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Waikato Thoroughbred Racing

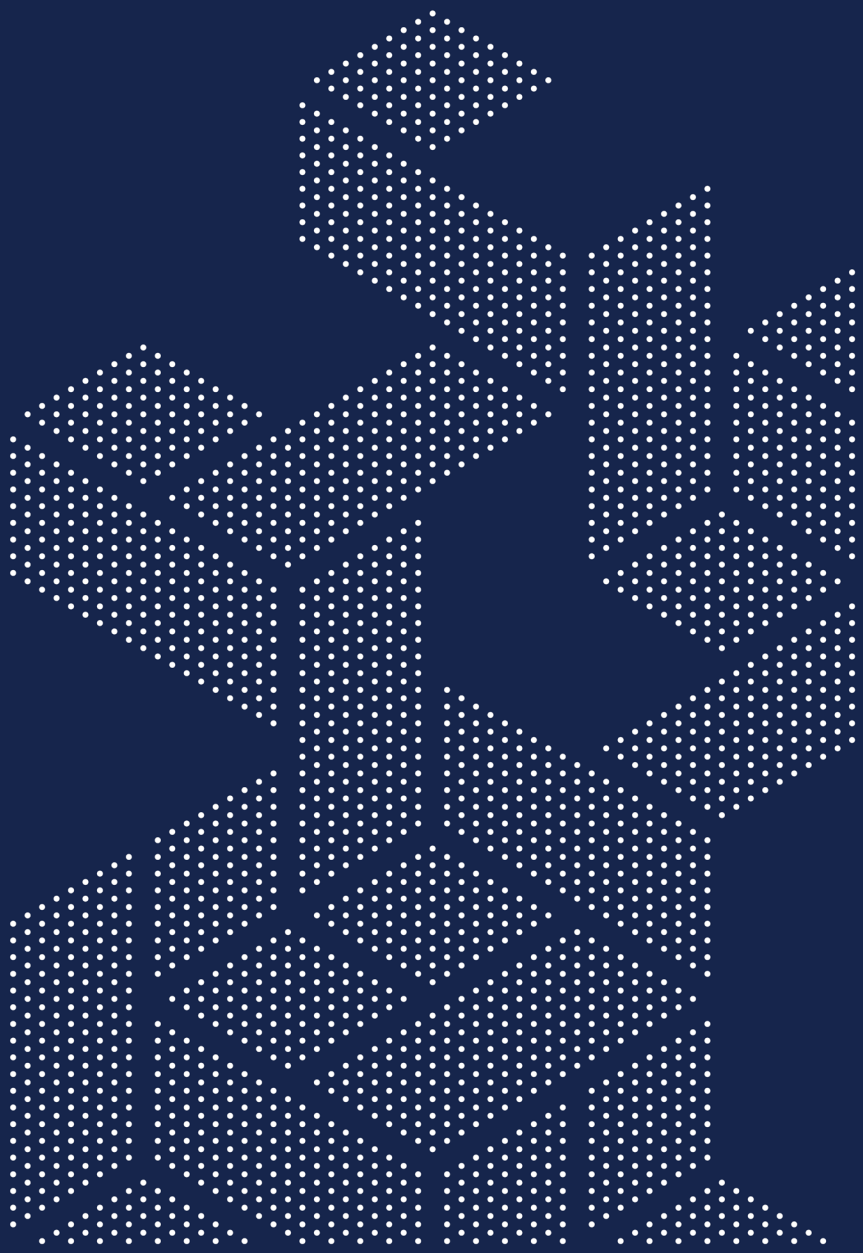
Hydrogeological Effects Assessment

ASSESSMENT OF EFFECTS

WGA253138

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September 2026



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1 EXECUTIVE SUMMARY

Waikato Thoroughbred Racing is seeking fast-track approvals for a new racing hub at Pencarrow Road, Tamahere. WGA has assessed the hydrogeological effects associated with groundwater abstraction for potable, stock, equine and track-conditioning supply; temporary construction dewatering; and long-term groundwater interception and diversion by below-ground infrastructure and water-storage ponds.

The proposed groundwater supply comprises two production bores. The calculated potable, stock and equine requirement is approximately 340 m^3/day (3.94 L/s) on a typical day and approximately **467 m^3/day** (5.40 L/s) on a peak race day. Further refinement of the minimum annual requirement by BCD indicated that the potable supply could be reduced to approximately 250 m^3/day as a long-term average based on 91,650 m^3/year. BCD's irrigation calculations indicate a peak seasonal irrigation groundwater requirement of up to **725 m^3/day** (8.39 L/s). A 24-hour constant-rate test of Test Bore 72_13219, screened in a deep aquifer from 51.5 m below ground level (Aquifer 3),at 345.6 m^3/day produced 1.9 m of drawdown and indicated a transmissivity of 75–106 m^2/day; storativity was conservatively assumed to range from 0.0001 to 0.001. A production rate of 500 m^3/day per bore is considered sustainable on the preliminary test results. Step-rate and three-day constant-rate testing of each production bore once drilled is recommended to confirm production capacity and aquifer parameters.

Groundwater abstraction modelling assessed concurrent operation of an irrigation take of 337 m^3/day (3.9 L/s) for 120 days and a potable, stock and equine supply take of 250 m^3/day (2.9 L/s) continuously for 10 years, giving a peak combined modelled abstraction of 587 m^3/day (6.8 L/s). These modelled rates are lower than the calculated operational requirements and were adopted to assess an operating regime that reduces interference effects on nearby bores. Predicted drawdown in pumped Aquifer 3 is up to 17.8 m at the irrigation bore and approximately 13.4 m at the potable-supply bore. Predicted drawdown in the shallow unconfined Aquifer 1 is no more than 0.4 m, while the conservative modelling of interference in Aquifer 2 predicts up to approximately 2.9 m near the bore. Interference was at or above 10% of the estimated available water column at four bores: 69_526, 69_1454, 69_1549, and 72_3229. Bore construction and water levels should be verified before confirming that effects on these users are no more than minor.

Temporary dewatering will be required for construction of the storage ponds, swales and wastewater infrastructure. For the ponds, the maximum modelled construction inflow is 876 m^3/day at the Main Pond, with a drawdown radius of approximately 19 m. For a 20 m swale construction section, the maximum modelled inflow is 86 m^3/day and the maximum drawdown radius is approximately 135 m. The wastewater pump-station wet wells have been assessed for a 15-day dewatering period, with target drawdown of 4.3 m at the Main WWPS and 3.9 m at the Trainer WWPS. Groundwater abstracted during dewatering will be discharged to the local drainage network and therefore returned to the same catchment in which it currently discharges. Effects on structures, groundwater users and surface-water features are expected to be localised and manageable, provided the proposed construction methodology, discharge controls and monitoring measures are implemented.

Long-term swale modelling predicts groundwater interception of approximately 352–459 m^3/day across the modelled swales, with the 0.25 m drawdown contour extending up to approximately 85 m. The Northern and Central East swales are predicted to cause no drawdown at the nearest assessed structures, while the Central West swale produces up to 0.6 m drawdown at the nearest structure. The ponds will be lined and are not expected to cause ongoing groundwater interception following construction.

The wastewater discharge assessment indicates that pathogen attenuation and protection of groundwater users depend on the final treatment performance, disposal loading and separation distances. These matters should be confirmed against the final wastewater design and incorporated into the consent conditions and operational monitoring requirements.

Pre-construction groundwater monitoring will use piezometers P2, P3, P7 and P8, together with one onsite barometric logger, recording at 15-minute intervals until construction commences. The same network will continue through relevant construction dewatering and groundwater-diversion works. Monitoring results will be compared against the baseline record, with unexpected or sustained changes triggering hydrogeological review and, where necessary, adjustment of dewatering methods or rates.

Overall, the modelling indicates that the proposed groundwater take, temporary construction dewatering and long-term groundwater interception can be managed without unacceptable hydrogeological effects, subject to the recommended production bore testing, verification of the four potentially affected bores, pre-construction and construction-phase groundwater monitoring, appropriate discharge controls and adaptive management. The results are conservative and represent upper-bound operating and dewatering scenarios; actual effects are expected to be lower where pumping is intermittent and works are staged.

2 INTRODUCTION

2.1 Background

A fast-track resource consent is being sought for the construction of a racetrack facility by Waikato Thoroughbred Racing.

WGA understands that the racetrack facility is proposed as a modern racing hub, intended to support the long-term future of the industry, providing infrastructure for New Zealand horse racing. The proposed site (Site) will provide for regional and national events. The Site is currently two dairy farms covering 163.9 ha and is bound on the northwest edge by State Highway 1 (Figure 1). During a normal day, up to 200 people are expected to be onsite, with numbers increasing to approximately 500 people during a standard race day and up to 5,000 people during feature race days. Horse numbers are expected to be a minimum of 200 on a typical day, increasing to 900 horses during an event.

A groundwater supply will be required for drinking water, stock water and track maintenance and conditioning of the proposed thoroughbred racetrack facility. In addition, there are proposed earthworks and wastewater disposal that could potentially influence local groundwater levels and quality.

2.2 Site Description

The Site is located in the Tamahere area, approximately 7 km southeast of the Hamilton urban boundary and 5 km northwest of Cambridge. It comprises approximately 163.9 ha of predominantly rural land at 592, 536 and 636 Pencarrow Road, 37 Hooker Road, and 38 and 90 Duncan Road. The Site is bounded by Pencarrow Road to the west, Hooker Road and Duncan Road to the south and southeast, and the Waikato Expressway/State Highway 1 along the northwestern margin. Existing access is available from Pencarrow Road, Hooker Road and Duncan Road. The Site location is shown in Figure 1.

The land is currently used primarily for dairy farming and comprises pasture, farm races, drains, scattered mature trees and several farm buildings. The Site is generally level to gently undulating, with an overall fall of approximately 4 m from the southeast towards the northwest. A low rolling hill, typical of the Waikato Basin, is present near Pencarrow Road along the western boundary. Existing ground levels are generally approximately 53 m to 57 m above mean sea level (RL) across the main Site and approximately 54.5 m to 59 m RL along the Pencarrow Road corridor.

Surface drainage is provided by a network of shallow farm drains and overland flow paths. Runoff generally flows from the southern and eastern parts of the Site towards the north and northwest. The Site is outside the Waikato District Council mapped flood-hazard areas; flood effects are addressed separately in the project flood assessment.

The Site is not connected to Council reticulated wastewater or stormwater networks. Existing wastewater and stormwater are understood to be managed locally through onsite systems, including septic tanks, disposal fields, soakage and farm drainage. Two existing groundwater-take consents support dairy-shed washdown and stock-water activities. These existing rural activities will be replaced progressively by the proposed racing hub, supporting equine, commercial, residential and associated infrastructure development.

2.3 Proposed Activity

This report provides assessment of the following proposed activities to be carried out in the development of the Site shown in Figure 2:

- A groundwater take to supply water to the development.
- Three storage wetlands, and a site wide swale system to manage stormwater and water storage.
- Two wastewater pump stations (WWPS) and site-wide wastewater pipeline infrastructure.
- Wastewater disposal areas.

The proposed development has three main groundwater related components:

1. **Drawdown from groundwater abstraction from bores for water supply and irrigation**.
2. **Short-term temporary dewatering during construction earthworks**, for proposed water storage ponds, the underpass and wastewater infrastructure (site-wide pipelines and wastewater pumping stations). Dewatering is anticipated to be localised and temporary, with groundwater managed and discharged in accordance with the site erosion and sediment control framework (namely the Erosion Sediment Control Plan (ESCP)) and relevant consent condition requirements.
3. **Long term drawdown effects of the proposed swales.**
4. **Water quality effects of the proposed wastewater disposal.**

The key groundwater effects to be considered in the hydrogeological assessment for the groundwater bore take include aquifer sustainability, effects on nearby groundwater users, effects on nearby streams and wetlands, and drawdown effects extending to neighbouring structures.

2.4 Proposed Site Water Use

Waikato Thoroughbred Racing intend to drill a deep production bore, or multiple bores, to provide a reliable water supply. WGA understands that the **potable, stock and equine** water requirements are:

- Typical daily potable, stock and equine demand; approximately 340 m^3/day (3.94 L/s), comprising approximately 290 m^3/day for potable, stock and other non-irrigation uses and 50 m^3/day for horse drinking water.
- Peak race-day potable, stock and equine demand of approximately 467 m^3/day (5.40 L/s), including approximately 127 m^3/day (1.47 L/s) for race-day visitors.
- Maximum annual potable, stock and equine abstraction volume of 91,650 m^3/year

Continuous potable, stock and equine abstraction rate adopted for groundwater modelling is 250 m^3/day (2.9 L/s), representing year-round demand (BCD pers. comm., 27 August 2026). This annual volume is based on:

- 320 days at 240 m^3/day, equivalent to 76,800 m^3/year.
- 45 days at 330 m^3/day, equivalent to 14,850 m^3/year.
- Total annual abstraction of 91,650 m^3/year, equivalent to an average of approximately 250 m^3/day.

In addition, Waikato Thoroughbred Racing is seeking groundwater from a separate production bore for **seasonal irrigation and track conditioning** requirements, as follows:

- Abstraction for up to 120 days, representing a four-month summer irrigation and track-conditioning season of up to 725 m^3/day based on BCD (2026) breakdown of irrigation requirements, equivalent to 87,000 m^3/year.
- Waikato Thoroughbred Racing are proposing that groundwater will be abstracted steadily to lined onsite storage ponds, rather than at short-duration peak irrigation rates, for subsequent irrigation and conditioning of the racing surfaces.

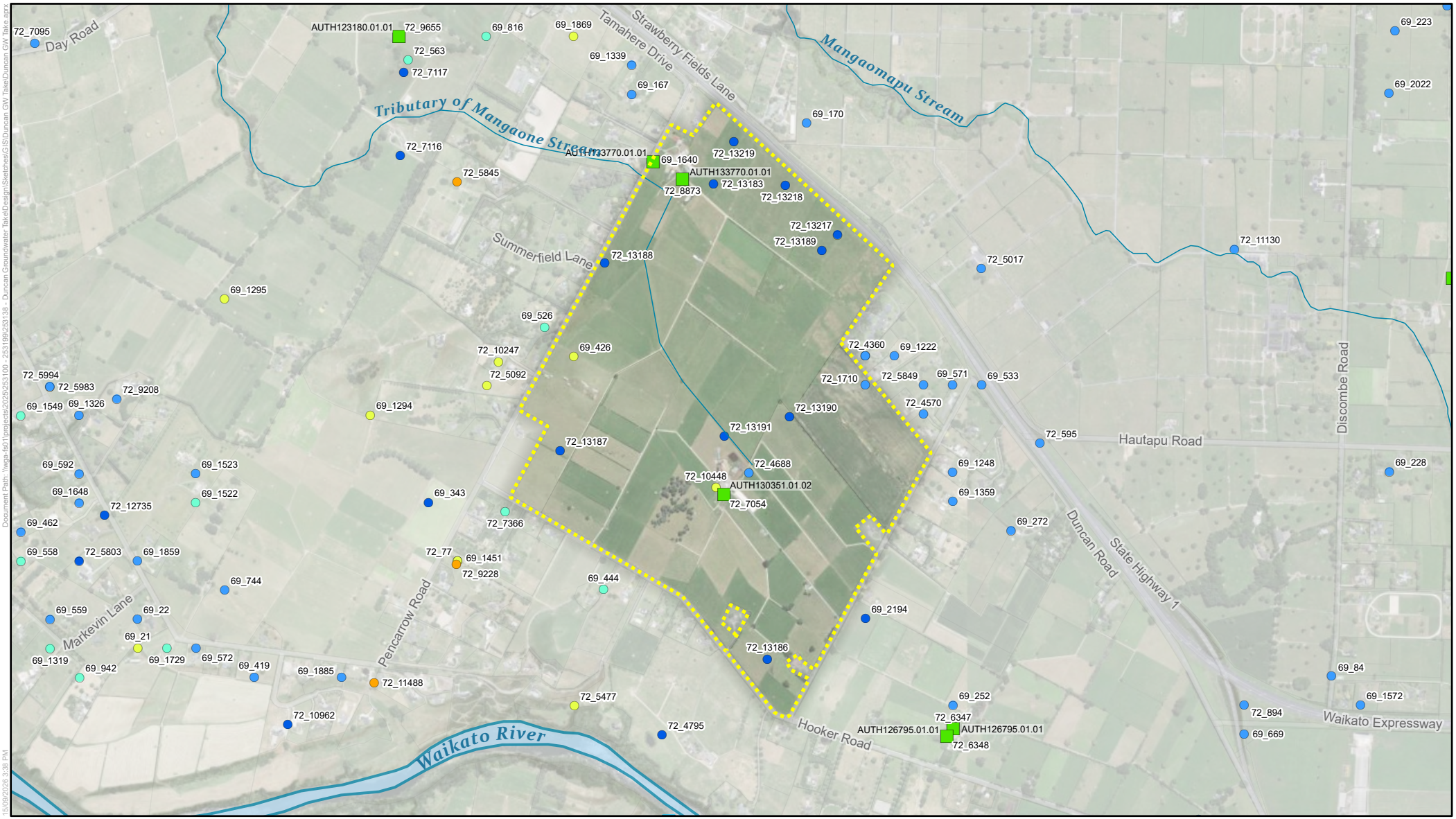
The modelled irrigation abstraction rate is 337 m^3/day (3.9 L/s), corresponding to a maximum modelled seasonal volume of 40,440 m^3 over 120 days. This is lower than the calculated peak seasonal irrigation requirement of up to 725 m^3/day because the higher rate produced unacceptable modelled interference at numerous nearby bores. The combined modelled abstraction is therefore 587 m^3/day (6.8 L/s) when the seasonal irrigation take operates concurrently with the continuous 250 m^3/day potable, stock and equine take. Waikato Thoroughbred Racing is investigating a deeper groundwater source, which may allow a higher irrigation abstraction rate without increasing effects on nearby bores.

The application is seeking up to the following volumes, if the deeper production bore can achieve a higher rate without causing an interference effect:

- Potable, stock and equine peak demand of approximately **467 m^3/day** and **91,650 m^3/year**
- Irrigation and track-conditioning water use of up to **725 m^3/day** and **87,000 m^3/year**

This hydrogeological assessment is for:

- The effects of operating a production bore and the supply of water for the development.
- The effects on construction and operation of stormwater and wastewater infrastructure for the proposed development.



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Figure **1**

Waikato Thorough Bred Fast Track Development

Location of the Proposed Development



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LEGEND

- Proposed Irrigation Bore
- Swale
- Streams

Contours (m)

- 51 - 54
- 54 - 56
- 56 - 58
- 58 - 60
- 60 - 61
- Track
- River
- Site Boundary
- Storage Ponds
- Wastewater Reticulation, Land Application or Treatment Area
- Wastewater Pipelines

N

0 100 200 300 400 m

Scale 1:12,500 @ A4

Coordinate System: NZGD 2000 New Zealand Transverse Mercator

WGA

Figure **2**
Ponds, Swales and Wastewater Infrastructure

Waikato Thoroughbred Fast Track Development

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3 EXISTING ENVIRONMENT

3.1 Rainfall

Rainfall data are recorded at the Ruakura Weather Station, which is located approximately 10 km north of the proposed development. The available record spans from November 2005 to the present, with a peak daily rainfall of 87.8 mm recorded during this period (Figure 3). The long-term mean annual rainfall at the station is approximately 1,121 mm.

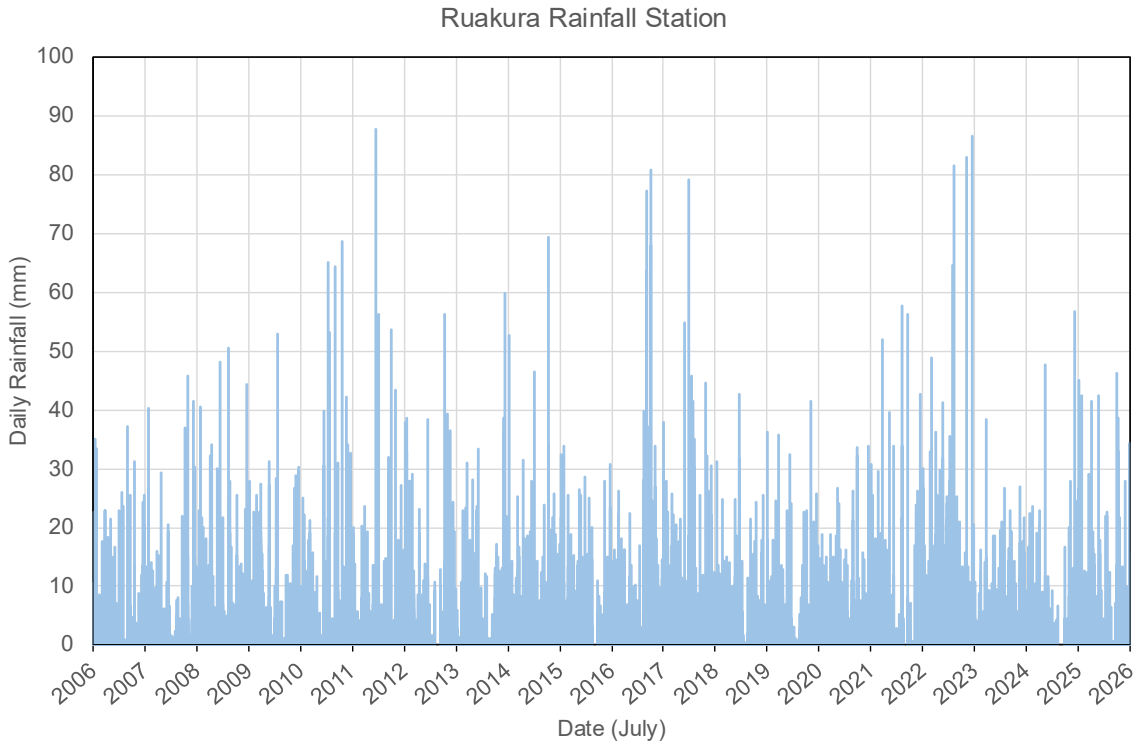


Figure 3: Rainfall Record at Ruakura Site

3.2 Surface Water Bodies and Catchment Boundary

The Site is within the lower Waikato River tributary catchment network near Tamahere. Surface drainage is controlled by shallow artificial watercourses and overland flow paths across the Site, which generally fall from the southern and eastern parts of the property towards the north, west and northwest. The principal onsite drainage line begins near Duncan Road, crosses the central part of the Site and flows west beneath Pencarrow Road before continuing through lower-lying land and ultimately connecting to the Mangaone Stream (Figure 4). The northern part of the Site drains to the artificial drainage corridor along the southern side of State Highway 1, which conveys flow westward before discharging toward the Mangaomapu Stream and then the Mangaone Stream catchment. Western and southern sub-catchments drain more directly west and southwest toward the Waikato River via minor local drainage paths. The ecological assessment (Awa 2026) records that the onsite watercourses have been classified, with Waikato Regional Council agreement, as artificial watercourses. However, they still provide hydraulic connectivity to downstream receiving environments, including the Mangaonua and Mangaone Stream systems and, ultimately, the Waikato River.





LEGEND

- Streams
- River
- Catchment
- Proposed Development Boundary

N

0 250 500 750 1,000 m

Scale 1:30,000 @ A4

Coordinate System: NZGD 2000 New Zealand Transverse Mercator

WGA

Figure **4**
Drainage and
Catchment Boundaries

Waikato Thoroughbred Fast Track Development

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3.3 Geology and Hydrogeology

3.3.1 Regional Geology

The Hamilton Basin, in which the Site is located, is a large tectonic basin centred on Hamilton City with an area of approximately 2,000 km^2 and traversed by the Waikato River. The basin is surrounded by ranges of Mesozoic (Manaia Hill Group) and Tertiary age rocks (Te Kuiti and Waitemata Groups). At depth, basement greywacke underlies the sedimentary deposits that infill the basin (GNS, 2005).

The basin is infilled with Tauranga Group alluvial sediments, dating from the Pliocene to the middle Holocene. The Tauranga Group sediments are up to 300 m thick and include gravels, sands, silt, muds and peats of fluvial, lacustrine and distal ignimbritic origin. Underlying the low hills within the basin are older ignimbrites, tephra fall deposits and alluvium of the Walton Subgroup (Lowe 2021; Figure 5).

The younger Hinuera Formation of the Tauranga Group underlies much of the Hamilton basin (Figure 5). This formation was deposited by braided river systems of the Waikato River, initiated by the supply of large volumes of sediment from volcanism in the Taupo Volcanic Zone (Petch, 1987). Within this formation, the most productive aquifers consist of well-sorted coarse sands and gravels. Discontinuous sequences of rhyolitic and pumiceous gravelly sands and gravels are interspersed with pumiceous silt, clay and peat layers. Lithological variability generally results in zones of higher permeability within the formation rather than a single, continuous aquifer (Scholfield, 1972). Low-lying areas comprise younger Holocene Swamp Deposits of the Tauranga Group, such as the Piako Subgroup, locally consisting of peat.

The Geology of the Waikato Area Map (Edbrooke, 2005) indicates that the Project area is predominantly underlain by sediments of the Hinuera Formation (Figure 6). Walton Subgroup sediments are exposed in elevated ridges along the northern site boundary extending to the north of the Site. An isolated outcrop of the Manaia Hill Group sandstone and siltstone basement rocks is located 350 m southwest of the site.

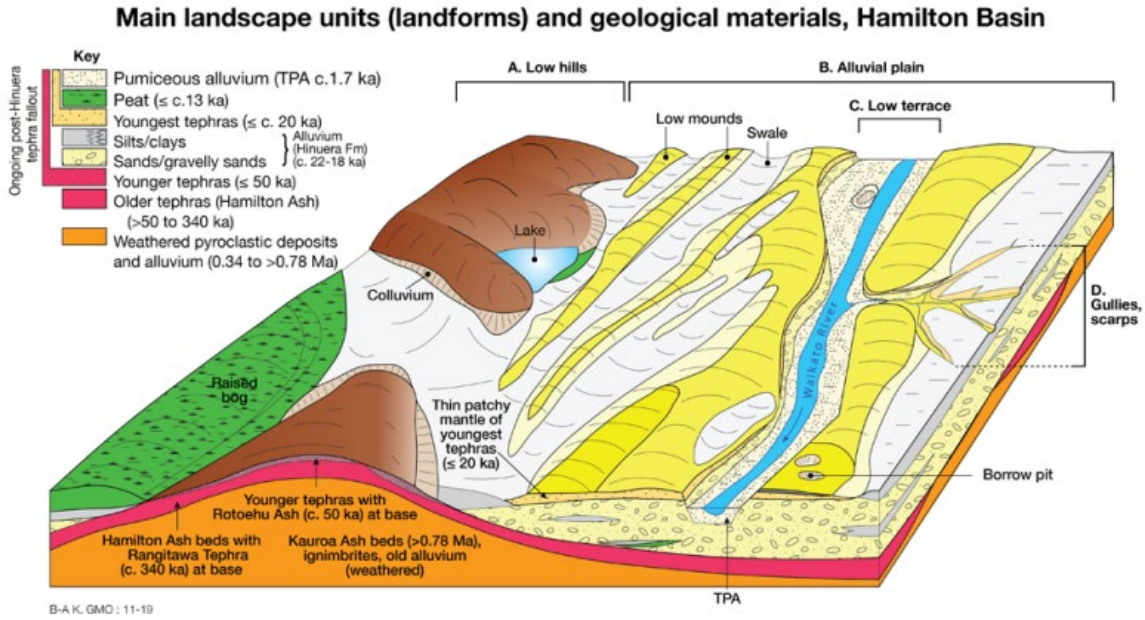


Figure 5: Geological Setting of the Hamilton Basin (Lowe, 2021)

3.3.2 Aquifer Geometry and Aquifer Characteristics

The Tauranga Group sediments contain the aquifers used most extensively across the Hamilton Basin. Within these formations, the most productive aquifers consist of well sorted coarse sands and gravels. Lithological variability generally results in a number of zones of higher permeability within each of these formations rather than single continuous aquifer (Schofield, 1972). The shallow aquifer in the Hinuera Formation is named the Hinuera Aquifer and is used locally as a water source by landowners.

Literature values for hydraulic conductivity in the Tauranga Group in the Hamilton Basin range from 0.5 m/day in the silts and peat layers to 13.5 m/day in the coarse gravelly sands. Aquifer transmissivity values derived from pumping tests range from 10 m^2/day to 1,000 m^2/day but are usually less than 100 m^2/day. Aquifer transmissivities tend to be lower in the deeper deposits as they are older, more intensely weathered and have a greater proportion of clays and silts (Marshall & Petch, 1985). Storativity values vary from 0.0001 for deeper, confined or semi-confined aquifers to 0.1 for shallower, unconfined aquifers in the Hamilton Basin (Petch & Marshall 1988). In some areas, these discontinuous aquifers may yield up to 30 L/s (Petch, 1987).

The piezometric surface in the Hamilton Basin is closely related to surface topography. Piezometric gradients beneath un-dissected areas of the Hamilton Basin surface have a gentle slope that steepens near incised stream channels. Groundwater is recharged from rainfall, predominantly during the winter when soil moisture deficits are satisfied. The shallow groundwater subsequently discharges to the incised streams and the Waikato River. Isotope analyses suggest that the groundwater flux is mainly through shallow aquifers. Groundwater in deeper aquifers within the basin is significantly older, having been dated at up to 6,500 years old (Marshall and Petch, 1985).

3.3.3 Local Shallow Lithology

BCD's shallow geotechnical investigations comprised 20 hand augers, generally advanced to between about 2 m and 3 m below ground level (bgl), together with 12 cone penetration tests (CPTs) extending to approximately 20 m bgl.

Across most of the site, the hand-auger logs record about 0.1 m to 0.3 m of topsoil over a shallow, commonly stiff to very stiff silt layer, generally about 0.3 m to 0.5 m thick (Table 1). This is underlain by highly variable, interbedded Hinuera Formation sands and silts. The sandy intervals range from fine to coarse, locally gravelly or pumice-bearing, and from loose to dense. Thin silt, clay and occasional organic-silt lenses occur within the sequence. The CPT profiles indicate that these interbedded sand-dominated deposits continue to approximately 17 m bgl (with a relatively consistent aquitard at approximately 12 to 17 m bgl), with laterally discontinuous silt and clay lenses. On the elevated low hill near the western boundary, HA06 encountered clayey silt attributed to the Walton Subgroup from beneath the topsoil to the 3 m investigation depth, and HA16 also encountered predominantly fine-grained Walton Subgroup materials. CPT logs indicate that the upper aquifer at the site extends to between 12 m and 17 m below ground surface, with a relatively continuous layer of interbedded silts and clays below.

Groundwater was generally encountered within the shallow Hinuera Formation aquifer at approximately 0.9 m to 2.8 m bgl during the November 2025 investigations, although deeper or absent groundwater was recorded locally in elevated ground.

Table 1: Shallow Site Stratigraphic Units

UNIT	DEPTH TO TOP OF SOIL UNIT (m)		THICKNESS (m)	
	Min	Max	Min	Max
Topsoil	0		0.1	0.5
Interbedded loose to dense sands and silts (Hinuera Formation)	0.1	0.5	>1.3	>19.5
Clayey silt (Hamilton Ash/Walton Subgroup)(1)	0.1	0.3	>2.4	>2.9

Note:

(1) Base of unit not encountered.





LEGEND

- Streams
- River
- Proposed Development Boundary

Geological Units

- OIS1 (Holocene) river deposits
- OIS1 (Holocene) river deposits (Taupo Pumice Alluvium)
- OIS14+-OIS12 (Early Pleistocene to Middle Pleistocene) river deposits and ignimbrite
- OIS3-OIS2 (Late Pleistocene) river deposits (Hinuera Formation)
- Manaia Hill Group sandstone and siltstone (Waipapa Composite Terrane)

N

0 250 500 750 1,000 m

Scale 1:30,000 @ A4

Coordinate System: NZGD 2000 New Zealand Transverse Mercator

WGA

Figure **6**
Surficial Geology

Waikato Thoroughbred Fast Track Development

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3.3.4 Local Hydrogeology

3.3.4.1 Hydrogeological Site Investigation

Ten shallow piezometers were installed by BCD across the Site during the November 2025 geotechnical investigation under Waikato Regional Council consent AUTH148356.01.01. Four piezometers were subsequently decommissioned on the request of the landowner. An overview of the currently operational piezometer as-built layouts is provided in Table 2.

Table 2: Piezometer Construction Details

PIEZOMETER	HA NUMBER	PROPOSED DEPTH (m bgl)	MEASURED DEPTH (m bgl)	CASING DIAMETER (mm)	SCREEN LENGTH (m)	EASTING (NZTM)	NORTHING (NZTM)
P2	HA09	2.0	2.0	30	1.6	1809890	5806999
P3	HA04	3.2	3.0	30	1.6	1810033	5806206
P7	HA03	2.0	2.0	30	1.6	1810113	5807065
P8	HA16	2.7	3.0	30	2.6	1809613	5807883
P9	HA15	4.2	4.1	30	3.7	1809429	5807053
P10	HA20	3.0	2.5	30	2.1	1810486	5806767

Typical piezometer construction comprised a 30 mm diameter hand slotted piezometer pipe, capped at the base, with a sand or pea-metal filter pack around the screened interval and bentonite clay above the filter pack. The headworks arrangements were a flush installation beneath a 300 mm × 300 mm paving stone, set 20 mm below ground surface within a 100 mm protective pipe and located approximately 200 mm to 400 mm from a fence line.

The piezometers were installed to depths between 2 m and 4.1 m bgl within the shallow Hinuera Formation sediments and were periodically monitored to characterise groundwater levels across the proposed development.

3.3.4.2 Groundwater Levels

Groundwater levels were recorded where encountered in the open hand auger and CPT holes during the geotechnical investigation. Test locations are shown in Figure 8, and recorded levels are presented in Table A1 in Appendix A and ranged from approximately 0.9 m to 4.2 m bgl, with the shallowest levels generally in the northeastern part of the Site and progressively deeper levels towards the west and south, partly reflecting variations in ground elevation. Converted to elevations, groundwater was generally between approximately 50.4 m RL and 56.2 m RL.

BCD monitored shallow groundwater levels across the Site in six piezometers using manual and continuous groundwater level readings collected at variable intervals over the monitoring period. The data recorded is illustrated in Figure 7.

The records indicate generally shallow groundwater, typically approximately 0.9 m to more than 2.8 m bgl, with most piezometers showing relatively modest temporal fluctuations of about 0.2 m to 0.3 m and short-term rises following rainfall. P3 and P9 became dry during the monitoring period, indicating that the water table fell below the bases of these shallow piezometers at times.

P2 shows greater and more rapid variability than the other piezometers, which appears to reflect the response of the shallow groundwater level to water levels in the adjacent drain rather than broader aquifer wide groundwater level changes. Accordingly, the P2 record should be interpreted as locally influenced by surface drainage conditions.

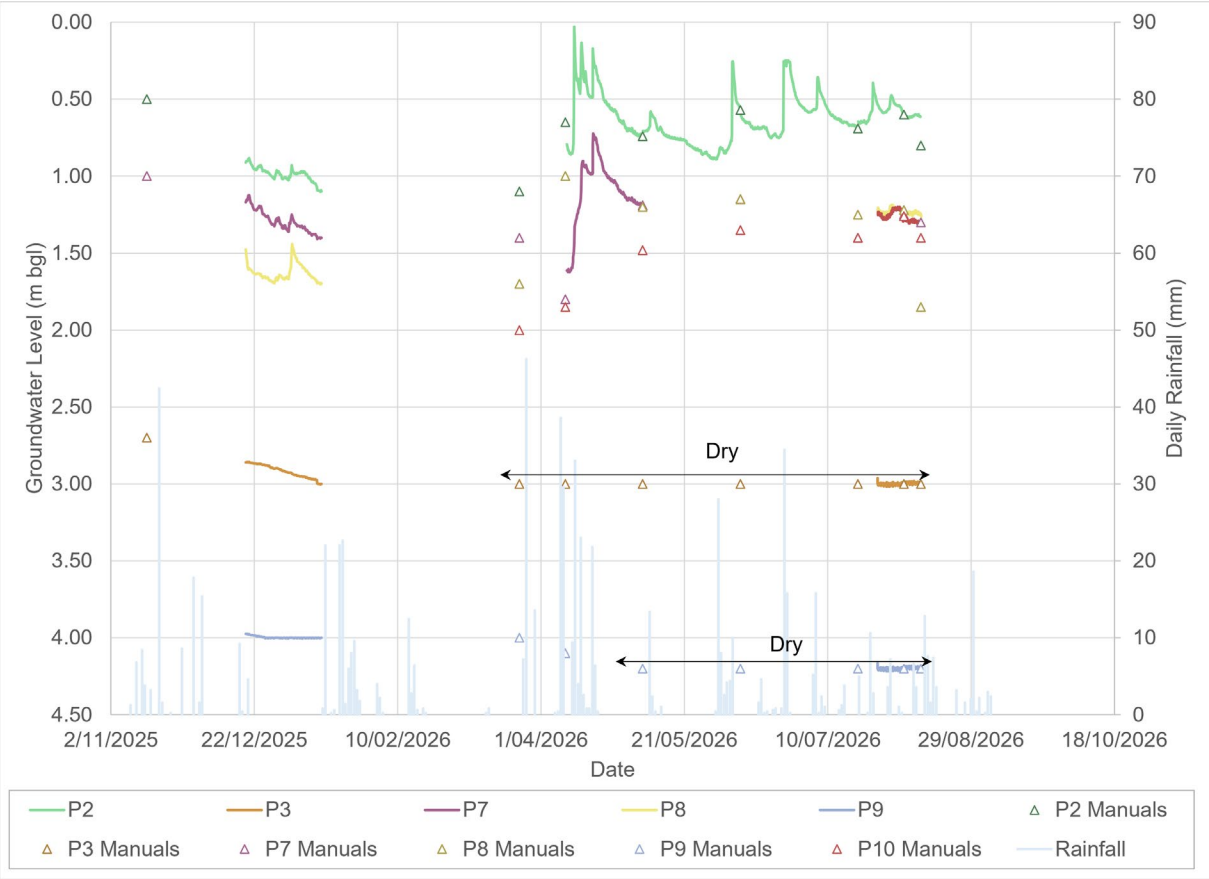
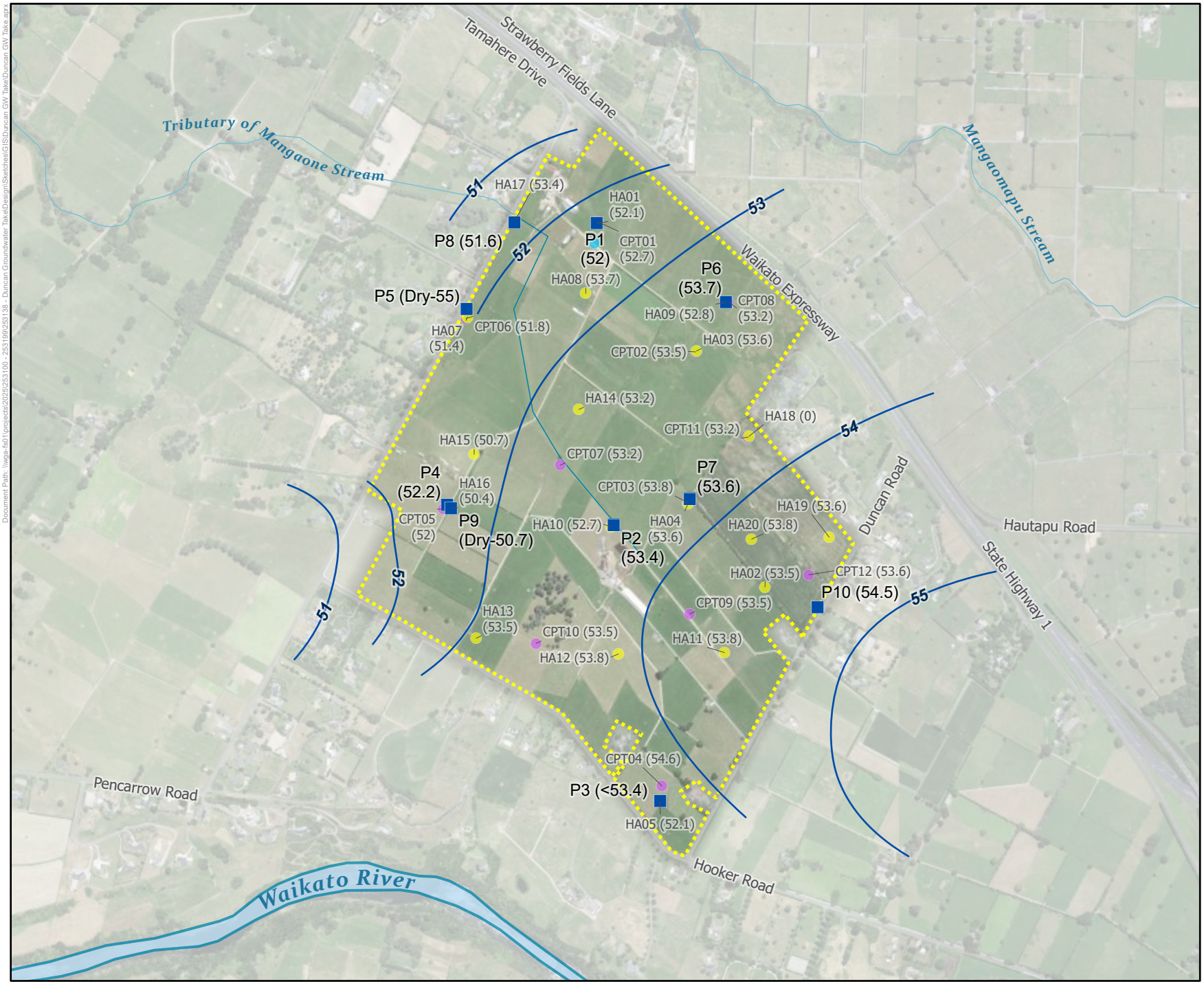


Figure 7: Monitored Groundwater Levels Supplied by BCD

Site observations and the geotechnical and hydrogeological investigations undertaken in the area were used to estimate a preliminary winter piezometric surface (Figure 8). WGA notes that some of the geotechnical groundwater data used to produce the piezometric surface does not align temporally. Consequently, the surface created is based on our interpretation of the data and general understanding of the Site hydrogeological setting.



LEGEND

- Piezometer
- Bore Hole
- CPT
- Hand Auger
- Piezometric Surface Contour (m RL)
- Streams
- Site Boundary
- River

Notes:
(xx) groundwater level measurement (m RL)

N

0 100 200 300 400 m

Scale 1:15,000 @ A4

Coordinate System: NZGD 2000 New Zealand Transverse Mercator

WGA

Figure **8**
Piezometric Surface

Waikato Thoroughbred Fast Track Development

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3.4 Hydrogeological Parameters

Hydraulic conductivity values for the Hinuera Formation sediments at the Site were determined by analysing slug tests performed in the four piezometers that contained a sufficient water column for testing (Table 3). Falling-head and rising-head tests were analysed where suitable data were available. The datasets were analysed using the Bouwer and Rice (1976) method in AQTESOLV Pro v4.5, developed by HydroSolve Inc. The analysis sheets are presented in Appendix B.

