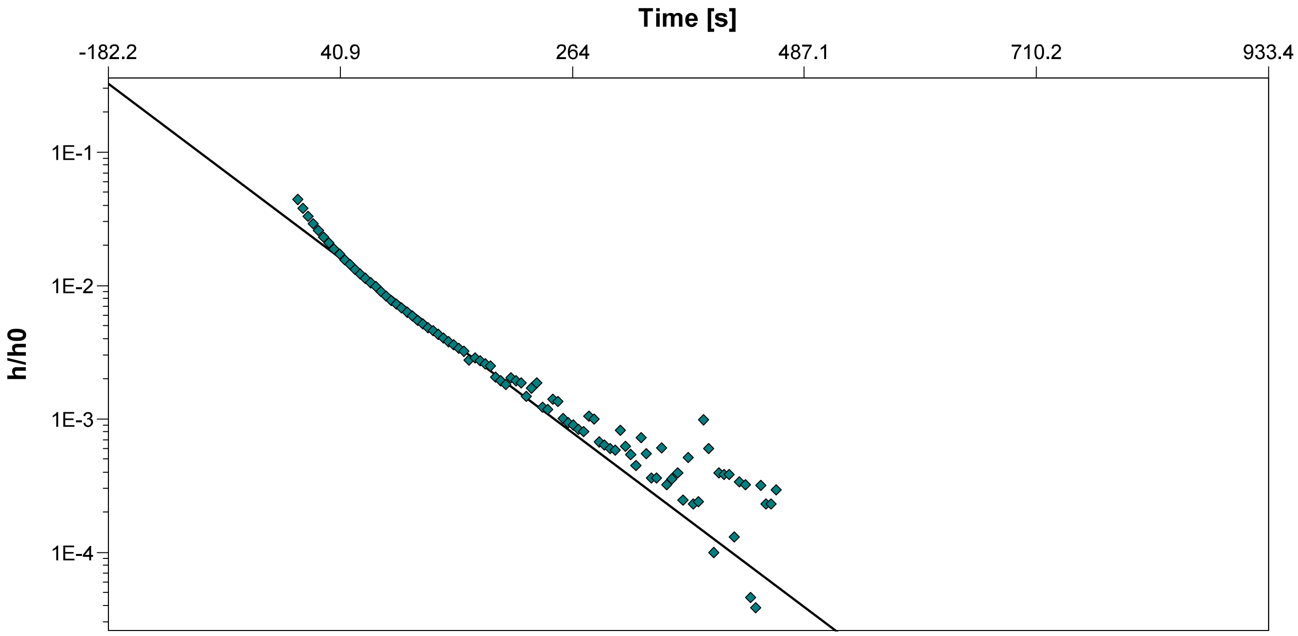


Calculation using Hvorslev

| Observation Well | Hydraulic Conductivity [m/s] | |
|---|---|---|
| PZ21 (SAND) | $1.24 \times 10^{-5}$ | |

| | |
|---|---|
| | **Slug Test Analysis Report** |
| | Project: Bell Road, Papamoa |
| | Number: 19630.000.001 |
| | Client: Bell Road Partnership |

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ21 (SAND) (FHT) | Test Well: PZ21 (SAND) |
| Test Conducted by: DRT | | Test Date: 21/08/2025 |
| Analysis Performed by: DRT | Bouwer & Rice | Analysis Date: 29/08/2025 |
| Aquifer Thickness: 7.00 m | | |



Calculation using Bouwer & Rice

| Observation Well | Hydraulic Conductivity [m/s] | |
|---|---|---|
| PZ21 (SAND) | $9.00 \times 10^{-6}$ | |

**Slug Test - Analyses Report**

Project: Bell Road, Papamoa

Number: 19630.000.001

Client: Bell Road Partnership

Location: Bell Road

Slug Test: PZ21 (SAND) (FHT)

Test Well: PZ21 (SAND)

Test Conducted by: DRT

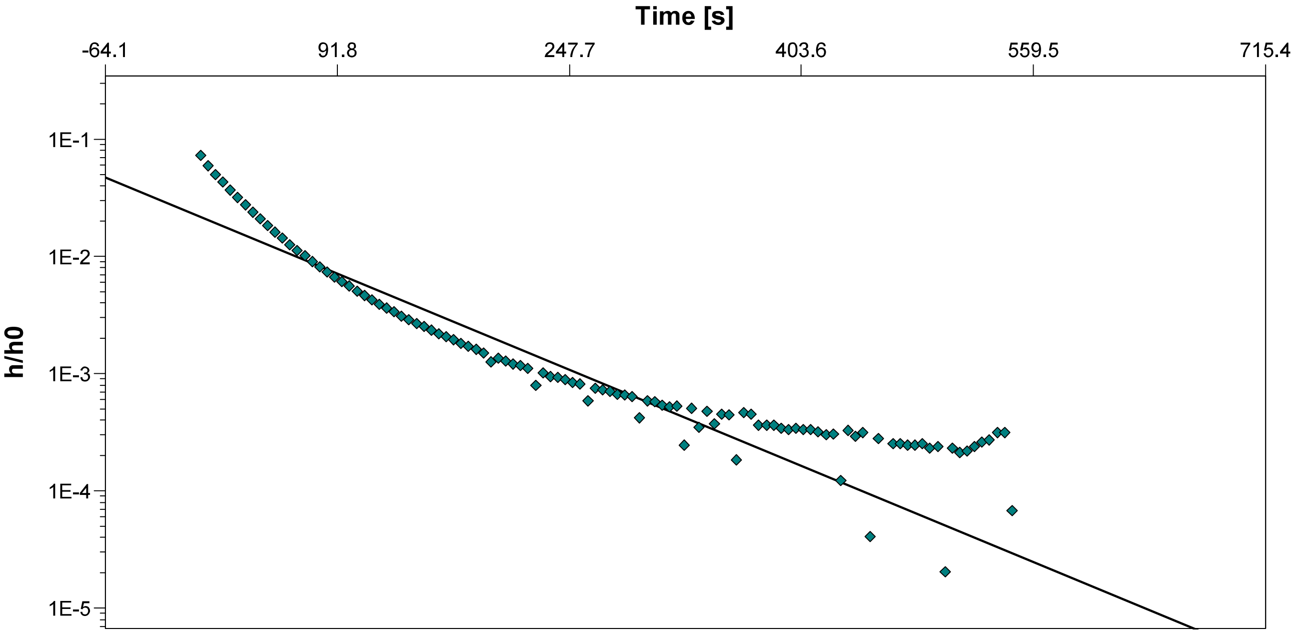
Test Date: 21/08/2025

Aquifer Thickness: 7.00 m

| | Analysis Name | Analysis Performed b | Analysis Date | Method name | Well | T [m²/s] | K [m/s] | S |
|---|---|---|---|---|---|---|---|---|
| 1 | Hvorslev | DRT | 29/08/2025 | Hvorslev | PZ21 (SAND) | | $1.24 \times 10^{-5}$ | |
| 2 | Bouwer & Rice | DRT | 29/08/2025 | Bouwer & Rice | PZ21 (SAND) | | $9.00 \times 10^{-6}$ | |

| | |
|---|---|
| | **Slug Test Analysis Report** |
| | Project: Bell Road, Papamoa |
| | Number: 19630.000.001 |
| | Client: Bell Road Partnership |

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ21 (SAND) (RHT) | Test Well: PZ21 (SAND) |
| Test Conducted by: DRT | | Test Date: 21/08/2025 |
| Analysis Performed by: DRT | Hvorslev | Analysis Date: 29/08/2025 |
| Aquifer Thickness: 7.00 m | | |

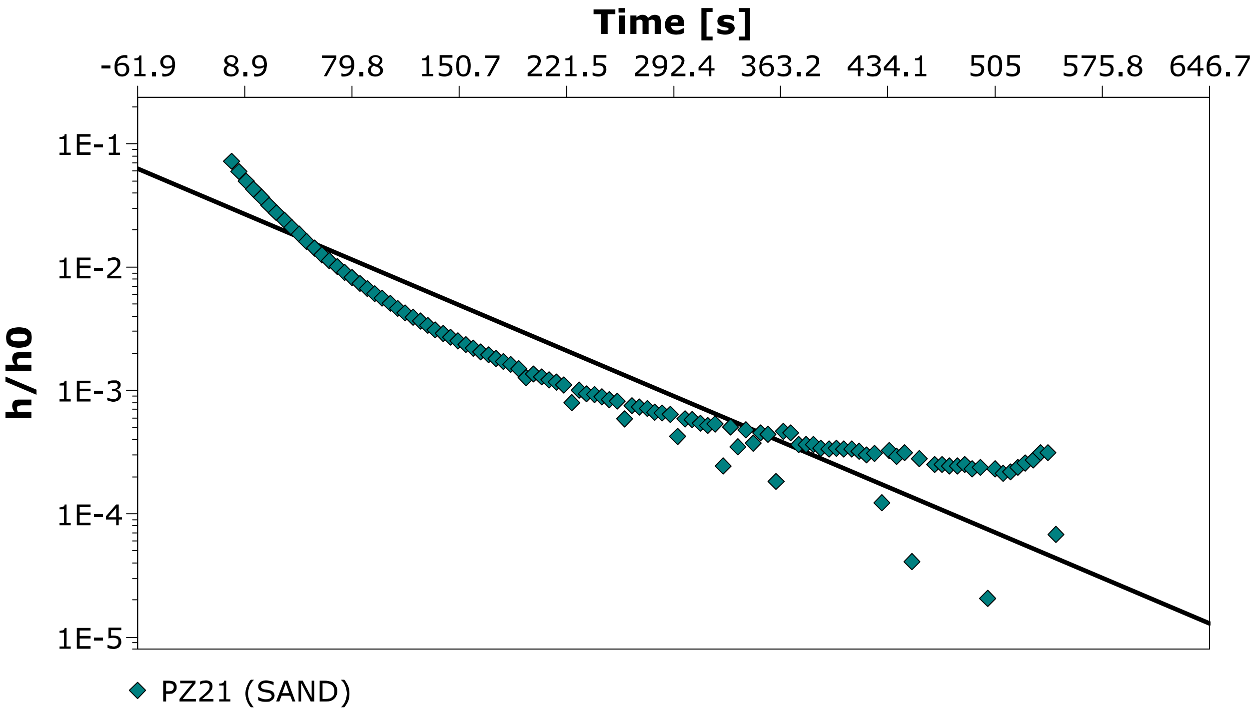


Calculation using Hvorslev

| Observation Well | Hydraulic Conductivity [m/s] | |
|---|---|---|
| PZ21 (SAND) | $1.25 \times 10^{-5}$ | |

| | |
|---|---|
| | **Slug Test Analysis Report** |
| | Project: Bell Road, Papamoa |
| | Number: 19630.000.001 |
| | Client: Bell Road Partnership |

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ21 (SAND) (RHT) | Test Well: PZ21 (SAND) |
| Test Conducted by: DRT | | Test Date: 21/08/2025 |
| Analysis Performed by: DRT | Bouwer & Rice | Analysis Date: 29/08/2025 |
| Aquifer Thickness: 7.00 m | | |



Calculation using Bouwer & Rice

| Observation Well | Hydraulic Conductivity [m/s] | |
|---|---|---|
| PZ21 (SAND) | $8.00 \times 10^{-6}$ | |

**Slug Test - Analyses Report**

Project: Bell Road, Papamoa

Number: 19630.000.001

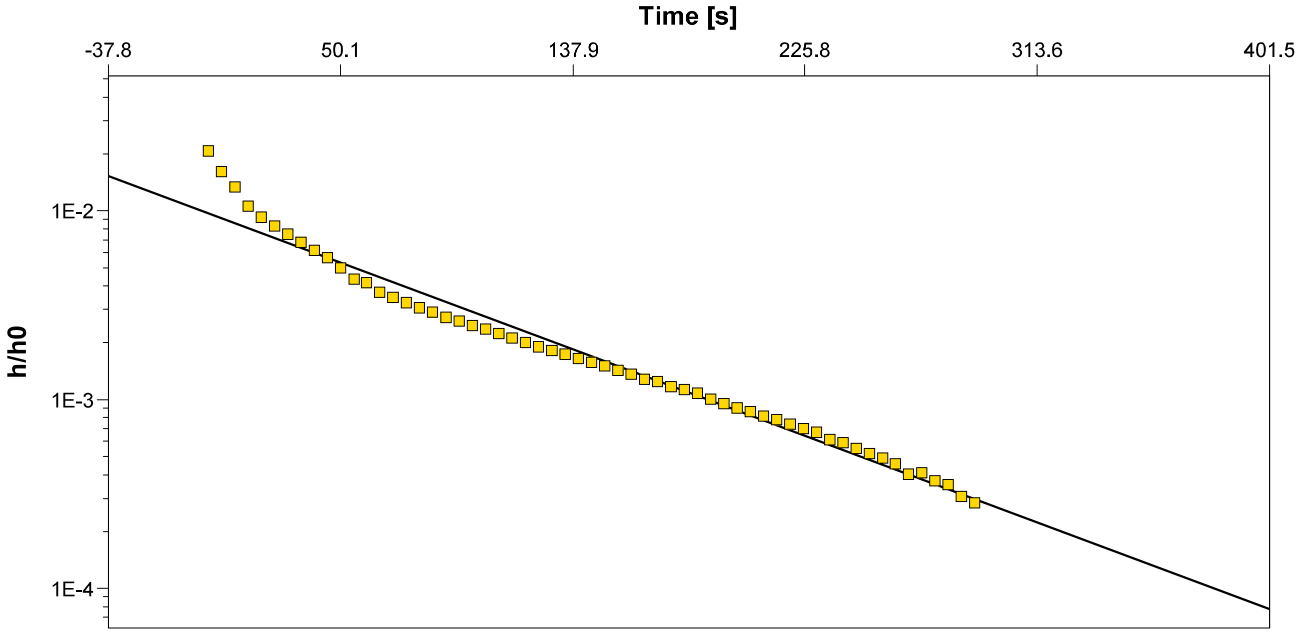
Client: Bell Road Partnership

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ21 (SAND) (RHT) | Test Well: PZ21 (SAND) |
| Test Conducted by: DRT | | Test Date: 21/08/2025 |
| Aquifer Thickness: 7.00 m | | |

| | Analysis Name | Analysis Performed b | Analysis Date | Method name | Well | T [m²/s] | K [m/s] | S |
|---|---|---|---|---|---|---|---|---|
| 1 | Hvorslev | DRT | 29/08/2025 | Hvorslev | PZ21 (SAND) | | $1.25 \times 10^{-5}$ | |
| 2 | Bouwer & Rice | DRT | 29/08/2025 | Bouwer & Rice | PZ21 (SAND) | | $8.00 \times 10^{-6}$ | |

| | |
|---|---|
| | **Slug Test Analysis Report** |
| | Project: Bell Road, Papamoa |
| | Number: 19630.000.001 |
| | Client: Bell Road Partnership |