Table 3: WGA Hydraulic Test Results Summary

PIEZOMETER ID	TESTED INTERVAL (m bgl)	SCREENED LITHOLOGY	HYDRAULIC CONDUCTIVITY [1] (m/second)
P2	0.4 to 2.0	SILT, to sandy SILT, to fine SAND, some silt	7.97×10^{-8} to 1.75×10^{-7}
P7	0.4 to 2.0	Interbedded sandy SILT and silty SAND	5.02×10^{-6} to 4.58×10^{-5}
P8	0.4 to 3.0	Sandy SILT, to SILT with increasing Clay content with depth	6.13×10^{-7} to 7.84×10^{-6}
P10	0.4 to 2.5	SAND, trace silt	4.40×10^{-6} to 1.45×10^{-5}

Notes:

[1] All analyses completed using the Bouwer & Rice (1976) method.

The WGA analysis results were used to derive appropriate hydraulic conductivity values for the principal sedimentary units, as summarised in Table 4. For lithological units without site-specific data, values derived from published literature and testing of comparable materials were adopted.

Table 4: Hydraulic Conductivities of the Site

MATERIAL NAME	HYDRAULIC CONDUCTIVITY RANGE (m/s)	HYDRAULIC CONDUCTIVITY ADOPTED FOR MODELLING (m/s)
Hinuera Formation - interbedded sands and silts	4.58×10^{-5} to 7.97×10^{-8}	$K_h = 4.6 \times 10^{-5}$ K_v[2] $= 4.6 \times 10^{-6}$
Walton Subgroup[1]	7.5×10^{-6} to 2×10^{-8} [3]	2×10^{-8}

Notes:

[1] No WGA testing undertaken – Range of values derived from tests in these materials in the surrounding area. Includes aquifer and aquitard layers.

[2] Vertical hydraulic conductivity, K_v, was assumed to be one tenth of horizontal hydraulic conductivity, K_h.

3.5 Catchment Water Balance

3.5.1 Recharge

The shallow groundwater and surface water bodies of the catchment are predominantly recharged directly by local rainfall.

WRC estimates that annual rainfall recharge (soil drainage) is 22% to 40% of rainfall in the Waikato region[1]. Based on an average annual rainfall for Waikato of 1,072 mm, this results in a recharge rate in the range of 235 mm to 428 mm. For the purposes of this assessment, a recharge value at the lower end of the estimated range (250 mm) was adopted for groundwater modelling purposes, to provide a conservative assessment of potential groundwater drawdown and associated dewatering effects.

3.5.2 Groundwater and Surface Water Flow Paths

The catchment is characterised by rural land use; infiltrating rainfall flows to drains and provides baseflows to surface water features. This groundwater–surface water interaction controls local groundwater flow paths and underpins the maintenance of baseflows through the catchment, ultimately contributing to flows in the Waikato River to the south and west, and to the Mangaone Gully system to the north and east.

Infiltrating rainfall onto the hills to the north and west flows to the base of the hills and towards the lower lying Site. The Walton Subgroup hills to the north of the Site, and the outcropping Manaia Group hill south-west of the Site also channel flow that is heading towards the Waikato River through a gap on the western corner of the Site.

3.6 Surrounding Groundwater Users

3.6.1 Nearby Bores

Water supply bores listed in Appendix C and groundwater abstraction consents within three kilometres of the Site are shown in Figure 9, with the consents summarised in Table 5. At the time of writing, according to the Waikato Regional Council (WRC) database, there are approximately 30 bores within one kilometre of the proposed racetrack development. Of these, nine are less than 20 m deep, eight are between 20 to 70 m deep, and four are between 70 to 100 m deep. The remaining bores have missing bore depth information.

3.6.2 Nearby Water Resource Consents and Flow Rates

Current groundwater abstraction from other nearby groundwater bores was reviewed. Maximum consented daily abstraction rates in the area (within 2.5 km) range from 28 to 1,620 m^3/day (Table 5). The lower flow rates relate to water being used for dairy shed washdown and stock water, and the higher flow rates relate to larger abstraction for irrigation/frost protection purposes.

According to the WRC database, there are two existing groundwater consents on the proposed development site: AUTH133770.01.01 and AUTH130351.01.02, both associated with dairy shed washdown activities. AUTH130351.01.02 allows a maximum daily abstraction rate of 28 m^3/day with an annual volume of 10,220 m^3. AUTH133770.01.01 allows a maximum daily abstraction rate of 45.4 m^3/day with an annual volume of 16,582 m^3. WGA understands that both of these consents, with a combined maximum daily volume of 73.4 m^3/day and 26,802 m^3/year, will be surrendered if the proposed development proceeds.

[1] https://www.waikatoregion.govt.nz/environment/water/groundwater-monitoring/flow5a-report/#:~:text=Pr%20%3D%20estimate%20of%20rainfall,percent)%20in%20the%20Waikato%20region.

Table 5: Nearby Groundwater Consents Within 2.5 km

CONSENT NUMBER	WATER USAGE	CONSENTED MAXIMUM DAILY FLOW RATE (m^3)	CONSENTED MAXIMUM DAILY FLOW RATE (L/s)
AUTH123180.01.01	Shed wash	28	0.32
AUTH126795.01.01	Stock water and shed wash	48.3	0.56
AUTH126565.01.01	Shed wash	21	0.24
AUTH126626.01.01	Shed wash	23.8	0.28
AUTH131504.01.01	Shed wash	23.1	0.27
AUTH133770.01.01	Stock water and shed wash	45.43	0.53
AUTH130351.01.02	Shed wash	28	0.32
AUTH144063.01.01	Irrigation	245	2.84
AUTH146846.01.01	Irrigation	588	6.81
AUTH143639.03.03	Irrigation	355	4.11
AUTH144754.01.01	Frost protection	1,620	18.75

Note: Retrieved from WRC database.

3.6.3 Groundwater Quality

The local aquifers contain some areas where the groundwater is characterised by elevated dissolved iron concentrations. Dissolved iron concentrations vary between aquifers and laterally within the same aquifer. The iron concentrations in water from a targeted aquifer will not be known until test bores are drilled and samples taken. Dissolved iron causes staining and taste effects but is not considered a health risk in potable water supplies. Removal of iron through water treatment is not a complicated process and usually involves aeration followed by filtration. In some instances, treatment processes can involve increasing the pH, chemical oxidation, followed by filtration, greensand filters or ion exchange.

Groundwater abstracted from deeper bores is characterised by low nutrient concentrations, which is beneficial as elevated nutrients can be problematic with respect to complying with the drinking water standards. For example, nitrate removal through water treatment is generally considered costly. As a result, it is generally more practical and cost effective to target deeper aquifers with low nutrient concentrations in the water, even if the water in these aquifers has elevated dissolved iron concentrations.

4 WATER SUPPLY GROUNDWATER TAKE

4.1 Introduction

In July 2026, Test Bore 72_13219 was drilled at the site of the proposed Waikato Thoroughbred Racing development, Pencarrow Road, Hamilton, with the purpose of supplying water to the proposed racecourse and residential blocks. This assessment summarises findings from the bore testing to date and evaluates production volumes that may be achieved with a larger diameter production bore.

As outlined in Section 2.4, the potable, stock and equine demand comprises water for the bloodstock precinct, commercial and event areas, trainers' facilities, equine support services, rural-residential lots, senior living, horse drinking water and race-day visitors. The calculated typical daily demand is approximately 340 m^3/day, comprising approximately 290 m^3/day for potable, stock and other non-irrigation uses and 50 m^3/day for horse drinking water. On a peak race day, visitor demand adds approximately 127 m^3/day, increasing the total potable, stock and equine demand to approximately 467 m^3/day.

WGA initially used these demands to inform the groundwater drawdown assessment and the proposed multi-bore supply strategy. A continuous long-term abstraction rate of 250 m^3/day for potable use has been adopted for groundwater modelling based on advice from BCD (Pers. Comm., 27 August 2026). This corresponds to a maximum annual volume of 91,650 m^3/year. Sensitivity analysis was undertaken to assess interference effects on nearby bores, and the proposed daily abstraction rates were subsequently reduced to 337 m^3/day for irrigation.

The Hamilton Basin features a complex hydrogeology with multiple aquifers separated by low-permeability aquitards. The production bore(s) target a deep confined aquifer, overlain by at least two significant fine-grained silt and clay layers. These confining layers reduce groundwater interaction with surface water and other overlying shallower aquifers, making the bore a naturally protected water source.

The Test Bore (72_13219) was drilled to a total depth of 120 m bgl. A screen was installed between 51.5 m and 65 m bgl across a confined aquifer within the Tauranga Group sediments, a geological unit within the Hamilton Basin. The bore diameter is 100 mm. The location of the Test Bore is shown in Figure 9.

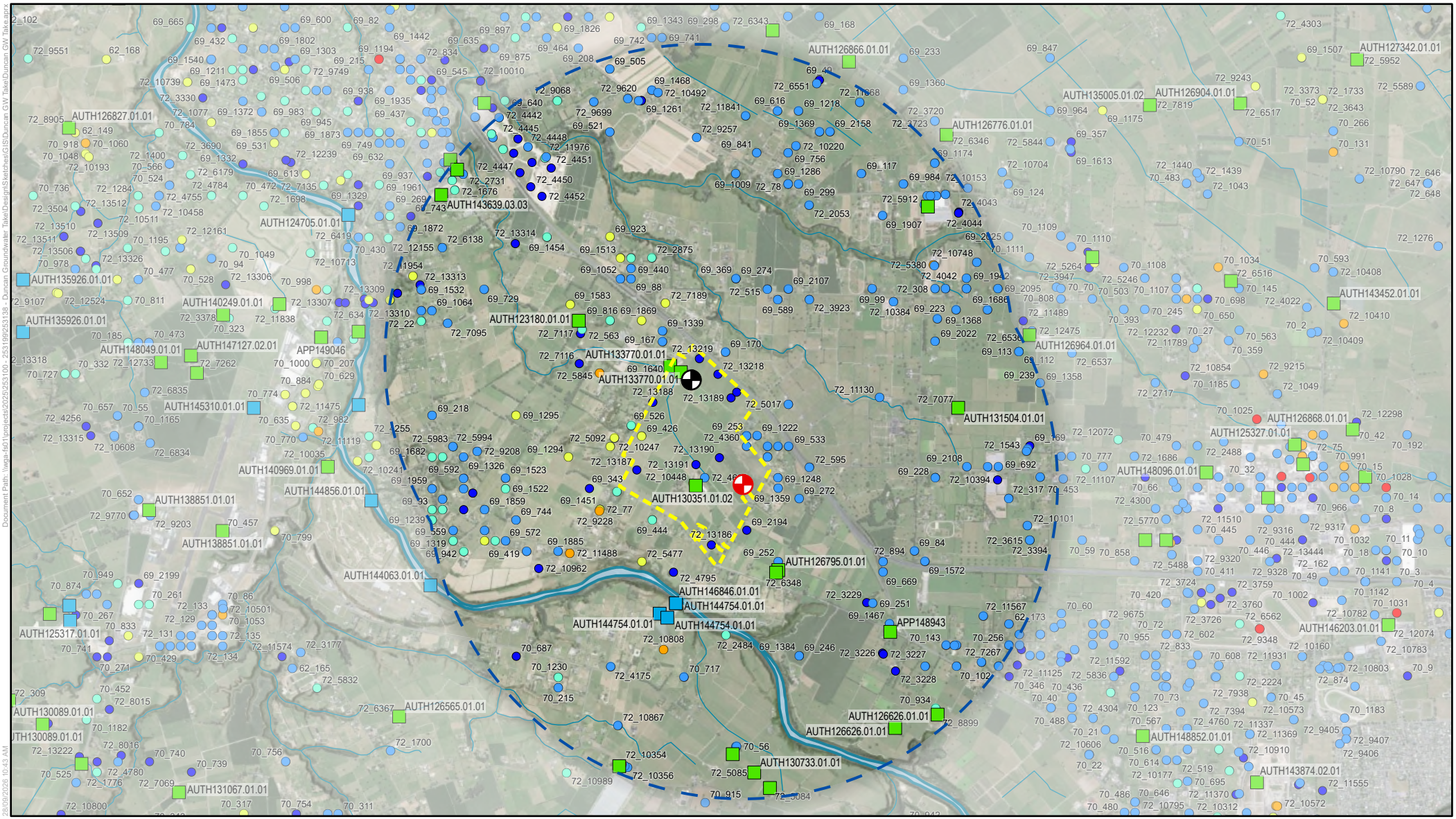
The broader Aquifer 3 sequence extends from approximately 45.7 m to 77.2 m bgl and comprises interbedded pumice sand, sand and silty sand. Aquitard 2, comprising approximately 4 m of fine silt, directly overlies Aquifer 3, with additional fine-grained units higher in the sequence further restricting downward groundwater and contaminant movement from the shallow groundwater system. The static water level in the bore was 10.95 m bgl. The lithological log for the bore is shown on Table 6.

Table 6: Lithological Log for Test Bore 72_13219

DEPTH (m bgl)		HYDROGEOLOGIST DESCRIPTION	HYDROGEOLOGICAL CHARACTERISTICS
From	To		
0	1.5	Topsoil	Aquifer 1
1.5	2.7	Brown gravelly sand	
2.7	5.7	Brown silt sand	
5.7	8	Grey gravelly sand	
8	14.2	Fine grey gravelly sand	
14.2	17.7	Fine grey silty clay	Aquitard 1
17.7	21.7	Light grey silty clay	
21.7	26.2	Green silty clay	
26.2	29.2	Brown fine silty sand	Aquifer 2
29.2	32.7	Brown fine sand	
32.7	38	Brown sand	
38	38.7	Brown fine sand	
38.7	41.7	White/brown pumice sand	
41.7	45.7	Brown fine silt, minor sand	Aquitard 2
45.7	50.7	Green pumice sand	Aquifer 3 (Pumped)
50.7	59.7	Green sand	
59.7	62.7	Green pumice sand	
62.7	65.7	Grey silty sand	
65.7	71.7	Grey pumice sand	
71.7	77.2	Grey pumice sand	
77.2	78.7	Peat	Aquitard 3
78.7	81	Grey silty clay	
81	86.7	Green silt	Aquifer 4
86.7	92.2	Red silt	
92.2	98.7	Grey fine silt with minor gravels	
98.7	120	Orange silt	

Note:

(1) Information sourced from Brown Bros Drilling and C. Howell





N

0 500 1,000 1,500 2,000 m

Scale 1:50,000 @ A4

Coordinate System: NZGD 2000 New Zealand Transverse Mercator

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Note: The information shown on this map is a copyright of WGA 2026

LEGEND

- Proposed Development Boundary
- 3km Buffer
- Ground water take
- Surface water take
- Proposed Irrigation Bore
- Test Bore: 72_13219

Nearby Bores (Depth (m))

- Unknown
- < 25
- 25 - 50
- 50 - 75
- 75 - 100
- > 100

WGA

Figure **9**
Location of Test Bore and Nearby Bores
Waikato Thoroughbred Fast Track Development

4.2 Pumping Test

The Test Bore was pumped at the maximum rate for the 100 mm bore diameter (14.4 m^3/hour; Figure 10). This test has been used to assess the aquifer parameters and potential for a production bore at the site of the Test Bore.

A 24-hour constant rate test was performed on the Test Bore at a discharge rate of 4 L/s (345.6 m^3/day). Recovery in the bore was observed for 12 hours after pumping cessation.

The bore experienced a maximum drawdown of 1.9 m at the end of the constant rate test. Long-term extrapolation suggests a drawdown of approximately 4.1 m after one year pumping continuously at the test rate (4 L/s). The bore did not fully recover within the 12-hour recovery period. Forward extrapolation of the bore recovery suggests that recovery would reach 99% of the original level within 3.5 days. The extrapolated drawdown and recovery are provided below in Figure 11 and Figure 12 respectively.

Drawdown and recovery data was analysed using the Theis (1935) and Cooper-Jacob (1946) methods respectively, standard techniques for confined aquifers. The measured drawdown curves were matched against type curves for a confined aquifer.

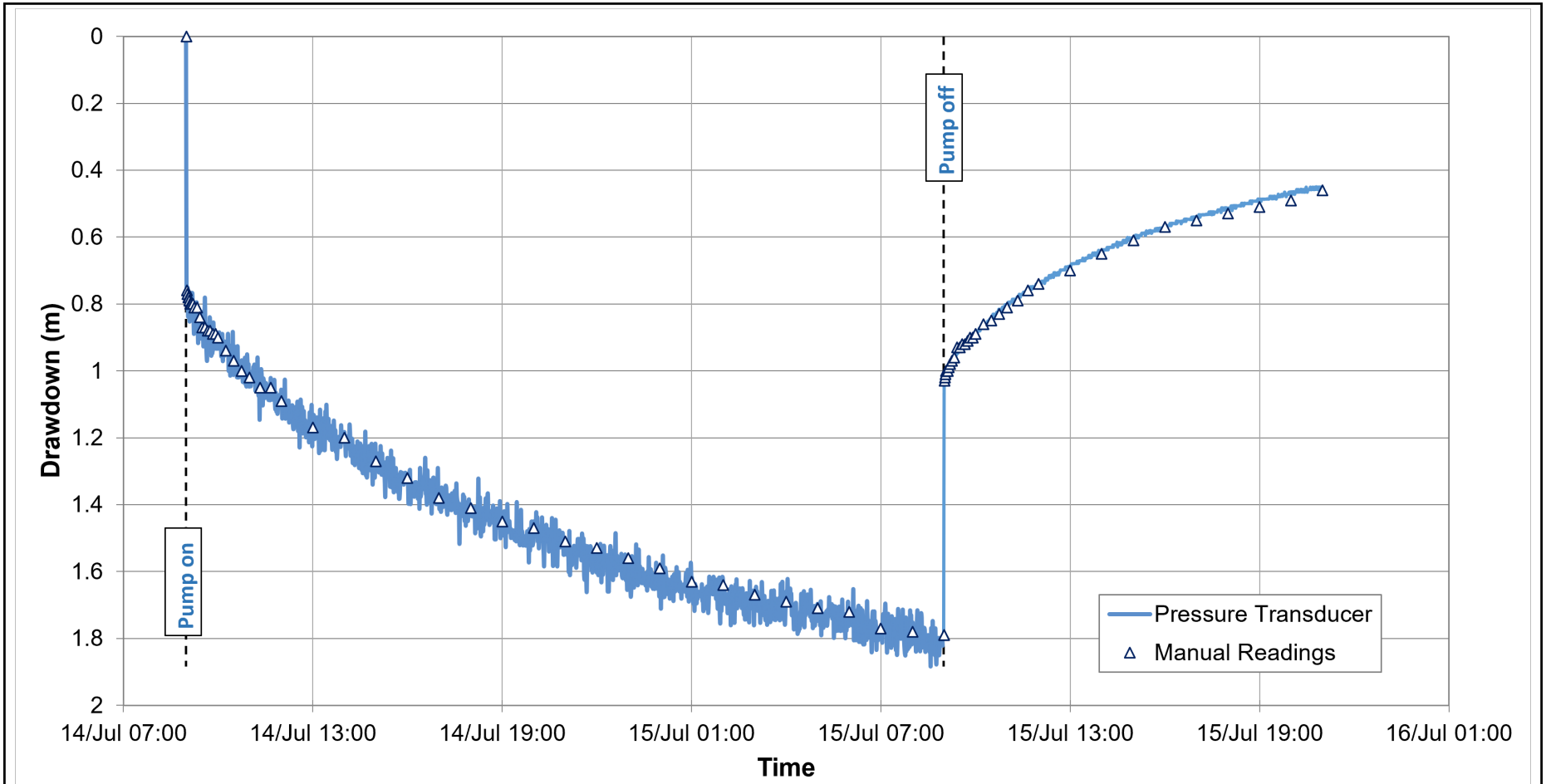
A confined aquifer was chosen as the bore log for the Production Bore has a 4 m layer of silt overlying the screened section of the bore, which WGA consider is having a confining effect on the pumped aquifer. Data analysis was undertaken on the constant rate pumping test data using the AQTESOLV Pro v4.5 software package from HydroSolve Inc., which is an industry standard pumping test analysis package.

The Theis (1935) solution is a common analytical method to determine aquifer properties of a confined aquifer. The following standard set of assumptions is incorporated in the Theis (1935) solution:

1. Aquifer has infinite areal extent
2. Aquifer is homogeneous and of uniform thickness
3. Control well is fully or partially penetrating
4. Flow to control well is horizontal when control well is fully penetrating
5. Aquifer is nonleaky confined
6. Flow is unsteady
7. Water is released instantaneously from storage with decline of hydraulic head
8. Diameter of a pumping well is very small so that storage in the well can be neglected

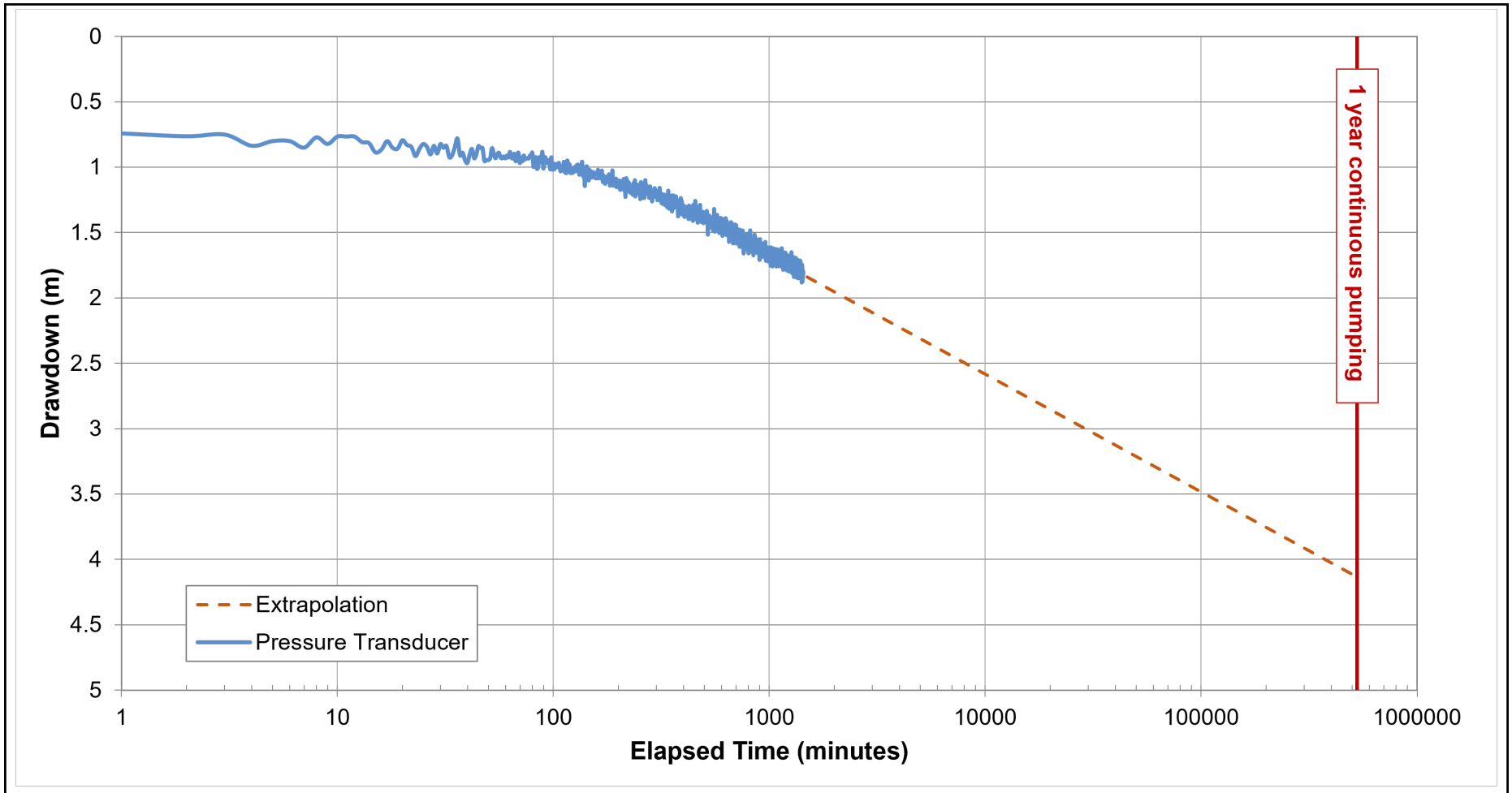
The Theis analytical solution matched the drawdown observed in Test Bore 72_13219 during pumping and the recovery response after pumping ceased. The pumping test analysis is presented in Appendix C.

The pumping and recovery analyses indicate a transmissivity of approximately 75 m^2/day to 106 m^2/day. Storativity could not be determined from the single-well test because no observation bore was monitored. A range of 0.0001 to 0.001, representative of confined aquifers, was therefore adopted for the predictive assessment. Test Bore 72_13219 could be used as an observation bore during future production-bore testing to obtain a site-specific storativity estimate.



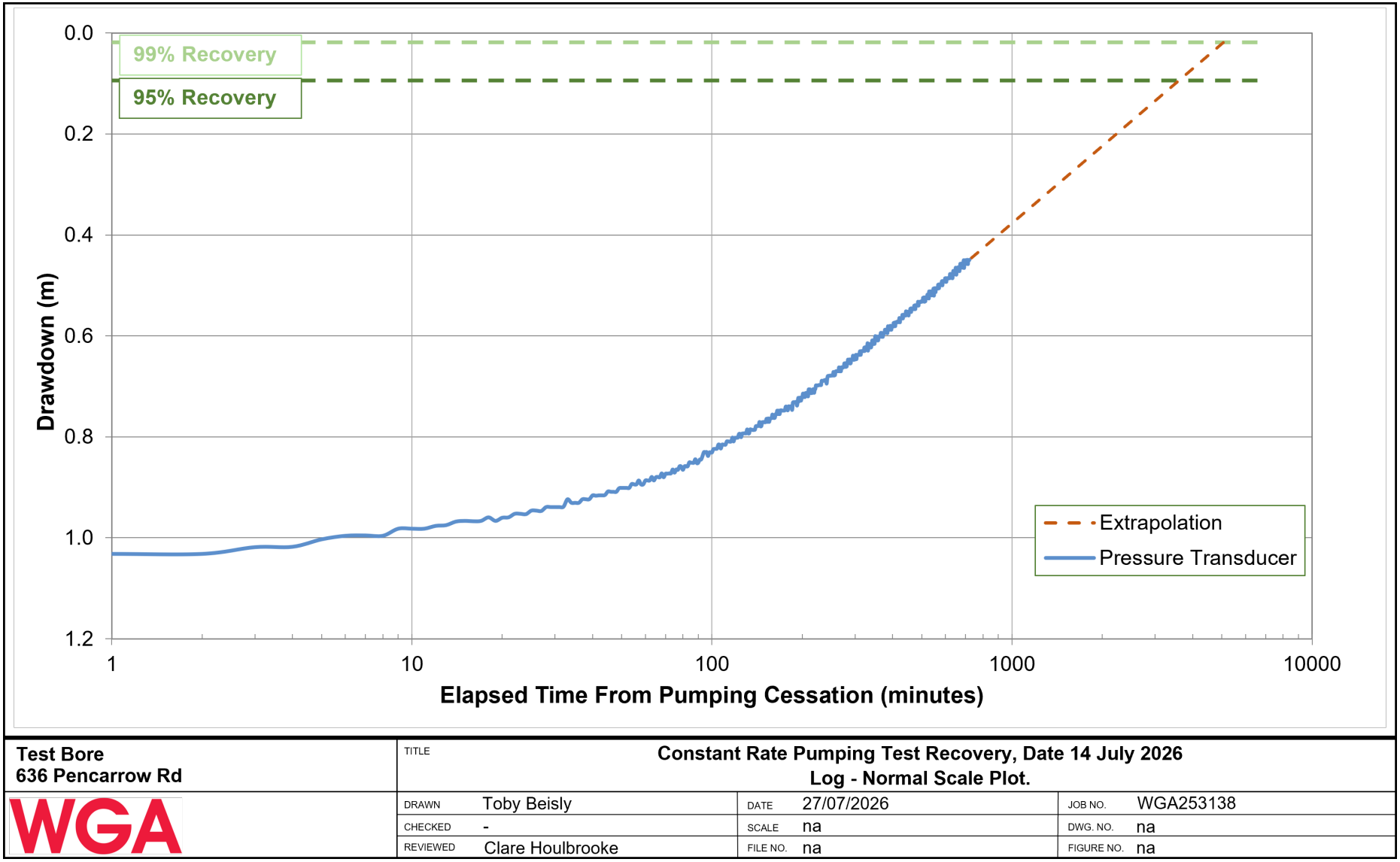
Test Bore **636 Pencarrow Rd**	TITLE	**Constant Rate Pumping Test Drawdown and Recovery, Date 14 July 2026** **Normal - Normal Scale Plot.**			
WGA	DRAWN	Toby Beisly	DATE	27/07/2026	JOB NO. WGA253138
	CHECKED	-	SCALE	na	DWG. NO. na
	REVIEWED	Clare Houlbrooke	FILE NO.	na	FIGURE NO. na

Figure 10: Constant Rate Pumping Test Drawdown and Recovery, Date 14 July 2026



Test Bore **636 Pencarrow Rd**	TITLE	**Constant Rate Pumping Test Drawdown Extrapolated, Date 14 July 2026** **Log - Normal Scale Plot.**			
WGA	DRAWN	Toby Beisly	DATE	27/07/2026	JOB NO. WGA253138
	CHECKED	-	SCALE	na	DWG. NO. na
	REVIEWED	Clare Houlbrooke	FILE NO.	na	FIGURE NO. na

Figure 11: Constant Rate Pumping Test Drawdown Extrapolated, Date 14 July 2026



Test Bore **636 Pencarrow Rd**	TITLE	**Constant Rate Pumping Test Recovery, Date 14 July 2026** **Log - Normal Scale Plot.**			
WGA	DRAWN	Toby Beisly	DATE	27/07/2026	JOB NO. WGA253138
	CHECKED	-	SCALE	na	DWG. NO. na
	REVIEWED	Clare Houlbrooke	FILE NO.	na	FIGURE NO. na

Figure 12: Constant Rate Pumping Test Recovery, Date 14 July 2026

4.3 Initial Estimated Aquifer Production Volumes

Given the estimated transmissivity and assumed storativity of the aquifer WGA has estimated potential maximum production volumes based on a range of parameters. These estimates are preliminary only and further testing would be required for any larger diameter production bore. Each of the curves in Figure 13 to Figure 15 represent a drawdown scenario based on a combination of aquifer parameters (transmissivity and storativity) within a defined range using a Monte Carlo distribution.

WGA used a range for transmissivity (60 – 110 m^2/day) and for storativity (0.001 – 0.0001) based on the Test Bore test results. Based on the aquifer depth, a maximum allowable drawdown has been taken as 28.1 m. This value accounts for 30% additional drawdown to account for well efficiency losses, a static water level of 10.95 m bgl, 2 m length for pump equipment and 2 m for seasonal fluctuations in water level. The drawdown has been calculated for one year of continuous pumping at three separate pumping rates, 500 m^3/day, 1,000 m^3/day and 1,500 m^3/day.

WGA note that our calculations of potential flow rates are high level and for initial assessments of potential capacity from a larger diameter bore. At higher pumping rates and during longer term operation the abstraction will draw on aquifer zones further from the bore, the conditions of which have not been observed during the 24-hour pumping test. The maximum viable production volume for the bore will depend on whether or not these conditions are favourable and the actual well efficiency of the final Production Bore.

Based on the forward projections of drawdown, a pumping rate of 500 m^3/day is considered sustainable based on the test results including under less favourable aquifer parameters further from the bore. The maximum estimates of drawdown for this scenario were under 15 m (Figure 13), which is well within the allowable drawdown.

To achieve a pumping rate of 1,000 m^3/day from a Production Bore in the location of the Test Bore, would require aquifer conditions further from the bore to remain similar to those observed in the 24-hour test. The majority of the drawdown estimates at this pumping rate fall within the allowable drawdown with the maximum estimate giving a drawdown of 32.1 m (Figure 14).

Drawdown estimates in the scenario with a pumping rate of 1,500 m^3/day ranged from 25 to 46 m (Figure 15), mostly outside the allowable drawdown. Therefore, to achieve this pumping rate from one Production Bore, aquifer conditions further from the production bore would need to be more favourable than those observed in the 24-hour test. Therefore, WGA consider that daily rates up to 1,000 m^3/day are possible based on the current information.

WGA note that the provided production estimates are based on preliminary data and a number of assumptions. Following the installation of any larger diameter bore a stepped rate test and three-day continuous pumping test is recommended to give a fuller understanding of the production capacity of the local aquifer.

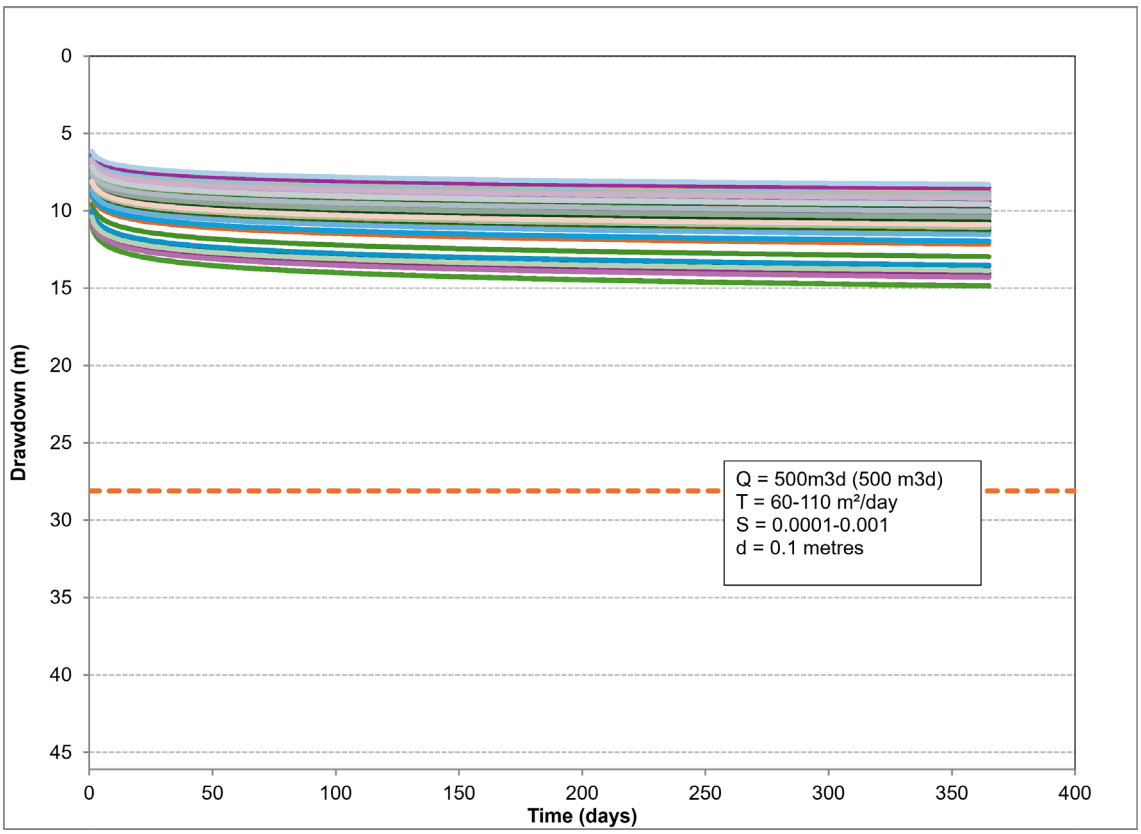


Figure 13: Estimated Drawdown in a Theoretical Bore with a Continuous Abstraction Rate of 500 m^3/day

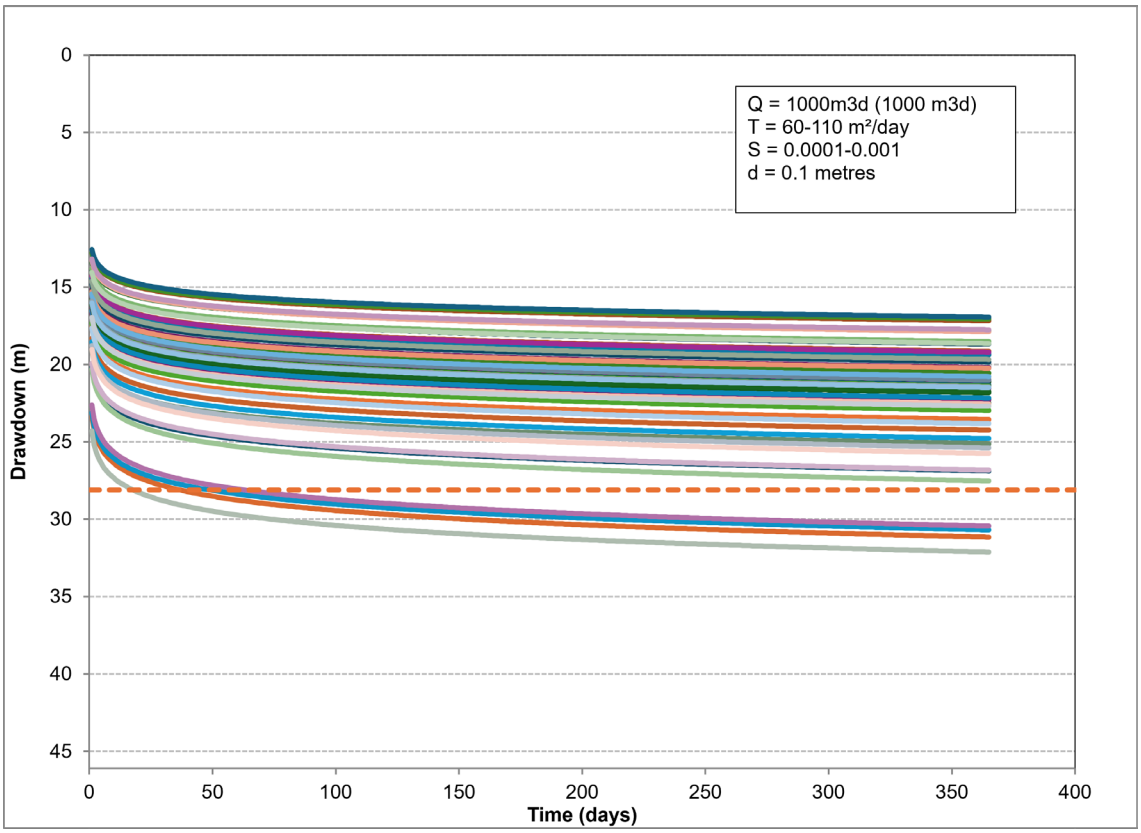


Figure 14: Estimated Drawdown in a Theoretical Bore with a Continuous Abstraction Rate of 1,000 m^3/day

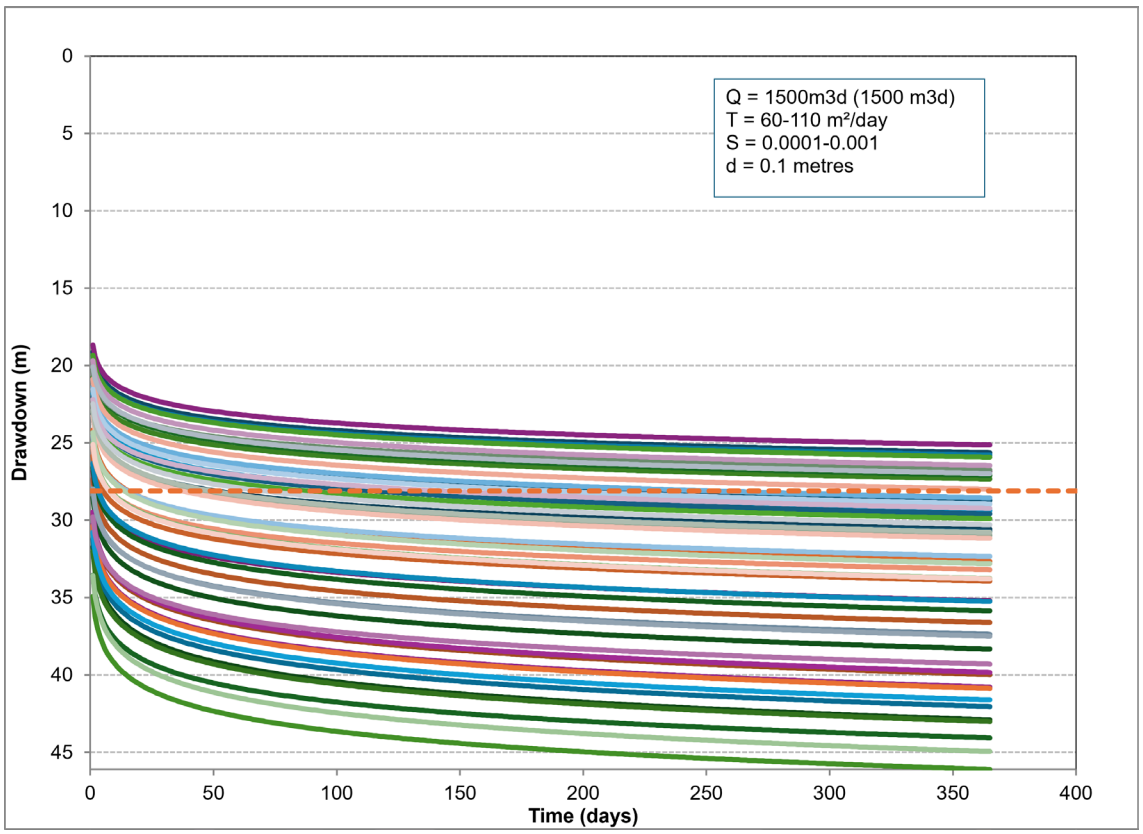


Figure 15: Estimated Drawdown in a Theoretical Bore with a Continuous Abstraction Rate of 1,500 m^3/day

4.4 Potential Impacts on Nearby Users

4.4.1 Introduction

A detailed analysis was carried out to determine the effects at the nearby bores. In this analysis the scenario for an irrigation supply is based on a new Irrigation Production Bore location, at a location further to the east of the site. As a conservative assumption WGA have assumed that this bore is also drawing water from Aquifer 3.

4.4.2 Drawdown Modelling Method

A more detailed assessment of interference effects was undertaken using the analytical solution of Hunt and Scott (2007) for flow to a well in a two-aquifer system, implemented using the Environment Canterbury two-aquifer system spreadsheet tool. This solution was selected in preference to a simple confined (Theis) solution because the local stratigraphy comprises a sequence of aquifers separated by leaky aquitards (Table 6), rather than a single, fully confined aquifer. The solution simulates a pumped aquifer overlain by an aquitard and a second aquifer and calculates drawdown in both the pumped aquifer and the overlying aquifer, together with the proportion of abstracted water derived from each. It therefore allows the degree of hydraulic connection between the pumped aquifer and shallower aquifers, and the resulting drawdown in overlying bores, to be assessed explicitly.

The pumped aquifer is Aquifer 3, screened between 51.5 m and 65.0 m bgl. Two configurations of the overlying system were assessed to bracket the range of aquitard thickness:

- **Case 1 – Aquifer 2 as the overlying aquifer**, separated from Aquifer 3 by Aquitard 2 (fine silt, 41.7 – 45.7 m bgl, 4 m thick).
- **Case 2 – Aquifer 1 as the overlying aquifer**, separated from Aquifer 3 by the thickness of Aquitard 1 (12 m of silty clay and silt).

4.4.3 Adopted Parameters and Scenarios

Deliberately conservative aquifer parameters were adopted because groundwater conditions may vary with distance from the bore. This approach provides an appropriate allowance for uncertainty in aquifer conditions beyond the area directly assessed by the 24-hour pumping test. A transmissivity of 30 m^2/day was applied to both the pumped aquifer and the overlying aquifer, which is well below the 75 – 106 m^2/day derived from the 24-hour constant rate test on Test Bore 72_13219. Adopting a lower transmissivity produces greater and more widespread drawdown, so the predicted interference effects are conservative relative to the conditions observed at the bore. Aquitard properties were assumed, as no vertical hydraulic conductivity testing has been undertaken at the site. The assumed values are consistent with the silt and clay lithologies logged during drilling and with published values for the Tauranga Group.

Table 7: Adopted Hunt and Scott Model Parameters

PARAMETER	CASE 1 (AQUIFER 2)	CASE 2 (AQUIFER 1)
Transmissivity of pumped aquifer (Aquifer 3)	30 m^2/day	30 m^2/day
Storativity of pumped aquifer	1 x 10^{-4}	1 x 10^{-4}
Transmissivity of overlying aquifer	30 m^2/day	30 m^2/day
Storage coefficient of overlying aquifer	0.001 (semi-confined)	0.1 (unconfined specific yield)
Aquitard thickness (b')	4 m	12 m
Aquitard vertical hydraulic conductivity (k')	0.001 m/day	0.0001 m/day
Leakage coefficient (k'/b')	2.5 x 10^{-4} day^{-1}	8.3 x 10^{-6} day^{-1}

The proposed supply comprises two production bores that will operate concurrently rather than as alternatives. The take has therefore been assessed as two components, with the combined effect of both bores operating at the same time representing the condition against which effects on neighbouring bores are assessed:

- **Take A – Irrigation:** 3.9 L/s (337 m^3/day) pumped continuously for 120 days, representing a four month summer irrigation and track-conditioning season, from a new bore location to the east of the Site.
- **Take B – Potable, stock and equine supply:** 2.9 L/s (250 m^3/day) pumped continuously for 10 years (3,652.5 days), representing long-term demand, from the Test Bore location.
- **Assessed condition (combined):** The sum of the drawdown predicted for Take A and Take B, representing the peak summer condition when the seasonal irrigation take is superimposed on the continuous long-term take.

The combined condition has been derived by summing the drawdown predicted for each take at each receptor. The Hunt and Scott (2007) solution, like the Theis solution on which it is based, is linear with respect to discharge, so drawdown from separate abstraction points within the same aquifer system can be added directly by superposition. This approach is valid provided the aquifer response remains within the linear range, which is the case here because the predicted drawdowns are small relative to the saturated thickness of Aquifer 3 and the aquifer remains confined throughout the modelled period.

The two takes are reported separately because they generate different drawdown signatures. The irrigation take produces larger drawdown close to the bore, reflecting the higher abstraction rate, but only over a limited season. The potable, stock and equine take produces smaller near-bore drawdown that propagates much further over the 10 year period as leakage from the overlying aquifers is progressively induced.

The modelled scenarios are deliberately conservative. The potable supply take assumes continuous abstraction at the maximum consented annual rate, sustained without interruption for 10 years. The irrigation take has been assessed at a daily abstraction rate of 337 m^3/day that has been applied continuously across the irrigation season.

In practice, demand will vary between years and seasons in response to climate, event schedules and operational requirements, and pumping will be intermittent rather than continuous. Periods of reduced or no abstraction will allow groundwater levels to partially recover, so the actual long-term drawdown and interference effects will be less than those predicted. The results should therefore be regarded as an upper bound on the effects likely to occur in practice.

Drilling the irrigation production bore to a greater depth may intersect a deeper, more confined aquifer, which would refine the aquitard parameters and may allow a greater irrigation volume to be supplied without a corresponding increase in effects on shallower bores. WGA has recommended drilling and testing at the proposed irrigation bore location and understands that this drilling is planned for October 2026.

4.4.4 Results

Predicted drawdown was calculated at radial distances of up to 3.0 km from each of the modelled production bores and applied to each of the surrounding bores identified in the WRC database. Each bore was assigned an aquifer. Predicted interference was compared with the available water column in each bore and assessed against the WRC internal guideline of 10% of the available water column for potential more than minor effects.

With the modelled potable supply take, predicted drawdown in the pumped aquifer is approximately 13.4 m at the bore, 4.2 m at 100 m and 1.3 m at 1,000 m (Figure 16, Figure 18). With the modelled irrigation take, drawdown in the pumped aquifer is higher at the bore (16.5 to 17.8 m) but attenuates more rapidly, to approximately 1.2 to 1.9 m at 1,000 m (Figure 17, Figure 19).

Predicted drawdown in the overlying aquifers is markedly lower and is strongly dependent on the assumed aquitard properties. Where the pumped aquifer is separated from the shallow unconfined aquifer by 12 m of silt and clay, predicted drawdown in Aquifer 1 does not exceed 0.4 m at any distance, and is less than 0.05 m under the irrigation abstraction. For Aquifer 2, where the aquitard is estimated to be 4 m thick separating Aquifer 3 from Aquifer 2, the predicted drawdown in the overlying aquifer reaches approximately 2.9 m near the bore under long-term potable abstraction.

The results indicate that the intervening aquitards substantially reduce predicted drawdown in shallow bores. Predicted interference effects are concentrated in bores screened within Aquifer 2 and Aquifer 3.

Predicted interference was at or above 10% of the available water column at four bores out of approximately 180 bores that were assessed (Appendix C), being 69_526, 69_1454, 69_1549 and 72_3229. Before effects can be confirmed as no more than minor, these four bores require bore-specific verification of construction details, static water levels and the aquifer targeted. Because WRC records do not consistently provide screen depth, casing base, bore depth, static water level or aquifer assignment, conservative assumptions have been used where information is missing or inconsistent.

Several nearby bores are also either owned by the applicant, screened in greywacke where no drawdown is expected, or appear to have erroneous casing information. Confirmation of actual bore construction and water levels, through landowner consultation and site verification for the four potentially affected bores, is therefore recommended to be included in consent conditions.

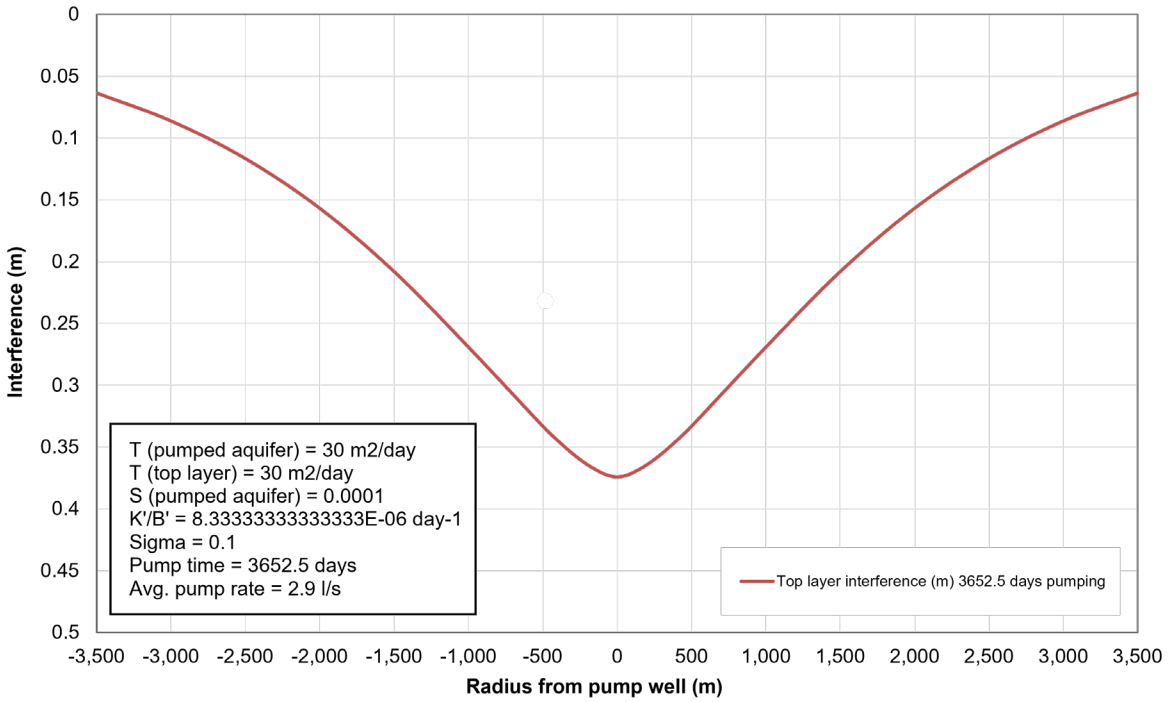


Figure 16: Drawdown in Aquifer 1 Potable Supply Take after 10 years

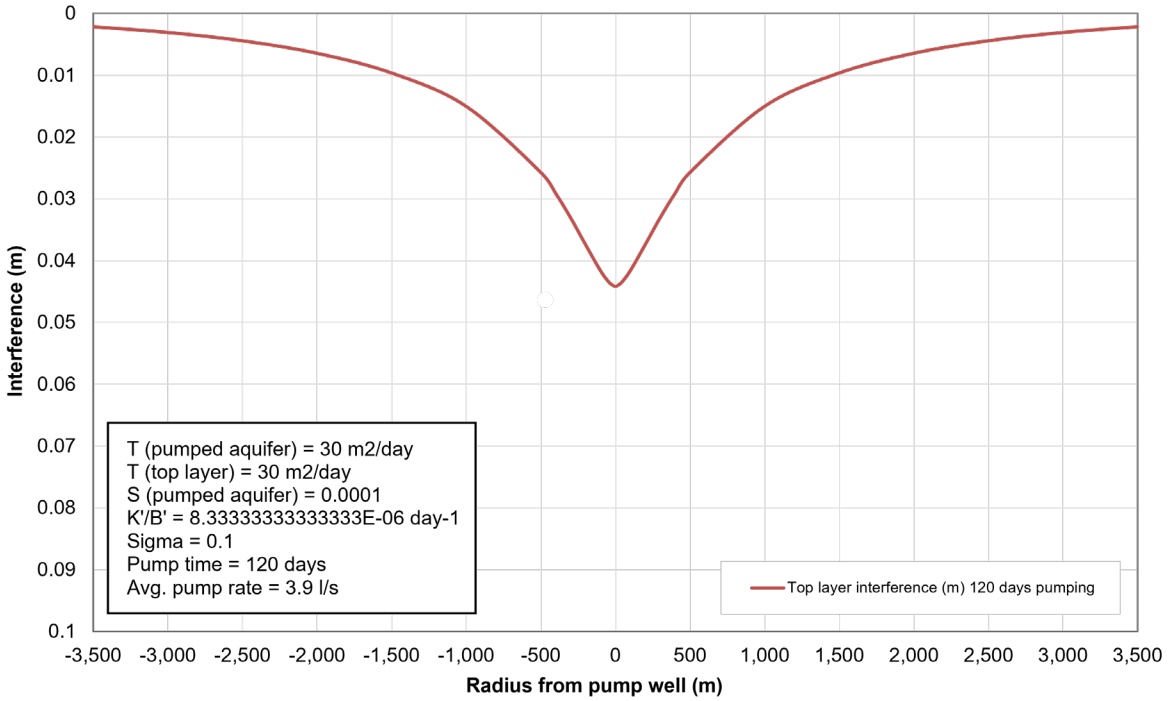


Figure 17: Drawdown in Aquifer 1 Irrigation Take after 120 days Pumping

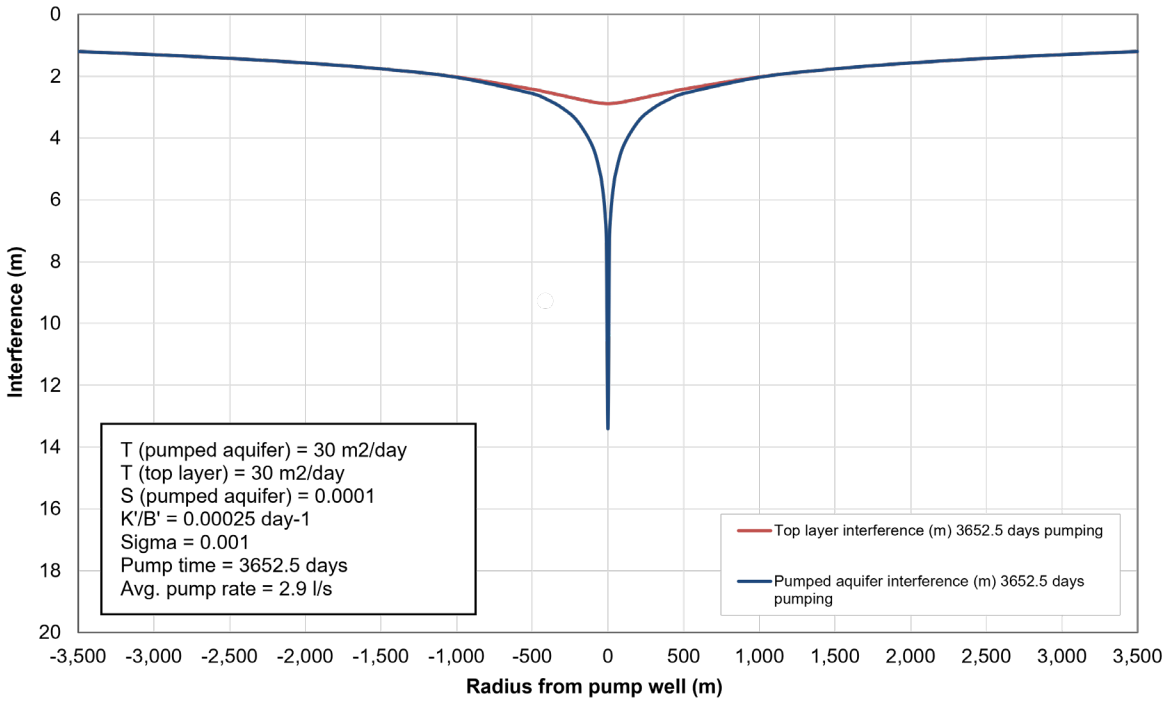


Figure 18: Drawdown in Aquifer 3 and Aquifer 2 Potable Supply take after 10 years Pumping

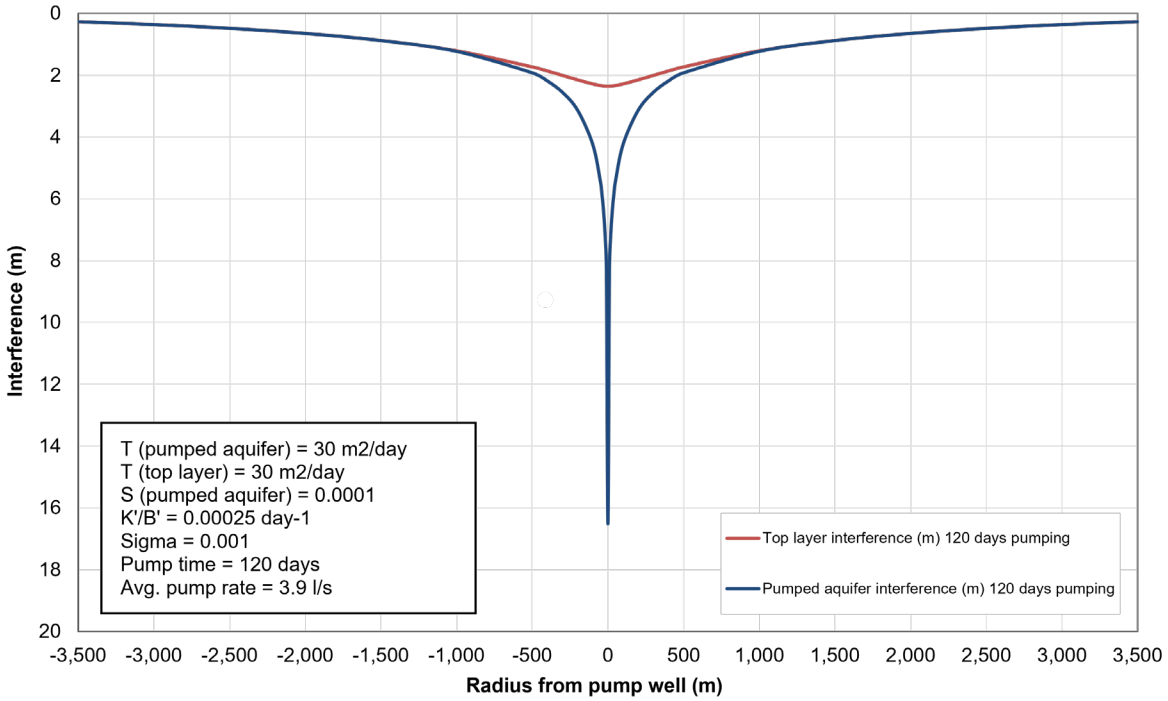


Figure 19: Drawdown in Pumped Aquifer 3 and Aquifer 2 Irrigation Take for 120 days

4.5 Effects on Surface Water

The proposed abstraction is from Aquifer 3, a deep confined Tauranga Group aquifer that is hydraulically separated from shallow groundwater and surface-water systems by multiple fine-grained silt, clay and peat units. The production bore is expected to target the same aquifer as Test Bore 72_13219, which is screened between 51.5 m and 65.0 m bgl.

The nearest major surface-water body is the Waikato River. Its water level near the Site is approximately 15.5 m RL, whereas ground levels at the Site are generally 53 m RL to 57 m RL. The river water level is therefore approximately 37.5 m to 41.5 m below the Site and is broadly coincident with the elevation of Aquitard 2 beneath the Site.

The Test Bore static water level was 10.95 m bgl, corresponding to an inferred potentiometric elevation of approximately 42 m RL to 46 m RL across the Site. This is approximately 26.5 m to 30.5 m above the nearby Waikato River level.

The river is interpreted to incise to approximately the level of Aquitard 2. Accordingly, the aquitard may be intersected, thinned or laterally truncated near the river, permitting hydraulic connection between the Waikato River and Aquifer 3.

Based on site ground levels of approximately 53 m RL to 57 m RL and the planned screen interval of 51.5 m to 65.0 m bgl, the production bore screen would extend from approximately -5.0 m RL to +5.5 m RL. The screen would therefore be approximately 10 m to 20.5 m below the nearby Waikato River level of 15.5 m RL.

On this basis, hydraulic connection between Aquifer 3 and the Waikato River is plausible where Aquitard 2 is intersected, thinned or laterally truncated near the river. The greywacke outcrop may act as a partial hydraulic barrier and limit the degree of connection locally. However, it is not assumed to fully isolate the production aquifer from the river. The proposed abstraction may therefore have an interception or depletion effect on the Waikato River. The magnitude and timing of this effect will depend on the continuity and hydraulic properties of Aquitard 2, the geometry of the greywacke high, the distance between the production bores and the river, and the operational pumping regime.

WGA understands that allocation is currently available from the Waikato River and the potential interception effects of the proposed groundwater abstraction will need to be considered within the applicable allocation regime.

5 DRAWDOWN EFFECT OF PONDS AND SWALES

5.1 Introduction

The proposed ponds are dual purpose, providing stormwater attenuation and treatment as well as irrigation storage for the racing surfaces. Multiple ponds are proposed across the Site. BCD has advised that the ponds consist of the Main Pond, the Harness Pond and Southern Pond. WGA understands that the final number and configuration of the ponds is still being resolved between flood management and irrigation supply requirements. BCD has confirmed that the ponds will all be lined with a heavy-duty PE lining.

The analytical assessments presented in this report are intended to evaluate groundwater inflows, dewatering requirements and groundwater drawdown effects.

5.2 Analytical Pit Modelling Methodology for Storage Ponds

An assessment of potential groundwater inflows to each of the planned Storage Ponds has been undertaken using a published method of estimating groundwater inflows to open-cut pits (Marinelli & Niccoli 2000). This methodology uses analogue equations to approximate horizontal groundwater inflows through the sides of the pit and vertical groundwater inflows through the floor of the pit (Appendix D).

This method adopts a conservative conceptual model in which the proposed ponds are assumed to be hydraulically connected to the underlying groundwater system. The analytical pit modelling applies the representative hydraulic conductivity of the Hinuera Formation (4.6×10^{-5} m/s) throughout the assessment.

The equations used by this methodology assume laterally radial groundwater flows toward a circular pit. In applying this methodology to the proposed storage ponds, their layouts needed to be simplified by converting them to equivalent circular ponds. This has been done by taking the full footprint area of each pond, including any forebays, assuming the area is circular and calculating the radius of the “equivalent” area. The effective radii for the ponds derived using this “equivalent” calculation, along with other model inputs, are presented in Table 8.

To assess inflows during the construction phase, the base of the pond has been modelled as dry. The models used surrounding winter groundwater levels as the starting conditions to identify peak inflow rates.

A recharge rate of 1.1×10^{-3} m/day, equivalent to approximately 400 mm/year, was adopted for the short-term construction scenarios to represent the potential for works to occur during wetter conditions. A recharge rate of 6.85×10^{-4} m/day, equivalent to approximately 250 mm/year, was adopted for the long-term operational scenarios, consistent with the conservative lower-end annual recharge estimate described in Section 3.5.1.

Table 8: Inputs for Pit Models Representing Each of the Storage Ponds

STORAGE POND	EXISTING GROUND ELEVATION (m RL)[1]	EXISTING GROUND-WATER LEVEL (m RL)[2]	DESIGN INVERT ELEVATION (m RL)[1]	DESIGN EXCAVATION DEPTH (m)[3]	DESIGN AREA AT RIM FLOW ELEVATION (m^2)[1]	EFFECTIVE RADIUS AT OVERFLOW ELEVATION (m)[4]
Main Pond	55.0	53.7	53.2	2.3	80,665.8	160.3
Harness Pond	54.7	52.7	53.0	2.2	10,682.3	58.3
Southern Pond	56.0	53.8	53.6	2.9	3,229.7	32.1

Notes:

[1] Existing and deepest design levels provided by BCD (2026).

[2] GWL estimated from piezometric surface

[3] Excavation depth 0.5m below invert level

[4] Assuming ponded area is treated as a circle.

5.3 Analytical Modelling Methodology for Stormwater Swales

An assessment of potential groundwater inflows to the stormwater swales has been undertaken using published methods of estimating groundwater inflows to a trench, McWhorter and Sunada (1977) to represent short term inflows during construction, and Hooghoudt (1940), as presented by Ritzema (1994), represent long term conditions.

The McWhorter and Sunada (1977) methodology models groundwater inflow to a trench by treating the excavation as a narrow line sink within a homogeneous, isotropic aquifer. Flow toward the trench is governed by Darcy's Law and represented using an analytical plane-source solution that accounts for transient drawdown and aquifer storage effects. Groundwater inflow is estimated from the induced hydraulic gradients, with inflow rates controlled by aquifer transmissivity, storage properties and the duration of dewatering.

The equations used by the Hooghoudt (1940) methodology assume steady-state, essentially horizontal groundwater flow toward a long, open trench within a homogeneous, isotropic unconfined aquifer underlain by a horizontal impermeable base. Uniform recharge is applied across the water table, and the resulting drawdown curve either side of the trench represents the balance between the recharge intercepted within the zone of influence and the horizontal flow toward the trench, with the drawdown extending until inflow and recharge are in equilibrium. The convergence of flow toward the base of the trench is represented by an “equivalent depth” beneath the invert, which converts the radial flow component near the trench into an equivalent horizontal flow component.

The deepest location at each swale section has been modelled to provide a conservative estimate of effects. Table 9 details the modelling inputs for these swale sections. A hydraulic conductivity of 4.6×10^{-5} m/s and an aquifer specific yield of 0.1 has been adopted at all modelling locations.

To assess inflows during the construction phase, the models assume a drawdown to 0.5 m below invert level to keep the base of the excavation dry. Surrounding winter groundwater levels have been used as the starting conditions to identify peak inflow rates. WGA have assumed the swale channels will be constructed in 20 m sections, and that each section will be completed within a period of 10 days.

For long-term scenarios, the swales have been modelled assuming a dry base, and summer surrounding groundwater levels. The drawdowns derived from the long-term drawdown assessment are intended to inform ground settlement calculations.

A recharge value of 1.1 x 10^{-3} m/day (400 mm/year) has been adopted for short term scenarios, to allow for the possibility of construction during winter conditions, with higher rainfall. For long term scenarios, a value of 6.85 x 10^{-4} m/day (250 mm/year) was adopted.

Table 9: Inputs for Trench Models Representing Swales

SWALE ID	AQUIFER SATURATED THICKNESS (m)[(1),(2)]	SWALE BASE WIDTH (m)	CRITICAL SECTION DRAWDOWN REQUIRED (m)	
			Construction[(3)]	Operation
North	15	5	0.7	0.2
Central West	15	5	1.1	0.6
Central East	15	5	0.8	0.3

Notes:

(1) GWL estimated from piezometric surface.

(2) Estimated from closest CPTs.

(3) Excavation depth 0.5m below invert level used for short term analysis.

5.4 Results

5.4.1 Drawdown Effects of Storage Ponds on Groundwater

The projected construction inflows and drawdown effects were modelled for the three ponds described in Section 5.1, as detailed in Appendix D and are summarised in Table 10. Groundwater inflows during construction were calculated on the assumption that the drawdown zones generated by the individual ponds do not overlap because the ponds will be constructed separately.

5.4.2 Drawdown effects of Swales on Groundwater

The projected inflows and drawdown effects have been modelled for each of the swales, (as detailed in Appendix D) and the results summarised in Table 11. The radius of groundwater drawdown influence and the groundwater inflows are presented separately for construction (dry trench) and for long-term (operational) effects in Table 11. The modelled inflows during construction are illustrated in Figure 20, Figure 21 and Figure 22. The extent of projected long-term drawdown influence is illustrated in Figure 23, with the ‘de-minimis’ contour being defined as 0.25 m drawdown.

Groundwater inflows at each swale channel were calculated on the assumption that the drawdown zones generated by individual swales and ponds do not overlap during construction..

5.4.3 Drawdown effects of Ponds and Swales on Surface Water

The three storage ponds are not expected to cause stream depletion effects during construction because their modelled drawdown extents are localised, as shown in Table 10. Construction dewatering will intercept groundwater that would otherwise contribute to the local drainage network. To maintain catchment flows, intercepted groundwater will be discharged to the receiving drainage network upstream of the works, subject to appropriate erosion, sediment and water-quality controls.

The Kahikatea Stand is located just beyond the modelled drawdown effect of the proposed Central West Swale.

Table 10: Projected Effects of Proposed Ponds

POND ID	SCENARIO	INITIAL SATURATED THICKNESS ABOVE BASIN BASE (m)	SATURATED THICKNESS AT BASIN WALL (m)	CALCULATED GROUNDWATER INFLOW (m^3/day)[2]	RADIUS OF DRAWDOWN INFLUENCE (m)[1,2]	DISTANCE TO NEAREST STRUCTURE (m)	DRAWDOWN AT NEAREST STRUCTURE (m)
Main Pond	Construction Dry	1.0	0	876	19	165	0
Harness Pond	Construction Dry	0.2	0	64	0	155	0
Southern Pond	Construction Dry	0.7	0	125	8	75	0

Notes:

(1) Radius of influence considered to be 0.25 m drawdown.

(2) Drawdown extents and inflows do not take account of changes in permeability such as Walton Subgroup sediments underlying the Hinuera Formation.

Table 11: Projected Effects of Proposed Swales

SWALE ID	SCENARIO	DRAIN BASE BELOW GROUNDWATER (m)	CALCULATED GROUNDWATER INFLOW (m^3/day)[2], [3]	RADIUS OF DRAWDOWN INFLUENCE (m)[1,2]	DISTANCE TO NEAREST STRUCTURE (m)	DRAWDOWN AT NEAREST STRUCTURE (m)
North	Construction	0.7	54	100	63	0.4
	Operation	0.2	457	0	63	0.6
Central West	Construction	1.1	86	135	27	0.9
	Operation	0.6	352	85	27	0.5
Central East	Construction	0.8	62	110	5	0.8
	Operation	0.3	459	17	5	0.3

Notes:

(1) Radius of influence considered to be 0.25 m drawdown.

(2) Drawdown extents and inflows do not take account of changes in permeability such as Walton Subgroup sediments underlying the Hinuera Formation.

(3) Short term inflow values represent inflows to a 20 m section of trench. Long term inflows represent inflows along the full swale length.

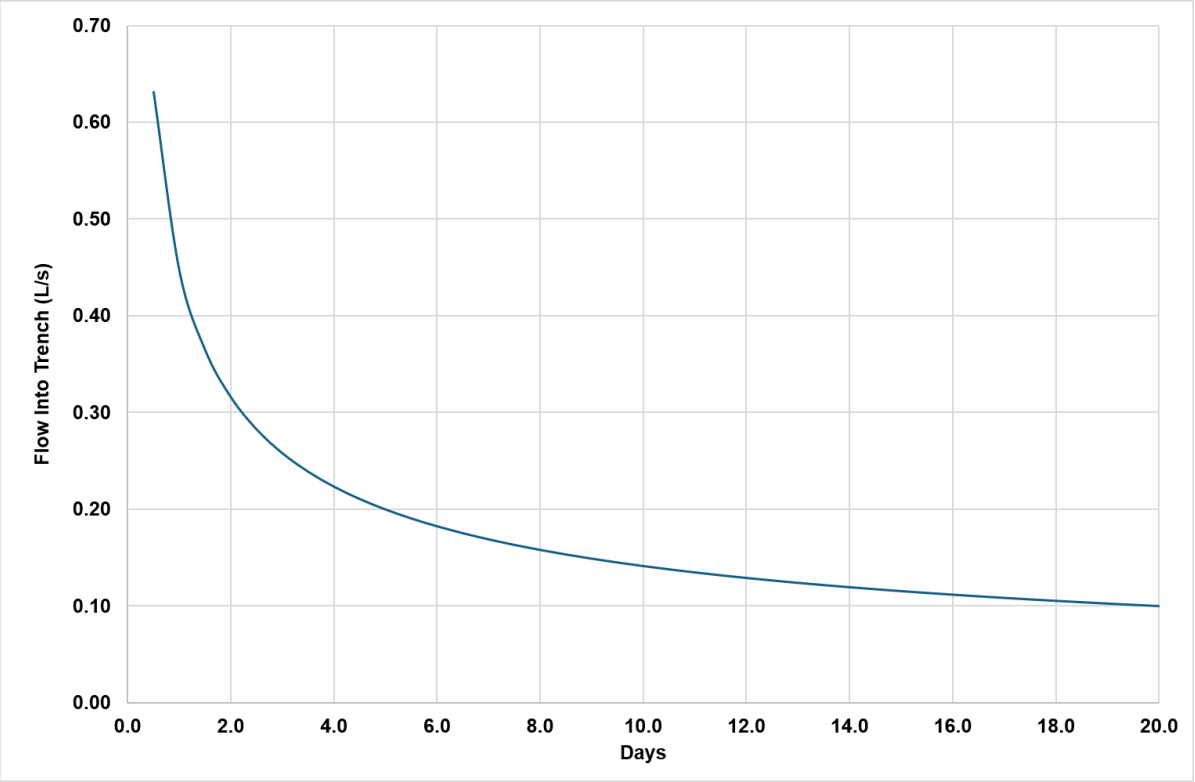


Figure 20: Construction Inflows to Northern Swale

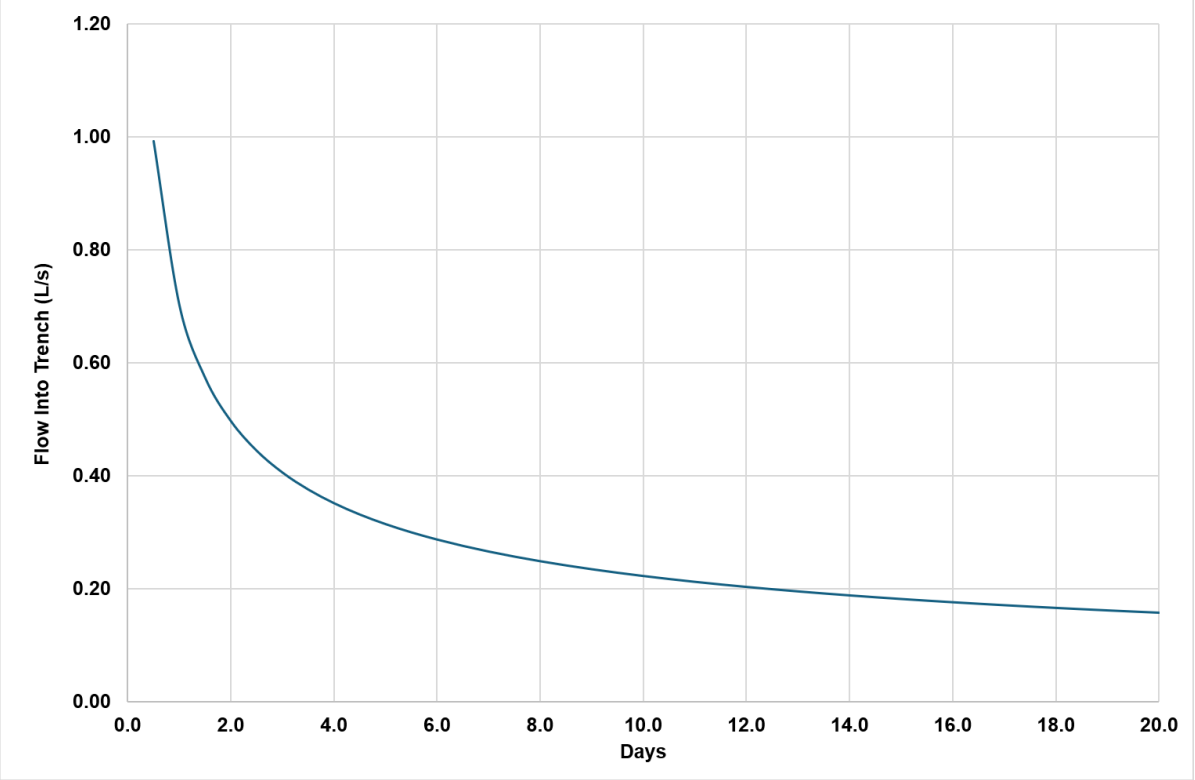


Figure 21: Construction Inflows to Central West Swale

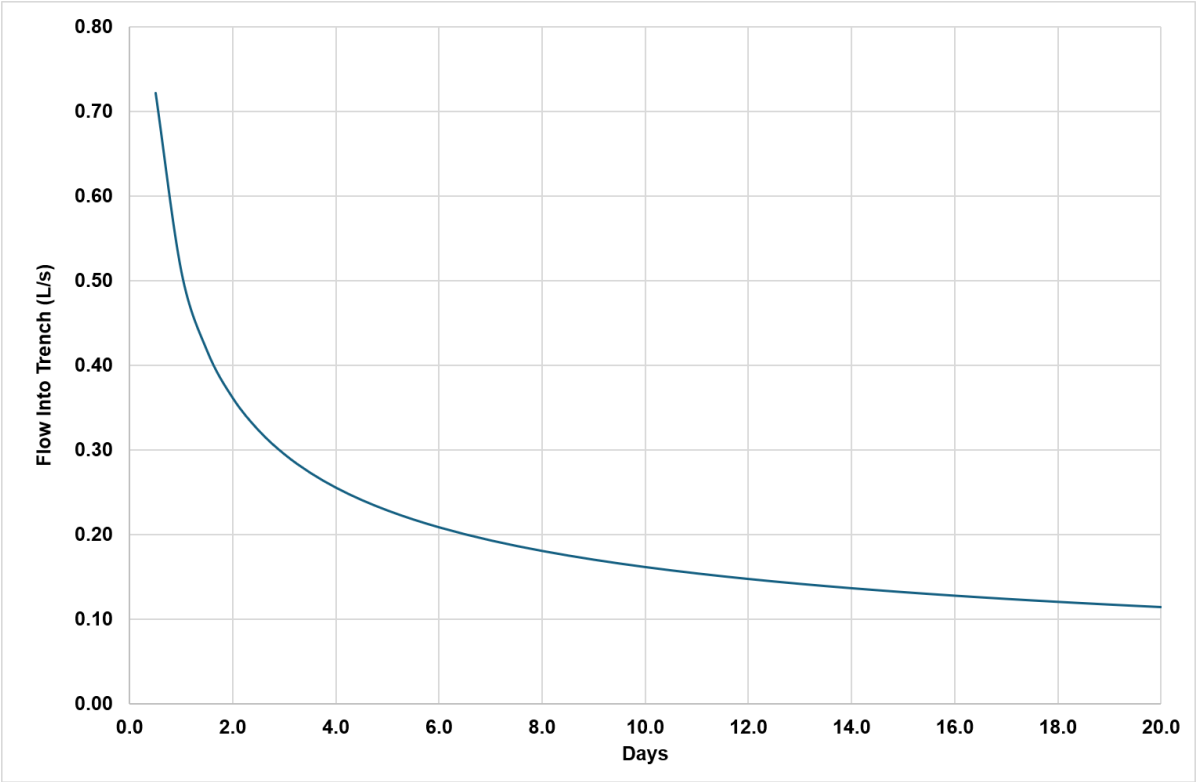


Figure 22: Construction Inflows to Central East Swale





LEGEND

- Swale
- Streams
- Track
- River
- Site Boundary
- Swale Drawdown Contour (m)

N

0 100 200 300 400 m

Scale 1:12,500 @ A4

Coordinate System: NZGD 2000 New Zealand Transverse Mercator

WGA

Figure **22**
Extent of Drawdown Influence for Swales

Waikato Thoroughbred Fast Track Development

Disclaimer: While all reasonable care has been taken to ensure the information contained on this map is up to date and accurate, no guarantee is given that the information portrayed is free from error or omission. Any relevance placed on such information shall be at the risk of the user.

Note: The information shown on this map is a copyright of WGA 2026

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6 DRAWDOWN EFFECTS OF WASTEWATER INFRASTRUCTURE

6.1 Introduction

WGA has modelled the drawdown effect of the dewatering required for the construction of proposed wastewater pump stations and the associated wastewater pipeline network. The modelling used hydrogeological parameters from onsite testing (Section 3.4) and the design provided by BCD.

6.2 Drawdown Effects of Wastewater Pump Station Installation

Two wastewater pumping stations (WWPS) are planned for construction, as shown in Figure 2 and Figure 24. BCD has provided indicative excavation depths and areas for the proposed WWPS designs (email correspondence, September 2026).

Earthworks associated with the wastewater pumping stations involve excavations below the groundwater table. The proposed wet wells have invert depths of approximately 5.4 m bgl at the Main WWPS and 4.3 m bgl at the Trainer WWPS. A wider, shallower excavation will be required around each shaft for the associated tanks and chamber surrounds, with final dimensions to be confirmed by BCD.

The schedule for the construction of these pumping stations remains undetermined at this stage. For the purposes of this assessment, it is assumed that installation of the WWPS wet well and related infrastructure will require 10 days. Therefore, dewatering has been modelled to be achieved within 15 days.

6.2.1 Modelling Methodology for Wet Well Installation

Excavation for the installation of each wet well is expected to be a circular shaft of approximately 1,800 mm diameter, extending to approximately 5.4 m (Main WWPS) and 4.3 m (Trainer WWPS) below existing ground level.

Due to the limited area of the excavation, it is expected that one bore will be sufficient to achieve the required dewatering for wet well construction. Multiple bores may be used for dewatering, but for modelling purposes, using one bore is a reasonable assumption. An excavation diameter of 5 m has been adopted in analysis to allow room for excavation around the wet well shaft and offsetting of the dewatering bore.

The Theis (1935) analytical solution was used to estimate drawdown caused by pumping for construction of the wastewater pump-station wet wells. The solution was applied using transmissivity values derived from onsite hydraulic testing, as described in Section 3.4, and a specific yield representative of the shallow unconfined aquifer. Table 12 presents the parameters adopted for each assessment.

Table 12: Input Parameters for Theis Assessment of Dewatering WWPS Wet Wells

WWPS ID	TRANSMISSIVITY IN UNCONFINED AQUIFER (m^2/day)	AQUIFER SPECIFIC YIELD	TARGET DRAWDOWN	EXCAVATION RADIUS (m)
Main WWPS	51.7	0.1	4.3	5
Trainer WWPS	63.6	0.1	3.9	5

6.2.2 Results - Wet Well Dewatering

The inflows and drawdown predicted from the Theis analysis of the wastewater pump station wet wells, as detailed in Appendix E, are outlined in Table 13. The extent of drawdown influence around each wet well location is shown in Figure 24.

Table 13: Projected Effects of WWPS Dewatering

PARAMETER	MAIN WWPS	TRAINER AREA WWPS
Modelled Excavation Depth (m)	5.9	4.8
Modelled Excavation Width (m)	4.0	4.0
Groundwater level	1.6	0.9
Targeted drawdown depth (m)(1)	4.3	3.9
Pumping rate to achieve dewatering within three days (L/s)	6	5.6
Pumping rate to achieve dewatering within 15 days (L/s)	4.5	4.3
Pumping rate to achieve dewatering within 15 days (m^3/d)	384	367
Calculated radius of influence after 15 days pumping (m)(2)	138	137
Distance to nearest settlement sensitive structure (m)	125	230
Calculated drawdown at nearest building after 15 days (m)(2)	0.3	0.0

Note:

(1) Targeted drawdown from groundwater level to 0.5m below invert level.

(2) Radius of influence is based on calculated drawdown of 0.25 m of the piezometric surface, on the assumption that pumping is continuous at a constant rate over a 10 day period.