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ04 (Peat&Sand) (FHT) | Test Well: PZ04 (Peat&Sand) |
| Test Conducted by: RN | | Test Date: 9/09/2025 |
| Analysis Performed by: RN | Hvorslev | Analysis Date: 9/09/2025 |
| Aquifer Thickness: 3.74 m | | |



Calculation using Hvorslev

| Observation Well | Hydraulic Conductivity [m/s] | |
|---|---|---|
| PZ04 (PeatSand) | $5.08 \times 10^{-6}$ | |

**Slug Test Analysis Report**

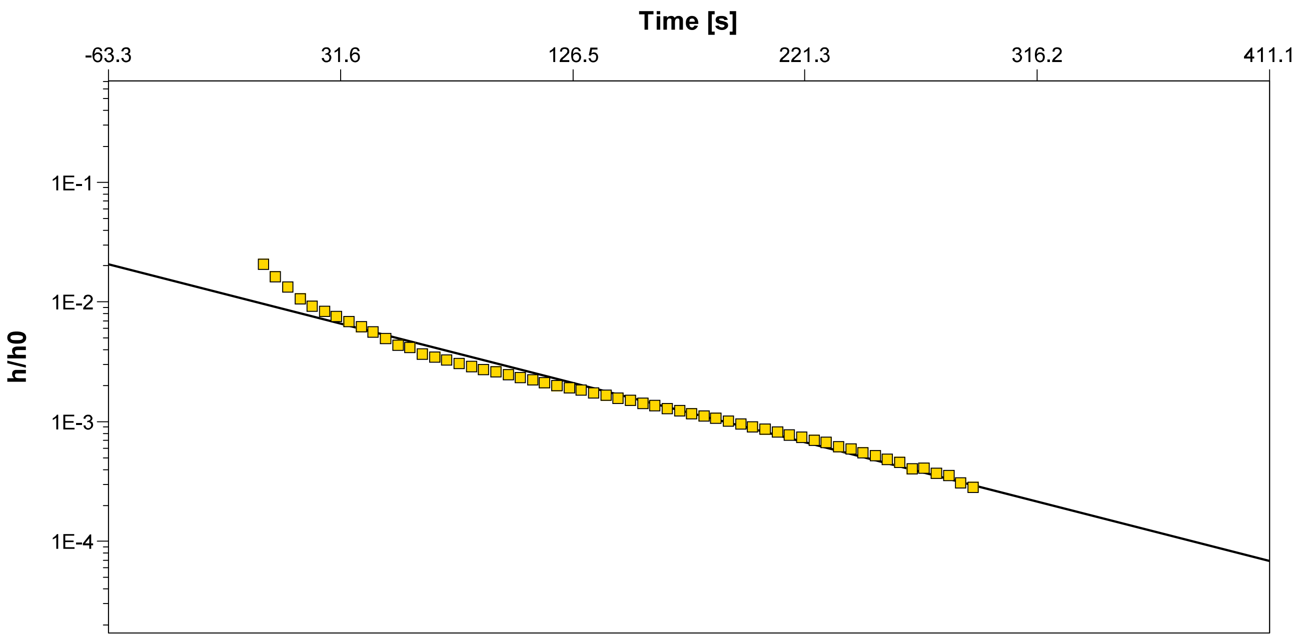
Project: Bell Road, Papamoa

Number: 19630.000.001

Client: Bell Road Partnership

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ04 (Peat&Sand) (FHT) | Test Well: PZ04 (Peat&Sand) |
| Test Conducted by: RN | | Test Date: 9/09/2025 |
| Analysis Performed by: RN | Bouwer & Rice | Analysis Date: 9/09/2025 |

Aquifer Thickness: 3.74 m



Calculation using Bouwer & Rice

| Observation Well | Hydraulic Conductivity [m/s] |
|---|---|
| PZ04 (PeatSand) | $3.94 \times 10^{-6}$ |

| | |
|---|---|
| | **Slug Test - Analyses Report** |
| | Project: Bell Road, Papamoa |
| | Number: 19630.000.001 |
| | Client: Bell Road Partnership |

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ04 (Peat&Sand) (FHT) | Test Well: PZ04 (Peat&Sand) |
| Test Conducted by: RN | | Test Date: 9/09/2025 |
| Aquifer Thickness: 3.74 m | | |

| | Analysis Name | Analysis Performed b | Analysis Date | Method name | Well | T [m²/s] | K [m/s] | S |
|---|---|---|---|---|---|---|---|---|
| 1 | Hvorslev | RN | 9/09/2025 | Hvorslev | PZ04 (Peat&Sand) | | $5.08 \times 10^{-6}$ | |
| 2 | Bouwer & Rice | RN | 9/09/2025 | Bouwer & Rice | PZ04 (Peat&Sand) | | $3.94 \times 10^{-6}$ | |

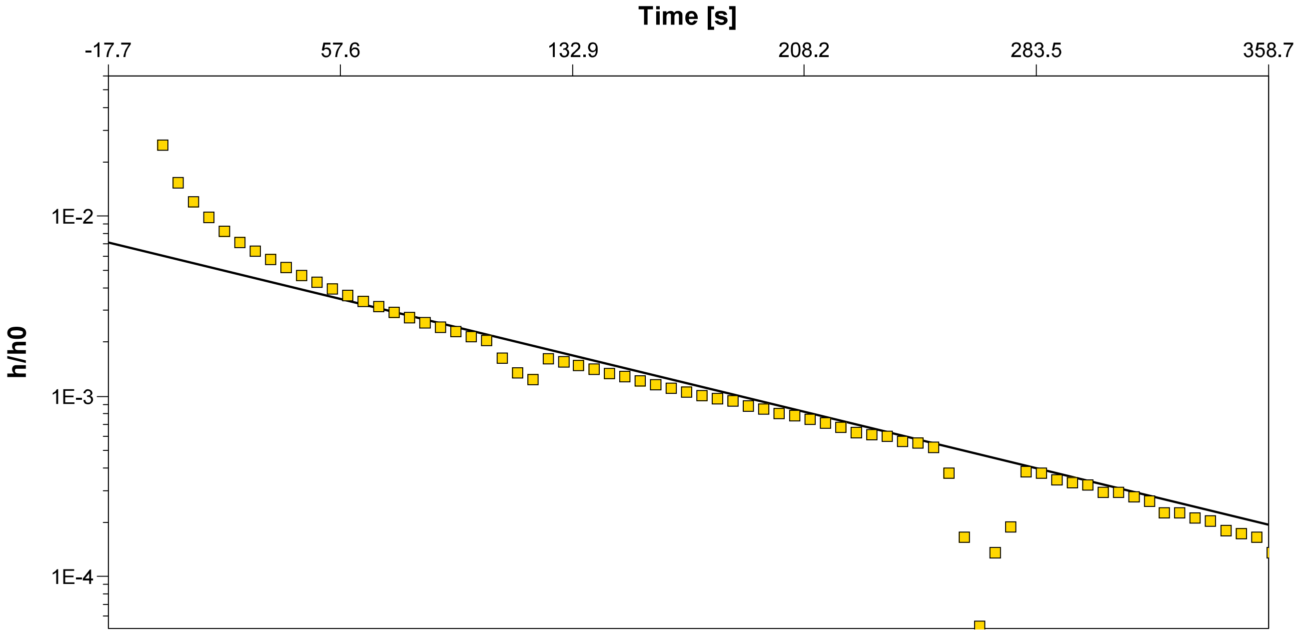
**Slug Test Analysis Report**

Project: Bell Road, Papamoa

Number: 19630.000.001

Client: Bell Road Partnership

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ04 (Peat&Sand) (RHT) | Test Well: PZ04 (Peat&Sand) |
| Test Conducted by: RN | | Test Date: 9/09/2025 |
| Analysis Performed by: RN | Hvorslev | Analysis Date: 9/09/2025 |
| Aquifer Thickness: 3.74 m | | |

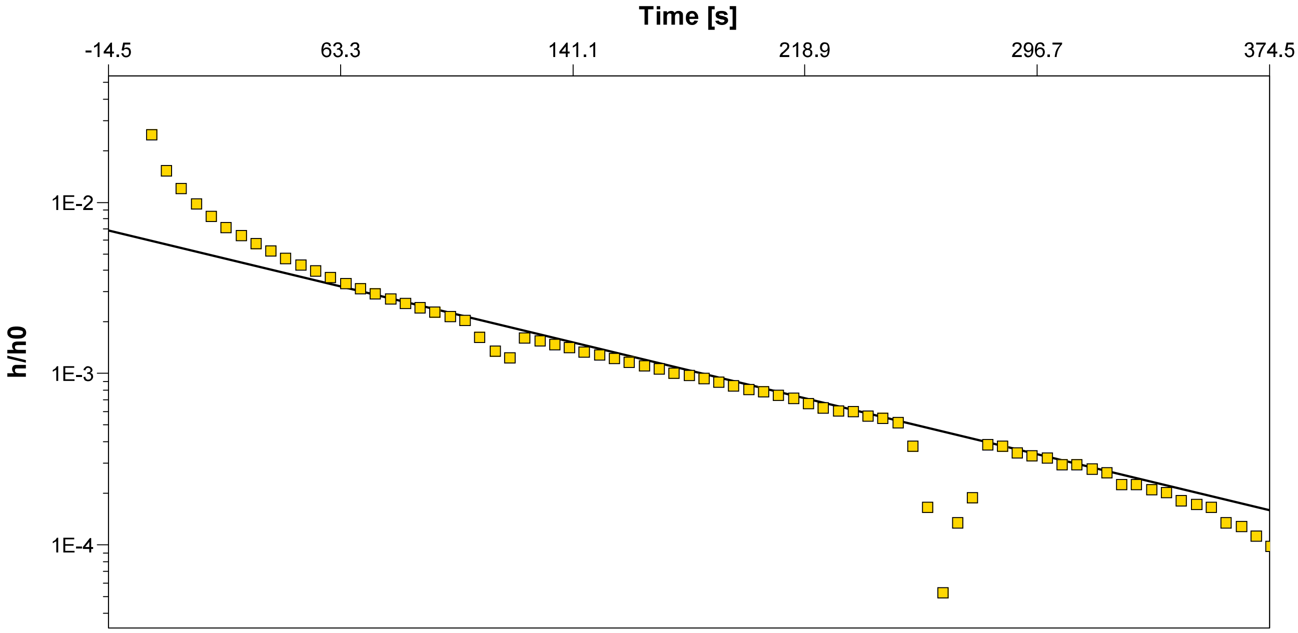


Calculation using Hvorslev

| Observation Well | Hydraulic Conductivity [m/s] | |
|---|---|---|
| PZ04 (PeatSand) | $4.04 \times 10^{-6}$ | |

| | |
|---|---|
| | **Slug Test Analysis Report** |
| | Project: Bell Road, Papamoa |
| | Number: 19630.000.001 |
| | Client: Bell Road Partnership |

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ04 (Peat&Sand) (RHT) | Test Well: PZ04 (Peat&Sand) |
| Test Conducted by: RN | | Test Date: 9/09/2025 |
| Analysis Performed by: RN | Bouwer & Rice | Analysis Date: 9/09/2025 |
| Aquifer Thickness: 3.74 m | | |



Calculation using Bouwer & Rice

| Observation Well | Hydraulic Conductivity [m/s] | |
|---|---|---|
| PZ04 (PeatSand) | $3.16 \times 10^{-6}$ | |

**Slug Test - Analyses Report**

Project: Bell Road, Papamoa

Number: 19630.000.001

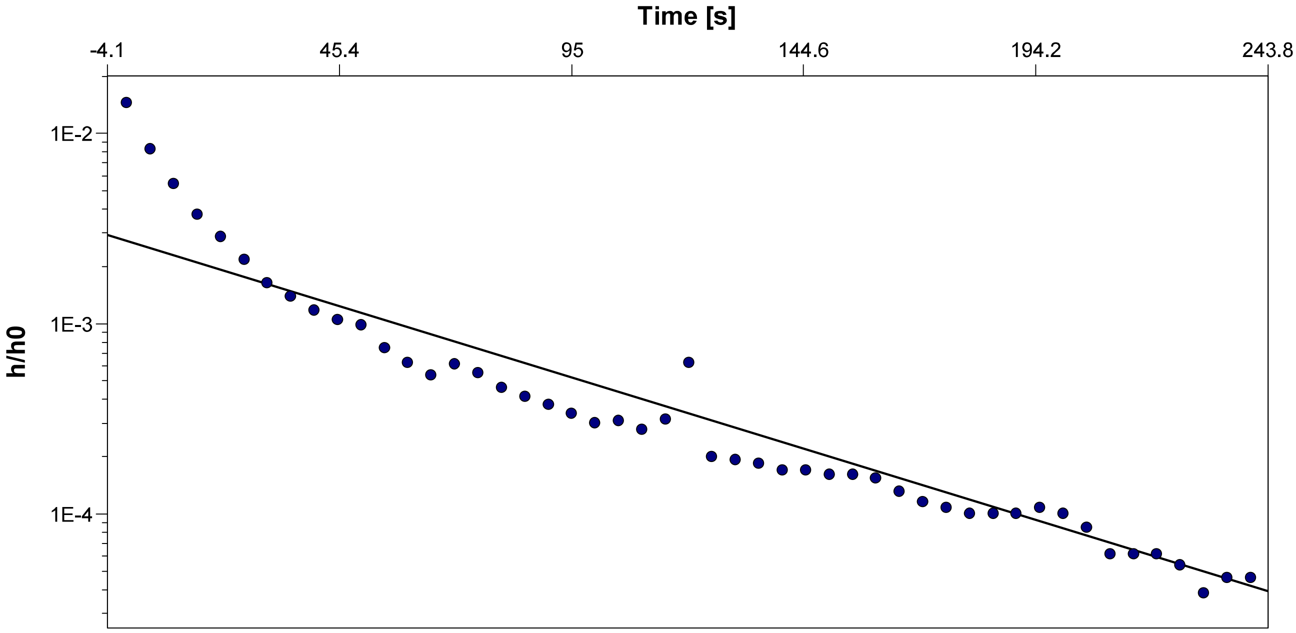
Client: Bell Road Partnership

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ04 (Peat&Sand) (RHT) | Test Well: PZ04 (Peat&Sand) |
| Test Conducted by: RN | | Test Date: 9/09/2025 |
| Aquifer Thickness: 3.74 m | | |

| | Analysis Name | Analysis Performed b | Analysis Date | Method name | Well | T [m²/s] | K [m/s] | S |
|---|---|---|---|---|---|---|---|---|
| 1 | Hvorslev | RN | 9/09/2025 | Hvorslev | PZ04 (Peat&Sand) | | $4.04 \times 10^{-6}$ | |
| 2 | Bouwer & Rice | RN | 9/09/2025 | Bouwer & Rice | PZ04 (Peat&Sand) | | $3.16 \times 10^{-6}$ | |

| | |
|---|---|
| | **Slug Test Analysis Report** |
| | Project: Bell Road, Papamoa |
| | Number: 19630.000.001 |
| | Client: Bell Road Partnership |

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ05 (Peat&Sand) (FHT) | Test Well: PZ05 (Peat&Sand) |
| Test Conducted by: RN | | Test Date: 9/09/2025 |
| Analysis Performed by: RN | Hvorslev | Analysis Date: 9/09/2025 |
| Aquifer Thickness: 3.65 m | | |

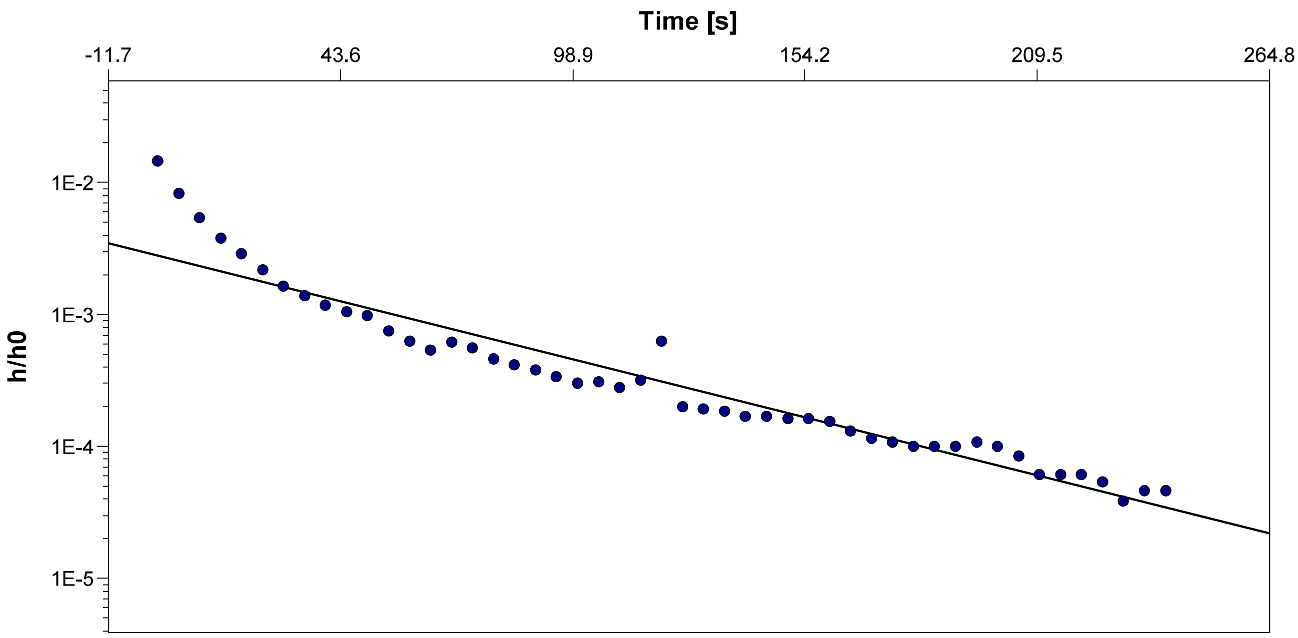


Calculation using Hvorslev

| Observation Well | Hydraulic Conductivity [m/s] | |
|---|---|---|
| PZ05 (PeatSand) | $7.47 \times 10^{-6}$ | |

| | |
|---|---|
| | **Slug Test Analysis Report** |
| | Project: Bell Road, Papamoa |
| | Number: 19630.000.001 |
| | Client: Bell Road Partnership |

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ05 (Peat&Sand) (FHT) | Test Well: PZ05 (Peat&Sand) |
| Test Conducted by: RN | | Test Date: 9/09/2025 |
| Analysis Performed by: RN | Bouwer & Rice | Analysis Date: 9/09/2025 |
| Aquifer Thickness: 3.65 m | | |



Calculation using Bouwer & Rice

| Observation Well | Hydraulic Conductivity [m/s] | |
|---|---|---|
| PZ05 (PeatSand) | $6.10 \times 10^{-6}$ | |

**Slug Test - Analyses Report**

Project: Bell Road, Papamoa

Number: 19630.000.001

Client: Bell Road Partnership

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ05 (Peat&Sand) (FHT) | Test Well: PZ05 (Peat&Sand) |
| Test Conducted by: RN | | Test Date: 9/09/2025 |
| Aquifer Thickness: 3.65 m | | |

| | Analysis Name | Analysis Performed b | Analysis Date | Method name | Well | T [m²/s] | K [m/s] | S |
|---|---|---|---|---|---|---|---|---|
| 1 | Hvorslev | RN | 9/09/2025 | Hvorslev | PZ05 (Peat&Sand) | | $7.47 \times 10^{-6}$ | |
| 2 | Bouwer & Rice | RN | 9/09/2025 | Bouwer & Rice | PZ05 (Peat&Sand) | | $6.10 \times 10^{-6}$ | |

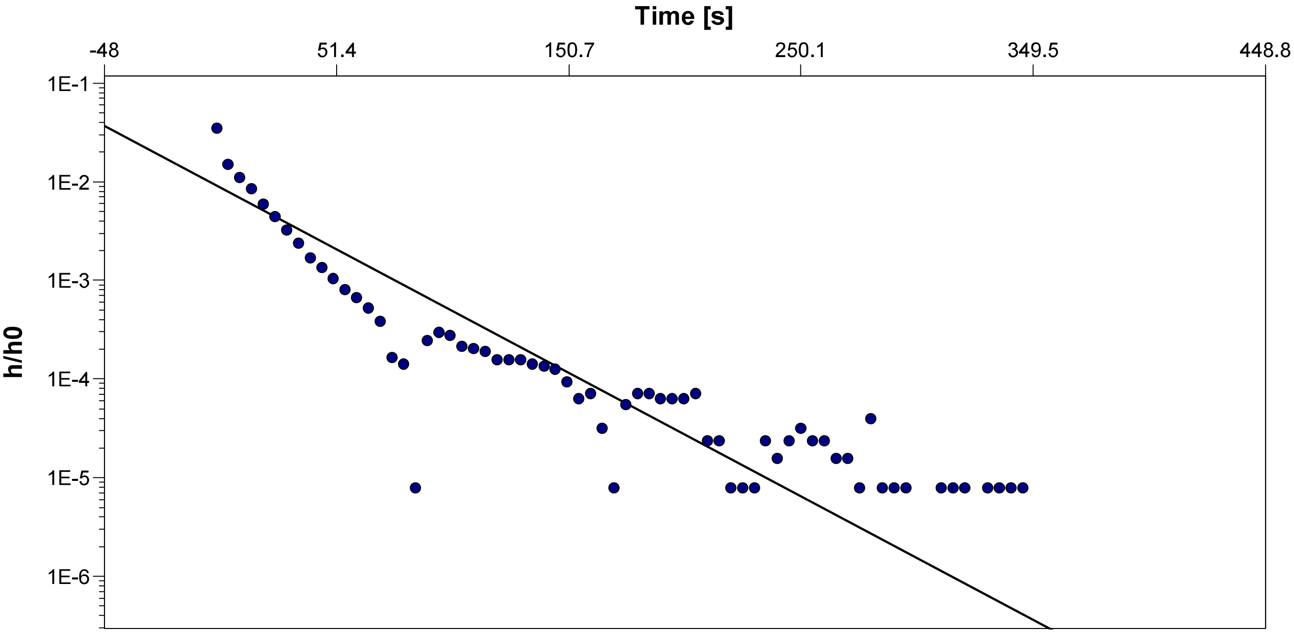
**Slug Test Analysis Report**

Project: Bell Road, Papamoa

Number: 19630.000.001

Client: Bell Road Partnership

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ05 (Peat&Sand) (RHT) | Test Well: PZ05 (Peat&Sand) |
| Test Conducted by: RN | | Test Date: 9/09/2025 |
| Analysis Performed by: RN | Hvorslev | Analysis Date: 9/09/2025 |
| Aquifer Thickness: 3.65 m | | |

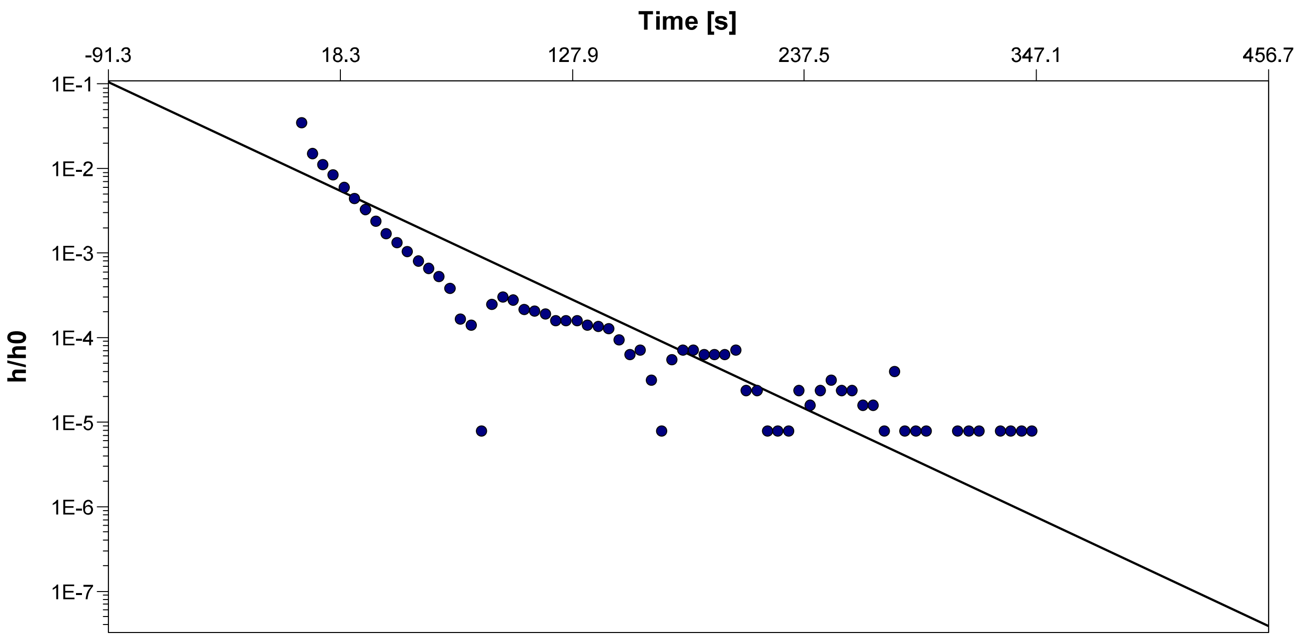


Calculation using Hvorslev

| Observation Well | Hydraulic Conductivity [m/s] | |
|---|---|---|
| PZ05 (PeatSand) | $1.25 \times 10^{-5}$ | |

| | |
|---|---|
| | **Slug Test Analysis Report** |
| | Project: Bell Road, Papamoa |
| | Number: 19630.000.001 |
| | Client: Bell Road Partnership |

| | | |
|---|---|---|
| Location: Bell Road | Slug Test: PZ05 (Peat&Sand) (RHT) | Test Well: PZ05 (Peat&Sand) |
| Test Conducted by: RN | | Test Date: 9/09/2025 |
| Analysis Performed by: RN | Bouwer & Rice | Analysis Date: 9/09/2025 |
| Aquifer Thickness: 3.65 m | | |