LEGEND

- Pumping Station
- Streams
- Track
- River
- Site Boundary
- Wastewater Pump Station Drawdown Contours (m)

N

0 100 200 300 400 m

Scale 1:12,500 @ A4

Coordinate System: NZGD 2000 New Zealand Transverse Mercator

WGA

Figure **24**
Extent of Drawdown Influence Wastewater Pump Station Wet Wells

Waikato Thoroughbred Fast Track Development

Disclaimer: While all reasonable care has been taken to ensure the information contained on this map is up to date and accurate, no guarantee is given that the information portrayed is free from error or omission. Any relevance placed on such information shall be at the risk of the user.

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6.3 Drawdown Effects of Wastewater Pipeline Trenching

6.3.1 Modelling Methodology for Wastewater Pipelines

The construction of the planned wastewater pipeline network requires the excavation of trenches below the water table, with the requirement that these trenches are maintained in a dry state until backfilling starts. WGA have assumed the wastewater pipeline will be constructed in 20 m sections, and that each section will be complete within a period of 10 days. This has been used as the basis of the groundwater drawdown and inflow modelling. The modelled pipeline network ranges in depth from approximately 0.65 m to 3.4 m below existing ground level (BCD Group, Drawing No's C-700 to C-709, Rev B) the depths advised remain subject to change through detailed design.

An analytical modelling approach was used to calculate temporary groundwater drawdown curves and expected inflows to the trenches for the wastewater pipeline. The analytical model used equations from McWhorter and Sunada (1977), which estimate transient drawdown and groundwater inflows into trenches.

WGA has identified the deepest sections of pipeline close to adjacent structures and roads for consideration of drawdown effects. Input parameters for the modelled pipeline sections are provided in Table 14. A hydraulic conductivity of 4.6 x 10^{-5} m/s and an aquifer specific yield of 0.1 has been adopted at all modelling locations.

Table 14: Input Parameters for Wastewater Pipeline Analytical Modelling

WASTEWATER PIPELINE	AQUIFER SATURATED THICKNESS (m)	LENGTH OF TRENCH SECTION (m)	NUMBER OF DAYS TRENCH OPEN	MAXIMUM TRENCH DRAWDOWN REQUIRED (m)
Trainers Wastewater Main – MH22 to MH23[(1)]	11.1	20	10	2.6
Main Harness Area Collector MH8 to MH11	11.4	20	10	2.9

Notes:

[(1)] MH = Manhole.

6.3.2 Results - Wastewater Pipelines

The projected inflows and drawdown effects have been modelled for the critical wastewater pipeline locations, and the results summarised in Table 15. The radius of drawdown influence is illustrated in Figure 25. The drawdown distribution is illustrated in Figure 26 and Figure 27. Projected groundwater inflows are shown in Figure 28 and Figure 29.

Table 15: Projected Effects During Construction of Wastewater Pipelines

WASTEWATER PIPELINE	CALCULATED GROUNDWATER INFLOW (m^3/day)	DISTANCE OF INFLUENCE (m)	DISTANCE TO NEAREST STRUCTURE (m)	DRAWDOWN AT NEAREST STRUCTURE (m)
Trainers Wastewater Main – MH22 to MH23	210	155	230	0.0
Main Harness Area Collector MH8 to MH11	238	163	115	0.7

Note: Distance of influence considered to be defined at 0.25 m drawdown.





LEGEND

- Proposed Irrigation Bore
- Pumping Station
- Streams
- Track
- River
- Site Boundary
- Storage Ponds
- Wastewater Reticulation, Land Application or Treatment Area
- Wastewater Pipelines
- Drawdown Contours (m)
- Open Trench for Wastewater Pipeline Installation

N

0 100 200 300 400 m

Scale 1:12,500 @ A4

Coordinate System: NZGD 2000 New Zealand Transverse Mercator

WGA

Figure **25**
Drawdown Contours
for Wastewater Pipeline
20m Trenches

Waikato Thoroughbred Fast Track Development

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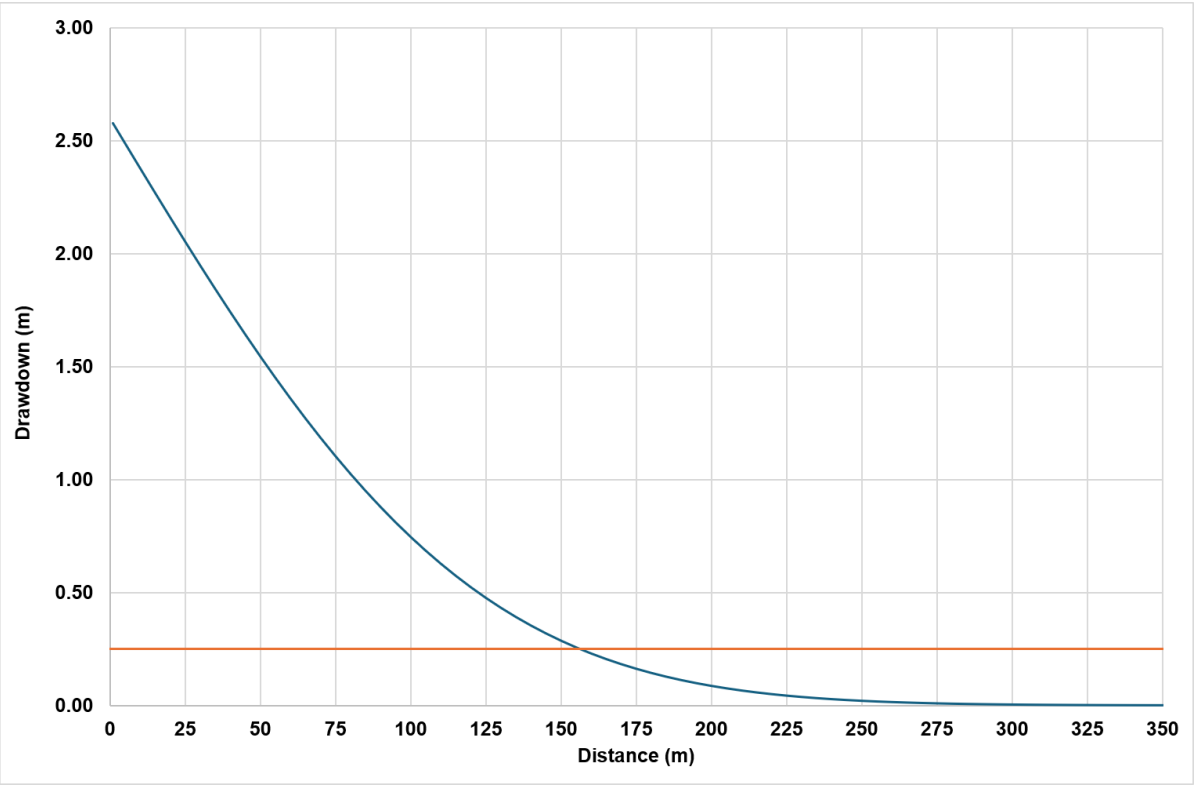


Figure 26: Projected Drawdown from a 20 m section of Wastewater Pipeline at Trainers Wastewater Main MH22 to MH23

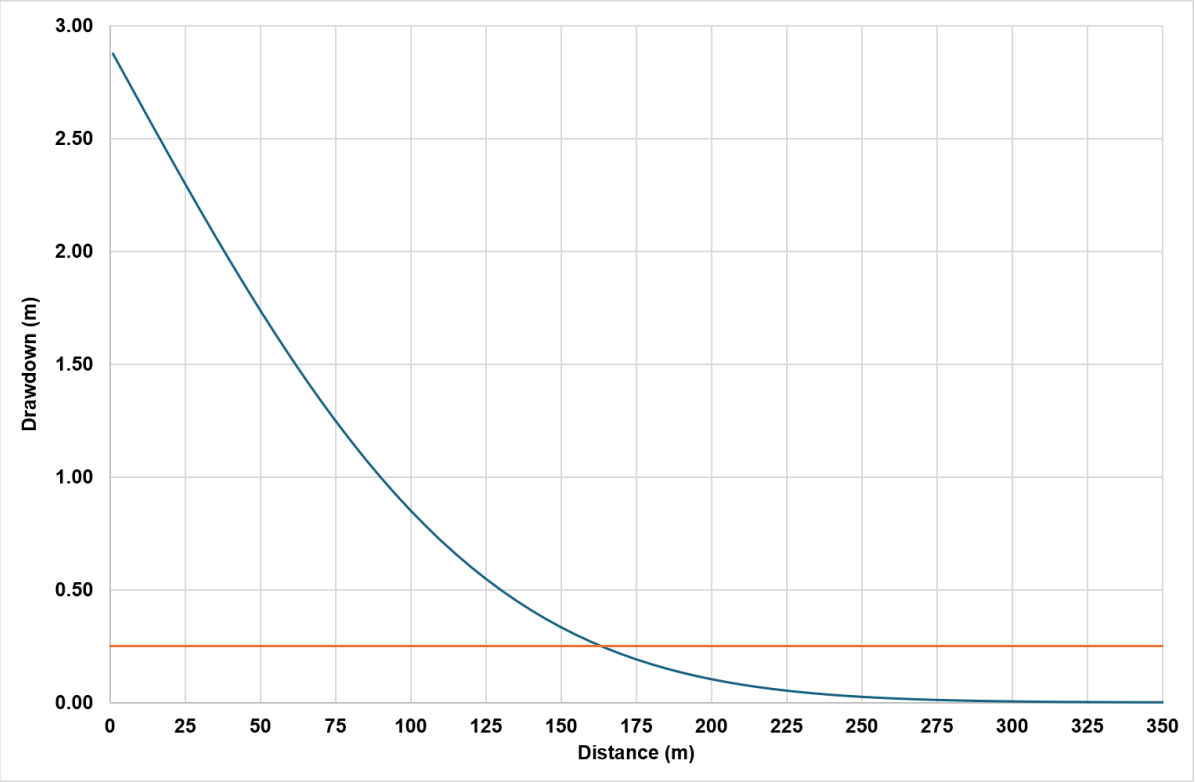


Figure 27: Projected Drawdown from a 20 m section of Wastewater Pipeline at Main Harness Collector MH8 to MH11

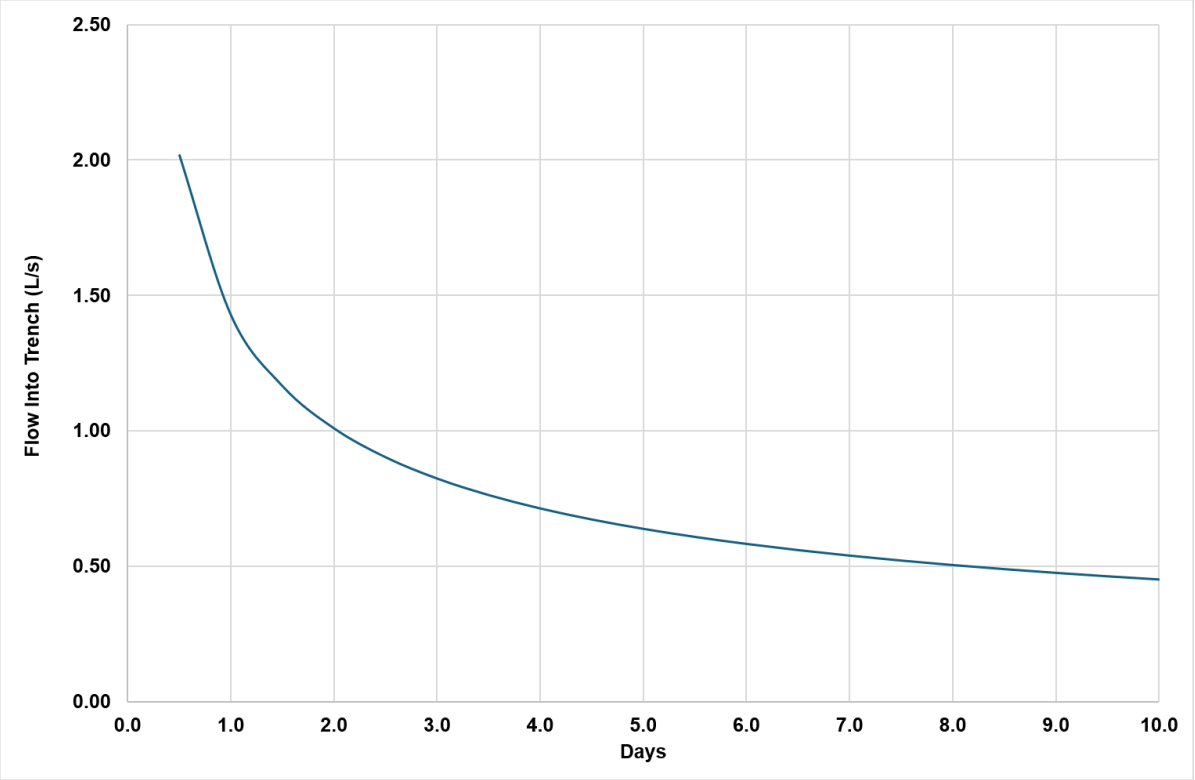


Figure 28: Projected Groundwater Inflows Over Time into a 20 m section of Wastewater Pipeline at Trainers Wastewater Main MH22 to MH23

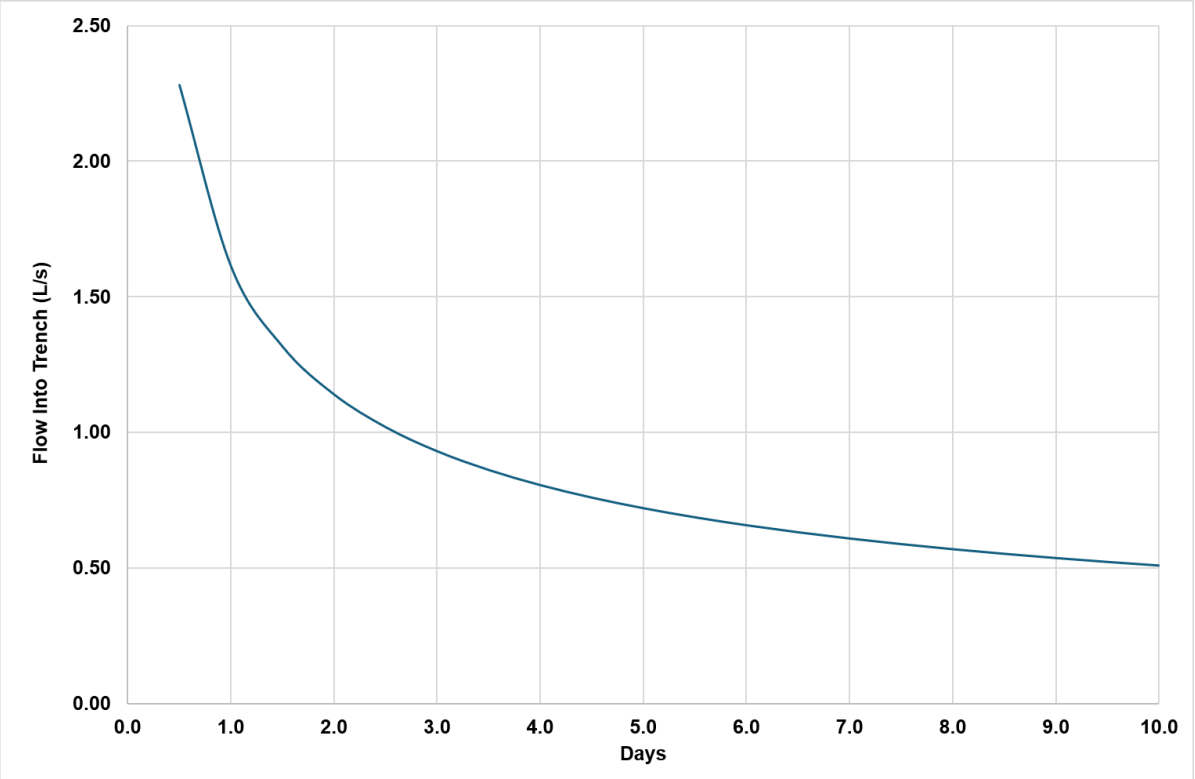


Figure 29: Projected Groundwater Inflows over Time into a 20 m section of Wastewater Pipeline at Main Harness Collector MH8 to MH11

7 DRAWDOWN EFFECTS OF UNDERPASS

7.1 Introduction

An underpass is to be constructed to allow safe passage for horses beneath the main track. The structure will be in the order of 140 m long, with ramps either side. It will be 5 m wide, with a maximum depth below the existing ground surface of 4.0 m. The location is shown in Figure 32.

7.2 Modelling Methodology for Underpass Excavation

An analytical modelling approach was used to calculate temporary groundwater drawdown curves and expected inflows to the trench for the underpass. The analytical model used equations from McWhorter and Sunada (1977), which estimate transient drawdown and groundwater inflows into trenches.

Input parameters for the underpass model are provided in Table 16. Summer groundwater levels were adopted to estimate the extent of drawdown from the excavation, while winter groundwater levels were used to estimate the maximum potential inflows during excavation. BCD have advised a construction window of 30 days is appropriate for modelling purposes (BCD, Pers comm. 2026).

Table 16: Input Parameters Underpass Drawdown Modelling

INPUT PARAMETER	SCENARIO	
	Summer	Winter
Groundwater Level (m bgl)	1.3	0.8
Hydraulic Conductivity (m/s)	4.6×10^{-5}	4.6×10^{-5}
Aquifer Saturated Thickness (m)	10.7	11.1
Aquifer Specific Yield	0.1	0.1
Width of Trench (m)	5	5
Length of Trench (m)	140	140
Number of Days Trench open (days)	30	30
Trench Drawdown Required (m)	3.3	3.3

7.3 Results

Projected groundwater inflows from the underpass excavation are shown in Figure 30. Peak inflows of **2,760 m^3/day** are predicted. Projected drawdown is summarised in Table 17, and Figure 31. The extent of drawdown influence is illustrated in Figure 32.

Table 17: Projected Distance of Drawdown Influence for Underpass Excavation

DISTANCE OF DRAWDOWN INFLUENCE (m)	DISTANCE TO NEAREST STRUCTURE (m)	DRAWDOWN AT NEAREST STRUCTURE (m)
300	230	0.65

Note: Distance of influence considered to be defined at 0.25 m drawdown.

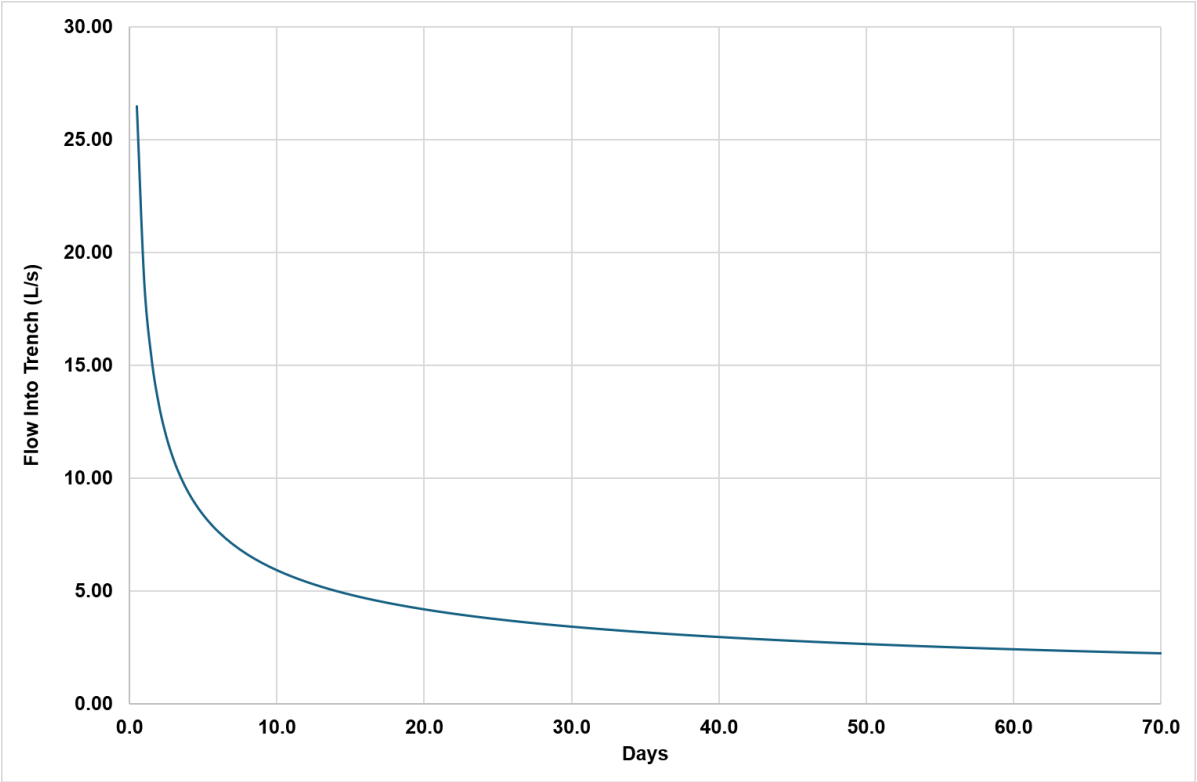


Figure 30: Groundwater Inflows to Underpass Excavation During Construction

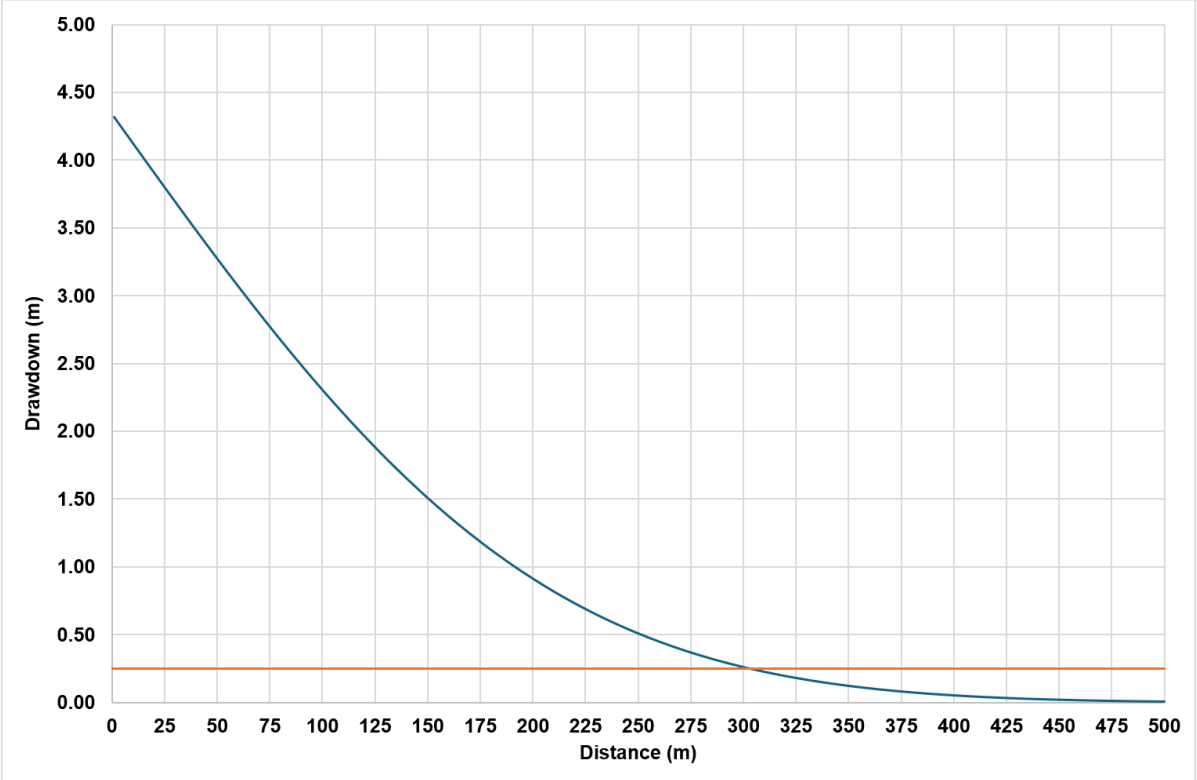


Figure 31: Groundwater Drawdown from Underpass Construction



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LEGEND

- Streams
- Underpass
- Site Boundary
- River
- Indicative Track
- Drawdown Contour (m)

N

0 100 200 300 400 m

Scale 1:12,500 @ A4

Coordinate System: NZGD 2000 New Zealand Transverse Mercator

WGA

Figure **32**
Underpass Location and
Extent of Drawdown Influence

Waikato Thoroughbred Fast Track Development

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8 WASTEWATER DISCHARGE EFFECTS ASSESSMENT

8.1 Proposed Wastewater Treatment Facilities

Wastewater generated by the racing facility, trainers' allotments, village centre, commercial equine activities and comprehensive retirement living precinct is proposed to be collected through a site wide gravity and pumped wastewater network and conveyed to a centralised onsite wastewater treatment plant. The current design basis provides for an average wastewater flow of approximately 356 m^3/day, with an average flow of approximately 4.1 L/s and a peak flow of approximately 27.6 L/s. Wastewater from the lifestyle residential precinct is proposed to be managed separately, with an indicative design flow of approximately 155 m^3/day.

The proposed treatment system comprises FILTEC modular membrane bioreactor plants. A 356 m^3/day MBR-N package is proposed for the main development, and a separate 155 m^3/day MBR-N package is proposed for the lifestyle residential precinct. WGA understands that the modular systems are pre-engineered and factory assembled, allowing additional treatment units to be installed in parallel if future expansion is required. The treatment process comprises preliminary screening, biological treatment, membrane filtration and disinfection. The MBR-N configuration also provides nitrogen and phosphorus removal. Indicative treated-effluent quality is less than 5 mg/L carbonaceous biochemical oxygen demand, less than 1 mg/L total suspended solids, less than 1 mg/L ammoniacal nitrogen, less than 4 mg/L total nitrogen, less than 0.5 mg/L total phosphorus and less than 100 cfu/100 mL E. coli (BCD 2026).

Treated effluent is proposed to be discharged to land within the designated wastewater disposal areas shown on the civil design drawings (BCD 2026). These areas comprise distributed land-application zones located within available open-space areas of the development and are intended to provide sufficient land area for disposal of the design flows while maintaining appropriate setbacks from buildings, property boundaries, waterways, drainage features, bores and other sensitive receptors. Effluent will be transferred from the treatment plants to the disposal areas through the proposed wastewater reticulation and pumping network and applied using subsurface disposal beds or fields.

BCD has advised a design land application rate of 50 mm/day, corresponding to a Class 3 soil category under Table L1 of AS/NZS 1547:2012 (BCD Group, pers. comm.). Applying this rate to the combined design flow of 511 m^3/day (356 m^3/day from the main development and 155 m^3/day from the lifestyle residential precinct) indicates a nominal wetted application area in the order of 10,220 m^2, exclusive of reserve areas. WGA notes that the assumed Class 3 soil category is to be confirmed against site-specific soil and soakage testing, given that the shallow profile logged across the Site comprises variable interbedded Hinuera Formation sands and silts with localised Walton Subgroup clayey silt.

Three disposal areas are proposed, one within the main track, one in the trainers' area and one for the senior living units. Each area has been assessed for E.coli and virus attenuation.

Final disposal-field locations, dimensions, loading rates, reserve areas and operating configuration will be confirmed through detailed design, site-specific soil and soakage testing, groundwater assessment and the resource-consent process.

8.2 Attenuation Modelling Methodology

8.2.1 Waste Water Quality

Typical abundances of faecal coliforms in raw sewage are in the range of 1 x 10^6 to 1 x 10^8 cfu/100 mL (Blanch et al. 2003). WGA has been advised by BCD that effluent discharged from the proposed wastewater treatment facility is to have E.coli counts below 200 cfu/100 mL.

8.2.2 E.coli Attenuation Factors

The digital soil map for New Zealand (SMap) indicates that the soil in the planned discharge areas either is an allophanic or gley soil. Based on the geological logs from the site topsoil depth and description is relatively uniform and indicative of allophanic soils. Given the information in the logs, WGA considers that the topsoil around the wastewater dispersal zones will have a thickness of approximately 0.2 m. WGA has assumed that the wastewater drip irrigation lines will be buried to a depth of between 0.1 m, leaving at least 0.1 m of topsoil below the drip lines.

Pang (2009) reports a mean E.coli attenuation rate of 5.34 log/m in water passing through allophanic soils. This attenuation rate has been applied for treated wastewater seepage through the 0.1 m thickness of topsoil before the seepage enters the vadose zone at all sites.

For this assessment the vadose zone underlying the topsoil has been taken to be the depth between the soil horizon and highest measured groundwater table level. The vadose zone thickness for each site is shown in Table 18.

For conservatism, the Hinuera Formation under the wastewater disposal field has been defined as a sand, rather than a silty sand or silt. An E.coli attenuation rate for vertical water seepage through a fine sand vadose zone has been defined as 0.05 log/m (Pang 2009). This attenuation rate has been applied for treated wastewater seepage through the vadose zone before the seepage enters the saturated aquifer zone.

After reaching the groundwater table, the infiltrated wastewater would be transported laterally in a northeast direction, with the faecal coliforms being further attenuated during this groundwater movement. The shallow aquifer (Hinuera Formation) has been interpreted as being primarily fine sands, for the purpose of this assessment. The attenuation rate for E.coli in the saturated zone of a fine sand aquifer has been defined as 0.05 log/m (Pang 2009).

Table 18: Wastewater disposal calculation inputs.

INPUT	MAIN TRACK AREA	TRAINER AREA	SENIOR LIVING AREA
Soil Type	Allophanic		
Drip line Depth (m)	0.1		
Topsoil depth (m)	0.2		
Topsoil seepage horizon (m)	0.1		
Topsoil attenuation rate (log/m)	5.34		
Vadose zone thickness (m)	1.2	0.7	1.9
Vadose zone attenuation rate (log/m)	0.05		
Saturated zone attenuation rate (log/m)	0.05		

8.2.3 Virus Attenuation Factors

For the assessment of the attenuation of viruses WGA has used ESR (2010) guidelines for separation distances based on viral transport between onsite domestic wastewater systems and bores was used to assess the necessary separation distance between each disposal field and a nearby bore. ESR (2010; Worksheet 2) determines the required separation distance between a wastewater discharge and a bore by calculating the virus attenuation still required after accounting for removal through treatment, the disposal field and the soil. It then uses hydrogeology specific virus transport modelling tables to identify the separation distance required to achieve this attenuation at a 95% confidence level.

Using the ESR (2010) guideline, rotavirus was adopted as the design virus because it is the most infectious of the waterborne viruses considered and therefore provides a conservative basis for determining groundwater protection requirements. A target virus reduction of 16.2 $\log_{10}$ was applied, corresponding to the reduction required to achieve an annual infection risk of approximately 1 in 10,000. Virus attenuation achieved through the wastewater disposal system was accounted for by applying a log reduction for the secondary treatment system (AWTS), UV treatment and a further reduction associated with disposal through a shallow dripper system and the allophanic underlying soil profile. The residual $\log_{10}$ reduction requirement was then assessed using the separation-distance tables for silt vadose zone overlying an alluvial sand aquifer. These tables incorporate virus removal during transport through both the unsaturated zone and groundwater. The required separation distance was determined by selecting the vadose zone thickness applicable to the site and identifying the minimum separation distance required to achieve the remaining $\log_{10}$ virus reduction target within the alluvial sand hydrogeological settings.

8.3 Results

8.3.1 E. coli Attenuation

E.coli attenuation calculations (Table 19) indicate that E.coli counts in the discharged treated wastewater become negligible shortly after it reaches the saturated zone of the shallow aquifer. The lateral distance from the discharge site at which faecal coliform counts drop below 1 cfu/100 mL is calculated to be approximately 35 m from the point of discharge for all sites. At a distance of 250 m from the discharge fields, counts were calculated to be effectively zero ($< 2 \times 10^{-11}$ cfu/100 mL). For E.coli concentrations to exceed the New Zealand drinking water standard of 1 cfu/100 mL at a distance of 250 m from the disposal field, a source concentration in the order of 1×10^{13} cfu/100 mL would be required. This value is far in excess of counts expected in raw wastewater, as indicated in Section 8.2.1.

All shallow bores located within a 250 m buffer of the disposal field will be owned by the applicant, and their use can be managed appropriately. There are no shallow bores or other potential receiving surface water systems located close enough to the discharge field to receive groundwater discharges carrying E.coli counts in excess of 1 cfu/100 mL. Therefore, the effects of the operation of the wastewater disposal sites are considered less than minor in terms of lateral movement of E. coli to nearby potential receptors.

A copy of the E.coli attenuation analysis sheets applied to these assessments are presented in Appendix F.

Table 19: Faecal Coliform Attenuation

ATTENUATION COMPONENT	E. COLI COUNT CFU/100 mL		
	Main Track Area	Trainer Area	Senior Living Area
Raw wastewater – no attenuation	1×10^6 to 1×10^8		
Wastewater treatment plant discharge	< 200		
Topsoil seepage horizon – 5.48 log/m	< 57	< 57	< 59
Vadose zone – 0.05 log/m	< 49	< 46	< 54
Saturated zone – 0.05 log/m	$< 2 \times 10^{-11}$	$< 2 \times 10^{-11}$	$< 2 \times 10^{-11}$

8.3.2 Virus Attenuation

Assessment of the wastewater disposal areas using the ESR (2010) methodology indicates that, following treatment through the secondary treatment system, UV disinfection, shallow dripper disposal and attenuation within the overlying allophanic soils, the remaining virus attenuation required within the subsurface is approximately 12 $\log_{10}$. For the assessed hydrogeological conditions, comprising a silt vadose zone overlying an alluvial sand aquifer, the guideline separation-distance tables indicate required wastewater disposal fields to water supply bore setback distances in the order of 300 m. Although calculated distances varied slightly between sites as a function of vadose zone thickness, all results were sufficiently similar to support adoption of a consistent separation distance of 300 m.

The calculated separation distances are considered conservative. The assessment was undertaken using the silt vadose zone and alluvial sand aquifer pathway identified in the ESR guidelines. Site investigations indicate that the alluvial sand deposits contain a proportion of pumiceous material, which is expected to enhance virus attenuation during subsurface transport. The ESR guidelines identify pumice sands as having substantially greater virus removal capacity than alluvial sands, resulting in significantly shorter modelled separation distances. As the hydrogeological assessment was based on the more conservative alluvial sand classification rather than a pumice sand classification, the adopted 300 m separation distance is considered precautionary and is likely to overestimate the setback required to achieve the target level of groundwater protection.

There are no nearby bores within 300 m of the senior living disposal field and the main track disposal field. The nearest bore (72_7210) on the database to the disposal field in the Trainer area is approximately 280 m away. However this bore is located upgradient of the proposed discharge and as described above, the separation distance of 300 m is considered conservative therefore the effects on nearby bores and surface water is expected to be less than minor.

9 GROUNDWATER DEWATERING EFFECTS ASSESSMENT

9.1 Effects Overview on Catchment Water Balance

The proposed development is predominantly located within the Mangaone Catchment where the catchment's hydrology is characterised by strong interaction between groundwater and drains. The groundwater contributes flows to drains and ultimately to the Waikato River. This connectivity influences local groundwater flow paths and supports the persistence of flow through the catchment.

The proposed development has three main groundwater-related components that have the potential to influence the catchment water balance (Figure 33):

1. Drawdown from groundwater abstraction for water supply and irrigation.
2. Short-term temporary dewatering during construction of the storage ponds, the underpass and wastewater infrastructure.
3. Long term effects from the proposed onsite swales.

Groundwater abstracted during temporary dewatering will be discharged to the local drainage network and therefore returned to the same catchment in which it currently discharges. Given the ponds are proposed to be lined they will not cause any long-term groundwater diversion. Once the ponds are formed surface water runoff will be captured and a portion will be used for irrigation.

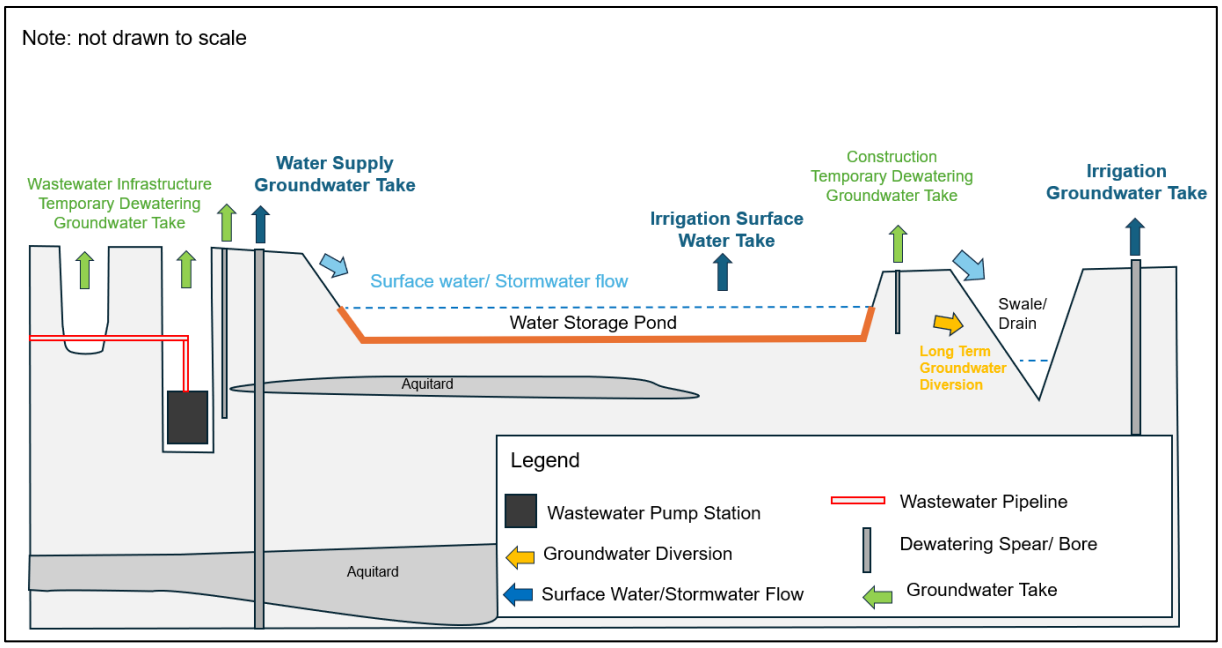


Figure 33: Conceptual Schematic of Water Flow Required and Induced by the Proposed Activities

9.2 Dewatering Effects Assessment

The combined rate of groundwater abstraction during the construction phase will vary as each of the ponds, the underpass, wastewater pumping stations and pipeline sections are progressively excavated, dewatered and backfilled. The construction programme has not been finalised, and some of these works may proceed concurrently. Individual excavations are nonetheless of short duration, with wet well dewatering assumed to be complete within 15 days and each 20 m pipeline trench within 10 days.

The rates summarised in Table 20 have been derived assuming winter groundwater levels as the starting condition and are presented as the sum of the inflows calculated for every excavation. This represents a conservative upper bound in which all works are assumed to be dewatering simultaneously, which is greater than the abstraction expected under any realistic construction sequence.

Based on current design, the largest construction groundwater abstraction is expected to occur for the underpass tunnel (**2,760 m^3/day** during construction). However, overlapping construction operations may increase cumulative dewatering rates up to **4,331 m^3/day** based on the largest pond, swale, wastewater pipeline, wet well and underpass occurring on the same day.

Pumped groundwater (where required during construction) and diverted inflows will be managed under a construction Water Management Plan and an Erosion and Sediment Control Plan for the project. Dewatering discharges will be directed to the local drainage network within the same catchment from which the groundwater is derived, so that the abstracted water is returned to the receiving environment rather than exported from the catchment.

Table 20: Summary of Construction Dewatering Flow Rates and Operational Cumulative Groundwater Diversions

EXCAVATION TYPE	TOTAL INFLOW (WINTER) CONSTRUCTION (m^3/day)	TOTAL INFLOW (WINTER) OPERATIONAL GROUNDWATER DIVERSION (m^3/day)	CUMULATIVE OPERATIONAL GROUNDWATER DIVERSION (m^3/day)
Storage Ponds See Section 5.4.1			
Main Pond	876	N/A	
Harness Pond	64		
Southern Pond	125		
Swales See Section 5.4.2			
Northern Swale	47 per 20 m section	457	1,268
Central East Swale	53 per 20 m section	459	
Central West Swale	73 per 20 m section	352	
WWPS See Section 6.2			
Main WWPS	384	N/A	
Trainers WWPS	367		
WW Pipelines See Section 6.3			
Trainers Area MH22 to MH23	210	N/A	
Main Harness Area MH8 to MH11	238		
Underpass See Section 7.3			
Underpass Tunnel	2,760 140 m length tunnel	N/A	

Notes: Table 20 combines results from different groundwater assessment methods, including steady state and transient dewatering analyses. These values are not directly comparable and should be interpreted with reference to the relevant methodology in Sections 5.4.1, 5.4.2, 6.2.2, 6.3.2 and 7.3.

9.2.1 Dewatering Effects on Nearby Bores

Of the surrounding groundwater users, there are nine current groundwater-related consents within 2.5 km of the Site (Figure 9). The furthest radial extent of dewatering impacts from proposed site activities assessed in this report is the 300 m extent of drawdown from construction of the underpass. The effect on the water column of the bores within 1 km of the underpass which are screened within the shallow aquifer is shown in Table 21. The projected change in available water column of all the bores is less than 6%. Therefore, effects of the construction dewatering on nearby bores will be minimal.

Table 21: Effects of Underpass Construction Drawdown on Nearby Bores

BORE ID	EASTING (NZTM)	NORTHING (NZTM)	BORE DEPTH (m)	CASING DEPTH (m)	STATIC WATER LEVEL (m bgl)	DISTANCE FROM PRODUCTION BORE (m)	AVAILABLE BORE WATER COLUMN (M)	PROJECTED DRAWDOWN (M)	PROJECTED CHANGE IN AVAILABLE WATER COLUMN (%)
72_4360	1810371	5807284	14	10	2	269	4	0.40	5.7%
69_1222	1810471	5807284	9	6	2	357	3	0.11	3.7%
72_5849	1810571	5807184	14	11	2	427	8	0.03	0%
72_4570	1810571	5807084	14	11	3.1	427	7	0.03	0%
69_571	1810671	5807184	14	11.58	2	526	9	0.00	0%
69_1248	1810671	5806884	6	3	2	581	0	0.00	-
69_533	1810771	5807184	7	5.4	2	626	2	0.00	0%
69_1359	1810672	5806784	11	6	2	631	3	0.00	0%
72_5017	1810770	5807584	12	6.5	2	768	4	0.00	0%
72_595	1810971	5806984	11.1	10	8	838	1	0.00	0%
69_272	1810872	5806684	10	6.0	2	854	3	0.00	0%
69_170	1810170	5808084	7.62	3.62	2	950	1	0.00	0%
69_1640	1809642	5807950	24	20	2	959	17	0.00	0%

Note: Groundwater levels are assumed to be 2 m bgl, consistent with observed shallow groundwater levels at the site.