Calculation using Bouwer & Rice

| Observation Well | Hydraulic Conductivity [m/s] | |
|---|---|---|
| PZ05 (PeatSand) | $9.00 \times 10^{-6}$ | |

**Slug Test - Analyses Report**

Project: Bell Road, Papamoa
Number: 19630.000.001
Client: Bell Road Partnership

Location: Bell Road
Slug Test: PZ05 (Peat&Sand) (RHT)
Test Well: PZ05 (Peat&Sand)
Test Conducted by: RN
Test Date: 9/09/2025
Aquifer Thickness: 3.65 m

|  | Analysis Name | Analysis Performed b | Analysis Date | Method name | Well | T [m²/s] | K [m/s] | S |
|---|---|---|---|---|---|---|---|---|
| 1 | Hvorslev | RN | 9/09/2025 | Hvorslev | PZ05 (Peat&Sand) |  | $1.25 \times 10^{-5}$ |  |
| 2 | Bouwer & Rice | RN | 9/09/2025 | Bouwer & Rice | PZ05 (Peat&Sand) |  | $9.00 \times 10^{-6}$ |  |

# Appendix 3:

## Soil Moisture Water Balance Model

# Recharge Modelling

## 1 Model Parameters

The soil moisture water balance model (WWLA SMWBM 4.5.0.0) is a deterministic lumped parameter model originally developed by Pitman (1976) to simulate river flows in South Africa. The code was reworked into a Windows environment and the functionality extended to include a surface ponding function, additional evaporation functions and an irrigation module.

This model uses daily data on rainfall and potential evaporation to estimate soil moisture and the movement of water throughout the catchment, whether under natural conditions or with irrigation. During dry spells, the model calculates changes once a day. When it rains, it switches to more frequent, hourly calculations.

The model incorporates parameters that characterise the catchment in terms of:

- interception storage,
- evaporation losses,
- soil moisture storage capacity,
- plant available water capacity,
- soil infiltration,
- sub-soil drainage,
- vadose zone vertical drainage,
- surface runoff (quickflow),
- stream baseflows (groundwater contribution); and
- the recession and / or attenuation of groundwater and surface water flow components, respectively.

## 2 Fundamental Operation

The fundamental operation of the model is as follows:

When rainfall occurs, rainfall must first fill a nominal interception storage (PI) before reaching the soil zone, where the net rainfall is assessed as part of the runoff / infiltration calculation. Water that penetrates the soil fills a nominal soil moisture storage zone (ST). This zone is subject to evapotranspiration via root uptake and direct evaporation (R) according to the mean monthly evaporation rate and current soil moisture deficits. The soil moisture zone provides a source of water for deeper percolation to the underlying aquifer, the rate of which is governed by the prevailing soil moisture status and the parameters FT and POW.

If disaggregated hourly rainfall is of greater intensity than the calculated hourly infiltration rate (ZMAX, ZMIN) surface runoff occurs. Surface runoff is also governed by two other factors, which are the prevailing soil moisture deficit and the proportion of impervious portions of the catchment directly linked to drainage pathways (AI). Rainfall of sufficient intensity and duration to fill the soil moisture storage results in excess rainfall that is allocated to either surface runoff or groundwater percolation depending on the slope and soakage characteristics of the catchment (DIV). Finally, the model produces daily summaries of the various components of the catchment water balance including interception loss, soil evaporation, percolation to groundwater and surface runoff.

Calibrated input parameters are presented in Table B1.

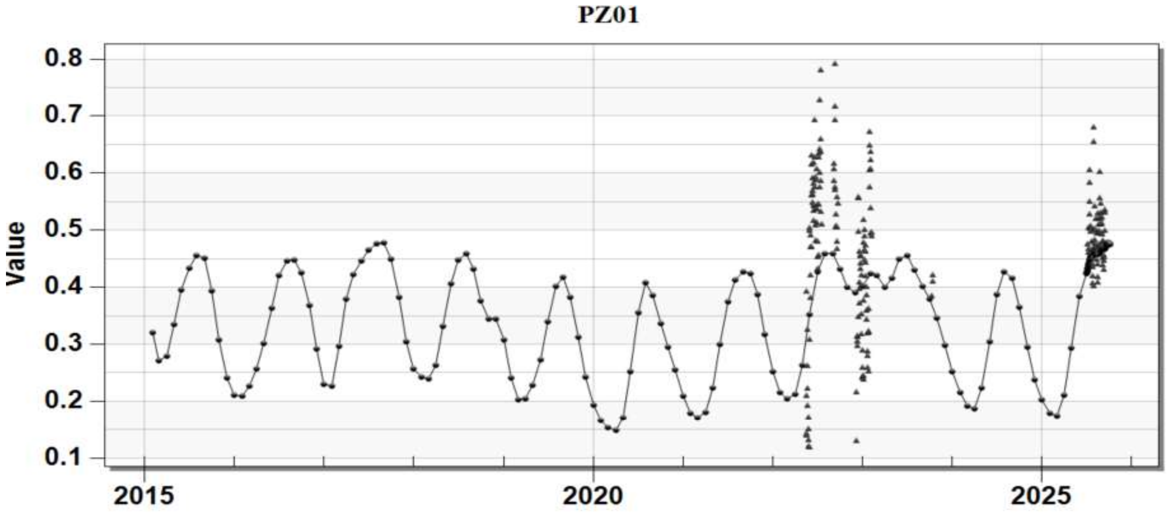
**Table B1: SMWBM Input Parameters**

| Parameter | Name | Parameter Values | | | Description |
|---|---|---|---|---|---|
| | | Peat | Dune Sand | Pumice Fill | |
| ST (mm) | Maximum soil water content | 250 | 1500 | 400 | ST defines the size of the soil moisture store in terms of a depth of water. |
| SL (mm) | Soil moisture content where drainage ceases | 0 | 0 | 0 | Soil moisture storage capacity below which sub-soil drainage ceases due to soil moisture retention. |
| ZMAX (mm/hr) | Maximum infiltration rate | 10 | 20 | 10 | ZMAX and ZMIN are nominal maximum and minimum infiltration rates in mm/hr used by the model to calculate the actual infiltration rate ZACT. ZMAX and ZMIN regulate the volume of water entering soil moisture storage and the resulting surface runoff. ZMIN is usually assigned zero. ZMAX is usually assigned the saturated infiltration rate from field testing. ZACT may be greater than ZMAX at the start of a rainfall event. ZACT is usually nearest to ZMAX when soil moisture is nearing maximum capacity. |
| ZMIN (mm/hr) | Minimum infiltration rate | 0 | 0 | 0 | |
| FT (mm/day) | Sub-soil drainage rate from soil moisture storage at full capacity | 1.2 | 4 | 0.6 | Together with POW, FT (mm/day) controls the rate of percolation to the underlying aquifer system from the soil moisture storage zone. FT is the maximum rate of percolation through the soil zone. |
| POW (>0) | Power of the soil moisture-percolation equation | 2 | 2 | 2 | POW determines the rate at which sub-soil drainage diminishes as the soil moisture content is decreased. POW therefore has significant effect on the seasonal distribution and reliability of drainage and hence baseflow, as well as the total yield from a catchment. |

| Parameter | Name | Parameter Values | | | Description |
|---|---|---|---|---|---|
| | | Peat | Dune Sand | Pumice Fill | |
| AI (-) | Impervious portion of catchment | 0.05 | 0.25 | 0.6 | AI represents the proportion of impervious zones of the catchment directly linked to drainage pathways. |
| R (0,1,10) | Evaporation-soil moisture relationship | 1 | 1 | 1 | Together with the soil moisture storage parameters ST and SL, R governs the evaporative process within the model. Three different relationships are available. The rate of evapotranspiration is estimated using either a linear (0,1) or power-curve (10) relationship relating evaporation to the soil moisture status of the soil. As the soil moisture capacity approaches full, evaporation occurs at a near maximum rate based on the mean monthly pan evaporation rate, and as the soil moisture capacity decreases, evaporation decreases according to the predefined function. |
| DIV (-) | Fraction of excess rainfall allocated directly to pond storage | 0.2 | 0.8 | 0.15 | DIV has values between 0 and 1 and defines the proportion of excess rainfall ponded at the surface due to saturation of the soil zone or rainfall exceeding the soils infiltration capacity to eventually infiltrate the soil, with the remainder (and typically majority) as direct runoff. |
| Kv (m/s) | Vertical hydraulic conductivity | - | - | - | Kv along with the VGn parameter and the soil moisture status governs the unsaturated hydraulic conductivity and travel times within the vadose zone. |
| TL (days) | Routing coefficient for surface runoff | - | - | - | TL defines the lag of surface water runoff. This is not necessary to define for this study as we are only interested in the groundwater percolation component of the water balance. |
| GL | Groundwater recession parameter | 20 | 20 | 20 | GL governs the lag in groundwater discharge or baseflow from a catchment. |