9.2.2 Effects on Nearby Surface Water Features

Construction dewatering may temporarily intercept shallow groundwater that would otherwise contribute to onsite drains and downstream surface-water features. The modelled 0.25 m drawdown extent is approximately 19 m for the Main Pond, up to 135 m for a 20 m swale section, up to 138 m for the wastewater pump-station wet wells, up to 163 m for a 20 m wastewater pipeline section, and 300 m for the underpass.

The storage ponds are shallow and the Kahikatea Stand lies just beyond the modelled Central West Swale drawdown extent. Groundwater intercepted during construction is to be discharged to the local drainage network, upstream of the relevant works where practicable, under the construction Water Management Plan and Erosion and Sediment Control Plan. This will return water to the same catchment and maintain receiving-water flows. The lined ponds will not cause long-term groundwater diversion.

Visual inspections of affected drains and stream reaches during dewatering are required as part of construction compliance checks. Unexpected or sustained changes are to trigger hydrogeological review and, where necessary, adjustment of dewatering methods, rates or discharge locations. Subject to these controls, effects on nearby surface-water features are expected to be temporary, localised and manageable, with no additional operational surface-water flow or level monitoring required.

10 MONITORING AND MITIGATION

10.1 Groundwater Monitoring

Monitoring of the four existing piezometers (P2, P3, P7 and P8) with pressure transducers shall continue prior to commencement of construction. One barometric pressure logger will be installed at a secure, representative onsite location. The instruments will record at 15-minute intervals until construction commences. Quarterly manual water-level measurements and downloads will verify logger readings. During the quarterly visits the piezometers and monitoring equipment will be inspected, data will be downloaded and reviewed regularly, and the results will establish and maintain the pre-construction groundwater baseline.

Following construction and development of each production bore, aquifer testing shall be undertaken before the bore is commissioned. Testing shall comprise a step-rate test to confirm bore efficiency and select an appropriate pumping rate, followed by a three-day constant-rate test at the proposed operational rate, with recovery monitored until water levels have substantially recovered or for a duration sufficient to support reliable analysis. The existing test bore(s) shall be used as observation bores during each test, where hydraulically suitable, with groundwater levels recorded before, during and after pumping. Pumping rate, production-bore water level, observation-bore water levels and barometric pressure shall be recorded at an interval appropriate to each testing phase. The combined pumping and recovery datasets shall be analysed to refine site-specific aquifer parameters, including transmissivity, storativity and leakage, and to assess bore efficiency, sustainable yield, hydraulic connection between production bores, and potential interference effects. The refined parameters shall be used to update the drawdown assessment and confirm the proposed operational pumping rates.

10.2 Surface Water Monitoring

Potential effects on surface waters are limited to localised interception of groundwater contributions during construction dewatering for ponds, swales, the underpass and wastewater infrastructure. These effects have been assessed as temporary and manageable through construction methodology and the appropriate placement of temporary water discharge points rather than ongoing operational controls.

Given the short duration of dewatering activities, the absence of predicted long-term surface water effects, and the implementation of erosion and sediment control measures under the project's construction management framework, additional surface water flow or level monitoring is not considered necessary for the operational phase of the development. Visual inspections of drains and stream reaches during construction, undertaken as part of standard environmental compliance checks, are considered sufficient to confirm expected performance.

11 CONCLUSIONS

This assessment has considered the potential groundwater effects associated with the construction and operation of the Waikato Thoroughbred Racing Development, including:

- Construction dewatering for storage ponds, swales, an underpass and wastewater infrastructure
- The long-term groundwater diversion associated with the swales
- Groundwater take for potable and equine water supply as well as irrigation
- Wastewater disposal to land

The application is seeking a groundwater take up to the following volumes, if a deeper production bore can achieve the higher rates without causing an interference effect on nearby bores:

- Potable, stock and equine peak demand of approximately **467 m^3/day**
- Irrigation and track-conditioning water use of up to **725 m^3/day**

The proposed groundwater takes for water supply and irrigation could affect the groundwater levels in four nearby bores by more than 10% of their available water columns, at reduced flow rates below the projected irrigation water requirements. Further investigation into a deeper irrigation bore is recommended to reduce effects on nearby bores and obtain the higher flow rates required.

The investigations, monitoring and analytical assessments undertaken indicate that the predicted groundwater effects of temporary dewatering during construction and long-term effects of the swales are localised and can be appropriately managed through the proposed development design and associated management measures. Accordingly, groundwater effects associated with the proposed construction development are assessed to be minimal.

Continuous groundwater monitoring within the four onsite piezometers is recommended until commencement of construction to identify seasonal variation in groundwater levels and to validate the preliminary winter piezometric interpretation used in this assessment.

Ongoing groundwater water monitoring may be required to identify the extent of drawdown affecting settlement sensitive structures, subject to assessment by BCD geotechnical engineers. Standard construction phase environmental controls will provide sufficient verification of other predicted effects.

Calculations of attenuation of viruses and E.coli from the proposed wastewater disposal fields was assessed. Separation distances of 300 m from downgradient wells is considered sufficient for all sites based on the attenuation of viruses. Given the proposed location of all three sites is to be at least 300 m from the nearest downgradient wells, the effects from wastewater disposal are considered to be less than minor.

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APPENDIX A
GROUNDWATER LEVELS

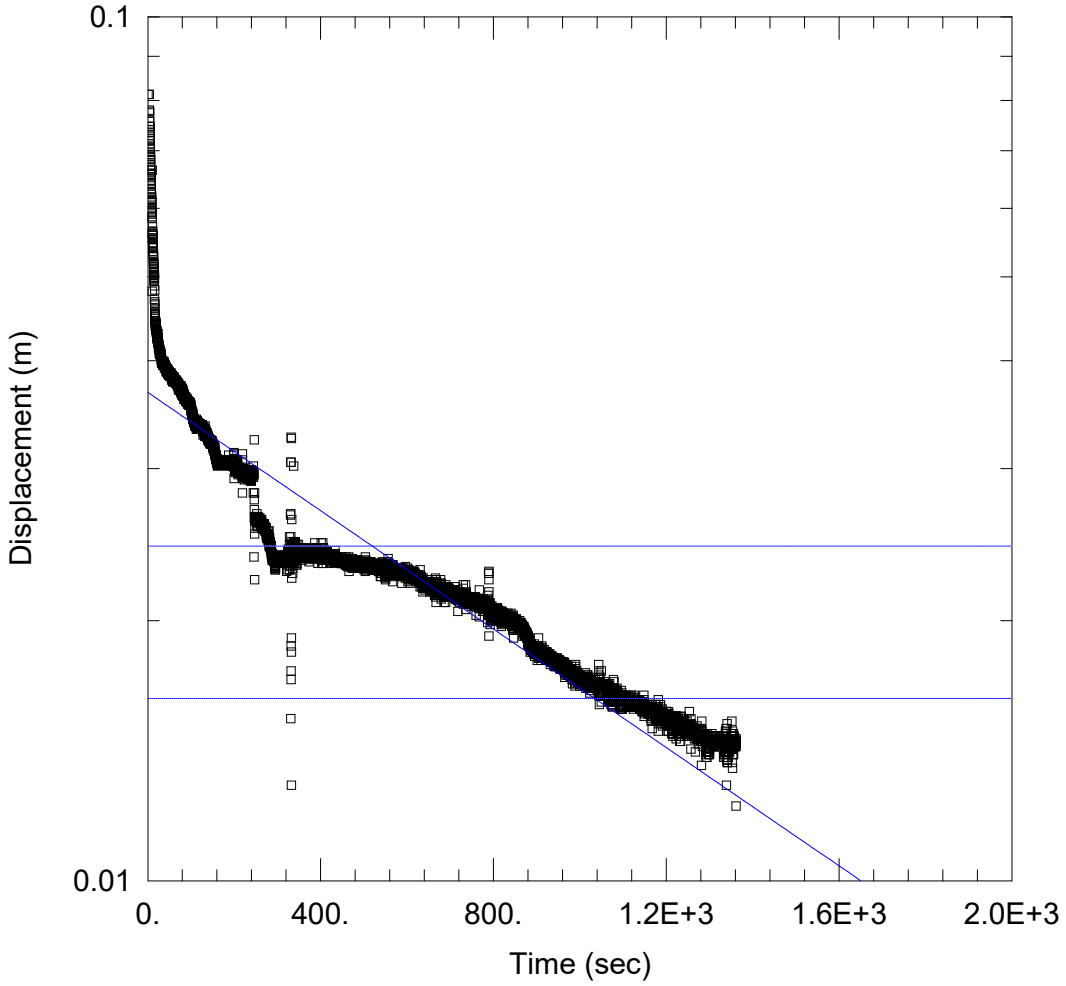


Table A1: Waikato Thoroughbred Racing Site Groundwater Levels

SITE NUMBER	GROUNDWATER LEVEL (m bgl)	GROUNDWATER LEVEL (m RL)
CPT01	0.9	52.7
CPT02	0.9	53.5
CPT03	0.9	53.8
CPT04	1.8	54.6
CPT05	2.9	52.0
CPT06	4.5	51.8
CPT07	1.3	53.2
CPT08	1.6	53.2
CPT09	1.1	53.5
CPT10	2.0	53.5
CPT11	1.8	53.2
CPT12	2.1	53.6
HA01	1.6	52.1
HA02	0.9	53.5
HA03	1.0	53.6
HA04	2.8	53.7
HA05	2.7	52.1
HA06	NE	NE
HA07	2.7	51.4
HA08	1.2	53.7
HA09	1.5	52.8
HA10	2.1	52.7
HA11	1.7	53.8
HA12	1.8	53.8
HA13	1.6	53.5
HA14	1.5	53.2
HA15	4.2	50.7
HA16	1.5	50.4
HA17	1.8	53.4
HA18	NE	NE
HA19	1.9	53.6
HA20	2.0	53.8

APPENDIX B
HYDRAULIC TESTING





WELL TEST ANALYSIS

Data Set: V:\...\P2 FHT 1.aqt
Date: 09/08/26 Time: 09:40:19

PROJECT INFORMATION

Company: WGA
Project: 253138
Location: Duncan Road Tamahere
Test Date: 25/08/26

AQUIFER DATA

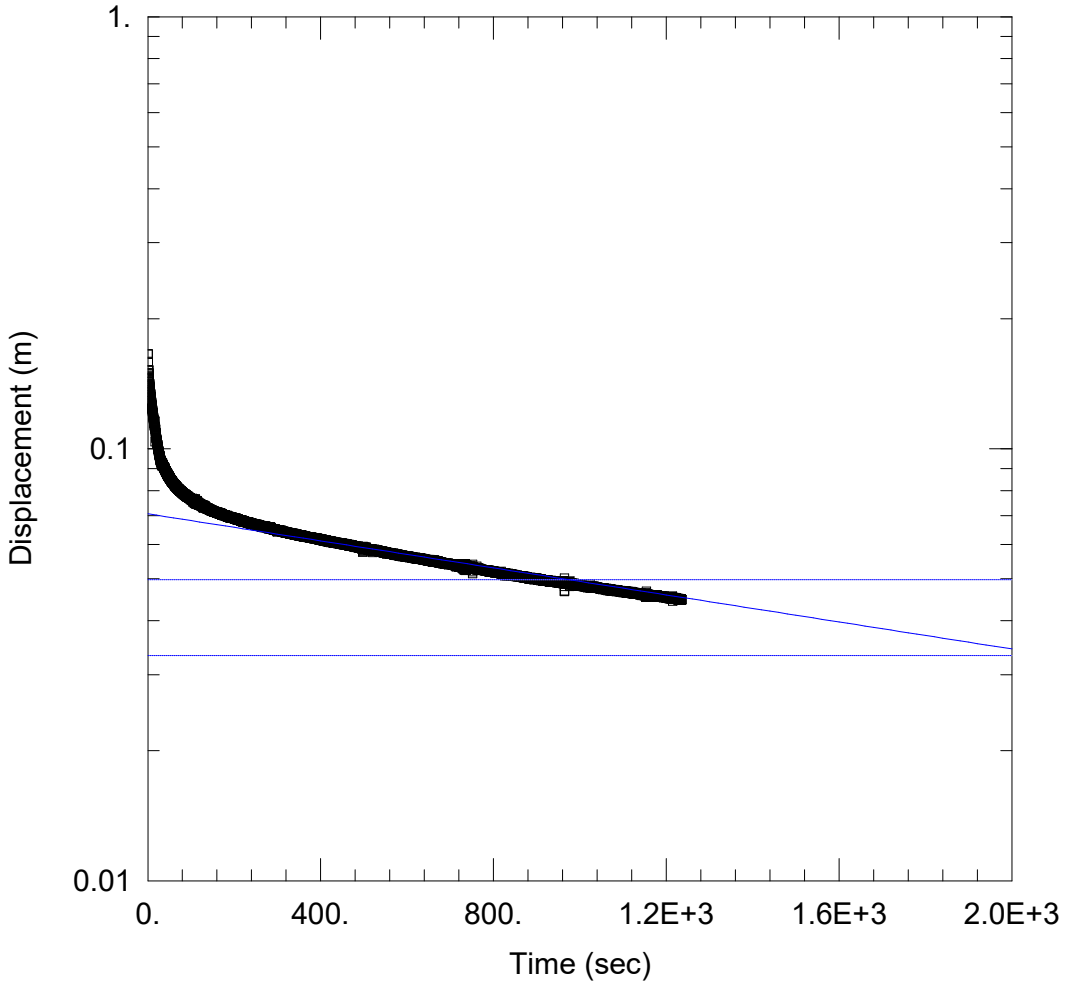
Saturated Thickness: 13.43 m Anisotropy Ratio (Kz/Kr): 1.

WELL DATA (P2 FHT 1)

Initial Displacement: 0.0813 m Static Water Column Height: 1.41 m
Total Well Penetration Depth: 1.41 m Screen Length: 1.41 m
Casing Radius: 0.015 m Well Radius: 0.02 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice
K = 1.746E-7 m/sec y0 = 0.03676 m



WELL TEST ANALYSIS

Data Set: V:\...\P2 RHT 1.aqt
Date: 09/08/26 Time: 09:44:01

PROJECT INFORMATION

Company: WGA
Project: 253138
Location: Duncan Road Tamahere
Test Date: 25/08/26

AQUIFER DATA

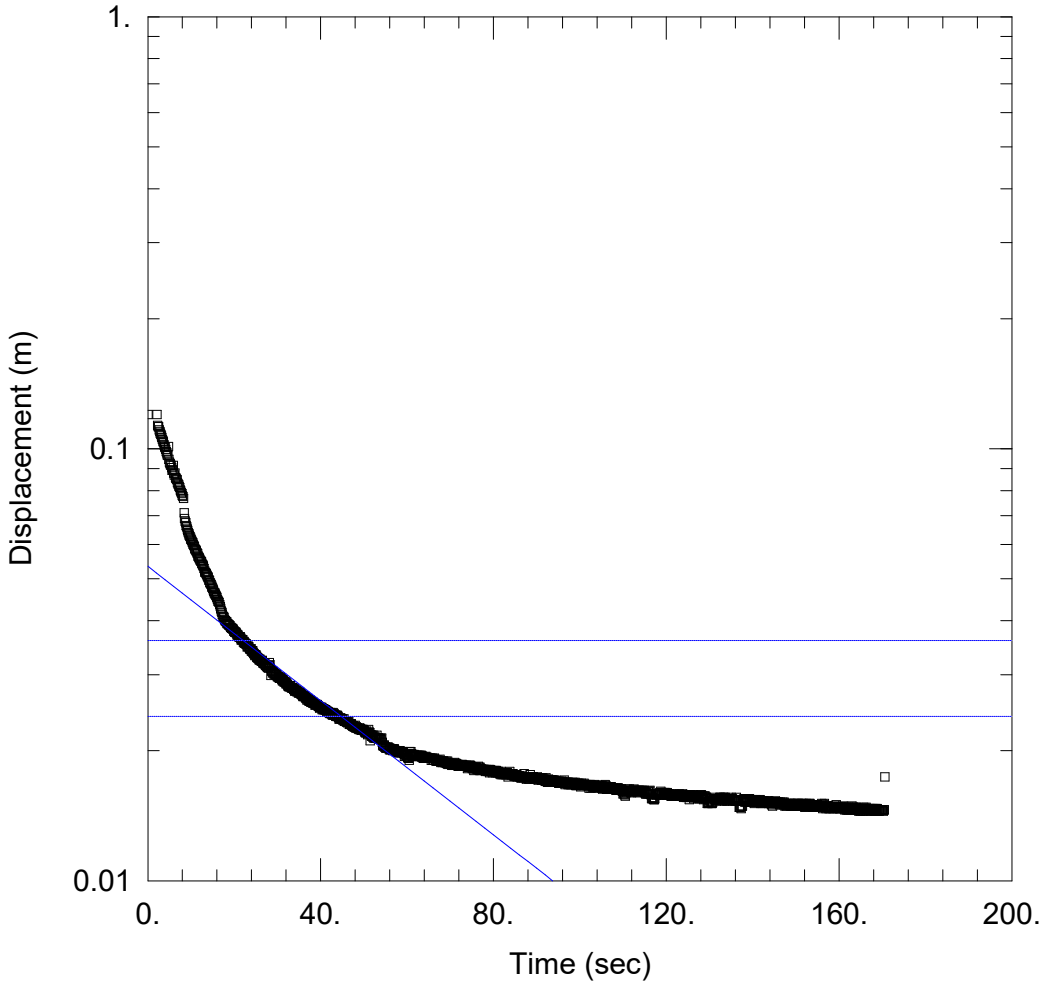
Saturated Thickness: 13.43 m Anisotropy Ratio (Kz/Kr): 1.

WELL DATA (P2 RHT 1)

Initial Displacement: 0.166 m Static Water Column Height: 1.41 m
Total Well Penetration Depth: 1.41 m Screen Length: 1.41 m
Casing Radius: 0.015 m Well Radius: 0.02 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice
K = 7.972E-8 m/sec y0 = 0.0707 m



WELL TEST ANALYSIS

Data Set: V:\...\P7 FHT 1.aqt
Date: 09/08/26 Time: 10:01:40

PROJECT INFORMATION

Company: WGA
Project: 253138
Location: Duncan Road Tamahere
Test Date: 25/08/26

AQUIFER DATA

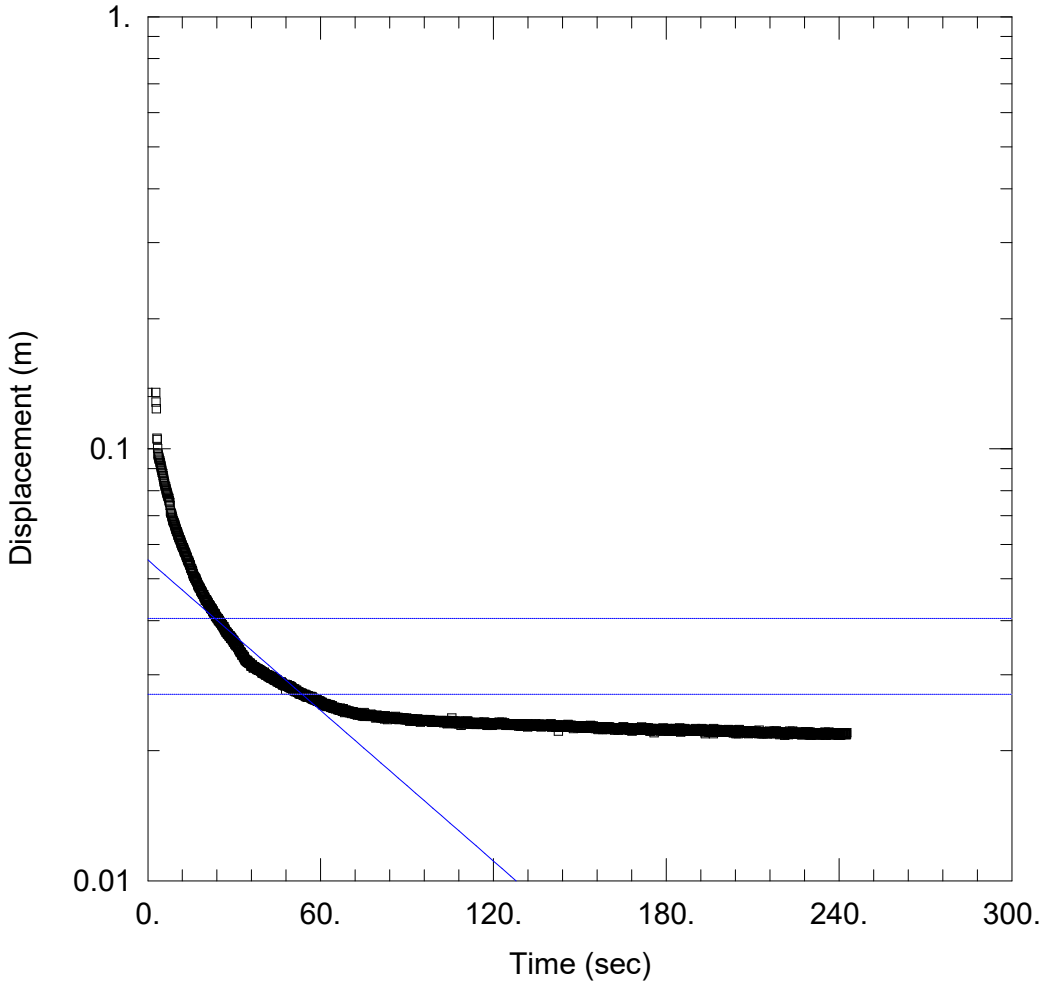
Saturated Thickness: 12.95 m Anisotropy Ratio (Kz/Kr): 1.

WELL DATA (P7 FHT 1)

Initial Displacement: 0.12 m Static Water Column Height: 0.95 m
Total Well Penetration Depth: 0.95 m Screen Length: 0.95 m
Casing Radius: 0.015 m Well Radius: 0.02 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice
K = 5.176E-6 m/sec y0 = 0.05343 m



WELL TEST ANALYSIS

Data Set: V:\...\P7 FHT 2.aqt
Date: 09/08/26 Time: 10:12:46

PROJECT INFORMATION

Company: WGA
Project: 253138
Location: Duncan Road Tamahere
Test Date: 25/08/26

AQUIFER DATA

Saturated Thickness: 12.95 m Anisotropy Ratio (Kz/Kr): 1.

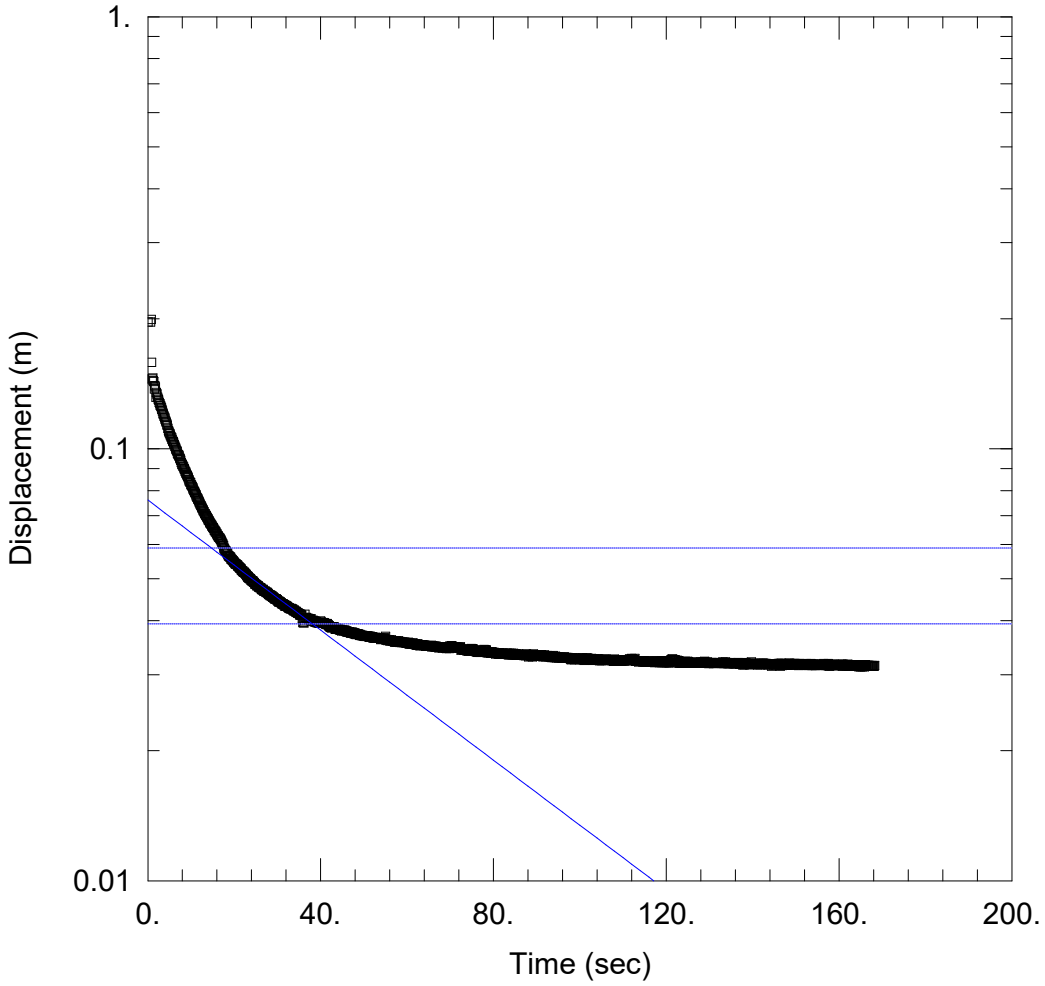
WELL DATA (P7 FHT 2)

Initial Displacement: 0.135 m Static Water Column Height: 0.95 m
Total Well Penetration Depth: 0.95 m Screen Length: 0.95 m
Casing Radius: 0.015 m Well Radius: 0.02 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice

K = 3.872E-6 m/sec y0 = 0.05531 m



WELL TEST ANALYSIS

Data Set: V:\...\P7 RHT 1.aqt
Date: 09/08/26 Time: 10:02:03

PROJECT INFORMATION

Company: WGA
Project: 253138
Location: Duncan Road Tamahere
Test Date: 25/08/26

AQUIFER DATA

Saturated Thickness: 12.95 m Anisotropy Ratio (Kz/Kr): 1.

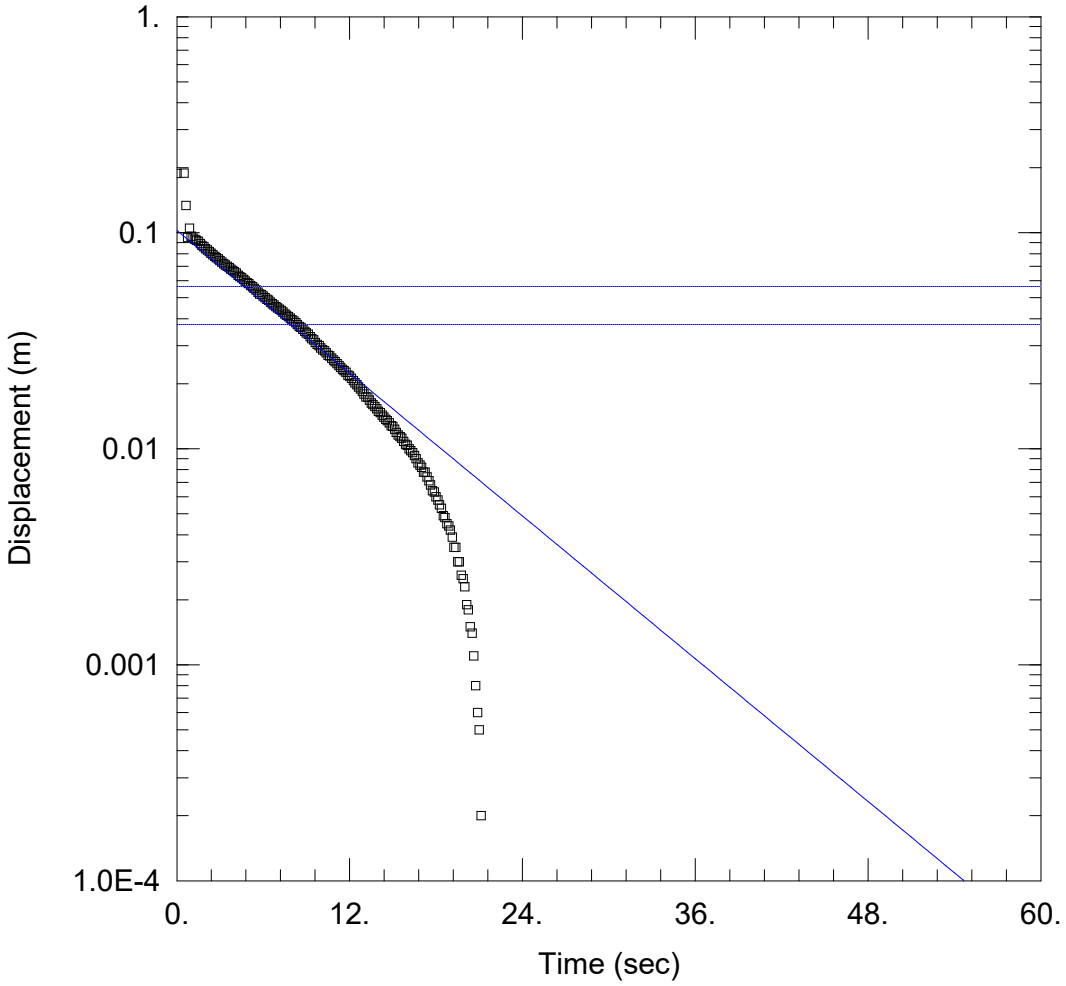
WELL DATA (P7 RHT 1)

Initial Displacement: 0.1965 m Static Water Column Height: 0.95 m
Total Well Penetration Depth: 0.95 m Screen Length: 0.95 m
Casing Radius: 0.015 m Well Radius: 0.02 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice

K = 5.018E-6 m/sec y0 = 0.0761 m



WELL TEST ANALYSIS

Data Set: V:\...\P7 RHT 2.aqt
Date: 09/09/26 Time: 09:30:47

PROJECT INFORMATION

Company: WGA
Project: 253138
Location: Duncan Road Tamahere
Test Date: 25/08/26

AQUIFER DATA

Saturated Thickness: 12.95 m Anisotropy Ratio (Kz/Kr): 1.

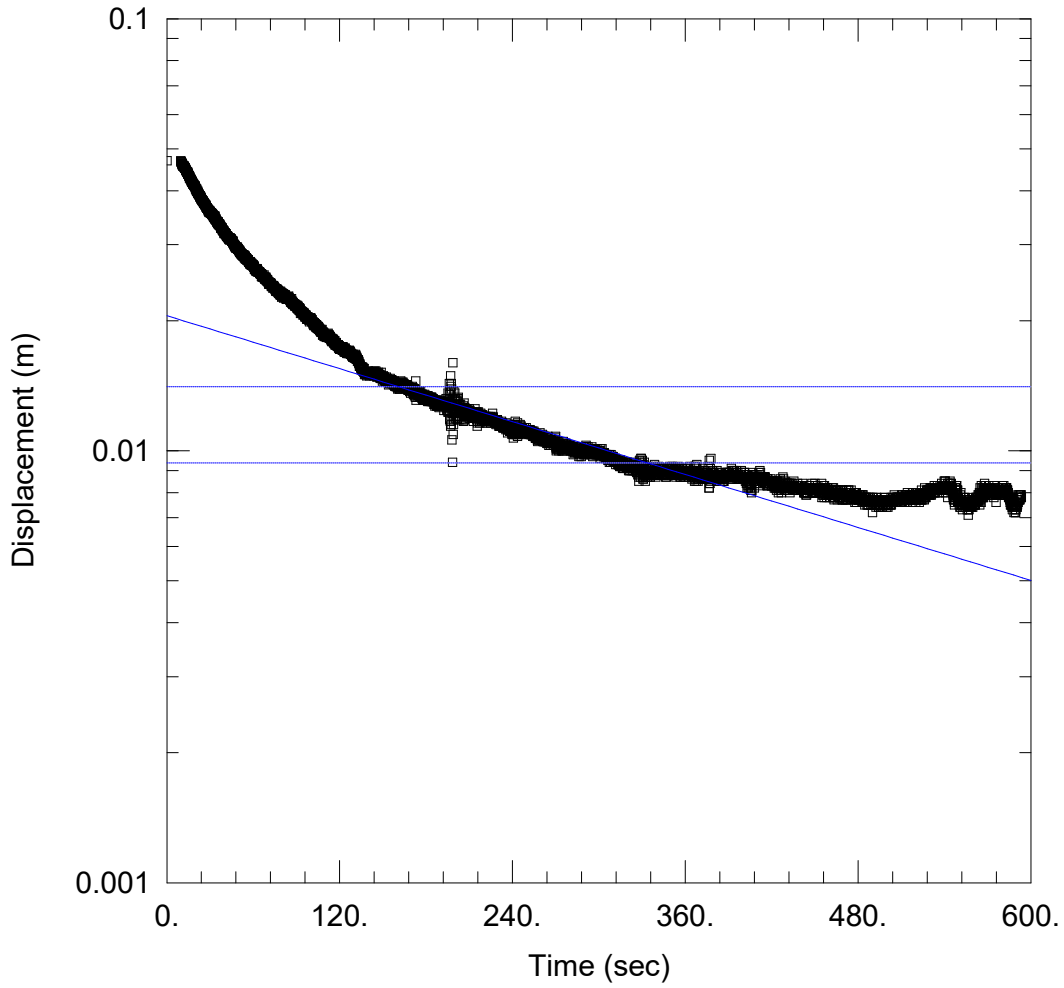
WELL DATA (P7 RHT 2)

Initial Displacement: 0.188 m Static Water Column Height: 0.95 m
Total Well Penetration Depth: 0.95 m Screen Length: 0.95 m
Casing Radius: 0.015 m Well Radius: 0.02 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice

K = 3.672E-5 m/sec y0 = 0.1026 m



WELL TEST ANALYSIS

Data Set: V:\...\P8 FHT 2.aqt
Date: 09/08/26 Time: 09:34:17

PROJECT INFORMATION

Company: WGA
Project: 253138
Location: Duncan Road Tamahere
Test Date: 25/08/26

AQUIFER DATA

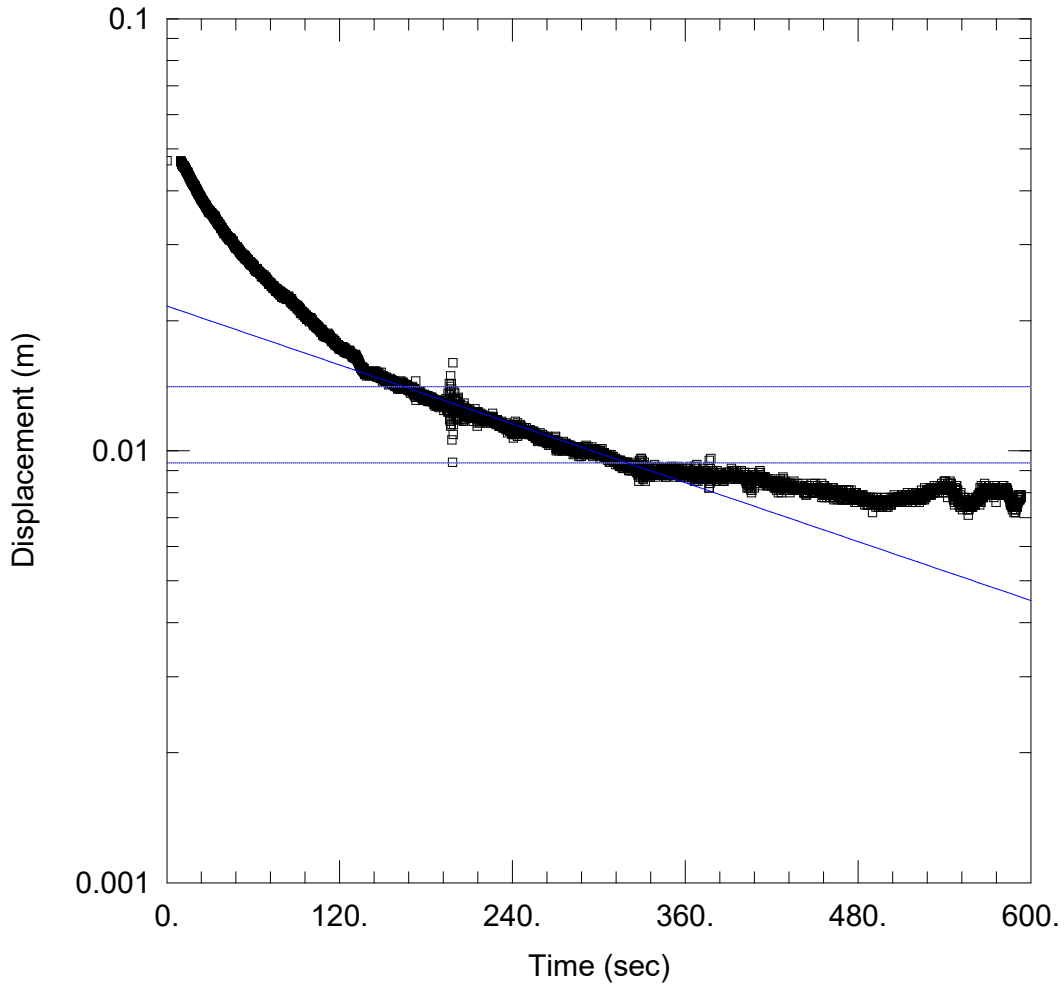
Saturated Thickness: 14.27 m Anisotropy Ratio (Kz/Kr): 1.

WELL DATA (P8 FHT 2)

Initial Displacement: 0.0469 m Static Water Column Height: 1.3 m
Total Well Penetration Depth: 1.3 m Screen Length: 1.3 m
Casing Radius: 0.015 m Well Radius: 0.02 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice
K = 5.506E-7 m/sec y0 = 0.02054 m



WELL TEST ANALYSIS

Data Set: V:\...\P8 FHT 2.aqt
Date: 09/08/26 Time: 11:30:33

PROJECT INFORMATION

Company: WGA
Project: 253138
Location: Duncan Road Tamahere
Test Date: 25/08/26

AQUIFER DATA

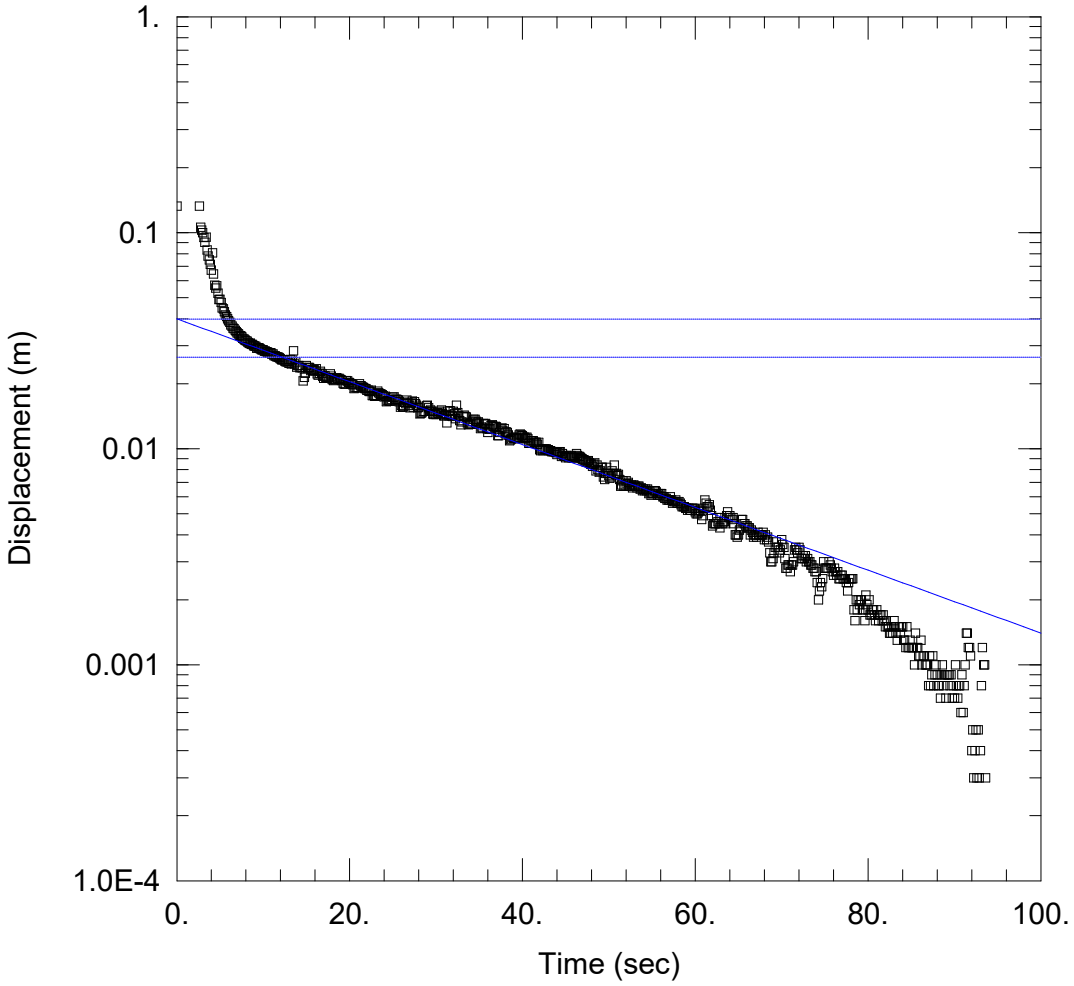
Saturated Thickness: 14.27 m Anisotropy Ratio (Kz/Kr): 1.

WELL DATA (P8 FHT 2)

Initial Displacement: 0.0469 m Static Water Column Height: 1.3 m
Total Well Penetration Depth: 1.3 m Screen Length: 1.3 m
Casing Radius: 0.015 m Well Radius: 0.02 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice
K = 6.126E-7 m/sec y0 = 0.02163 m



WELL TEST ANALYSIS

Data Set: V:\...\P8 RHT 1.aqt
Date: 09/09/26 Time: 09:32:03

PROJECT INFORMATION

Company: WGA
Project: 253138
Location: Duncan Road Tamahere
Test Date: 25/08/26

AQUIFER DATA

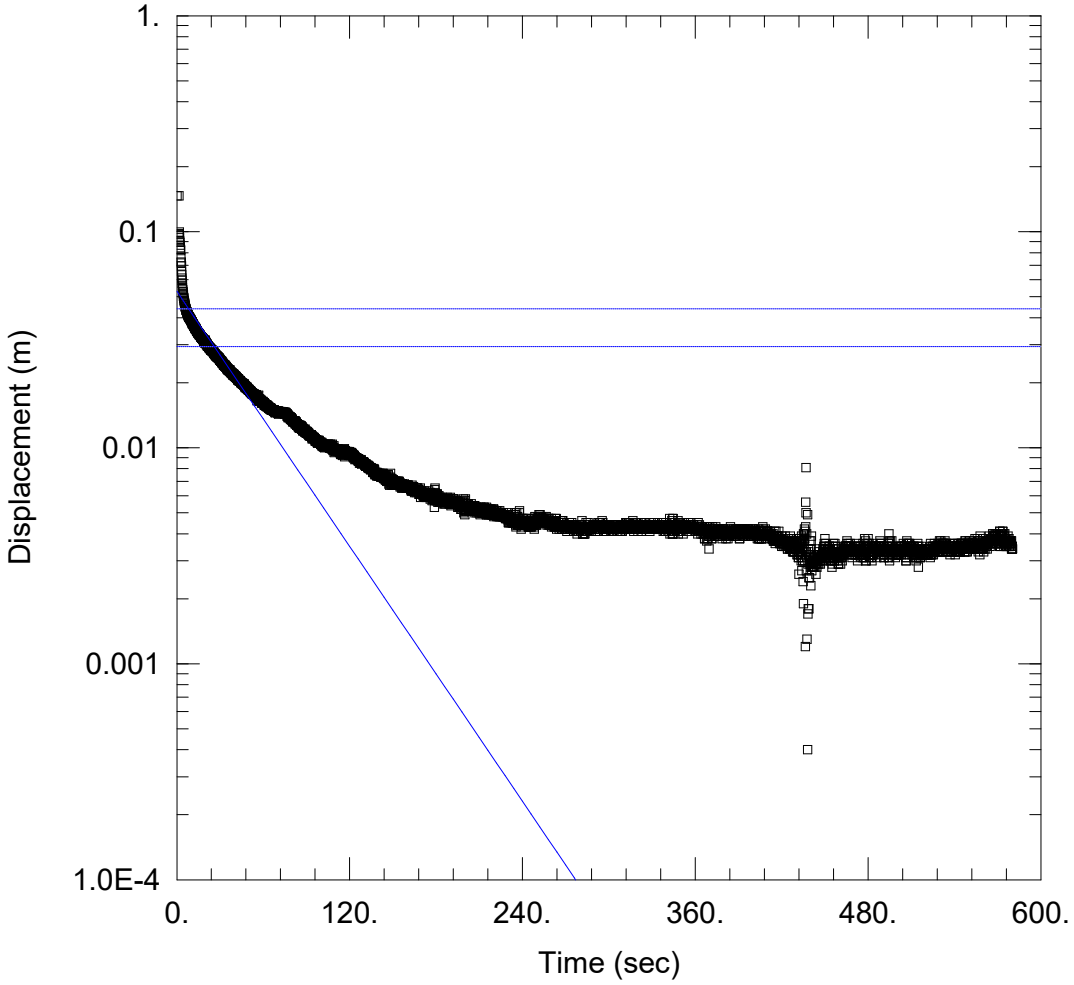
Saturated Thickness: 14.27 m Anisotropy Ratio (Kz/Kr): 1.

WELL DATA (P8 RHT 1)

Initial Displacement: 0.1327 m Static Water Column Height: 1.3 m
Total Well Penetration Depth: 1.3 m Screen Length: 1.3 m
Casing Radius: 0.015 m Well Radius: 0.02 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice
K = 7.844E-6 m/sec y0 = 0.03996 m



WELL TEST ANALYSIS

Data Set: V:\...\P8 RHT 2.aqt
Date: 09/09/26 Time: 09:33:22

PROJECT INFORMATION

Company: WGA
Project: 253138
Location: Duncan Road Tamahere
Test Date: 25/08/26

AQUIFER DATA

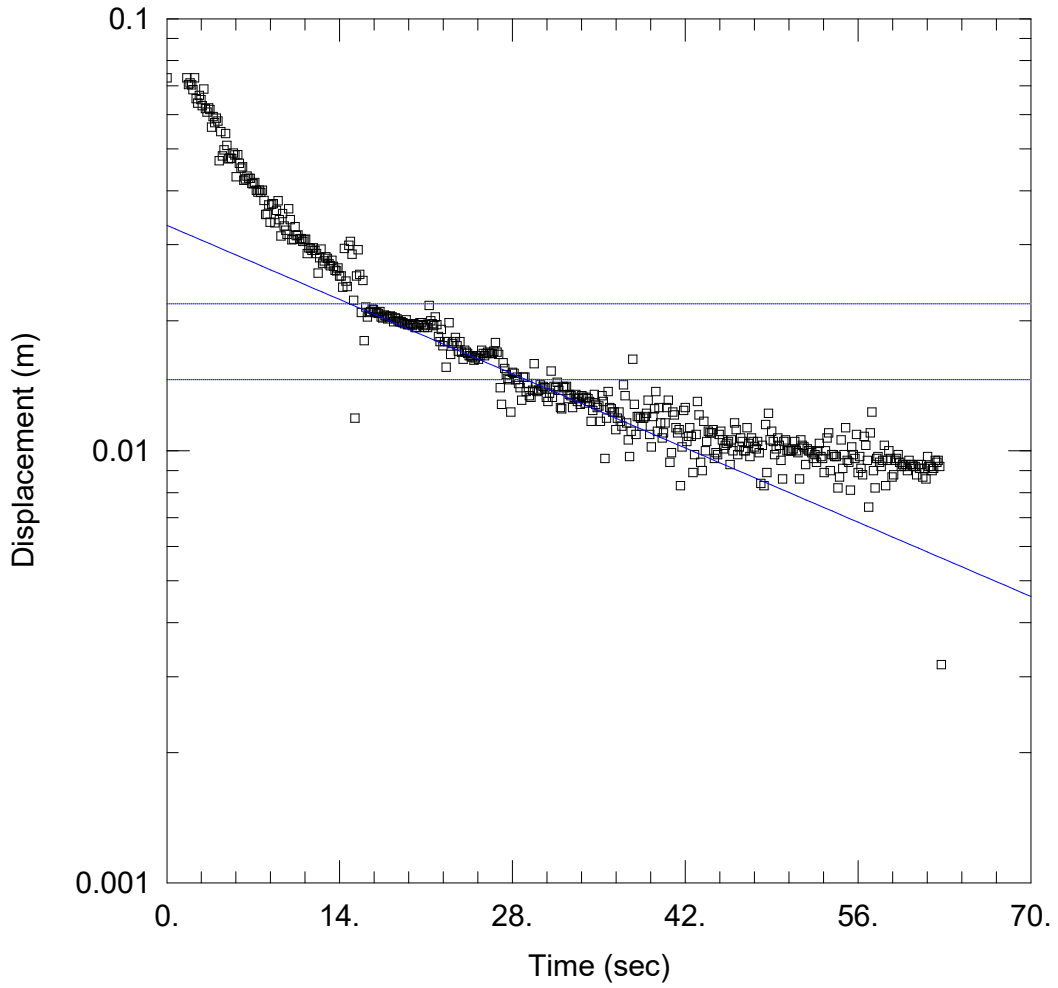
Saturated Thickness: 14.27 m Anisotropy Ratio (Kz/Kr): 1.

WELL DATA (P8 RHT 2)

Initial Displacement: 0.1468 m Static Water Column Height: 1.3 m
Total Well Penetration Depth: 1.3 m Screen Length: 1.3 m
Casing Radius: 0.015 m Well Radius: 0.02 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice
K = 5.304E-6 m/sec y0 = 0.05303 m



WELL TEST ANALYSIS

Data Set: V:\...\P10 FHT 1.aqt
Date: 09/08/26 Time: 11:29:17

PROJECT INFORMATION

Company: WGA
Project: 253138
Location: Duncan Road Tamahere
Test Date: 25/08/26

AQUIFER DATA

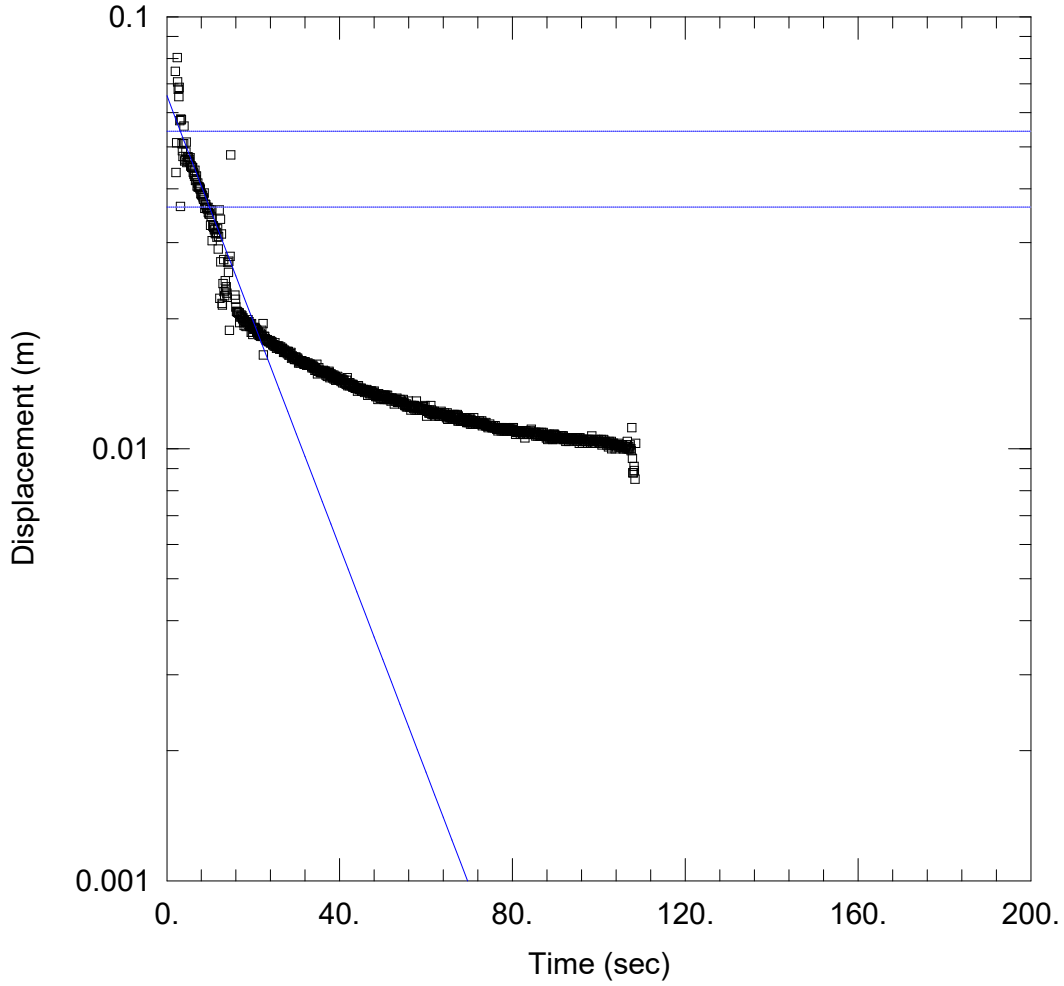
Saturated Thickness: 14.84 m Anisotropy Ratio (Kz/Kr): 1.

WELL DATA (P10 FHT 1)

Initial Displacement: 0.073 m Static Water Column Height: 1.34 m
Total Well Penetration Depth: 1.34 m Screen Length: 1.34 m
Casing Radius: 0.015 m Well Radius: 0.02 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice
K = 6.481E-6 m/sec y0 = 0.03327 m



WELL TEST ANALYSIS

Data Set: V:\...\P10 FHT 2_CMH.aqt
Date: 09/09/26 Time: 09:34:59

PROJECT INFORMATION

Company: WGA
Project: 253138
Location: Duncan Road Tamahere
Test Date: 25/08/26

AQUIFER DATA

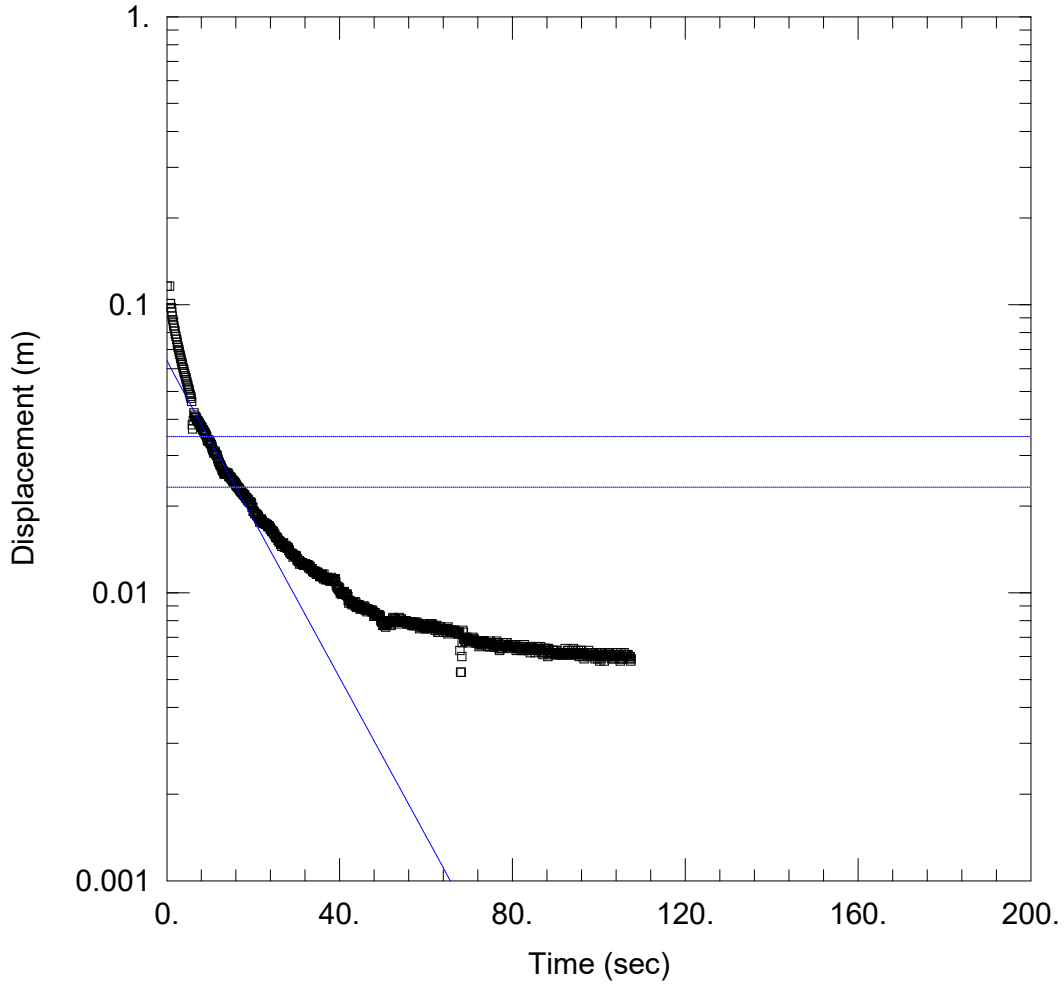
Saturated Thickness: 14.84 m Anisotropy Ratio (Kz/Kr): 1.

WELL DATA (P10 FHT 2)

Initial Displacement: 0.1812 m Static Water Column Height: 1.34 m
Total Well Penetration Depth: 1.34 m Screen Length: 1.34 m
Casing Radius: 0.015 m Well Radius: 0.02 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice
K = 1.378E-5 m/sec y0 = 0.06569 m



WELL TEST ANALYSIS

Data Set: V:\...\P10 RHT 1.aqt
Date: 09/09/26 Time: 09:36:16

PROJECT INFORMATION

Company: WGA
Project: 253138
Location: Duncan Road Tamahere
Test Date: 25/08/26

AQUIFER DATA

Saturated Thickness: 14.84 m Anisotropy Ratio (Kz/Kr): 1.

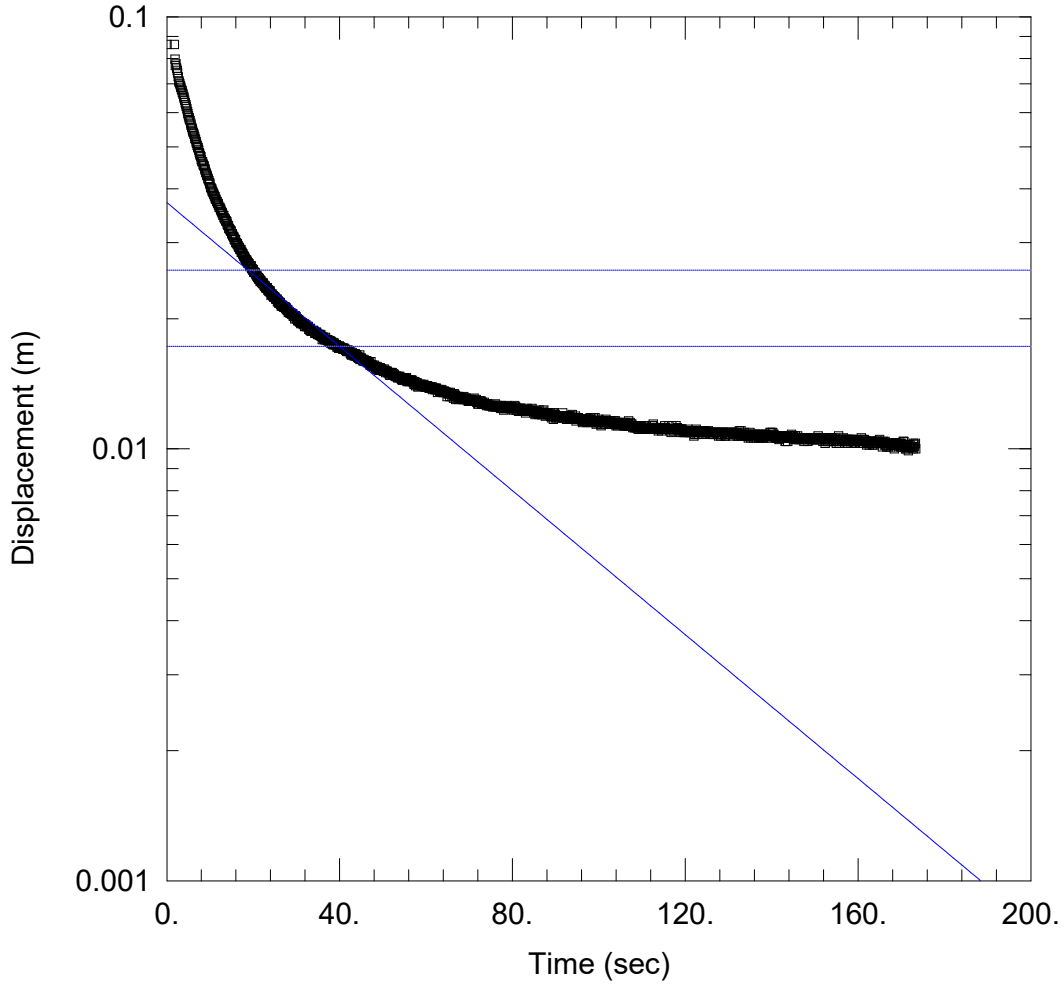
WELL DATA (P10 RHT 1)

Initial Displacement: 0.1162 m Static Water Column Height: 1.34 m
Total Well Penetration Depth: 1.34 m Screen Length: 1.34 m
Casing Radius: 0.015 m Well Radius: 0.02 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice

K = 1.452E-5 m/sec y0 = 0.06391 m



WELL TEST ANALYSIS

Data Set: V:\...\P10 RHT 2.aqt
Date: 09/08/26 Time: 09:27:28

PROJECT INFORMATION

Company: WGA
Project: 253138
Location: Duncan Road Tamahere
Test Date: 25/08/26

AQUIFER DATA

Saturated Thickness: 14.84 m Anisotropy Ratio (Kz/Kr): 1.

WELL DATA (P10 RHT 2)

Initial Displacement: 0.0863 m Static Water Column Height: 1.34 m
Total Well Penetration Depth: 1.34 m Screen Length: 1.34 m
Casing Radius: 0.015 m Well Radius: 0.02 m

SOLUTION

Aquifer Model: Unconfined Solution Method: Bouwer-Rice
K = 4.4E-6 m/sec y0 = 0.03715 m

APPENDIX C
GROUNDWATER TAKE EFFECTS ON NEARBY BORES



Table 1: Nearby Bores in Aquifer 1 (up to 25 m deep)

Bore ID	Easting (NZTM)	Northing (NZTM)	Bore Depth (m)	Casing Depth (m)	Assumptions	Static Water Level (m bgl)	Distance from Test Bore (m)	Distance From Potential Irrigation Bore (m)	Available Bore Water Column (m)	Potable Supply Bore		Irrigation Bore		Combined Drawdown	
										Projected Drawdown (m)	Change in Available Water (%)	Projected Drawdown (m)	Change in Available Water (%)	Projected Drawdown (m)	Change in Available Water (%)
69_1640	1809642	5807950	24	20	2, 3, 7	2.0	241	1,332	18.0	0.36	2.0%	0.01	0.1%	0.37	2.1%
69_170	1810170	5808084	7.62	3.62	2, 3, 6, 7	2.0	422	1,279	5.62	0.34	6.1%	0.01	0.2%	0.35	6.3%
69_1339	1809569	5808283	7.3	5.3	6, 7	2.0	540	1,658	5.3	0.33	6.2%	0.01	0.2%	0.34	6.4%
72_4360	1810371	5807284	14	10		4.8	751	468	5.2	0.3	5.8%	0.03	0.5%	0.33	6.3%
69_1222	1810471	5807284	9	6	7	2.0	824	484	4.0	0.29	7.3%	0.03	0.7%	0.32	8.0%
72_5017	1810770	5807584	12	6.5	7	2.0	956	878	4.5	0.27	6.1%	0.02	0.4%	0.29	6.5%
72_5849	1810571	5807184	14	11	7	2.0	965	432	9.0	0.27	3.0%	0.03	0.3%	0.3	3.4%
72_4570	1810571	5807084	14	11		3.1	1,033	351	7.9	0.26	3.4%	0.03	0.4%	0.3	3.7%
69_571	1810671	5807184	14	11.58	7	2.0	1,043	492	9.58	0.26	2.8%	0.03	0.3%	0.29	3.0%
69_369	1810269	5808784	11.4	7.4	2, 3, 7	2.0	1,056	1,968	5.4	0.26	4.9%	0.01	0.1%	0.27	5.0%
69_88	1809268	5808782	9.3	5.3	2, 3, 6, 7	2.0	1,123	2,240	7.3	0.25	3.5%	0.01	0.1%	0.26	3.5%
69_533	1810771	5807184	7	5.4	7	2.0	1,124	564	3.4	0.25	7.5%	0.02	0.7%	0.28	8.2%
69_589	1810669	5808584	16.7	14.3	7	2.0	1,127	1,797	12.3	0.25	2.1%	0.01	0.1%	0.26	2.1%
72_515	1810569	5808684	15	12	6	11.2	1,131	1,881	3.8	0.25	6.6%	0.01	0.2%	0.26	6.8%
69_1052	1809168	5808782	10.5	6.5	7	2.0	1,177	2,290	4.5	0.25	5.5%	0.01	0.1%	0.25	5.6%
69_440	1809268	5808882	17.6	13.6	2, 3, 7	2.0	1,210	2,328	11.6	0.24	2.1%	0.01	0.0%	0.25	2.1%
72_2875	1809468	5808983	26	15	6	15.0	1,224	2,336	11.0	0.24	2.2%	0.01	0.0%	0.25	2.2%
69_1248	1810671	5806884	6	3	6, 7	2.0	1,248	335	4.0	0.24	5.9%	0.03	0.8%	0.27	6.7%
69_2107	1810769	5808685	12.8	11.5		8.8	1,269	1,916	2.7	0.24	8.7%	0.01	0.3%	0.24	9.0%
72_7366	1809134	5806749	36.8	15		4.4	1,282	1,211	10.6	0.23	2.2%	0.01	0.1%	0.25	2.3%
69_1359	1810672	5806784	11	6	7	2.0	1,325	331	4.0	0.23	5.7%	0.03	0.8%	0.26	6.5%
72_3923	1810969	5808585	11.75	8.75	6	9.3	1,363	1,876	2.45	0.22	9.1%	0.01	0.3%	0.23	9.4%
72_595	1810971	5806984	11.1	10	6	8.0	1,403	650	3.1	0.22	7.1%	0.02	0.7%	0.24	7.8%
69_272	1810872	5806684	10	6	2, 3, 7	2.0	1,531	545	4.0	0.2	5.1%	0.02	0.6%	0.23	5.7%
72_11130	1811639	5807650	21	12		3.5	1,804	1,541	8.5	0.18	2.1%	0.01	0.1%	0.19	2.2%
69_252	1810673	5806084	5.5	1.5	2, 3, 6, 7	2.0	1,922	804	3.5	0.16	4.7%	0.02	0.5%	0.18	5.2%
69_99	1811569	5808686	5.48	1.48	2, 3, 6, 7	2.0	1,932	2,235	3.48	0.16	4.7%	0.01	0.2%	0.17	4.8%
72_6347	1810673	5806003	12.5	8.5	7	2.0	1,995	878	6.5	0.16	2.4%	0.02	0.3%	0.17	2.7%
69_1523	1808071	5806880	24.38	19.2	7	2.0	2,004	2,273	17.2	0.16	0.9%	0.01	0.0%	0.16	0.9%
72_6348	1810652	5805978	15	11	2, 3, 7	2.0	2,009	894	9.0	0.16	1.7%	0.02	0.2%	0.17	1.9%
72_2053	1810968	5809485	10.5	6.5	6	6.0	2,012	2,740	4.5	0.16	3.5%	0.0	0.1%	0.16	3.5%
69_1009	1810267	5809784	12.8	11.5	7	2.0	2,012	2,968	9.5	0.16	1.6%	0.0	0.0%	0.16	1.7%
72_10384	1811772	5808564	10	4	6	5.0	2,069	2,257	5.0	0.15	3.0%	0.01	0.1%	0.16	3.1%
69_1885	1808572	5806181	8.53	4.53	2, 3, 7	2.0	2,072	1,882	2.53	0.15	5.9%	0.01	0.3%	0.16	6.2%
69_453	1810767	5809685	8.8	5.7		3.6	2,084	2,899	2.1	0.15	7.1%	0.0	0.2%	0.15	7.3%
72_78	1810767	5809685	12.8	9.8	7	2.0	2,084	2,899	7.8	0.15	1.9%	0.0	0.0%	0.15	2.0%
69_299	1810867	5809685	5.49	1.49	2, 3, 6, 7	2.0	2,130	2,915	3.49	0.15	4.2%	0.0	0.1%	0.15	4.3%
69_1286	1810667	5809785	12	9		5.7	2,133	2,986	3.3	0.15	4.4%	0.0	0.1%	0.15	4.5%
69_744	1808171	5806480	14	10	2, 3, 7	2.0	2,141	2,198	8.0	0.14	1.8%	0.01	0.1%	0.15	1.9%

Bore ID	Easting (NZTM)	Northing (NZTM)	Bore Depth (m)	Casing Depth (m)	Assumptions	Static Water Level (m bgl)	Distance from Test Bore (m)	Distance From Potential Irrigation Bore (m)	Available Bore Water Column (m)	Potable Supply Bore		Irrigation Bore		Combined Drawdown	
										Projected Drawdown (m)	Change in Available Water (%)	Projected Drawdown (m)	Change in Available Water (%)	Projected Drawdown (m)	Change in Available Water (%)
72_9208	1807800	5807136	19	3.5	6	7.1	2,153	2,563	11.9	0.14	1.2%	0.0	0.0%	0.15	1.2%
69_729	1807868	5808680	16.5	13	6	12.0	2,155	3,098	4.5	0.14	3.2%	0.0	0.1%	0.15	3.2%
69_841	1810467	5809985	5.4	1.4	2, 3, 6, 7	2.0	2,256	3,170	3.4	0.14	4.0%	0.0	0.1%	0.14	4.1%
69_419	1808272	5806180	18.5	16	7	2.0	2,269	2,167	14.0	0.13	1.0%	0.01	0.0%	0.14	1.0%
69_1326	1807670	5807080	23.47	19.66	7	2.0	2,295	2,686	17.66	0.13	0.7%	0.0	0.0%	0.14	0.8%
72_9257	1809894	5810132	9	6		3.5	2,315	3,345	2.5	0.13	5.2%	0.0	0.1%	0.13	5.3%
69_1859	1807871	5806580	11.58	7.58	2, 3, 7	2.0	2,328	2,483	5.58	0.13	2.3%	0.0	0.1%	0.13	2.4%
69_572	1808072	5806280	23.7	19.7	2, 3, 7	2.0	2,345	2,334	17.7	0.13	0.7%	0.01	0.0%	0.13	0.8%
69_2022	1812170	5808187	17	13		8.0	2,356	2,284	5.0	0.13	2.5%	0.01	0.1%	0.13	2.6%
69_756	1810767	5809985	20	15.3	7	2.0	2,356	3,196	13.3	0.13	1.0%	0.0	0.0%	0.13	1.0%
72_5983	1807570	5807179	21	12.6	7	2.0	2,361	2,797	10.6	0.13	1.2%	0.0	0.0%	0.13	1.2%
72_5994	1807570	5807179	21	12.6	7	2.0	2,361	2,797	10.6	0.13	1.2%	0.0	0.0%	0.13	1.2%
69_592	1807671	5806879	12.8	10.05	7	2.0	2,366	2,673	8.05	0.13	1.6%	0.0	0.0%	0.13	1.6%
72_7095	1807518	5808358	17	13		4.0	2,387	3,218	9.0	0.12	1.4%	0.0	0.0%	0.13	1.4%
69_1907	1811668	5809386	10	6.5	7	2.0	2,407	2,891	4.5	0.12	2.7%	0.0	0.1%	0.13	2.8%
69_1648	1807671	5806779	12.8	8.8	2, 3, 7	2.0	2,407	2,672	6.8	0.12	1.8%	0.0	0.1%	0.13	1.9%
69_223	1812191	5808401	7.5	2.92	6	4.1	2,420	2,434	3.4	0.12	3.6%	0.0	0.1%	0.13	3.7%
69_22	1807871	5806380	14.6	10.6	2, 3, 7	2.0	2,440	2,510	8.6	0.12	1.4%	0.0	0.1%	0.13	1.5%
72_10220	1810842	5810048	13.1	7.1		2.88	2,444	3,269	4.22	0.12	2.9%	0.0	0.1%	0.12	2.9%
72_308	1812169	5808687	15	6.5	6	10.0	2,483	2,614	5.0	0.12	2.4%	0.0	0.1%	0.12	2.4%
69_521	1809066	5810183	24	21.6	7	2.0	2,490	3,600	19.6	0.12	0.6%	0.0	0.0%	0.12	0.6%
69_218	1807370	5807479	18.96	18.96		12.23	2,496	3,046	6.73	0.12	1.7%	0.0	0.0%	0.12	1.8%
69_228	1812172	5806886	6.1	2.1	2, 3, 6, 7	2.0	2,508	1,830	4.1	0.12	2.8%	0.01	0.2%	0.12	3.0%
72_894	1811673	5806085	12.8	12.4		8.2	2,520	1,518	4.2	0.12	2.7%	0.01	0.2%	0.12	3.0%
72_11975	1808449	5809935	28.5	10	7	2.0	2,535	3,648	8.0	0.11	1.4%	0.0	0.0%	0.12	1.5%
72_11976	1808459	5809945	23.5	19.5	7	2.0	2,538	3,652	17.5	0.11	0.7%	0.0	0.0%	0.12	0.7%
69_117	1811467	5809786	4.97	0.97	2, 3, 6, 7	2.0	2,552	3,175	2.97	0.11	3.8%	0.0	0.1%	0.12	3.9%
72_7077	1812389	5807551	5.5	1.5	2, 3, 6, 7	2.0	2,560	2,174	3.5	0.11	3.2%	0.01	0.2%	0.12	3.4%
69_1064	1807368	5808479	16.4	13.7	7	2.0	2,562	3,408	11.7	0.11	1.0%	0.0	0.0%	0.11	1.0%
72_4042	1812162	5808909	15	11	2, 3, 7	2.0	2,563	2,772	9.0	0.11	1.2%	0.0	0.0%	0.12	1.3%
72_11841	1810361	5810338	10.5	6.5		4.0	2,573	3,521	2.5	0.11	4.5%	0.0	0.1%	0.11	4.6%
69_975	1812269	5808687	14	11.3		8.5	2,577	2,684	2.8	0.11	4.0%	0.0	0.1%	0.12	4.1%
69_669	1811673	5805985	12.8	11.2	7	2.0	2,590	1,569	9.2	0.11	1.2%	0.01	0.1%	0.12	1.3%
69_1368	1812370	5808487	12	6.5		4.0	2,614	2,626	2.5	0.11	4.4%	0.0	0.2%	0.11	4.5%
72_22	1807268	5808379	36	26	7	2.0	2,635	3,449	24.0	0.11	0.4%	0.0	0.0%	0.11	0.5%
69_1733	1807268	5808479	19.5	13		10.0	2,658	3,495	3.0	0.11	3.5%	0.0	0.1%	0.11	3.6%
69_1369	1811067	5810186	6	4.5	6	4.2	2,666	3,446	1.8	0.11	5.9%	0.0	0.1%	0.11	6.0%
69_1959	1807371	5806779	24.38	20.38	2, 3, 7	2.0	2,681	2,972	18.38	0.1	0.6%	0.0	0.0%	0.11	0.6%
69_84	1811973	5806186	6	2	2, 3, 6, 7	2.0	2,683	1,748	4.0	0.1	2.6%	0.01	0.2%	0.11	2.8%
69_559	1807571	5806379	21.33	19.8	7	2.0	2,689	2,806	17.8	0.1	0.6%	0.0	0.0%	0.11	0.6%
72_6138	1807467	5809080	15	13	7	2.0	2,691	3,660	11.0	0.1	0.9%	0.0	0.0%	0.11	1.0%

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										Projected Drawdown (m)	Change in Available Water (%)	Projected Drawdown (m)	Change in Available Water (%)	Projected Drawdown (m)	Change in Available Water (%)
69_616	1810666	5810385	4.5	4.5	7	2.0	2,696	3,583	2.5	0.1	4.2%	0.0	0.1%	0.11	4.2%
72_10624	1809342	5810482	19.5	13		2.5	2,711	3,799	10.5	0.1	1.0%	0.0	0.0%	0.1	1.0%
69_2158	1811167	5810186	11	6.5	7	2.0	2,714	3,468	4.5	0.1	2.3%	0.0	0.0%	0.1	2.3%
72_4448	1808285	5810041	10	6	2, 3, 7	2.0	2,715	3,825	4.0	0.1	2.6%	0.0	0.0%	0.1	2.6%
69_1532	1807268	5808679	17.37	12.5	7	2.0	2,715	3,595	10.5	0.1	1.0%	0.0	0.0%	0.1	1.0%
69_251	1811574	5805685	8.78	4.78	2, 3, 7	2.0	2,747	1,672	2.78	0.1	3.6%	0.01	0.3%	0.11	3.9%
72_9620	1809238	5810502	11	8		6.0	2,752	3,847	2.0	0.1	5.0%	0.0	0.1%	0.1	5.1%
69_984	1811868	5809687	9.4	7.6	7	2.0	2,756	3,250	5.6	0.1	1.8%	0.0	0.0%	0.1	1.8%
69_2108	1812472	5806987	20.1	17		6.4	2,757	2,136	10.6	0.1	0.9%	0.01	0.1%	0.11	1.0%
72_10492	1809526	5810557	8.5	6	6	5.1	2,758	3,828	3.4	0.1	2.9%	0.0	0.1%	0.1	3.0%
69_1218	1810866	5810385	5.7	1.7	2, 3, 6, 7	2.0	2,764	3,606	3.7	0.1	2.7%	0.0	0.1%	0.1	2.7%
69_180	1812469	5808687	6.38	2.38	2, 3, 6, 7	2.0	2,766	2,831	4.38	0.1	2.3%	0.0	0.1%	0.1	2.4%
69_181	1812469	5808687	6.31	2.31	2, 3, 6, 7	2.0	2,766	2,831	4.31	0.1	2.3%	0.0	0.1%	0.1	2.4%
72_5912	1812068	5809487	8	4.5	7	2.0	2,782	3,179	2.5	0.1	3.9%	0.0	0.1%	0.1	4.1%
72_801	1812100	5809458	10	6	2, 3, 7	2.0	2,790	3,172	4.0	0.1	2.5%	0.0	0.1%	0.1	2.5%
69_1468	1809466	5810583	7	5	7	2.0	2,791	3,867	3.0	0.1	3.3%	0.0	0.1%	0.1	3.3%
69_1942	1812469	5808787	7.5	7.5	7	2.0	2,799	2,898	5.5	0.1	1.8%	0.0	0.1%	0.1	1.8%
72_9699	1808913	5810471	19.5	13		10.3	2,812	3,924	2.7	0.1	3.6%	0.0	0.1%	0.1	3.6%
69_1572	1812073	5806086	8	6	6	5.0	2,823	1,878	3.0	0.1	3.2%	0.01	0.2%	0.1	3.4%
69_1384	1810874	5805184	7.39	3.39	2, 3, 7	2.0	2,828	1,717	1.39	0.1	6.9%	0.01	0.6%	0.1	7.5%
72_1177	1812095	5809573	10	6	2, 3	4.7	2,856	3,266	1.3	0.09	7.3%	0.0	0.2%	0.1	7.4%
72_10489	1812125	5809575	9	6.5	6	5.7	2,881	3,284	3.3	0.09	2.8%	0.0	0.1%	0.1	2.9%
72_1676	1807587	5809630	27	21.4		17.0	2,894	3,938	4.4	0.09	2.1%	0.0	0.0%	0.09	2.1%
69_1686	1812670	5808487	8.5	5.5		4.0	2,905	2,864	1.5	0.09	6.1%	0.0	0.2%	0.09	6.3%
72_6494	1812182	5809575	7.75	3.75	2, 3, 6	4.92	2,926	3,315	2.83	0.09	3.2%	0.0	0.1%	0.09	3.3%
72_2731	1807666	5809780	19	13	6	16.0	2,931	3,993	3.0	0.09	3.0%	0.0	0.1%	0.09	3.1%
72_10748	1812568	5808940	11.8	5.8		3.78	2,947	3,075	2.02	0.09	4.4%	0.0	0.1%	0.09	4.6%
72_1247	1812672	5806987	13.5	11.7		2.3	2,949	2,335	9.4	0.09	0.9%	0.01	0.1%	0.09	1.0%
72_9068	1808565	5810482	25.5	22.5		20.4	2,955	4,074	2.1	0.09	4.2%	0.0	0.1%	0.09	4.3%
72_10754	1807562	5809722	26.5	20.5	7	2.0	2,972	4,022	18.5	0.09	0.5%	0.0	0.0%	0.09	0.5%
72_4444	1807950	5810128	10	6	2, 3, 7	2.0	2,987	4,085	4.0	0.09	2.2%	0.0	0.0%	0.09	2.2%
69_1467	1811734	5805492	21	13		12.0	2,997	1,921	1.0	0.09	8.7%	0.01	0.7%	0.09	9.3%
69_692	1812772	5806987	18.7	16		4.3	3,045	2,435	11.7	0.08	0.7%	0.0	0.0%	0.09	0.8%
69_112	1812970	5808088	5.23	1.23	2, 3, 6, 7	2.0	3,139	2,918	3.23	0.08	2.5%	0.0	0.1%	0.08	2.6%
69_113	1812970	5808088	5.49	5.49	7	2.0	3,139	2,918	3.49	0.08	2.3%	0.0	0.1%	0.08	2.4%
72_317	1812872	5806687	4.8	4.8		3.0	3,233	2,532	1.8	0.07	4.2%	0.0	0.2%	0.08	4.4%
72_1543	1813042	5807138	6	4	7	2.0	3,271	2,718	2.0	0.07	3.7%	0.0	0.2%	0.08	3.9%
72_1542	1813052	5807158	6	4	7	2.0	3,276	2,730	2.0	0.07	3.7%	0.0	0.2%	0.08	3.8%
69_239	1813171	5807788	5.48	1.48	2, 3, 6, 7	2.0	3,328	2,990	3.48	0.07	2.0%	0.0	0.1%	0.07	2.1%
69_1283	1813172	5806888	5.8	5.7	7	2.0	3,456	2,830	3.7	0.07	1.8%	0.0	0.1%	0.07	1.9%
72_10101	1813050	5806426	14	11		3.0	3,496	2,735	8.0	0.06	0.8%	0.0	0.0%	0.07	0.8%

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										Projected Drawdown (m)	Change in Available Water (%)	Projected Drawdown (m)	Change in Available Water (%)	Projected Drawdown (m)	Change in Available Water (%)
70_143	1812274	5805286	18.2	14.9	7	2.0	3,510	2,464	12.9	0.06	0.5%	0.0	0.0%	0.07	0.5%
70_146	1812075	5805085	8.2	7		4.5	3,528	2,449	2.5	0.06	2.5%	0.0	0.2%	0.07	2.7%
72_12187	1812353	5805285	17	11		5.0	3,566	2,527	6.0	0.06	1.0%	0.0	0.1%	0.07	1.1%
72_3394	1813073	5806287	9	7		2.6	3,574	2,781	4.4	0.06	1.4%	0.0	0.1%	0.06	1.5%
72_3615	1813073	5806287	9	7		2.6	3,574	2,781	4.4	0.06	1.4%	0.0	0.1%	0.06	1.5%
70_33	1812374	5805286	8.83	4.83	2, 3, 7	2.0	3,580	2,543	2.83	0.06	2.1%	0.0	0.2%	0.07	2.3%
72_11567	1812629	5805564	14	7		3.27	3,583	2,607	3.73	0.06	1.6%	0.0	0.1%	0.06	1.7%
70_102	1812375	5805086	9.8	6.9	7	2.0	3,725	2,669	4.9	0.06	1.1%	0.0	0.1%	0.06	1.2%
62_173	1812874	5805487	7.15	3.15	2, 3, 7	2.0	3,823	2,859	1.15	0.05	4.6%	0.0	0.3%	0.06	4.9%
72_7267	1812613	5805122	23	9.5		3.4	3,865	2,833	6.1	0.05	0.8%	0.0	0.1%	0.05	0.9%
70_934	1812175	5804685	25.9	22.9		11.9	3,905	2,811	11.0	0.05	0.5%	0.0	0.0%	0.05	0.5%
70_256	1812875	5805287	7	4	7	2.0	3,949	2,958	2.0	0.05	2.4%	0.0	0.2%	0.05	2.6%
72_3933	1812974	5805387	13	8.5		7.0	3,964	2,995	1.5	0.05	3.2%	0.0	0.2%	0.05	3.4%
72_8899	1812193	5804623	18	13	7	2.0	3,966	2,870	11.0	0.05	0.4%	0.0	0.0%	0.05	0.5%
70_807	1812775	5805087	5.7	3.7	7	2.0	4,007	2,985	1.7	0.05	2.8%	0.0	0.2%	0.05	3.0%