# APPENDIX 4:

## Numerical Model Calibration Plots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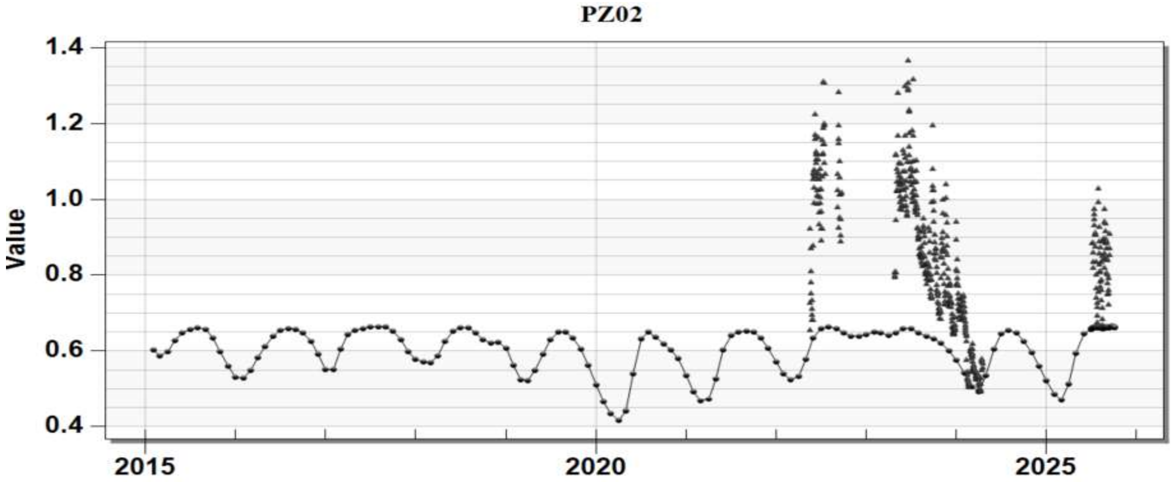
2015
2020
2025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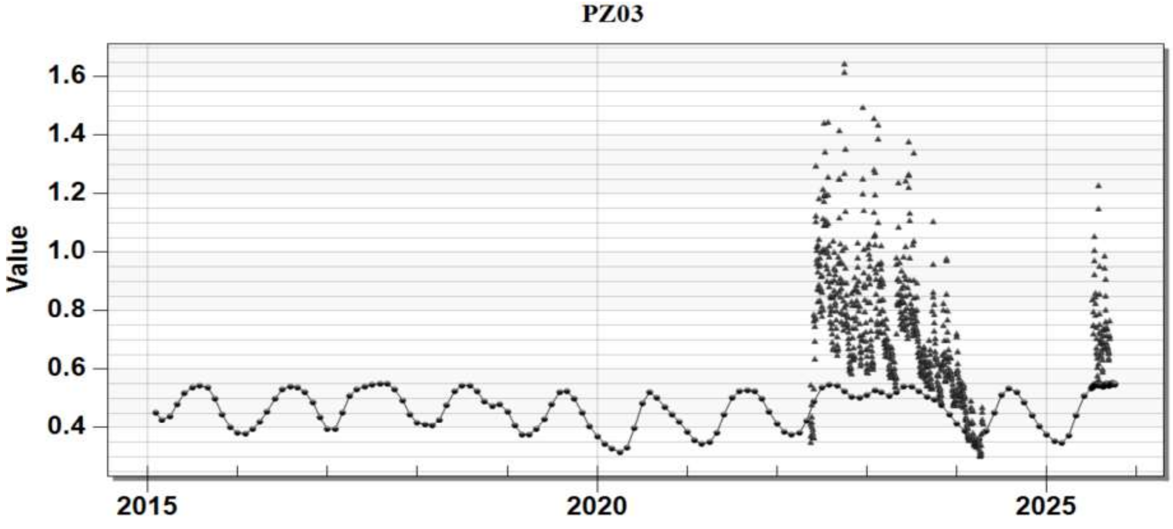
2020
2025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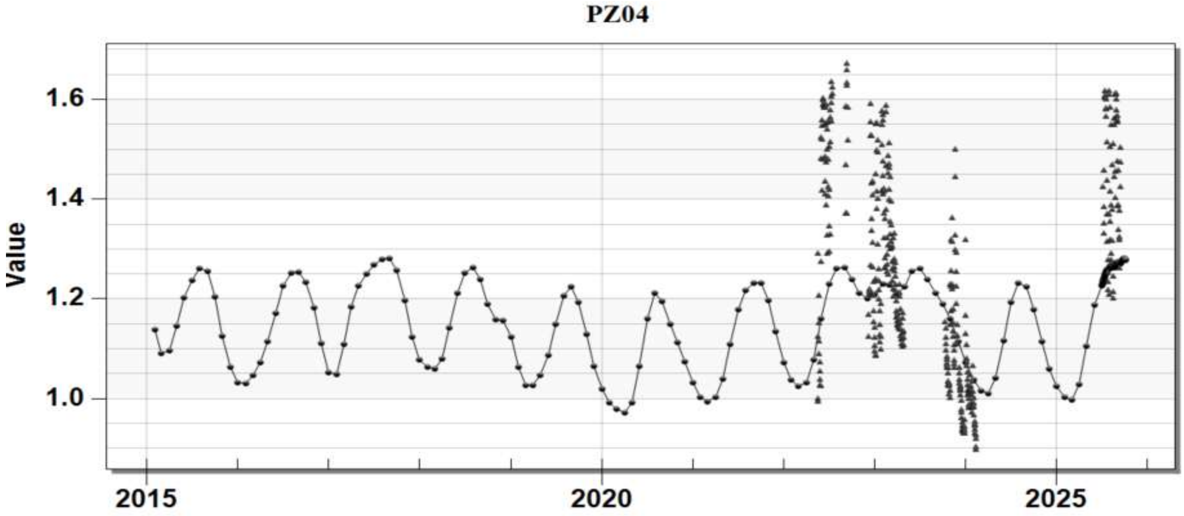
2020
2025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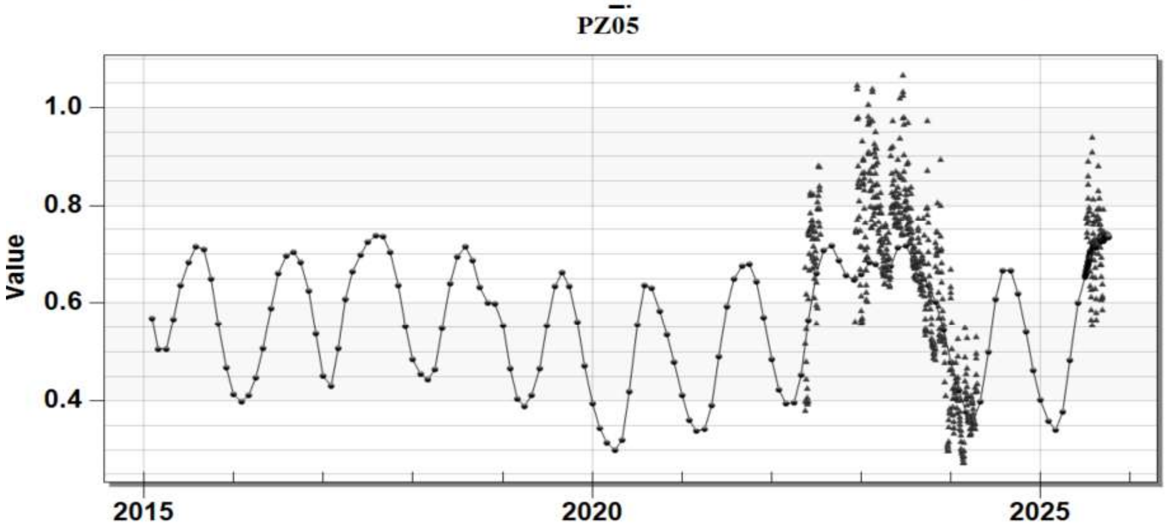
2020
2025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2020
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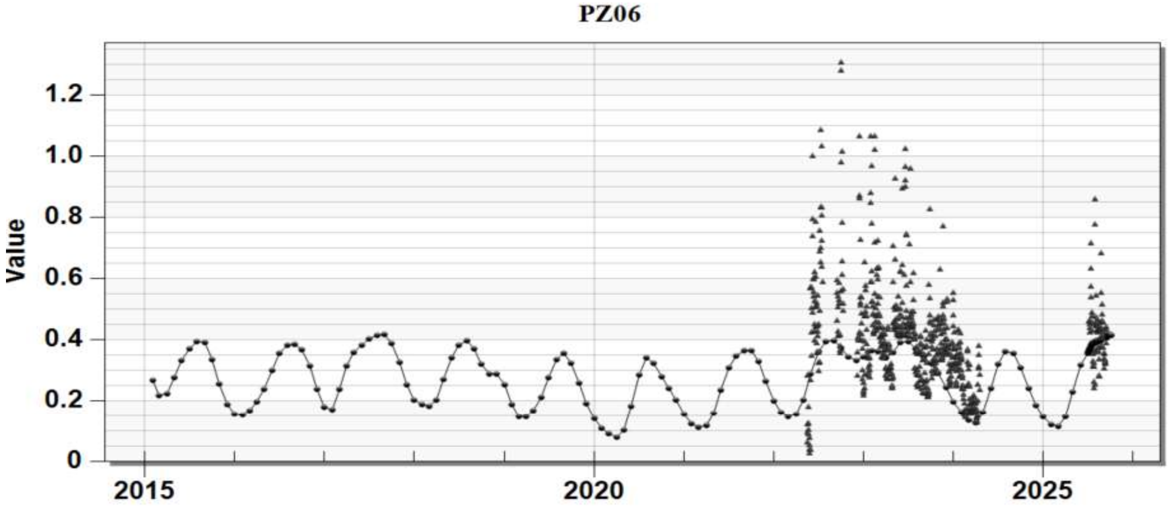
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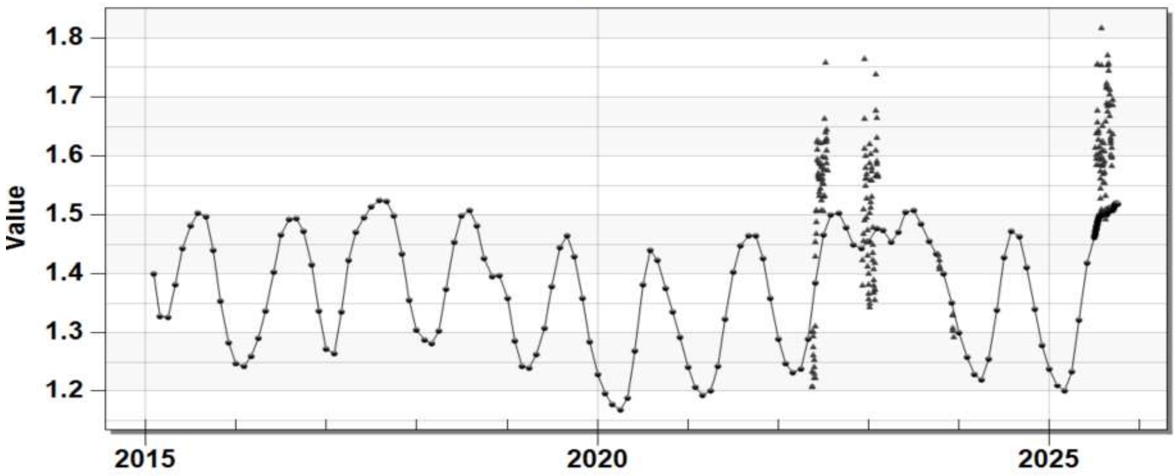
0
2015
2020
2025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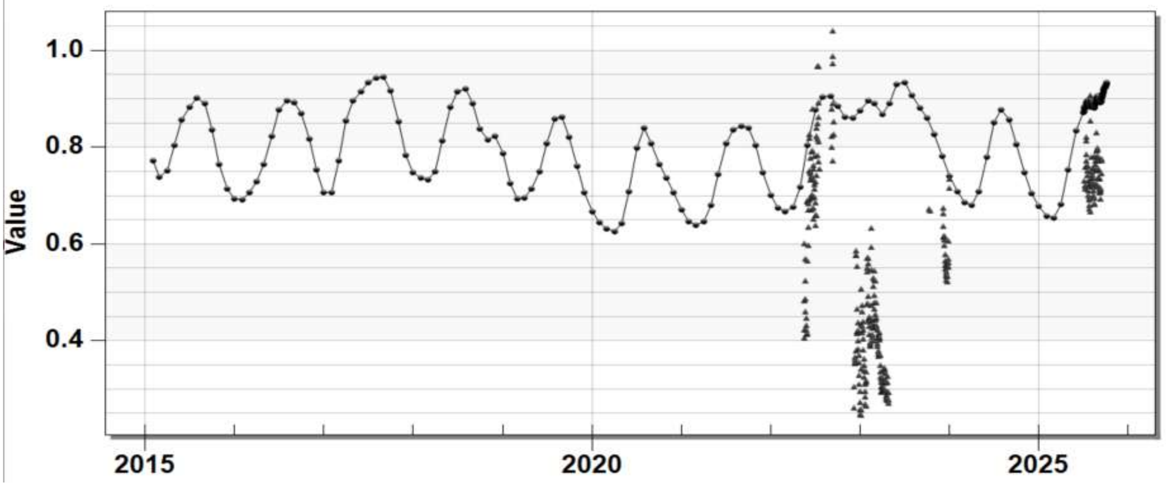
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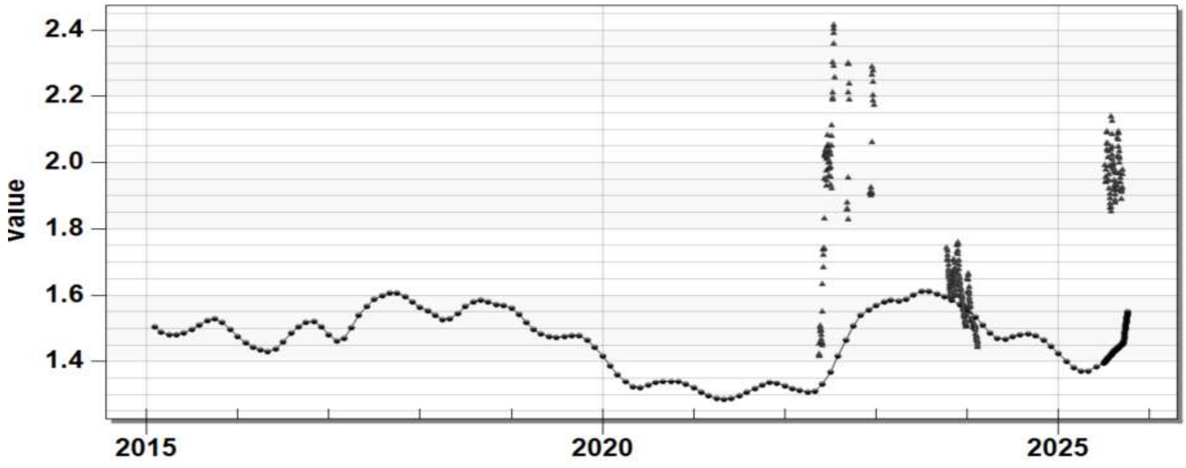
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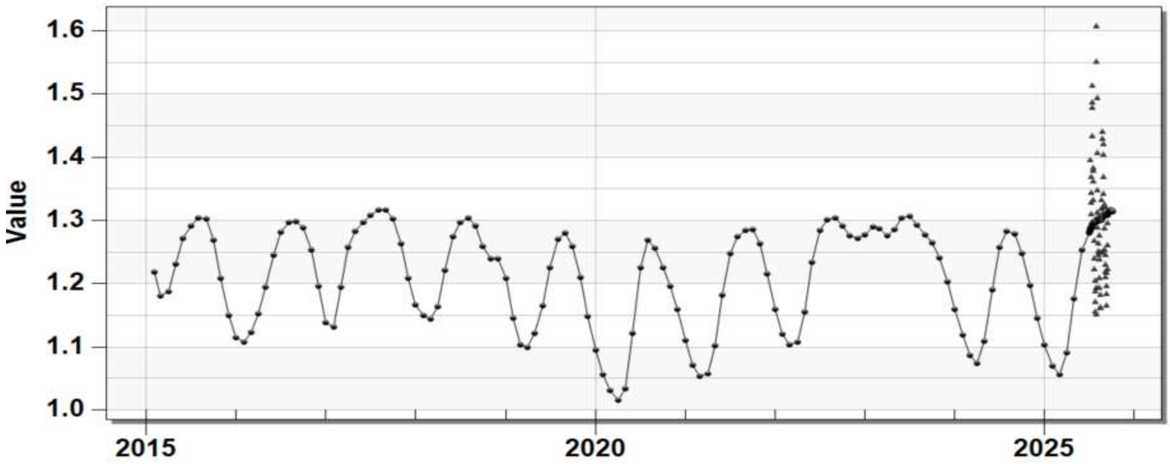