Note: A number of assumptions have been made as indicated in the assumptions column

- 2: Screen base unknown, assumed to be 1 m above the bore depth allowing for a 1 m sump
- 3: Screen top and casing base unknown, assumed to be 3 m above the screen base (i.e. a 3 m screened interval)
- 6: Assumed the base of the available water column to be the base of the bore
- 7: Static water level unknown, assumed a static water level of 2 m bgl

Table C2: Nearby bores in Aquifer 2

Bore ID	Easting (NZTM)	Northing (NZTM)	Bore Depth (m)	Casing Depth (m)	Assumptions	Static Water Level (m bgl)	Distance from Test Bore (m)	Distance From Potential Irrigation Bore (m)	Available Bore Water Column (m)	Potable Supply Bore		Irrigation Bore		Combined Drawdown	
										Projected Drawdown (m)	Change in Available Water (%)	Projected Drawdown (m)	Change in Available Water (%)	Projected Drawdown (m)	Change in Available Water (%)
69_526	1809270	5807382	46.9	41.4		18.5	719	1,213	22.9	2.22	9.7%	1.05	4.6%	3.27	14.3%
69_443	1809268	5808982	47.24	39.61		13.47	1,299	2,417	26.14	1.85	7.1%	0.51	1.9%	2.36	9.0%
69_1454	1808467	5809181	40	26	7	10.9	1,937	3,018	15.1	1.59	10.5%	0.36	2.4%	1.95	12.9%
69_1729	1807972	5806280	36.57	32.57	2, 3, 7	10.9	2,421	2,431	21.67	1.44	6.7%	0.5	2.3%	1.95	9.0%
69_1549	1807470	5807079	33	19.5	7	10.9	2,485	2,885	8.6	1.43	16.6%	0.39	4.5%	1.81	21.1%
69_942	1807672	5806179	38.71	32.31	7	10.9	2,720	2,746	21.41	1.37	6.4%	0.42	2.0%	1.79	8.3%
69_93	1807371	5806579	33	33	7	10.9	2,765	2,982	22.1	1.36	6.1%	0.37	1.7%	1.72	7.8%

Note: A number of assumptions have been made as indicated in the assumptions column

- 2: Screen base unknown, assumed to be 1 m above the bore depth allowing for a 1 m sump
- 3: Screen top and casing base unknown, assumed to be 3 m above the screen base (i.e. a 3 m screened interval)
- 7: Static water level unknown, assumed a static water level of 10.9 m bgl (Test Bore SWL)

Table C3: Bores targeting Aquifer 3 or deeper

Bore ID	Easting (NZTM)	Northing (NZTM)	Bore Depth (m)	Casing Depth (m)	Assumptions	Static Water Level (m bgl)	Distance from Test Bore (m)	Distance From Potential Irrigation Bore (m)	Available Bore Water Column (m)	Potable Supply Bore		Irrigation Bore		Combined Drawdown	
										Projected Drawdown (m)	Change in Available Water (%)	Projected Drawdown (m)	Change in Available Water (%)	Projected Drawdown (m)	Change in Available Water (%)
69_1869	1809369	5808382	62.5	51.5	7	10.95	737	1,843	40.55	2.25	5.6%	0.72	1.8%	2.97	7.3%
72_7189	1809596	5808550	73.35	58		11.4	773	1,887	46.6	2.22	4.8%	0.7	1.5%	2.92	6.3%
72_5845	1808969	5807882	93	71	7	9.78	876	1,738	61.22	2.13	3.5%	0.76	1.2%	2.89	4.7%
72_10247	1809111	5807263	75	64.7		5.5	918	1,310	59.2	2.1	3.5%	1.0	1.7%	3.09	5.2%
69_816	1809069	5808382	47.2	43.2	2, 3, 7	10.95	958	2,018	32.25	2.07	6.4%	0.65	2.0%	2.71	8.4%
72_5092	1809071	5807182	72	45.5	7	10.95	1,000	1,323	34.55	2.04	5.9%	0.99	2.9%	3.02	8.8%
69_274	1810269	5808784	65	61	1, 2, 3, 7	10.95	1,056	1,968	50.05	2.0	4.0%	0.67	1.3%	2.66	5.3%
72_7116	1808774	5807973	65	61	1, 2, 3, 7	10.95	1,080	1,949	50.05	1.98	4.0%	0.67	1.3%	2.66	5.3%
72_7117	1808786	5808259	65	61	1, 2, 3, 7	10.95	1,145	2,122	50.05	1.94	3.9%	0.61	1.2%	2.55	5.1%
72_9655	1808769	5808382	100	67	7	9.78	1,213	2,220	57.22	1.9	3.3%	0.57	1.0%	2.48	4.3%
69_1513	1809168	5808982	64	39.01	7	10.95	1,346	2,463	28.06	1.83	6.5%	0.5	1.8%	2.33	8.3%
69_1583	1808689	5808537	55	45	7	10.95	1,360	2,386	34.05	1.83	5.4%	0.52	1.5%	2.35	6.9%
69_1294	1808671	5807081	60.96	56.96	2, 3, 7	10.95	1,384	1,693	46.01	1.81	3.9%	0.78	1.7%	2.6	5.6%
69_343	1808871	5806781	65	61	1, 2, 3, 7	10.95	1,421	1,472	50.05	1.8	3.6%	0.9	1.8%	2.7	5.4%
69_1451	1808971	5806581	69.5	58.5	7	10.95	1,513	1,392	47.55	1.75	3.7%	0.95	2.0%	2.7	5.7%
72_77	1808971	5806581	65	45.5	7	10.95	1,513	1,392	34.55	1.75	5.1%	0.95	2.7%	2.7	7.8%
72_9228	1808967	5806568	97	58		2.0	1,526	1,398	56.0	1.75	3.1%	0.94	1.7%	2.69	4.8%
69_2194	1810372	5806383	65	61	1, 2, 3, 7	10.95	1,529	435	50.05	1.75	3.5%	2.09	4.2%	3.84	7.7%
69_923	1809068	5809182	67	39.01	7	10.95	1,569	2,687	28.06	1.73	6.2%	0.44	1.6%	2.17	7.7%
69_1295	1808170	5807480	60.96	56.96	2, 3, 7	10.95	1,706	2,272	46.01	1.67	3.6%	0.56	1.2%	2.23	4.9%
72_4795	1809673	5805982	65	61	1, 2, 3, 7	10.95	1,843	1,071	50.05	1.62	3.2%	1.17	2.3%	2.8	5.6%
72_11488	1808684	5806162	83.4	83.4		19.1	2,021	1,784	64.3	1.56	2.4%	0.74	1.2%	2.31	3.6%
72_4452	1808418	5809569	65	61	1, 2, 3, 7	10.95	2,258	3,358	50.05	1.49	3.0%	0.3	0.6%	1.79	3.6%
72_10962	1808387	5806018	65	61	1, 2, 3, 7	10.95	2,315	2,113	50.05	1.47	2.9%	0.61	1.2%	2.08	4.2%
72_4450	1808311	5809660	65	61	1, 2, 3, 7	10.95	2,396	3,495	50.05	1.45	2.9%	0.27	0.5%	1.72	3.4%
72_4451	1808506	5809839	65	61	1, 2, 3, 7	10.95	2,424	3,537	50.05	1.44	2.9%	0.27	0.5%	1.71	3.4%
69_21	1807872	5806280	54	50	2, 3, 7	10.95	2,500	2,529	39.05	1.42	3.6%	0.48	1.2%	1.9	4.9%
72_5803	1807671	5806579	65	61	1, 2, 3, 7	10.95	2,500	2,683	50.05	1.42	2.8%	0.44	0.9%	1.86	3.7%
72_5380	1812154	5808914	87	72		7.3	2,558	2,771	64.7	1.41	2.2%	0.42	0.6%	1.82	2.8%
72_4447	1808208	5809795	65	61	1, 2, 3, 7	10.95	2,566	3,664	50.05	1.4	2.8%	0.25	0.5%	1.65	3.3%
72_4449	1808323	5809892	65	61	1, 2, 3, 7	10.95	2,572	3,679	50.05	1.4	2.8%	0.25	0.5%	1.65	3.3%
69_1682	1807370	5807079	46.9	42.9	2, 3, 7	10.95	2,581	2,985	31.95	1.4	4.4%	0.37	1.2%	1.77	5.5%
69_558	1807471	5806579	48.77	36.58	7	10.95	2,676	2,882	25.63	1.38	5.4%	0.39	1.5%	1.77	6.9%
72_3229	1811516	5805692	65	61	1, 2, 3	43.78	2,705	1,625	17.22	1.37	8.0%	0.82	4.7%	2.19	12.7%
69_1261	1809366	5810483	65	61	1, 2, 3, 7	10.95	2,708	3,794	50.05	1.37	2.7%	0.23	0.5%	1.6	3.2%
69_219	1809366	5810483	65	61	1, 2, 3, 7	10.95	2,708	3,794	50.05	1.37	2.7%	0.23	0.5%	1.6	3.2%
69_1319	1807571	5806279	48.77	39.6	7	10.95	2,744	2,824	28.65	1.36	4.7%	0.41	1.4%	1.77	6.2%
72_4446	1808148	5809980	65	61	1, 2, 3, 7	10.95	2,748	3,850	50.05	1.36	2.7%	0.22	0.4%	1.58	3.2%
72_802	1812100	5809471	88	79		7.6	2,798	3,183	71.4	1.35	1.9%	0.33	0.5%	1.68	2.3%
69_1239	1807371	5806479	48.77	44.77	2, 3, 7	10.95	2,811	2,991	33.82	1.34	4.0%	0.37	1.1%	1.71	5.1%

Bore ID	Easting (NZTM)	Northing (NZTM)	Bore Depth (m)	Casing Depth (m)	Assumptions	Static Water Level (m bgl)	Distance from Test Bore (m)	Distance From Potential Irrigation Bore (m)	Available Bore Water Column (m)	Potable Supply Bore		Irrigation Bore		Combined Drawdown	
										Projected Drawdown (m)	Change in Available Water (%)	Projected Drawdown (m)	Change in Available Water (%)	Projected Drawdown (m)	Change in Available Water (%)
69_246	1810874	5805184	65	61	1, 2, 3, 7	10.95	2,828	1,717	50.05	1.34	2.7%	0.77	1.5%	2.11	4.2%
72_4445	1808189	5810118	65	61	1, 2, 3, 7	10.95	2,833	3,942	50.05	1.34	2.7%	0.21	0.4%	1.55	3.1%
72_11954	1807188	5808808	64.32	57		30.88	2,834	3,731	26.12	1.34	5.1%	0.24	0.9%	1.58	6.0%
72_5798	1806968	5808579	60	52	7	10.95	2,974	3,807	41.05	1.31	3.2%	0.23	0.6%	1.54	3.7%
72_10394	1812765	5806863	65	61	1, 2, 3, 7	10.95	3,074	2,422	50.05	1.29	2.6%	0.51	1.0%	1.8	3.6%
72_3226	1811667	5805210	65	61	1, 2, 3	12.34	3,182	2,082	48.66	1.26	2.6%	0.62	1.3%	1.89	3.9%
72_3227	1811675	5805200	65	61	1, 2, 3	12.3	3,195	2,095	48.7	1.26	2.6%	0.62	1.3%	1.88	3.9%
69_169	1813072	5807188	65	61	1, 2, 3, 7	10.95	3,290	2,754	50.05	1.24	2.5%	0.42	0.8%	1.66	3.3%
69_1358	1813171	5807788	65	61	1, 2, 3, 7	10.95	3,328	2,990	50.05	1.23	2.5%	0.37	0.7%	1.6	3.2%
72_3228	1811791	5805039	65	61	1, 2, 3	10.08	3,393	2,293	50.92	1.22	2.4%	0.55	1.1%	1.77	3.5%

Note: A number of assumptions have been made as indicated in the assumptions column

- 1: Depth unknown, assumed to be the same depth as the test bore
- 2: Screen base unknown, assumed to be 1 m above the bore depth allowing for a 1 m sump
- 3: Screen top and casing base unknown, assumed to be 3 m above the screen base (i.e. a 3 m screened interval)
- 7: Static water level unknown, assumed a static water level of 10.95 m bgl based on the Test Bore SWL.

Table C4: Bores owned by the applicant, geotechnical investigation bores, and bores targeting greywacke material

Bore ID	Easting (NZTM)	Northing (NZTM)	Bore Depth (m)	Casing Depth (m)	Assumptions	Aquifer No.	Static Water Level (m bgl)	Distance from Test Bore (m)	Distance From Potential Irrigation Bore (m)	Available Bore Water Column (m)	Potable Supply Bore		Irrigation Bore		Combined Drawdown		Comment
											Projected Drawdown (m)	Change in Available Water (%)	Projected Drawdown (m)	Change in Available Water (%)	Projected Drawdown (m)	Change in Available Water (%)	
72_8873	1809743	5807891	65	61	1, 2, 3, 7	3	10.95	124	1,230	50.05	4.02	8.0%	1.05	2.1%	5.07	10.1%	Proposed/Owned by Applicant
69_167	1809569	5808183	5.2	1.2	2, 3, 6, 7	1	2.0	457	1,570	3.2	0.34	10.6%	0.01	0.3%	0.35	10.9%	Presumed unused, small dimeter, only 5 m deep
69_426	1809370	5807282	64	43.2	6	3	29.2	714	1,078	34.8	2.28	6.5%	1.17	3.4%	3.44	9.9%	Proposed/Owned by Applicant?
69_253	1810371	5807284	3.5	-0.5	2, 3, 6, 7	1	2.0	751	468	1.5	0.3	20.1%	0.03	1.8%	0.33	21.9%	Old bore, Presumed unused, small dimeter, only 3.5 m deep
72_1710	1810371	5807184	7.5	6.5		1	3.8	825	368	2.7	0.29	10.8%	0.03	1.1%	0.32	11.9%	Proposed/Owned by Applicant?
72_4688	1809971	5806883	14	9		1	4.0	943	378	5.0	0.28	5.5%	0.03	0.6%	0.31	6.1%	Proposed/Owned by Applicant?
72_10448	1809859	5806832	61.26	56.5	7	3	10.95	985	484	45.55	2.05	4.5%	1.96	4.3%	4.01	8.8%	Proposed/Owned by Applicant?
72_7054	1809886	5806808	7	3	2, 3, 7	1	2.0	1,010	457	1.0	0.27	26.8%	0.03	2.7%	0.29	29.5%	Proposed/Owned by Applicant?
72_563	1808801	5808301	50	40.5		2	23.0	1,149	2,140	17.5	1.93	11.0%	0.6	3.4%	2.53	14.5%	Greywacke
69_444	1809472	5806482	31	16	6	2	17.86	1,386	933	13.14	1.81	13.8%	1.27	9.6%	3.08	23.4%	In greywacke, no expected drawdown
72_5477	1809372	5806082	51	25.5		3	14.5	1,798	1,218	11.0	1.64	14.9%	1.06	9.6%	2.7	24.5%	In greywacke, no expected drawdown
69_1522	1808071	5806780	25.3	19.51	6	1	22.0	2,053	2,272	3.3	0.15	4.6%	0.01	0.2%	0.16	4.8%	Geotech
69_462	1807471	5806679	2.5	2.5	7	1	2.0	2,631	2,875	0.5	0.11	21.6%	0.0	0.7%	0.11	22.3%	Old bore, Presumed unused, small dimeter, only 2.5 m deep
72_11977	1808047	5810019	26.5	23.5	7	2	14.49	2,841	3,940	9.01	1.34	14.8%	0.21	2.3%	1.54	17.1%	Geotech

Note: A number of assumptions have been made as indicated in the assumptions column

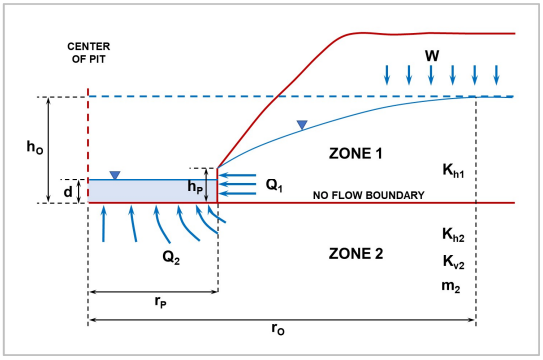
- 1: Depth unknown, assumed to be the same depth as the test bore
- 2: Screen base unknown, assumed to be 1 m above the bore depth allowing for a 1 m sump
- 3: Screen top and casing base unknown, assumed to be 3 m above the screen base (i.e. a 3 m screened interval)
- 6: Assumed the base of the available water column to be the base of the bore
- 7: Static water level unknown, assumed a static water appropriate to the targeted aquifer

APPENDIX D
DRAWDOWN MODELLING FOR PONDS AND SWALES



INFLOW TO OPEN PIT - APPROXIMATION

CLIENT:	Waikato Thoroughbred Racing
PROJECT:	Waikato Thoroughbred Racing Development
Project #:	WGA253138
Location:	Main Pond
Date:	Wednesday, 9 September 2026
Document #:	WGA253138-RP-HG-0003



A $$h_o = \sqrt{h_p^{\,2} + \frac{W}{K_{h1}}\left[r_o^2 \ln\left(\frac{r_o}{r_p}\right) - \frac{\left(r_o^2 - r_p^2\right)}{2}\right]},$$

B $$Q_1 = W\pi\left(r_o^2 - r_p^2\right),$$

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

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

W	1E-08	**m/s**	recharge flux
K_{h1}	4.58E-05	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	4.58E-05	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	4.58E-06	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	1	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	160.28	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_O is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Inflow from Zone 1

Radius of influence r_o calculated by iterating equation A	217	m
Known h_O	1	m
h_O calculated using Equation A	1.0	m
Q_1	8.6E-04	m^3/s
	7.4E+01	m^3/day

Inflow from Zone 2

Anisotropy parameter calculated using equation D	3.162278	
Q_2	9.3E-03	m^3/s
	8.0E+02	m^3/day

Total Pit Inflow **Q_{TOT}**	1.0E-02	m^3/s
	876.41	m^3/day

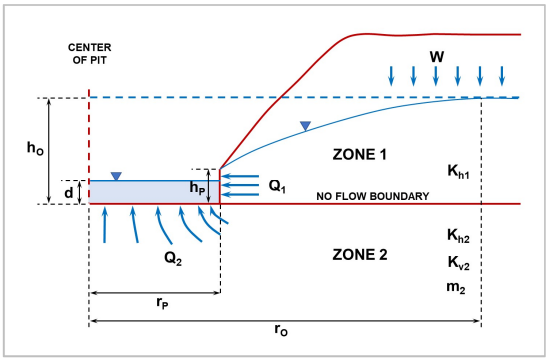
Reference: Marinelli, F. & Niccoli W. L. 2000. Simple analytical equations for estimating ground water inflow to a mine pit. Ground Water 38, no. 2: 311-314.

Analysed by: ECA 10/09/2026
Checked by: CHO 15/09/2026

Notes:

GROUNDWATER DRAWDOWN AROUND OPEN PIT - APPROXIMATION

CLIENT:	Waikato Thoroughbred Racing
PROJECT:	Waikato Thoroughbred Racing Development
Project #:	WGA253138
Location:	Main Pond
Date:	Wednesday, 9 September 2026
Document #:	WGA253138-RP-HG-0003



$$dH_{Zone\,1} = h_0 - \sqrt{h_p^2 + \frac{W}{k_{h1}}\left[r_0^2 Ln\left(\frac{r}{r_p}\right) - \frac{\left(r^2 - r_p^2\right)}{2}\right]}$$

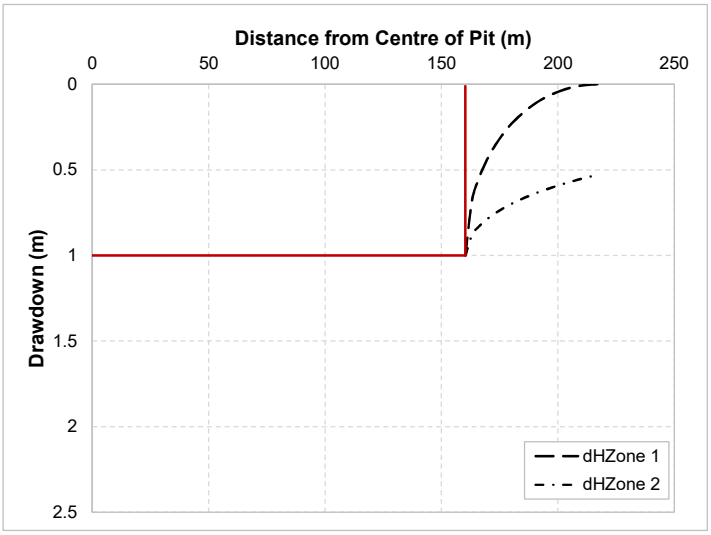
$$dH_{Zone2} = \frac{2(h_0 - d)}{\pi} sin^{-1}\left(\frac{r_p}{r}\right)$$

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

W	1.27E-08	**m/s**	recharge flux
K_{h1}	4.58E-05	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	4.58E-05	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	4.58E-06	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	1	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	160.28	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_o is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Radius of influence r_o calculated by iterating equation A in analysis sheet 1 217 m



Drawdown curves

r	$dH_{Zone\,1}$	$dH_{Zone\,2}$
160.28	1	1.0
163.1	0.7	0.9
166.0	0.6	0.8
168.8	0.5	0.8
171.7	0.4	0.8
174.5	0.3	0.7
177.4	0.3	0.7
180.2	0.2	0.7
183.1	0.2	0.7
185.9	0.2	0.7
188.8	0.1	0.6
191.6	0.1	0.6
194.5	0.1	0.6
197.4	0.06	0.6
200.2	0.04	0.6
203.1	0.03	0.6
205.9	0.02	0.6
208.8	0.01	0.6
211.6	0.00	0.5
214.5	0.00	0.5
217.3	0.00	0.5

Reference: Marinelli, F. & Niccoli W. L. 2000. Simple analytical equations for estimating ground water inflow to a mine pit. Ground Water 38, no. 2: 311-314.

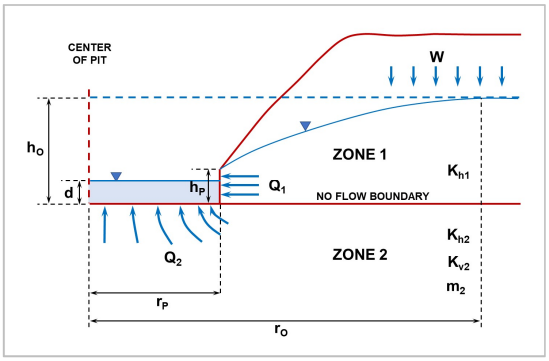
Analysed by: ECA 10/09/2026
Checked by: CHO 15/09/2026

Notes: The drawdown curves apply along the no-flow boundary line at an elevation equivalent to the base of the pit.
The dH_{Zone2} line does not take into account recharge from above.
Drawdown projections are initial approximations only.

GROUNDWATER DRAWDOWN AT BORES LOCATED CLOSE TO OPEN PIT - APPROXIMATION

WGA

CLIENT:	Waikato Thoroughbred Racing
PROJECT:	Waikato Thoroughbred Racing Development
Project #:	WGA253138
Location:	Main Pond
Date:	Wednesday, 9 September 2026
Document #:	WGA253138-RP-HG-0003



$$dH_{Zone\,1} = h_0 - \sqrt{h_p^2 + \frac{W}{k_{h1}}\left[r_0^2 Ln\left(\frac{r}{r_p}\right) - \frac{(r^2 - r_p^2)}{2}\right]}$$

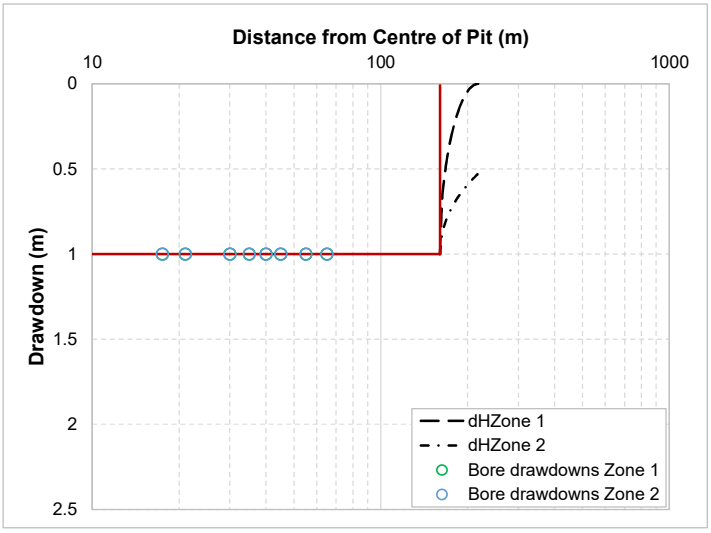
$$dH_{Zone2} = \frac{2(h_0 - d)}{\pi} sin^{-1}\left(\frac{r_p}{r}\right)$$

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

W	1.27E-08	**m/s**	recharge flux
K_{h1}	4.58E-05	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	4.58E-05	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	4.58E-06	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	1	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	160.28	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_o is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Radius of influence r_o calculated by iterating equation A in analysis sheet 1 217 m
Radial distance of nearby bore from centre of pit = r.



Drawdown in bores

r	$dH_{Zone\,1}$	$dH_{Zone\,2}$
17.5	1.0	1.0
21.0	1.0	1.0
30.0	1.0	1.0
35.0	1.0	1.0
40.0	1.0	1.0
45.0	1.0	1.0
55.0	1.0	1.0
65.0	1.0	1.0

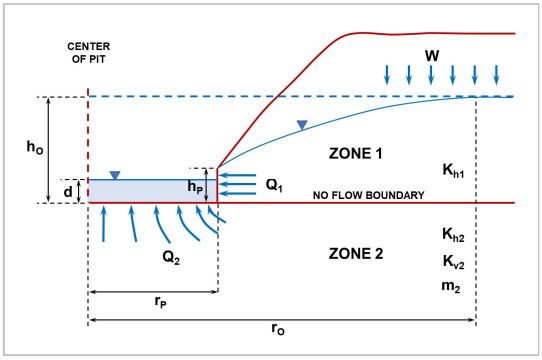
Reference: Marinelli, F. & Niccoli W. L. 2000. Simple analytical equations for estimating ground water inflow to a mine pit. Ground Water 38, no. 2: 311-314.

Analysed by: ECA 10/09/2026
Checked by: CHO 15/09/2026

Notes: The drawdown curves apply along the no-flow boundary line at an elevation equivalent to the base of the pit.
The dH_{Zone2} line does not take into account recharge from above.
Drawdown projections are initial approximations only.

INFLOW TO OPEN PIT - APPROXIMATION

CLIENT:	Waikato Thoroughbred Racing
PROJECT:	Waikato Thoroughbred Racing Development
Project #:	WGA253138
Location:	Harness Pond
Date:	Wednesday, 9 September 2026
Document #:	WGA253138-RP-HG-0003



A $$h_o = \sqrt{h_p^{\,2} + \frac{W}{K_{h1}}\left[r_o^2 \ln\left(\frac{r_o}{r_p}\right) - \frac{\left(r_o^2 - r_p^2\right)}{2}\right]},$$

B $$Q_1 = W\pi\left(r_o^2 - r_p^2\right),$$

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

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

W	1E-08	**m/s**	recharge flux
K_{h1}	4.58E-05	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	4.58E-05	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	4.58E-06	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	0.2	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	58.3	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_O is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Inflow from Zone 1

Radius of influence r_o calculated by iterating equation A	70	m
Known h_O	0.2	m
h_O calculated using Equation A	0.2	m
Q_1	6.0E-05	m^3/s
	5.2E+00	m^3/day

Inflow from Zone 2

Anisotropy parameter calculated using equation D	3.162278	
Q_2	6.8E-04	m^3/s
	5.8E+01	m^3/day

Total Pit Inflow **Q_{TOT}**	7.4E-04	m^3/s
	63.52	m^3/day

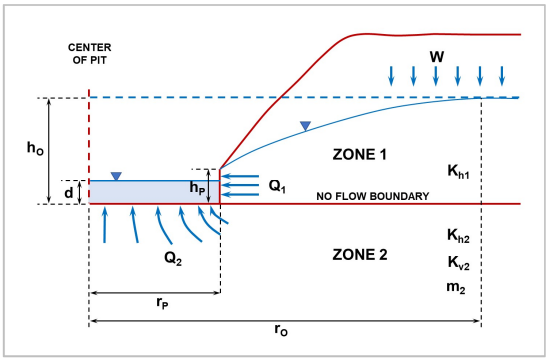
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Analysed by: ECA 10/09/2026
Checked by: CHO 15/09/2026

Notes:

GROUNDWATER DRAWDOWN AROUND OPEN PIT - APPROXIMATION

CLIENT:	Waikato Thoroughbred Racing
PROJECT:	Waikato Thoroughbred Racing Development
Project #:	WGA253138
Location:	Harness Pond
Date:	Wednesday, 9 September 2026
Document #:	WGA253138-RP-HG-0003



$$dH_{Zone\,1} = h_0 - \sqrt{h_p^2 + \frac{W}{k_{h1}}\left[r_0^2 Ln\left(\frac{r}{r_p}\right) - \frac{\left(r^2 - r_p^2\right)}{2}\right]}$$

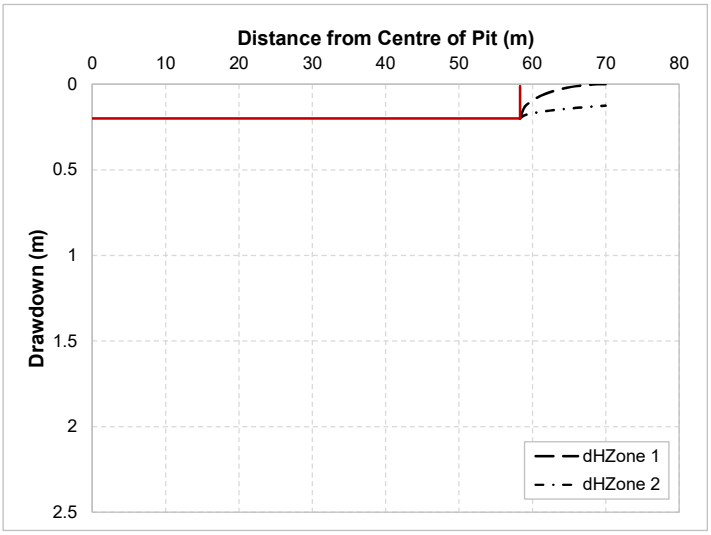
$$dH_{Zone2} = \frac{2(h_0 - d)}{\pi} sin^{-1}\left(\frac{r_p}{r}\right)$$

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

W	1.27E-08	**m/s**	recharge flux
K_{h1}	4.58E-05	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	4.58E-05	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	4.58E-06	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	0.2	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	58.3	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_o is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Radius of influence r_o calculated by iterating equation A in analysis sheet 1 70 m



Drawdown curves

r	$dH_{Zone\,1}$	$dH_{Zone\,2}$
58.3	0.2	0.2
58.9	0.1	0.2
59.5	0.1	0.2
60.1	0.1	0.2
60.6	0.1	0.2
61.2	0.1	0.2
61.8	0.1	0.2
62.4	0.0	0.2
63.0	0.0	0.2
63.6	0.0	0.1
64.1	0.0	0.1
64.7	0.0	0.1
65.3	0.0	0.1
65.9	0.01	0.1
66.5	0.01	0.1
67.1	0.01	0.1
67.6	0.00	0.1
68.2	0.00	0.1
68.8	0.00	0.1
69.4	0.00	0.1
70.0	0.00	0.1

Reference: Marinelli, F. & Niccoli W. L. 2000. Simple analytical equations for estimating ground water inflow to a mine pit. Ground Water 38, no. 2: 311-314.

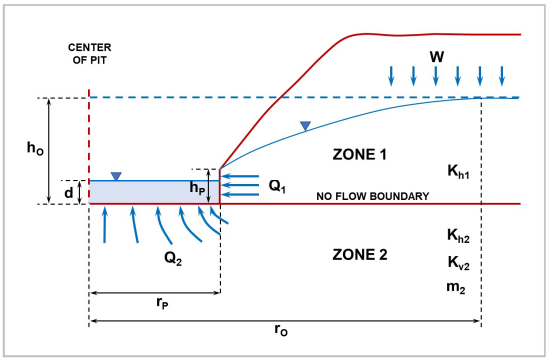
Analysed by: ECA 10/09/2026
Checked by: CHO 15/09/2026

Notes: The drawdown curves apply along the no-flow boundary line at an elevation equivalent to the base of the pit.
The dH_{Zone2} line does not take into account recharge from above.
Drawdown projections are initial approximations only.

GROUNDWATER DRAWDOWN AT BORES LOCATED CLOSE TO OPEN PIT - APPROXIMATION

WGA

CLIENT:	Waikato Thoroughbred Racing
PROJECT:	Waikato Thoroughbred Racing Development
Project #:	WGA253138
Location:	Harness Pond
Date:	Wednesday, 9 September 2026
Document #:	WGA253138-RP-HG-0003



$$dH_{Zone\,1} = h_0 - \sqrt{h_p^2 + \frac{W}{k_{h1}}\left[r_0^2 Ln\left(\frac{r}{r_p}\right) - \frac{(r^2 - r_p^2)}{2}\right]}$$

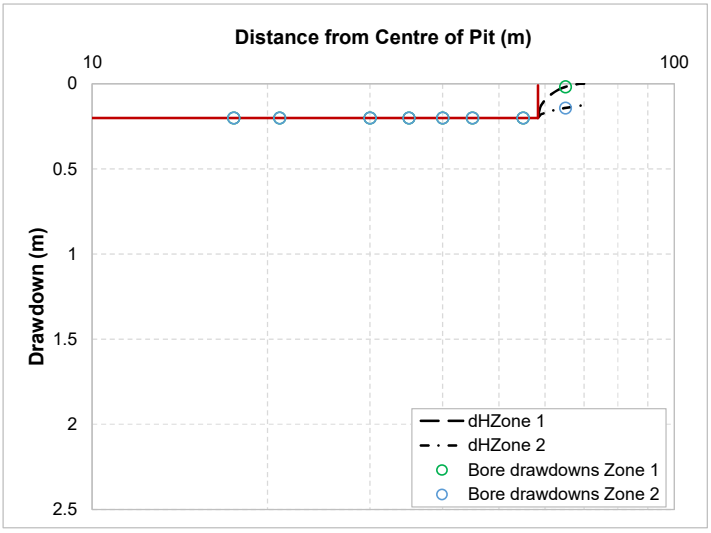
$$dH_{Zone2} = \frac{2(h_0 - d)}{\pi} sin^{-1}\left(\frac{r_p}{r}\right)$$

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

W	1.27E-08	**m/s**	recharge flux
K_{h1}	4.58E-05	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	4.58E-05	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	4.58E-06	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	0.2	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	58.3	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_o is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Radius of influence r_o calculated by iterating equation A in analysis sheet 1 70 m
Radial distance of nearby bore from centre of pit = r.



Drawdown in bores

r	$dH_{Zone\,1}$	$dH_{Zone\,2}$
17.5	0.2	0.2
21.0	0.2	0.2
30.0	0.2	0.2
35.0	0.2	0.2
40.0	0.2	0.2
45.0	0.2	0.2
55.0	0.2	0.2
65.0	0.0	0.1

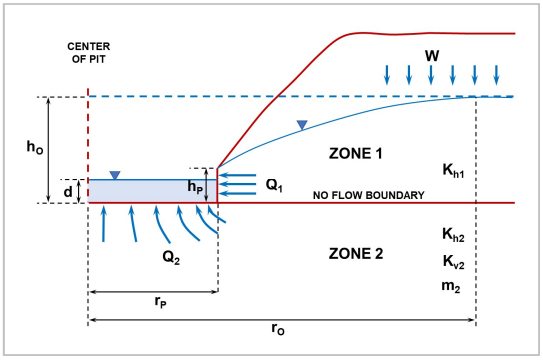
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Analysed by: ECA 10/09/2026
Checked by: CHO 15/09/2026

Notes: The drawdown curves apply along the no-flow boundary line at an elevation equivalent to the base of the pit.
The dH_{Zone2} line does not take into account recharge from above.
Drawdown projections are initial approximations only.

INFLOW TO OPEN PIT - APPROXIMATION

CLIENT:	Waikato Thoroughbred Racing
PROJECT:	Waikato Thoroughbred Racing Development
Project #:	WGA253138
Location:	Southern Pond
Date:	Wednesday, 9 September 2026
Document #:	WGA253138-RP-HG-0003



A $$h_o = \sqrt{h_p^{\,2} + \frac{W}{K_{h1}}\left[r_o^2 \ln\left(\frac{r_o}{r_p}\right) - \frac{\left(r_o^2 - r_p^2\right)}{2}\right]},$$

B $$Q_1 = W\pi\left(r_o^2 - r_p^2\right),$$

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

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

W	1E-08	**m/s**	recharge flux
K_{h1}	4.58E-05	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	4.58E-05	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	4.58E-06	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	0.7	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	32.1	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_O is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Inflow from Zone 1

Radius of influence r_o calculated by iterating equation A	69	m
Known h_O	0.7	m
h_O calculated using Equation A	0.7	m
Q_1	1.5E-04	m^3/s
	1.3E+01	m^3/day

Inflow from Zone 2

Anisotropy parameter calculated using equation D	3.162278	
Q_2	1.3E-03	m^3/s
	1.1E+02	m^3/day

Total Pit Inflow **Q_{TOT}**	1.4E-03	m^3/s
	125.28	m^3/day

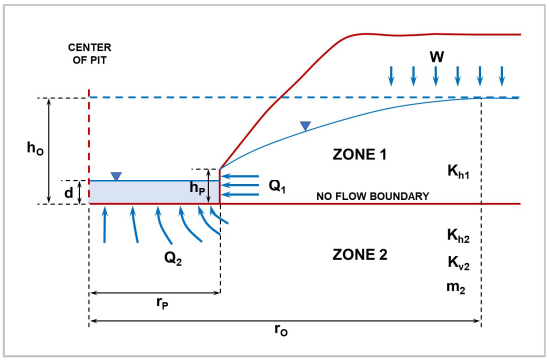
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Analysed by: ECA 10/09/2026
Checked by: CHO 15/09/2026

Notes:

GROUNDWATER DRAWDOWN AROUND OPEN PIT - APPROXIMATION

CLIENT:	Waikato Thoroughbred Racing
PROJECT:	Waikato Thoroughbred Racing Development
Project #:	WGA253138
Location:	Southern Pond
Date:	Wednesday, 9 September 2026
Document #:	WGA253138-RP-HG-0003



$$dH_{Zone\,1} = h_0 - \sqrt{h_p^2 + \frac{W}{k_{h1}}\left[r_0^2 Ln\left(\frac{r}{r_p}\right) - \frac{\left(r^2 - r_p^2\right)}{2}\right]}$$

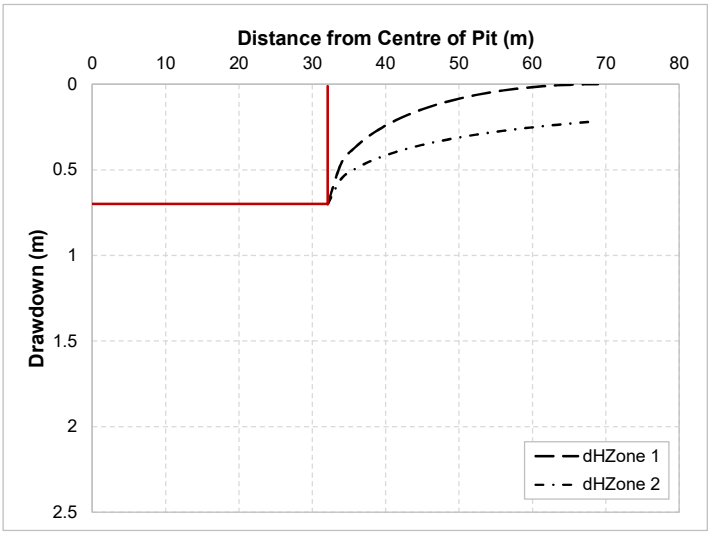
$$dH_{Zone2} = \frac{2(h_0 - d)}{\pi} sin^{-1}\left(\frac{r_p}{r}\right)$$

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

W	1.27E-08	**m/s**	recharge flux
K_{h1}	4.58E-05	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	4.58E-05	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	4.58E-06	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	0.7	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	32.1	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_o is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Radius of influence r_o calculated by iterating equation A in analysis sheet 1 69 m



Drawdown curves

r	$dH_{Zone\,1}$	$dH_{Zone\,2}$
32.1	0.7	0.7
33.9	0.5	0.6
35.8	0.4	0.5
37.6	0.3	0.5
39.5	0.3	0.4
41.3	0.2	0.4
43.1	0.2	0.4
45.0	0.1	0.4
46.8	0.1	0.3
48.7	0.1	0.3
50.5	0.1	0.3
52.4	0.1	0.3
54.2	0.0	0.3
56.0	0.04	0.3
57.9	0.03	0.3
59.7	0.02	0.3
61.6	0.01	0.2
63.4	0.01	0.2
65.2	0.00	0.2
67.1	0.00	0.2
68.9	0.00	0.2

Reference: Marinelli, F. & Niccoli W. L. 2000. Simple analytical equations for estimating ground water inflow to a mine pit. Ground Water 38, no. 2: 311-314.

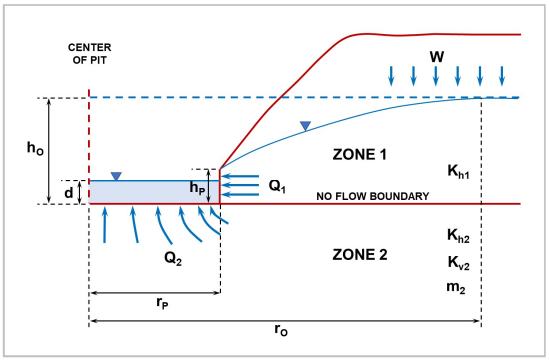
Analysed by: ECA 10/09/2026
Checked by: CHO 15/09/2026

Notes: The drawdown curves apply along the no-flow boundary line at an elevation equivalent to the base of the pit.
The dH_{Zone2} line does not take into account recharge from above.
Drawdown projections are initial approximations only.