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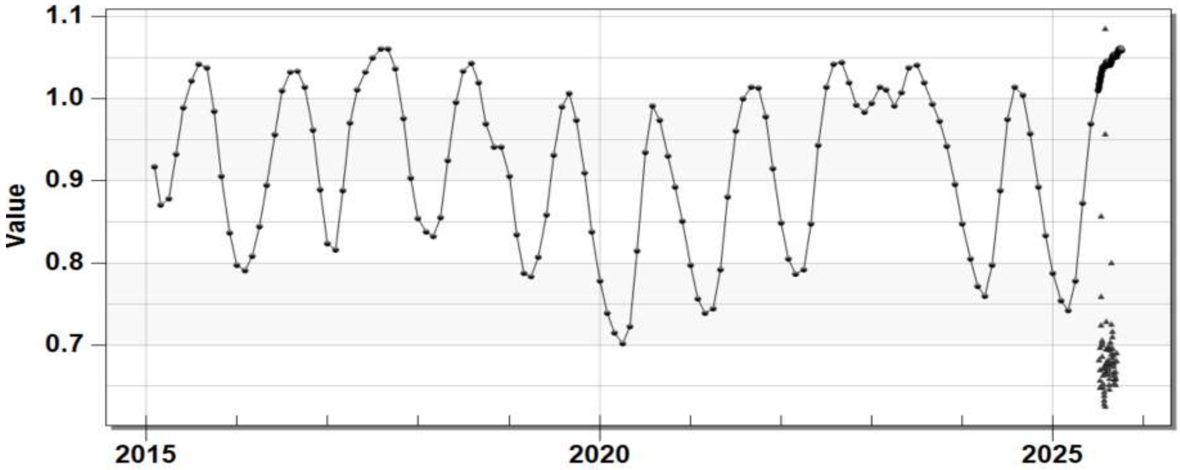

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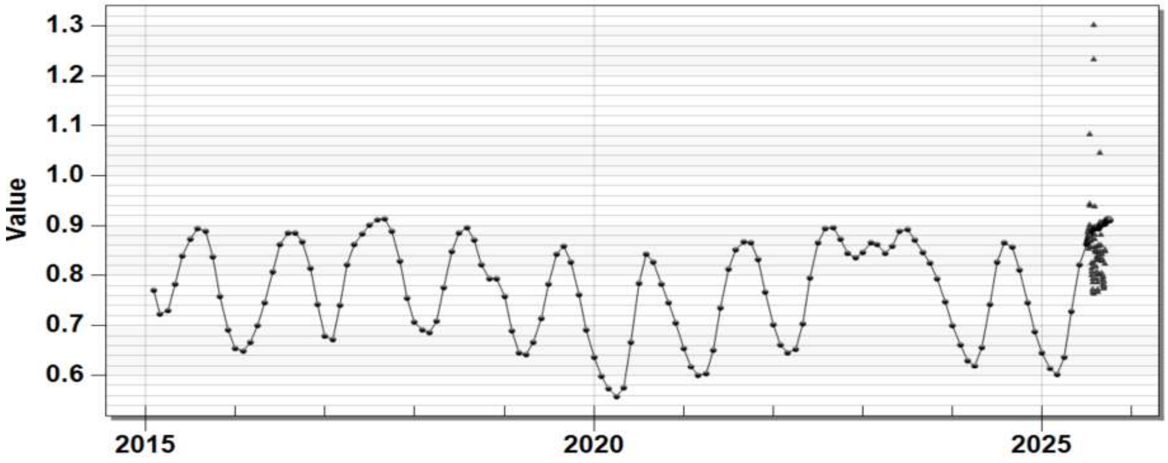

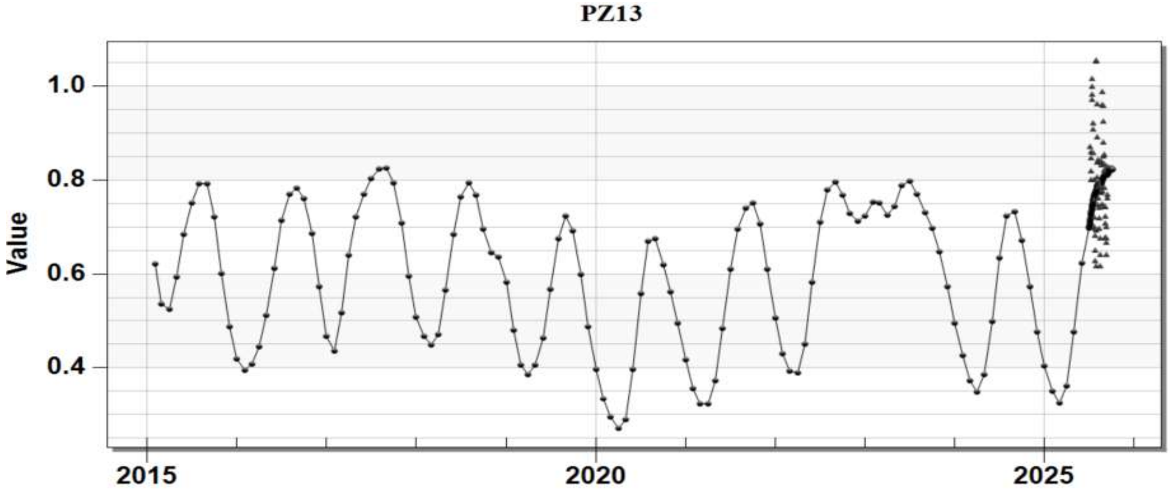
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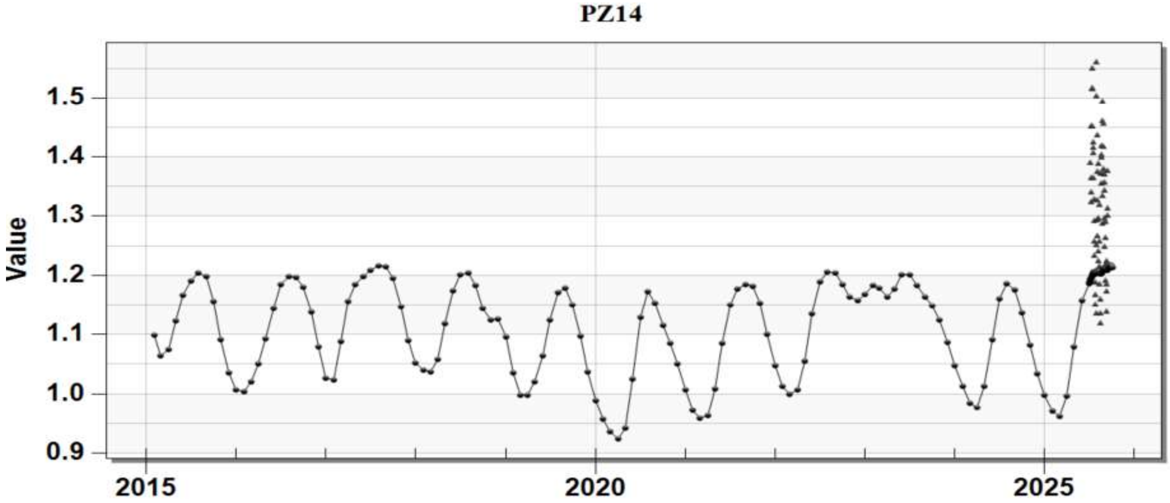
2020
2025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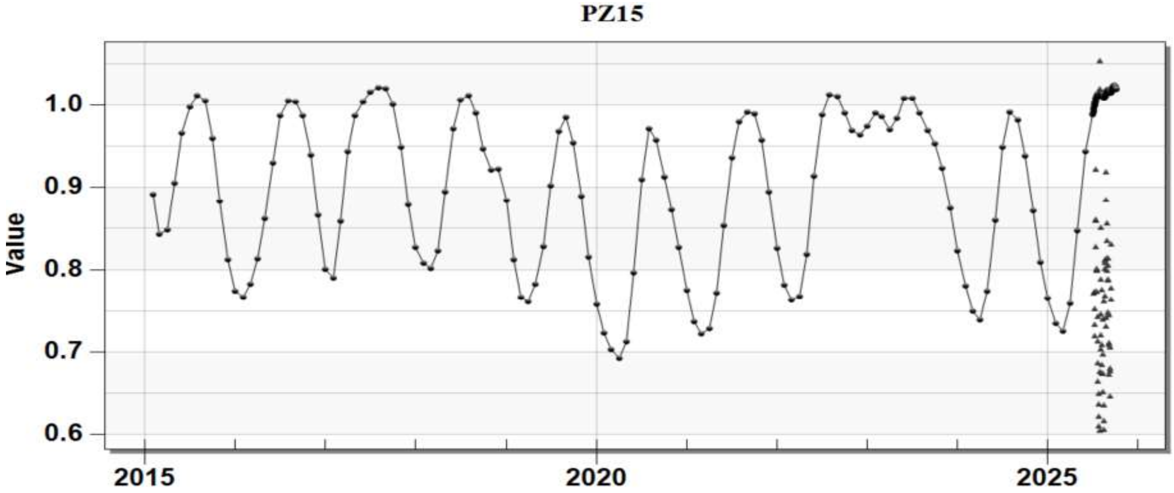
2020
2025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2020
2025

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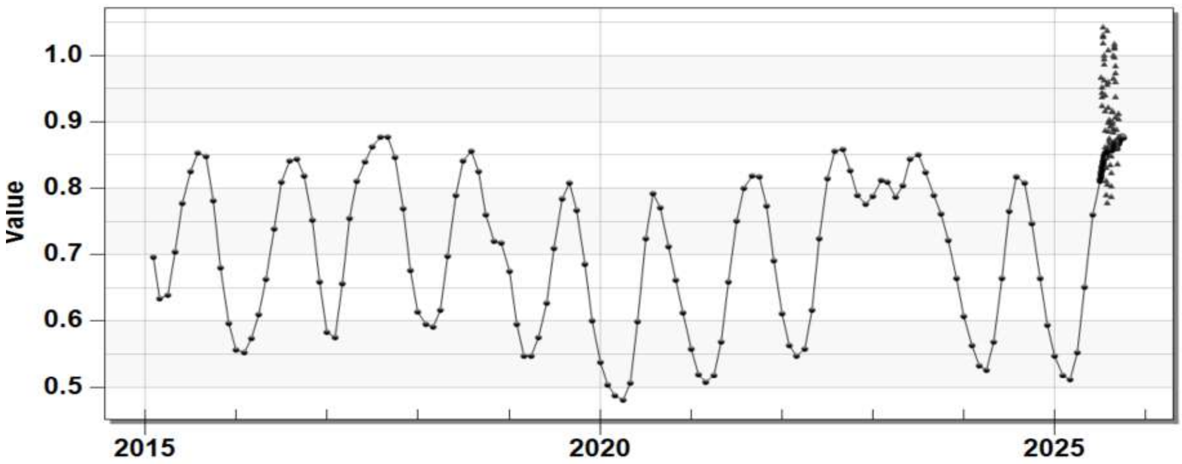

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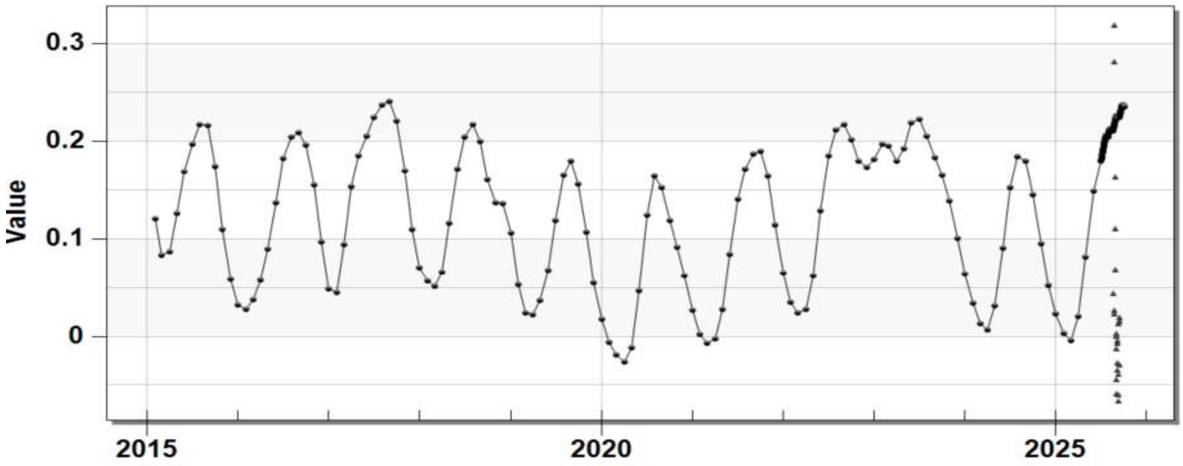

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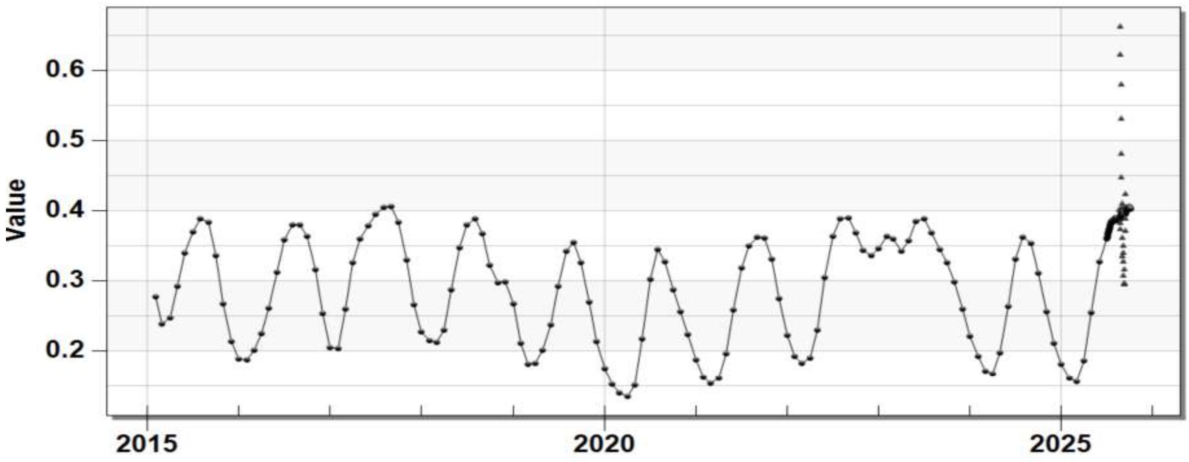

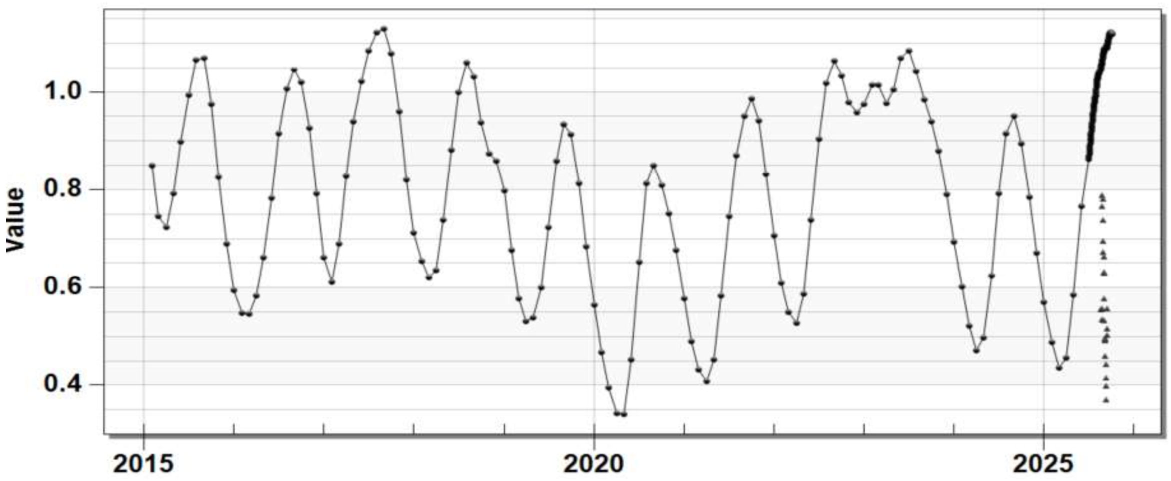

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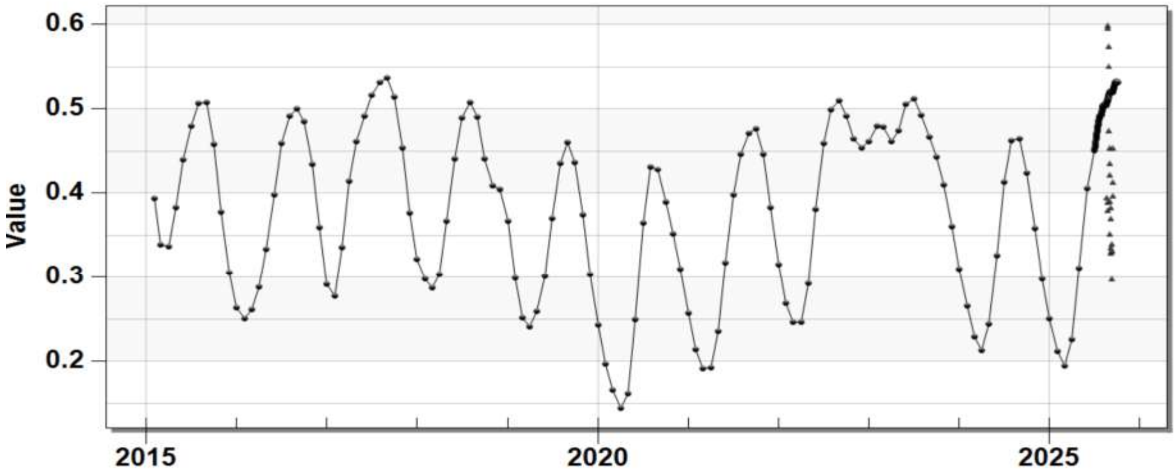

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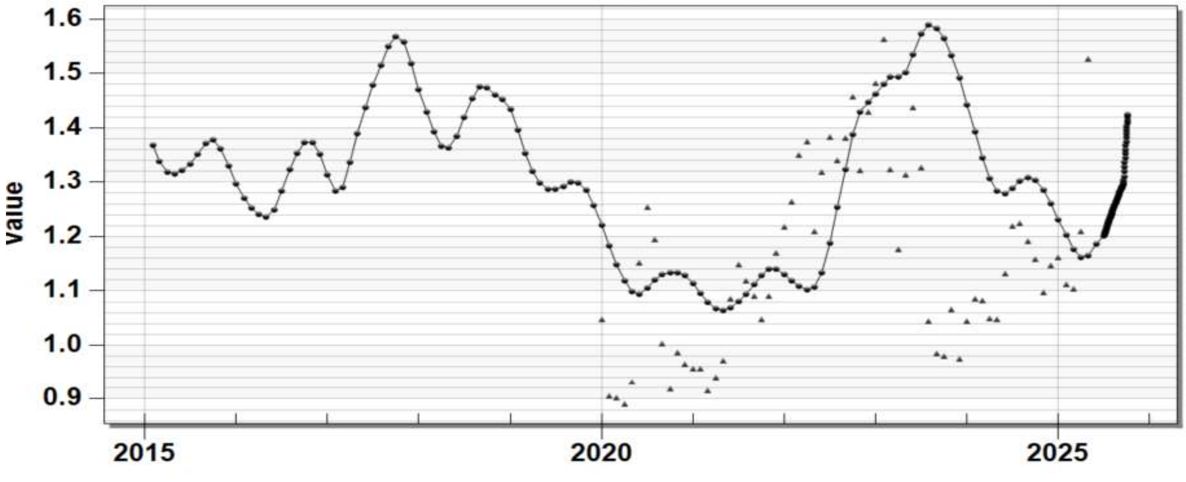

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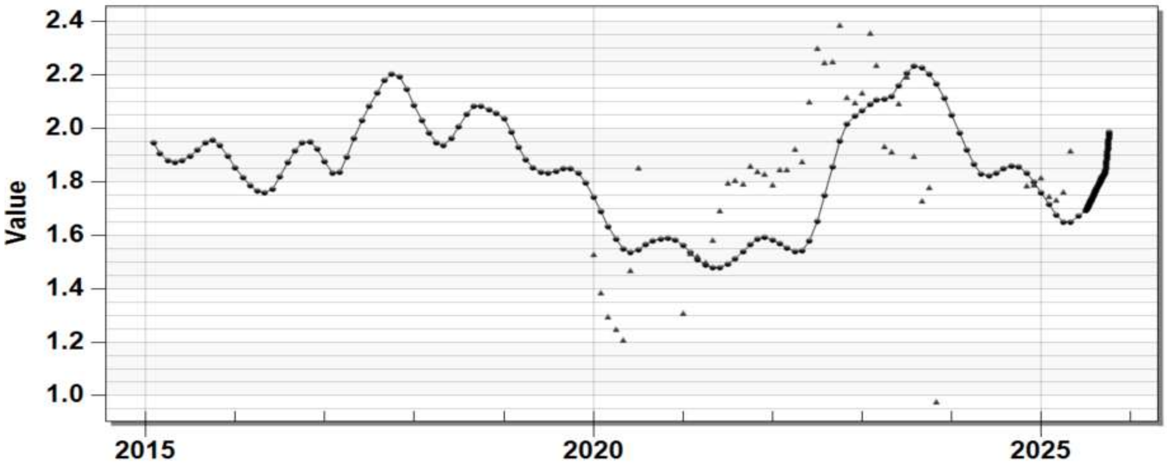
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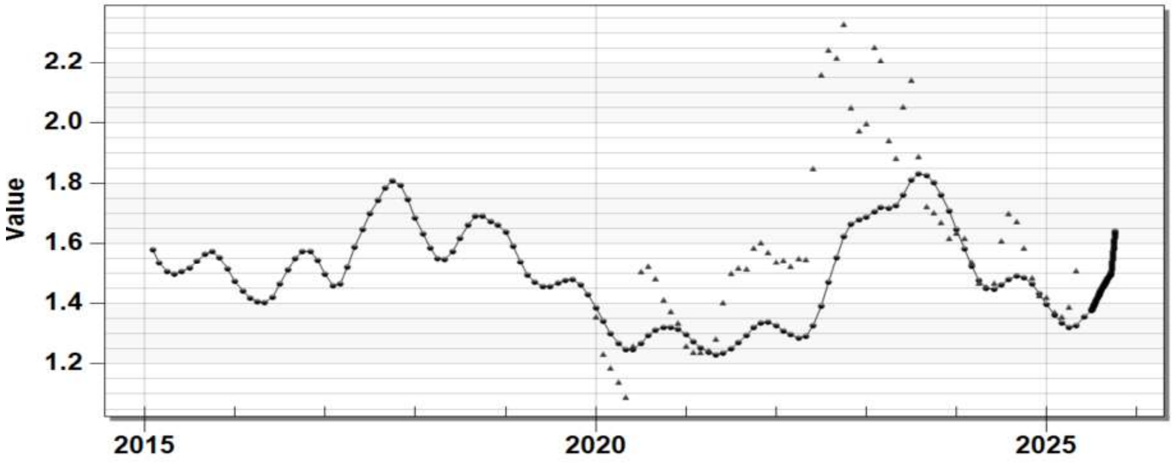
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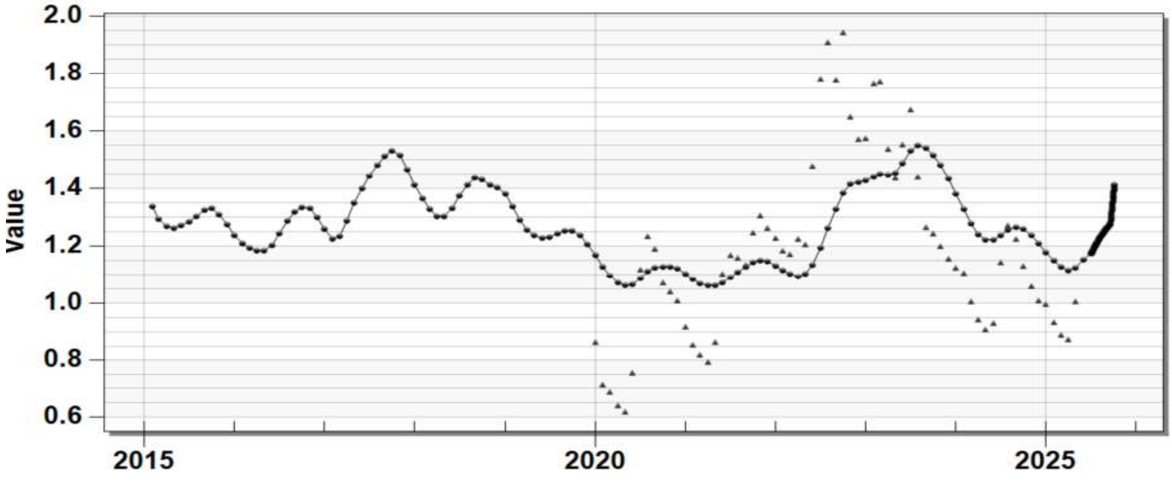
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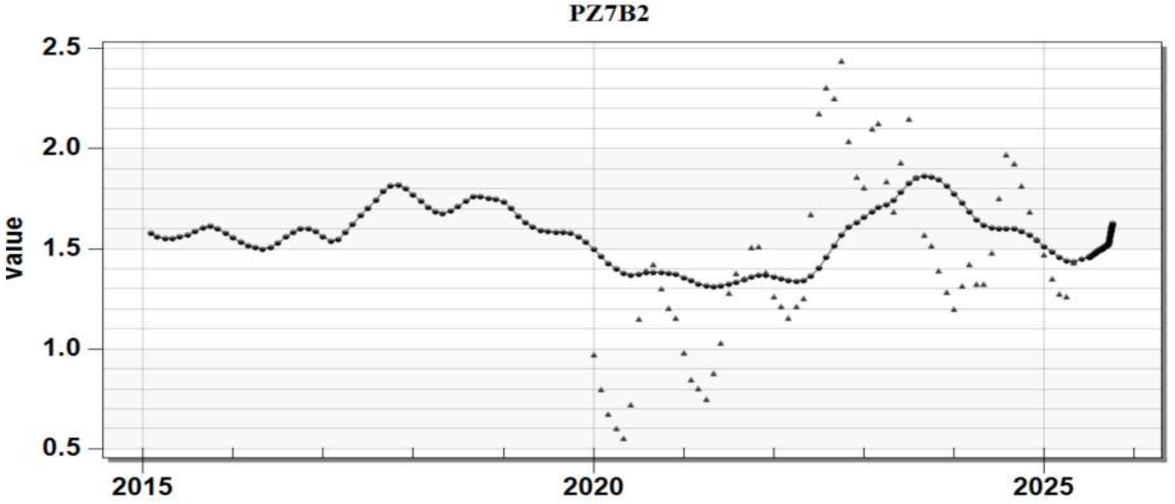
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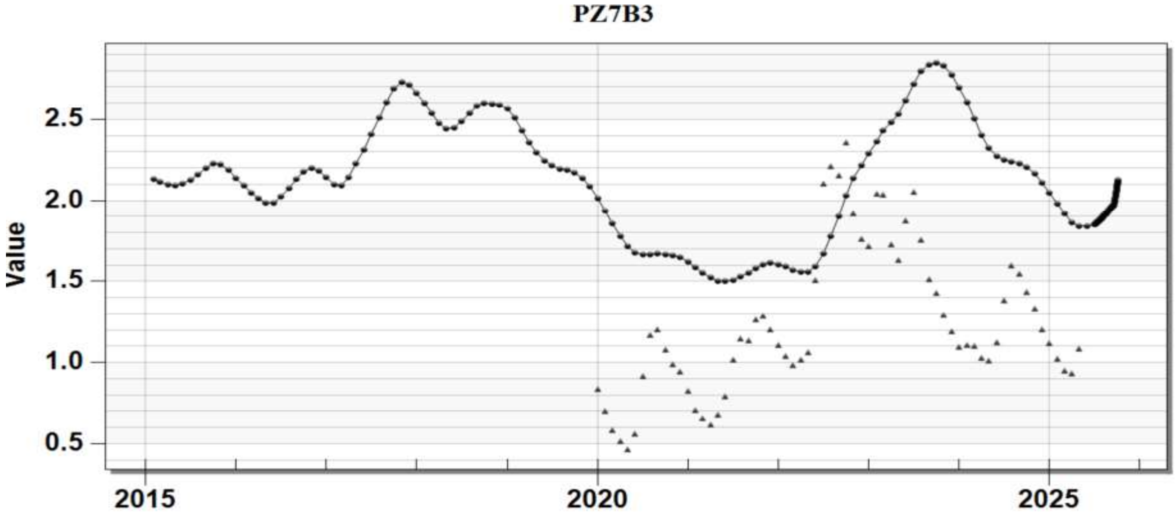
2015
2020
2025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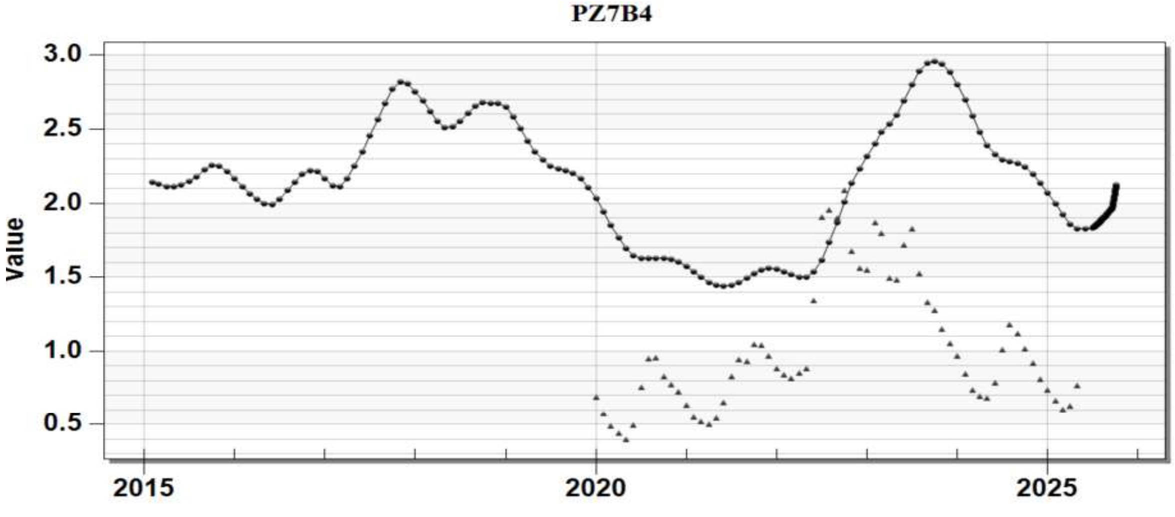
2015
2020
2025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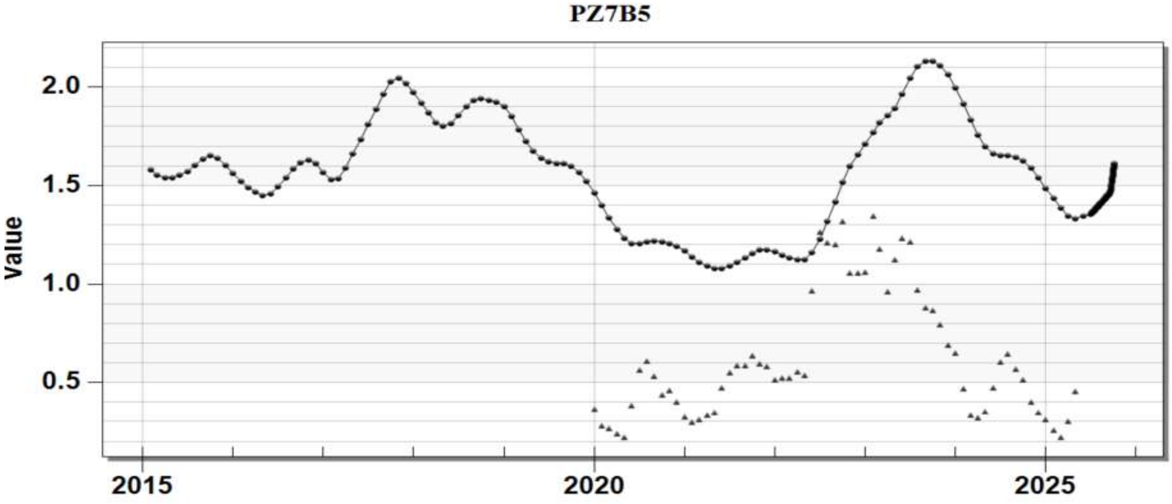
2020
2025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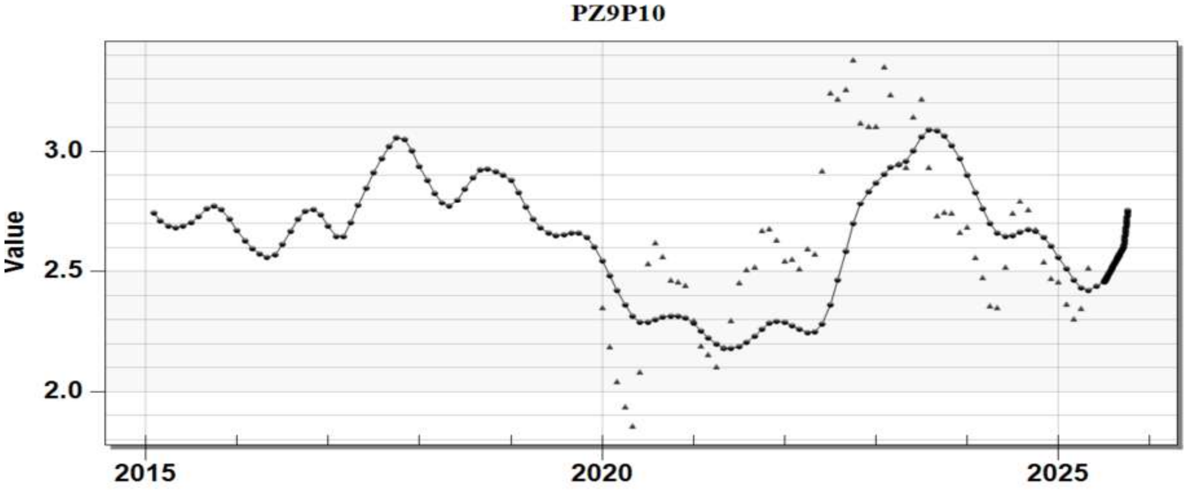
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2020
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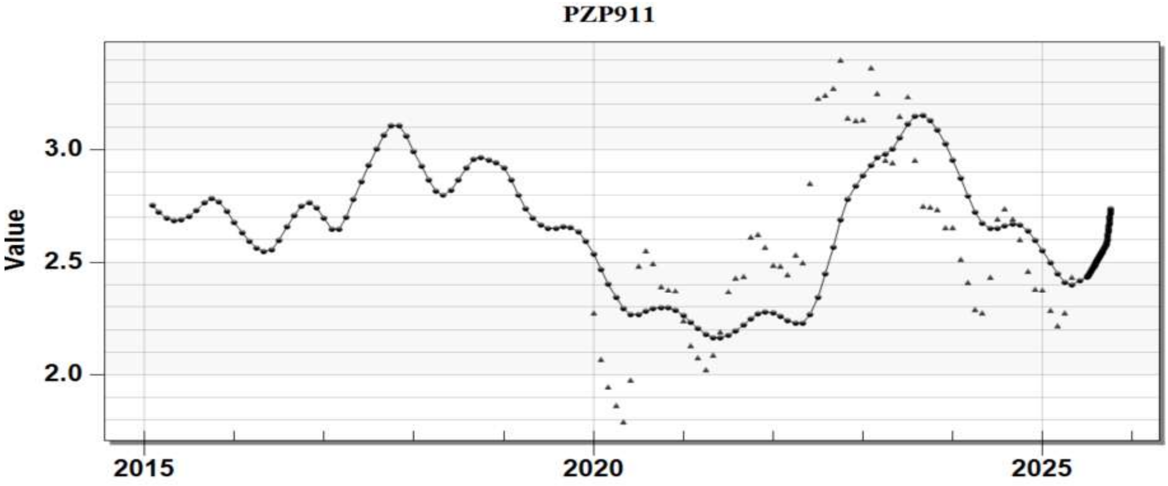
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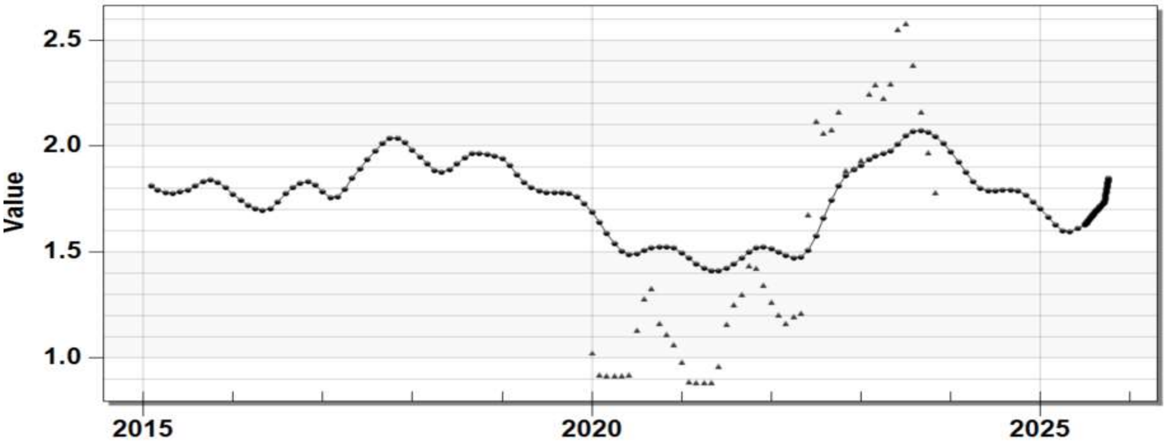
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2020
2025

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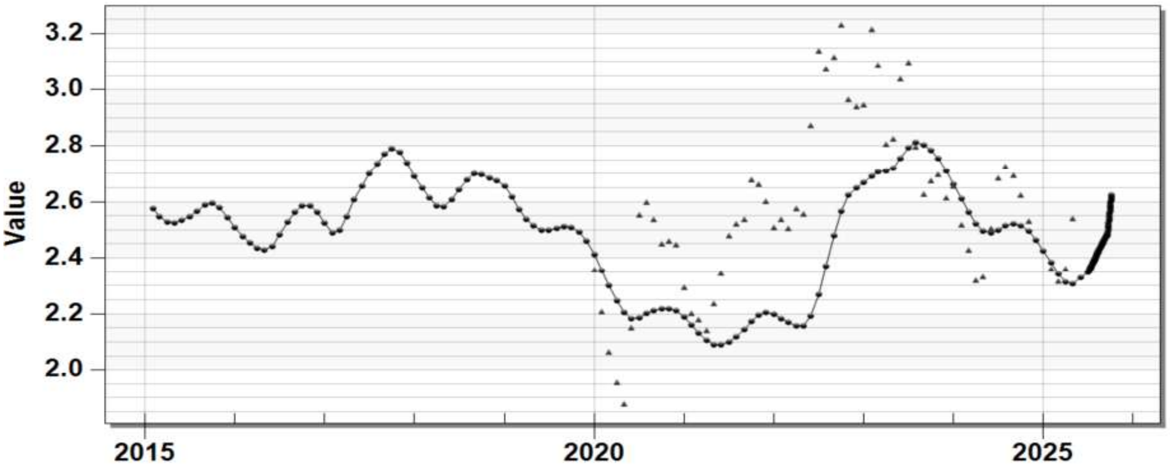
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2020
2025