GROUNDWATER DRAWDOWN AT BORES LOCATED CLOSE TO OPEN PIT - APPROXIMATION

WGA

CLIENT:	Waikato Thoroughbred Racing
PROJECT:	Waikato Thoroughbred Racing Development
Project #:	WGA253138
Location:	Southern Pond
Date:	Wednesday, 9 September 2026
Document #:	WGA253138-RP-HG-0003



$$dH_{Zone\,1} = h_0 - \sqrt{h_p^2 + \frac{W}{k_{h1}}\left[r_0^2 Ln\left(\frac{r}{r_p}\right) - \frac{(r^2 - r_p^2)}{2}\right]}$$

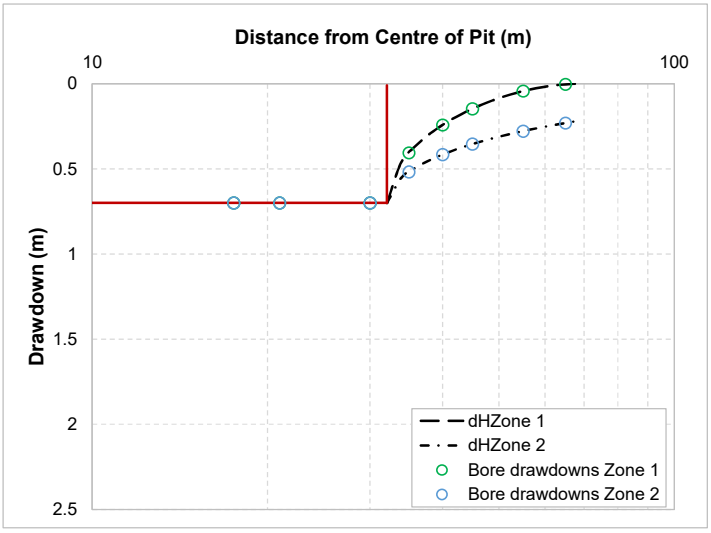
$$dH_{Zone2} = \frac{2(h_0 - d)}{\pi} sin^{-1}\left(\frac{r_p}{r}\right)$$

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

W	1.27E-08	**m/s**	recharge flux
K_{h1}	4.58E-05	**m/s**	horizontal hydraulic conductivity in Zone 1
K_{h2}	4.58E-05	**m/s**	horizontal hydraulic conductivity in Zone 2
K_{v2}	4.58E-06	**m/s**	vertical hydraulic conductivity in Zone 2
h_o	0.7	**m**	initial saturated thickness above the base of Zone 1
h_p	0	**m**	saturated thickness at the pit wall
r_p	32.1	**m**	effective pit radius
d	0	**m**	depth of the pit lake

r_o is the radius of influence measured from the center of the pit beyond which groundwater drawdown is nil.

Radius of influence r_o calculated by iterating equation A in analysis sheet 1 69 m
Radial distance of nearby bore from centre of pit = r.



Drawdown in bores

r	$dH_{Zone\,1}$	$dH_{Zone\,2}$
17.5	0.7	0.7
21.0	0.7	0.7
30.0	0.7	0.7
35.0	0.4	0.5
40.0	0.2	0.4
45.0	0.1	0.4
55.0	0.0	0.3
65.0	0.0	0.2

Reference: Marinelli, F. & Niccoli W. L. 2000. Simple analytical equations for estimating ground water inflow to a mine pit. Ground Water 38, no. 2: 311-314.

Analysed by: ECA 10/09/2026
Checked by: CHO 15/09/2026

Notes: The drawdown curves apply along the no-flow boundary line at an elevation equivalent to the base of the pit.
The dH_{Zone2} line does not take into account recharge from above.
Drawdown projections are initial approximations only.

HOOGHOUDT EQUATION — GROUNDWATER DRAWDOWN & DRAIN INFLOW CALCULATOR

Open Trench Drainage — Impervious Layer Far Below Drain Invert

INPUT PARAMETERS

Parameter	Value	Unit
Drain spacing (L)	**270**	m
Hydraulic conductivity above drain (Ka)	**3.97**	m/day
Hydraulic conductivity below drain (Kb)	**3.97**	m/day
Depth of drain invert below ground (Dd)	**2.2**	m
Steady-state watertable depth midway (Dw)	**2**	m
Depth of impervious layer below ground	**17**	m
Trench bottom width (W)	**5**	m
Trench side slopes (H:V)	**1**	—
Water depth in trench	**0**	m
Trench length	**1500**	m
Recharge rate (R)	**0.0011**	m/day
No. of series terms for equiv. depth	**20**	—

INTERMEDIATE CALCULATIONS

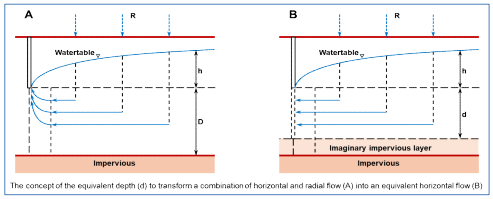
Parameter	Value	Unit
Depth of imperv. layer below drain (D)	14.800	m
Watertable height above drain level (h)	0.200	m
Trench wet perimeter (u)	5.000	m
Equivalent drain radius (r_o = u/π)	1.592	m
x = 2πD / L	0.3444	—
F(x) correction factor	4.2603	—
Equivalent depth (d)	12.853	m

DRAWDOWN PROFILE — STEADY-STATE WITH RECHARGE R (both sides of trench)

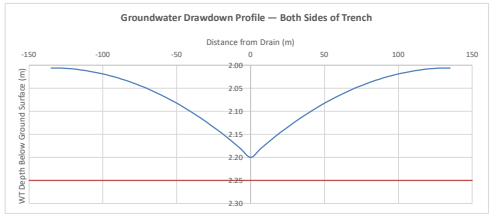
Position relative to drain (x)	WT height above drain (y)	WT depth below surface
m	*m*	*m*
-135.000	0.194	2.006
-128.250	0.194	2.006
-121.500	0.192	2.008
-114.750	0.190	2.010
-108.000	0.187	2.013
-101.250	0.182	2.018
-94.500	0.177	2.023
-87.750	0.171	2.029
-81.000	0.163	2.037
-74.250	0.155	2.045
-67.500	0.146	2.054
-60.750	0.136	2.064
-54.000	0.125	2.075
-47.250	0.113	2.087
-40.500	0.099	2.101
-33.750	0.085	2.115
-27.000	0.070	2.130
-20.250	0.054	2.146
-13.500	0.037	2.163
-6.750	0.019	2.181
0.000	0.000	2.200
6.750	0.019	2.181
13.500	0.037	2.163
20.250	0.054	2.146
27.000	0.070	2.130
33.750	0.085	2.115
40.500	0.099	2.101
47.250	0.113	2.087
54.000	0.125	2.075
60.750	0.136	2.064
67.500	0.146	2.054
74.250	0.155	2.045
81.000	0.163	2.037
87.750	0.171	2.029
94.500	0.177	2.023
101.250	0.182	2.018
108.000	0.187	2.013
114.750	0.190	2.010
121.500	0.192	2.008
128.250	0.194	2.006
135.000	0.194	2.006

MAIN RESULTS

Drainage discharge Q (Hooghoudt)	**0.001129**	m/day
Drainage discharge Q	**1.129**	mm/day
Flow component below drain level	0.001120	m/day
Flow component above drain level	0.000009	m/day
Total inflow per m of trench — both sides (Q×L)	0.3047	m³/d/m
Total inflow to full trench — both sides	**457.10**	m³/day
Total inflow to full trench — both sides	**5.291**	L/s
Actual midpoint WT height (from R)	0.194	m
Actual midpoint WT depth below surface	2.006	m
Adequacy check (Q vs R)	**ADEQUATE: Q ≥ R**	



The concept of the equivalent depth (d) to transform a combination of horizontal and radial flow (A) into an equivalent horizontal flow (B)



SOURCE REFERENCES

1. Hooghoudt, S.B. (1940). Bijdragen tot de kennis van eenige natuurkundige grootheden van den grond, No.7. Bodemkundig Instituut, Groningen, Netherlands.
2. Ritzema, H.P. (1994). Subsurface flow to drains. Ch.8 in Drainage Principles and Applications, ILRI Publ.16, pp.236–304. Wageningen. ISBN 90-70754-33-9.
3. Van der Molen, W.H. & Wesseling, J. (1991). A closed form solution for Hooghoudt equivalent depth. Agricultural Water Management, 19, pp.1–16.
4. Van Schilfgaarde, J. (1957). Approximate solutions to drainage flow problems. In: Luthin (Ed.), Drainage of Agricultural Lands, pp.79–112.
5. Smedema, L.K. & Rycroft, D.W. (1983). Land Drainage — Planning and Design of Agricultural Drainage Systems. Batsford, London.

LIMITATIONS & ASSUMPTIONS

1. STEADY-STATE ONLY: Assumes recharge rate = drainage discharge. Not valid for transient conditions (storm events, fluctuating WT). For transient analysis, use Glover-Dumm or De Zeeuw-Hellinga equations.
2. TWO-DIMENSIONAL FLOW: Assumes infinitely long trench — identical flow in every cross-section perpendicular to drain. End effects and 3D flow near trench terminations are ignored.
3. UNIFORM RECHARGE: Rainfall/irrigation recharge is assumed spatially uniform across the entire drained area.
4. HOMOGENEOUS & ISOTROPIC SOIL: Assumes uniform K in all directions within each layer. Does not account for anisotropy (Kv≠Kh), macropore flow, or spatial variability.
5. DUPUIT-FORCHHEIMER ASSUMPTIONS: Streamlines assumed horizontal, equipotentials vertical. Less accurate when D >> L/4.
6. EQUIVALENT DEPTH APPROXIMATION: Radial flow convergence near drain approximated by replacing actual D with smaller d. Accuracy decreases for very deep impervious layers (d maxes out at D ≈ ¼L).
7. NO ENTRANCE RESISTANCE: Assumes water enters trench freely. Valid only if trench backfill K ≥ 10× surrounding soil K (Smedema & Rycroft 1983).
8. FLAT TERRAIN: Derived for horizontal ground surface. Can be adapted for slopes (Oosterbaan 1975) but standard form assumes flat conditions.
9. SINGLE AQUIFER: Assumes single aquifer bounded below by impervious layer. Does not model leaky aquitards or confined systems.
10. PARABOLIC WT PROFILE: Dupuit approximation may underestimate actual WT height near drain by ~10% vs. exact potential flow (Kirkham 1958).

EQUATIONS USED

Hooghoudt Equation: $Q = [\,8 \cdot Kb \cdot d \cdot h + 4 \cdot Ka \cdot h^2\,] / L^2$ where h = Dd − Dw (watertable height above drain level)

Equivalent Depth (Van der Molen & Wesseling 1991): $d = \pi L / \{\,8 \cdot [\,\ln(L/(\pi \cdot r_o)) + F(x)\,]\,\}$ where x = 2πD/L

F(x) series (x ≥ 0.5): $F(x) = \Sigma\ 4 \cdot \exp(-2n \cdot x) / [\,n \cdot (1 - \exp(-2n \cdot x))\,]$ for n = 1, 3, 5, ...

F(x) approx (x < 0.5): $F(x) = \pi^2/(4x) + \ln(x/(2\pi))$

Equivalent radius for open trench: $r_o = u / \pi$ where u = wet perimeter of trench cross-section

Drawdown profile (Dupuit with recharge): $\frac{1}{2} \cdot Ka \cdot y^2 + Kb \cdot d \cdot y = R/2 \cdot x \cdot (L - x)$, solved as $y(x) = [\,-Kb \cdot d + \sqrt{((Kb \cdot d)^2 + Ka \cdot R \cdot x \cdot (L - x))}\,] / Ka$

DESIGN MODE — SOLVE FOR REQUIRED DRAIN SPACING (L) GIVEN RECHARGE RATE (R)

Iterative solution: $L^2 = [8 \cdot Kb \cdot d(L) \cdot h + 4 \cdot Ka \cdot h^2] / R$, where d depends on L. Successive substitution converges in ~10 iterations.

Iteration	L (m)	x = 2πD/L	F(x)	d (m)	L_new (m)	Converged?
0	293.876	0.31643	4.8091	12.992	275.469	no
1	284.673	0.32666	4.5967	12.941	274.931	no
2	279.802	0.33235	4.4847	12.913	274.633	no
3	277.218	0.33544	4.4254	12.897	274.472	no
4	275.845	0.33711	4.3940	12.889	274.384	no
5	275.115	0.33801	4.3773	12.885	274.338	no
6	274.726	0.33849	4.3684	12.882	274.313	no
7	274.520	0.33874	4.3636	12.881	274.300	no
8	274.410	0.33888	4.3611	12.880	274.293	no
9	274.351	0.33895	4.3598	12.880	274.289	no
10	274.320	0.33899	4.3591	12.880	274.287	no
11	274.303	0.33901	4.3587	12.880	274.286	no
12	274.295	0.33902	4.3585	12.880	274.285	no
13	274.290	0.33903	4.3584	12.880	274.285	no
14	274.287	0.33903	4.3583	12.880	274.285	no
15	274.286	0.33903	4.3583	12.880	274.285	no
16	274.285	0.33903	4.3583	12.880	274.285	YES
17	274.285	0.33903	4.3583	12.880	274.285	YES
18	274.285	0.33903	4.3583	12.880	274.285	YES
19	274.285	0.33903	4.3583	12.880	274.285	YES
20	274.285	0.33903	4.3583	12.880	274.284	YES
21	274.285	0.33903	4.3583	12.880	274.284	YES

DESIGN MODE RESULTS

Required drain spacing (L)	**274.28**	m
Corresponding equivalent depth (d)	12.880	m
Recharge rate (R) used	0.001096	m/day
Verification: Q at design L	0.001096	m/day
Q = R check	**VERIFIED ✓**	

HOOGHOUDT EQUATION — GROUNDWATER DRAWDOWN & DRAIN INFLOW CALCULATOR

Open Trench Drainage — Impervious Layer Far Below Drain Invert

INPUT PARAMETERS

Parameter	Value	Unit
Drain spacing (L)	**480**	m
Hydraulic conductivity above drain (Ka)	**3.97**	m/day
Hydraulic conductivity below drain (Kb)	**3.97**	m/day
Depth of drain invert below ground (Dd)	**2.6**	m
Steady-state watertable depth midway (Dw)	**2**	m
Depth of impervious layer below ground	**17**	m
Trench bottom width (W)	**5**	m
Trench side slopes (H:V)	**1**	—
Water depth in trench	**0**	m
Trench length	**650**	m
Recharge rate (R)	**0.0011**	m/day
No. of series terms for equiv. depth	**20**	—

INTERMEDIATE CALCULATIONS

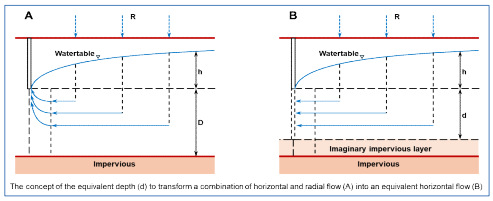
Parameter	Value	Unit
Depth of imperv. layer below drain (D)	14.400	m
Watertable height above drain level (h)	0.600	m
Trench wet perimeter (u)	5.000	m
Equivalent drain radius (r_o = u/π)	1.592	m
x = 2πD / L	0.1885	—
F(x) correction factor	9.5834	—
Equivalent depth (d)	13.323	m

DRAWDOWN PROFILE — STEADY-STATE WITH RECHARGE R (both sides of trench)

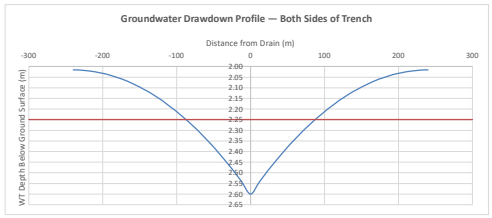
Position relative to drain (x)	WT height above drain (y)	WT depth below surface
m	*m*	*m*
-240.000	0.584	2.016
-228.000	0.582	2.018
-216.000	0.578	2.022
-204.000	0.571	2.029
-192.000	0.561	2.039
-180.000	0.548	2.052
-168.000	0.532	2.068
-156.000	0.514	2.086
-144.000	0.492	2.108
-132.000	0.468	2.132
-120.000	0.440	2.160
-108.000	0.410	2.190
-96.000	0.377	2.223
-84.000	0.340	2.260
-72.000	0.301	2.299
-60.000	0.259	2.341
-48.000	0.213	2.387
-36.000	0.165	2.435
-24.000	0.113	2.487
-12.000	0.058	2.542
0.000	0.000	2.600
12.000	0.058	2.542
24.000	0.113	2.487
36.000	0.165	2.435
48.000	0.213	2.387
60.000	0.259	2.341
72.000	0.301	2.299
84.000	0.340	2.260
96.000	0.377	2.223
108.000	0.410	2.190
120.000	0.440	2.160
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192.000	0.561	2.039
204.000	0.571	2.029
216.000	0.578	2.022
228.000	0.582	2.018
240.000	0.584	2.016

MAIN RESULTS

Drainage discharge Q (Hooghoudt)	**0.001127**	m/day
Drainage discharge Q	**1.127**	mm/day
Flow component below drain level	0.001102	m/day
Flow component above drain level	0.000025	m/day
Total inflow per m of trench — both sides (Q×L)	0.5408	m³/d/m
Total inflow to full trench — both sides	**351.55**	m³/day
Total inflow to full trench — both sides	**4.069**	L/s
Actual midpoint WT height (from R)	0.584	m
Actual midpoint WT depth below surface	2.016	m
Adequacy check (Q vs R)	**ADEQUATE: Q ≥ R**	



The concept of the equivalent depth (d) to transform a combination of horizontal and radial flow (A) into an equivalent horizontal flow (B)



SOURCE REFERENCES

1. Hooghoudt, S.B. (1940). Bijdragen tot de kennis van eenige natuurkundige grootheden van den grond, No.7. Bodemkundig Instituut, Groningen, Netherlands.
2. Ritzema, H.P. (1994). Subsurface flow to drains. Ch.8 in Drainage Principles and Applications, ILRI Publ.16, pp.236–304. Wageningen. ISBN 90-70754-33-9.
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LIMITATIONS & ASSUMPTIONS

1. STEADY-STATE ONLY: Assumes recharge rate = drainage discharge. Not valid for transient conditions (storm events, fluctuating WT). For transient analysis, use Glover-Dumm or De Zeeuw-Hellinga equations.
2. TWO-DIMENSIONAL FLOW: Assumes infinitely long trench — identical flow in every cross-section perpendicular to drain. End effects and 3D flow near trench terminations are ignored.
3. UNIFORM RECHARGE: Rainfall/irrigation recharge is assumed spatially uniform across the entire drained area.
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5. DUPUIT-FORCHHEIMER ASSUMPTIONS: Streamlines assumed horizontal, equipotentials vertical. Less accurate when D >> L/4.
6. EQUIVALENT DEPTH APPROXIMATION: Radial flow convergence near drain approximated by replacing actual D with smaller d. Accuracy decreases for very deep impervious layers (d maxes out at D ≈ ¼L).
7. NO ENTRANCE RESISTANCE: Assumes water enters trench freely. Valid only if trench backfill K ≥ 10× surrounding soil K (Smedema & Rycroft 1983).
8. FLAT TERRAIN: Derived for horizontal ground surface. Can be adapted for slopes (Oosterbaan 1975) but standard form assumes flat conditions.
9. SINGLE AQUIFER: Assumes single aquifer bounded below by impervious layer. Does not model leaky aquitards or confined systems.
10. PARABOLIC WT PROFILE: Dupuit approximation may underestimate actual WT height near drain by ~10% vs. exact potential flow (Kirkham 1958).

EQUATIONS USED

Hooghoudt Equation: $Q = [\,8 \cdot Kb \cdot d \cdot h + 4 \cdot Ka \cdot h^2\,] / L^2$ where h = Dd − Dw (watertable height above drain level)

Equivalent Depth (Van der Molen & Wesseling 1991): $d = \pi L / \{\,8 \cdot [\,\ln(L/(\pi \cdot r_0)) + F(x)\,]\,\}$ where $x = 2\pi D/L$

F(x) series (x ≥ 0.5): $F(x) = \Sigma\, 4 \cdot \exp(-2n \cdot x) / [\,n \cdot (1 - \exp(-2n \cdot x))\,]$ for n = 1, 3, 5, ...

F(x) approx (x < 0.5): $F(x) = \pi^2/(4x) + \ln(x/(2\pi))$

Equivalent radius for open trench: $r_o = u / \pi$ where u = wet perimeter of trench cross-section

Drawdown profile (Dupuit with recharge): $\tfrac{1}{2} \cdot Ka \cdot y^2 + Kb \cdot d \cdot y = R/2 \cdot x \cdot (L - x)$, solved as $y(x) = [\,-Kb \cdot d + \sqrt{((Kb \cdot d)^2 + Ka \cdot R \cdot x \cdot (L - x))}\,] / Ka$

DESIGN MODE — SOLVE FOR REQUIRED DRAIN SPACING (L) GIVEN RECHARGE RATE (R)

Iterative solution: $L^2 = [8 \cdot Kb \cdot d(L) \cdot h + 4 \cdot Ka \cdot h^2] / R$, where d depends on L. Successive substitution converges in ~10 iterations.

Iteration	L (m)	x = 2πD/L	F(x)	d (m)	L_new (m)	Converged?
0	505.581	0.17896	10.2291	13.374	487.617	no
1	496.599	0.18219	10.0021	13.357	487.310	no
2	491.955	0.18391	9.8848	13.348	487.147	no
3	489.551	0.18482	9.8242	13.343	487.062	no
4	488.307	0.18529	9.7928	13.340	487.017	no
5	487.662	0.18553	9.7765	13.339	486.994	no
6	487.328	0.18566	9.7681	13.338	486.982	no
7	487.155	0.18573	9.7637	13.338	486.976	no
8	487.066	0.18576	9.7615	13.338	486.973	no
9	487.019	0.18578	9.7603	13.338	486.971	no
10	486.995	0.18579	9.7597	13.338	486.970	no
11	486.983	0.18579	9.7594	13.338	486.970	no
12	486.976	0.18580	9.7592	13.338	486.970	no
13	486.973	0.18580	9.7591	13.338	486.969	no
14	486.971	0.18580	9.7591	13.338	486.969	no
15	486.970	0.18580	9.7591	13.338	486.969	YES
16	486.970	0.18580	9.7591	13.338	486.969	YES
17	486.970	0.18580	9.7591	13.338	486.969	YES
18	486.969	0.18580	9.7591	13.338	486.969	YES
19	486.969	0.18580	9.7591	13.338	486.969	YES
20	486.969	0.18580	9.7591	13.338	486.969	YES
21	486.969	0.18580	9.7591	13.338	486.969	YES

DESIGN MODE RESULTS

Required drain spacing (L)	**486.97**	m
Corresponding equivalent depth (d)	13.338	m
Recharge rate (R) used	0.001096	m/day
Verification: Q at design L	0.001096	m/day
Q = R check	**VERIFIED ✓**	

HOOGHOUDT EQUATION — GROUNDWATER DRAWDOWN & DRAIN INFLOW CALCULATOR

Open Trench Drainage — Impervious Layer Far Below Drain Invert

INPUT PARAMETERS

Parameter	Value	Unit
Drain spacing (L)	**330**	m
Hydraulic conductivity above drain (Ka)	**3.97**	m/day
Hydraulic conductivity below drain (Kb)	**3.97**	m/day
Depth of drain invert below ground (Dd)	**2.3**	m
Steady-state watertable depth midway (Dw)	**2**	m
Depth of impervious layer below ground	**17**	m
Trench bottom width (W)	**5**	m
Trench side slopes (H:V)	**1**	—
Water depth in trench	**0**	m
Trench length	**1200**	m
Recharge rate (R)	**0.0011**	m/day
No. of series terms for equiv. depth	**20**	—

INTERMEDIATE CALCULATIONS

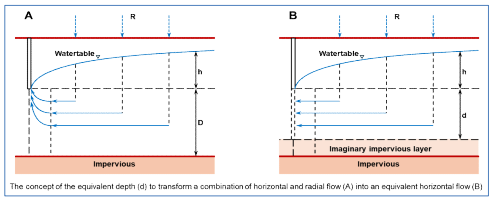
Parameter	Value	Unit
Depth of imperv. layer below drain (D)	14.700	m
Watertable height above drain level (h)	0.300	m
Trench wet perimeter (u)	5.000	m
Equivalent drain radius (r_o = u/π)	1.592	m
x = 2πD / L	0.2799	—
F(x) correction factor	5.7044	—
Equivalent depth (d)	13.098	m

DRAWDOWN PROFILE — STEADY-STATE WITH RECHARGE R (both sides of trench)

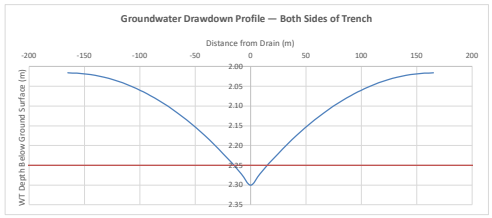
Position relative to drain (x)	WT height above drain (y)	WT depth below surface
m	*m*	*m*
-165.000	0.284	2.016
-156.750	0.283	2.017
-148.500	0.281	2.019
-140.250	0.277	2.023
-132.000	0.273	2.027
-123.750	0.266	2.034
-115.500	0.259	2.041
-107.250	0.249	2.051
-99.000	0.239	2.061
-90.750	0.227	2.073
-82.500	0.213	2.087
-74.250	0.199	2.101
-66.000	0.182	2.118
-57.750	0.165	2.135
-49.500	0.146	2.154
-41.250	0.125	2.175
-33.000	0.103	2.197
-24.750	0.079	2.221
-16.500	0.054	2.246
-8.250	0.028	2.272
0.000	0.000	2.300
8.250	0.028	2.272
16.500	0.054	2.246
24.750	0.079	2.221
33.000	0.103	2.197
41.250	0.125	2.175
49.500	0.146	2.154
57.750	0.165	2.135
66.000	0.182	2.118
74.250	0.199	2.101
82.500	0.213	2.087
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123.750	0.266	2.034
132.000	0.273	2.027
140.250	0.277	2.023
148.500	0.281	2.019
156.750	0.283	2.017
165.000	0.284	2.016

MAIN RESULTS

Drainage discharge Q (Hooghoudt)	**0.001159**	m/day
Drainage discharge Q	**1.159**	mm/day
Flow component below drain level	0.001146	m/day
Flow component above drain level	0.000013	m/day
Total inflow per m of trench — both sides (Q×L)	0.3825	m³/d/m
Total inflow to full trench — both sides	**459.00**	m³/day
Total inflow to full trench — both sides	**5.312**	L/s
Actual midpoint WT height (from R)	0.284	m
Actual midpoint WT depth below surface	2.016	m
Adequacy check (Q vs R)	**ADEQUATE: Q ≥ R**	



The concept of the equivalent depth (d) to transform a combination of horizontal and radial flow (A) into an equivalent horizontal flow (B)



SOURCE REFERENCES

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LIMITATIONS & ASSUMPTIONS

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2. TWO-DIMENSIONAL FLOW: Assumes infinitely long trench — identical flow in every cross-section perpendicular to drain. End effects and 3D flow near trench terminations are ignored.
3. UNIFORM RECHARGE: Rainfall/irrigation recharge is assumed spatially uniform across the entire drained area.
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EQUATIONS USED

Hooghoudt Equation: $Q = [\,8 \cdot Kb \cdot d \cdot h + 4 \cdot Ka \cdot h^2\,] / L^2$ where h = Dd − Dw (watertable height above drain level)

Equivalent Depth (Van der Molen & Wesseling 1991): $d = \pi L / \{\,8 \cdot [\,\ln(L/(\pi \cdot r_0)) + F(x)\,]\,\}$ where $x = 2\pi D/L$

F(x) series (x ≥ 0.5): $F(x) = \Sigma\, 4 \cdot \exp(-2n \cdot x) / [\,n \cdot (1 - \exp(-2n \cdot x))\,]$ for n = 1, 3, 5, ...

F(x) approx (x < 0.5): $F(x) = \pi^2/(4x) + \ln(x/(2\pi))$

Equivalent radius for open trench: $r_o = u / \pi$ where u = wet perimeter of trench cross-section

Drawdown profile (Dupuit with recharge): $½ \cdot Ka \cdot y^2 + Kb \cdot d \cdot y = R/2 \cdot x \cdot (L - x)$, solved as $y(x) = [\,-Kb \cdot d + \sqrt{((Kb \cdot d)^2 + Ka \cdot R \cdot x \cdot (L - x))}\,] / Ka$

DESIGN MODE — SOLVE FOR REQUIRED DRAIN SPACING (L) GIVEN RECHARGE RATE (R)

Iterative solution: $L^2 = [8 \cdot Kb \cdot d(L) \cdot h + 4 \cdot Ka \cdot h^2] / R$, where d depends on L. Successive substitution converges in ~10 iterations.

Iteration	L (m)	x = 2πD/L	F(x)	d (m)	L_new (m)	Converged?
0	359.319	0.25705	6.4026	13.215	340.884	no
1	350.102	0.26382	6.1823	13.180	340.437	no
2	345.269	0.26751	6.0671	13.161	340.194	no
3	342.731	0.26949	6.0067	13.151	340.063	no
4	341.397	0.27054	5.9750	13.146	339.994	no
5	340.696	0.27110	5.9583	13.143	339.957	no
6	340.327	0.27139	5.9495	13.141	339.938	no
7	340.132	0.27155	5.9449	13.140	339.928	no
8	340.030	0.27163	5.9425	13.140	339.923	no
9	339.976	0.27167	5.9412	13.140	339.920	no
10	339.948	0.27170	5.9405	13.140	339.918	no
11	339.933	0.27171	5.9402	13.140	339.918	no
12	339.925	0.27171	5.9400	13.140	339.917	no
13	339.921	0.27172	5.9399	13.140	339.917	no
14	339.919	0.27172	5.9398	13.140	339.917	no
15	339.918	0.27172	5.9398	13.140	339.917	no
16	339.917	0.27172	5.9398	13.140	339.917	YES
17	339.917	0.27172	5.9398	13.140	339.917	YES
18	339.917	0.27172	5.9398	13.140	339.917	YES
19	339.917	0.27172	5.9398	13.140	339.917	YES
20	339.917	0.27172	5.9398	13.140	339.917	YES
21	339.917	0.27172	5.9398	13.140	339.917	YES

DESIGN MODE RESULTS

Required drain spacing (L)	**339.92**	m
Corresponding equivalent depth (d)	13.140	m
Recharge rate (R) used	0.001096	m/day
Verification: Q at design L	0.001096	m/day
Q = R check	**VERIFIED ✓**	

APPENDIX E

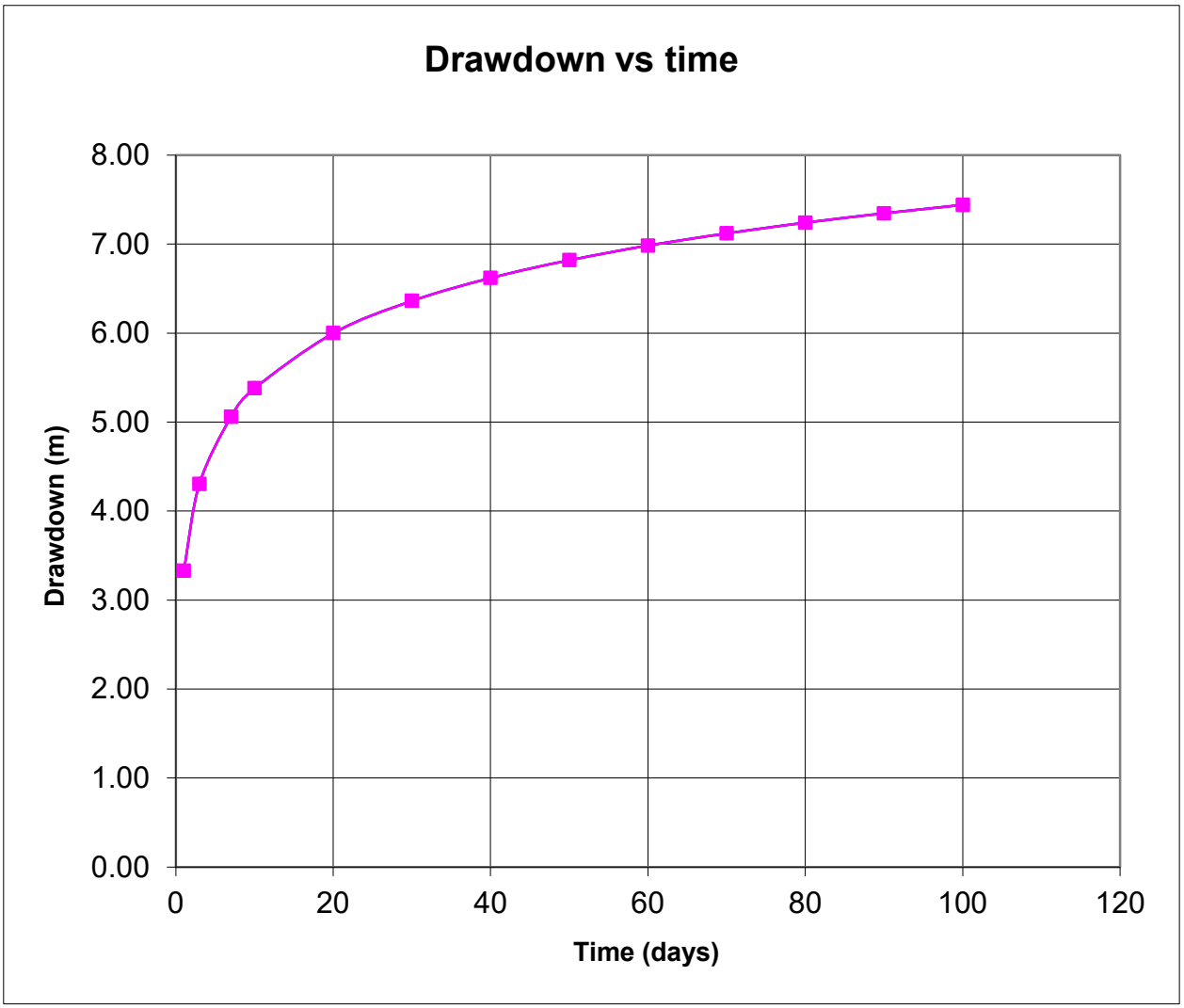
DRAWDOWN MODELLING FOR WASTEWATER INFRASTRUCTURE



Time-drawdown calculations using Theis equation

Aquifer parameters		
T	45.31	m2/d
S	0.1	
B		
Pumping rate		
Q	5.9	l/s

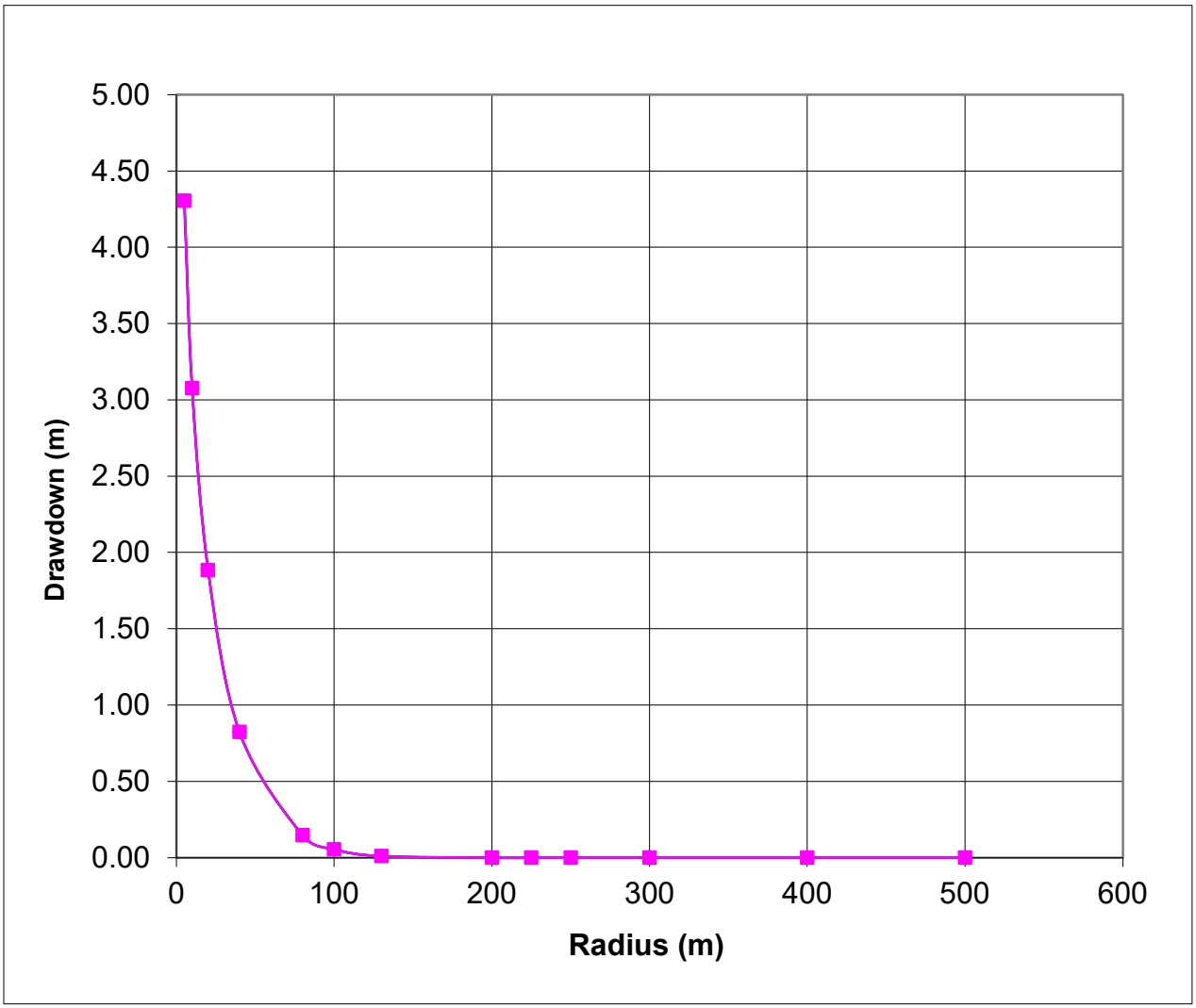
Radius (m)	5	5	5
Time (days)	Drawdown (m)	Drawdown (m)	Drawdown (m)
1	3.331	3.331	3.331
3	4.306	4.306	4.306
7	5.062	5.062	5.062
10	5.381	5.381	5.381
20	6.001	6.001	6.001
30	6.364	6.364	6.364
40	6.621	6.621	6.621
50	6.821	6.821	6.821
60	6.984	6.984	6.984
70	7.122	7.122	7.122
80	7.242	7.242	7.242
90	7.347	7.347	7.347
100	7.441	7.441	7.441



Distance-drawdown calculations using Theis equation

Aquifer parameters		
T	45.31	m2/d
S	0.1	
B		
Pumping rate		
Q	5.9	l/s

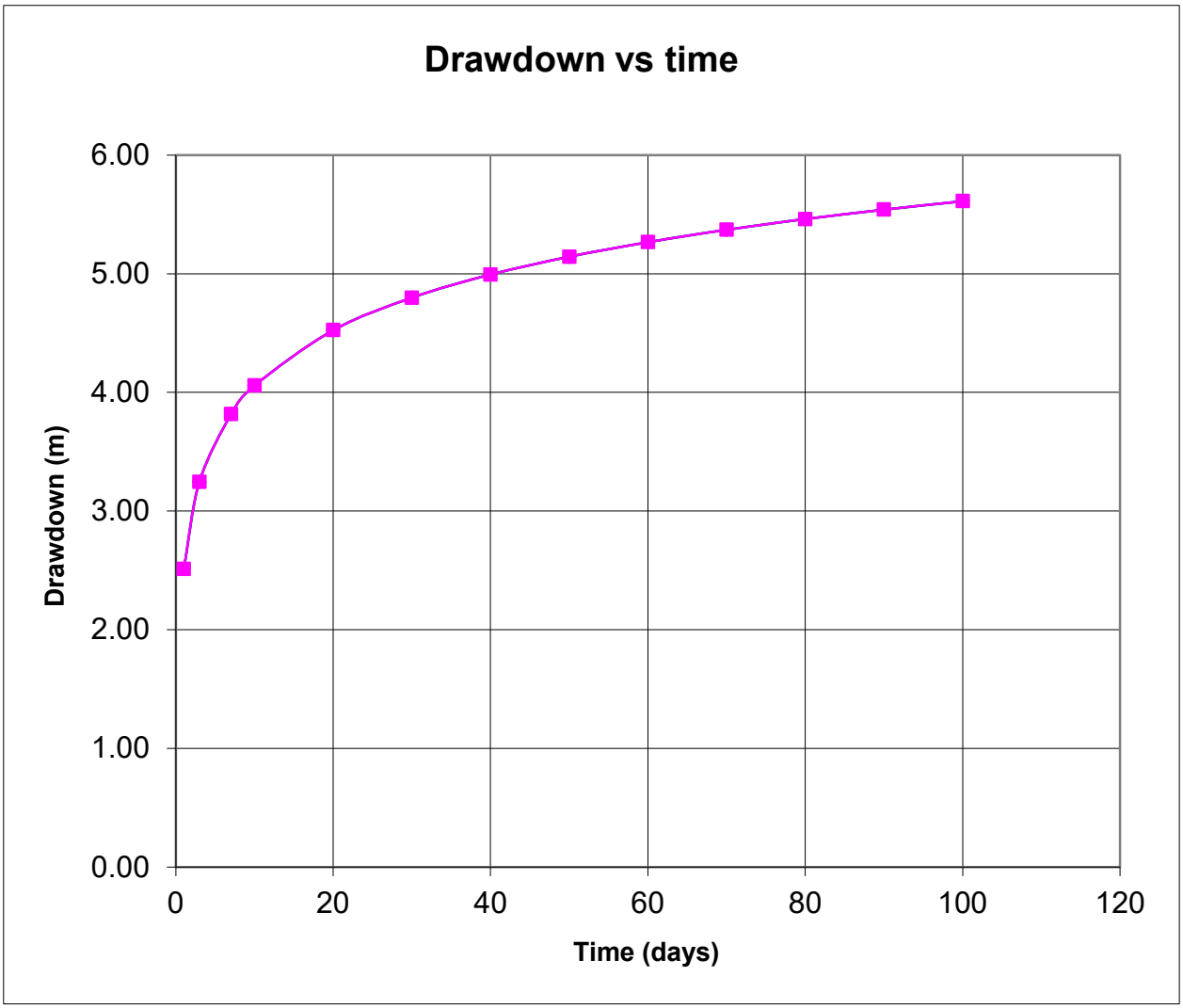
Time (days)	3	3	3
Radius (m)	Drawdown (m)	Drawdown (m)	Drawdown (m)
5	4.306	4.306	4.306
10	3.077	3.077	3.077
20	1.884	1.884	1.884
40	0.824	0.824	0.824
80	0.147	0.147	0.147
100	0.055	0.055	0.055
130	0.010	0.010	0.010
200	0.000	0.000	0.000
225	0.000	0.000	0.000
250	0.000	0.000	0.000
300	-	-	-
400	-	-	-
500	-	-	-



Time-drawdown calculations using Theis equation

Aquifer parameters		
T	45.31	m2/d
S	0.1	
B		
Pumping rate		
Q	4.45	l/s

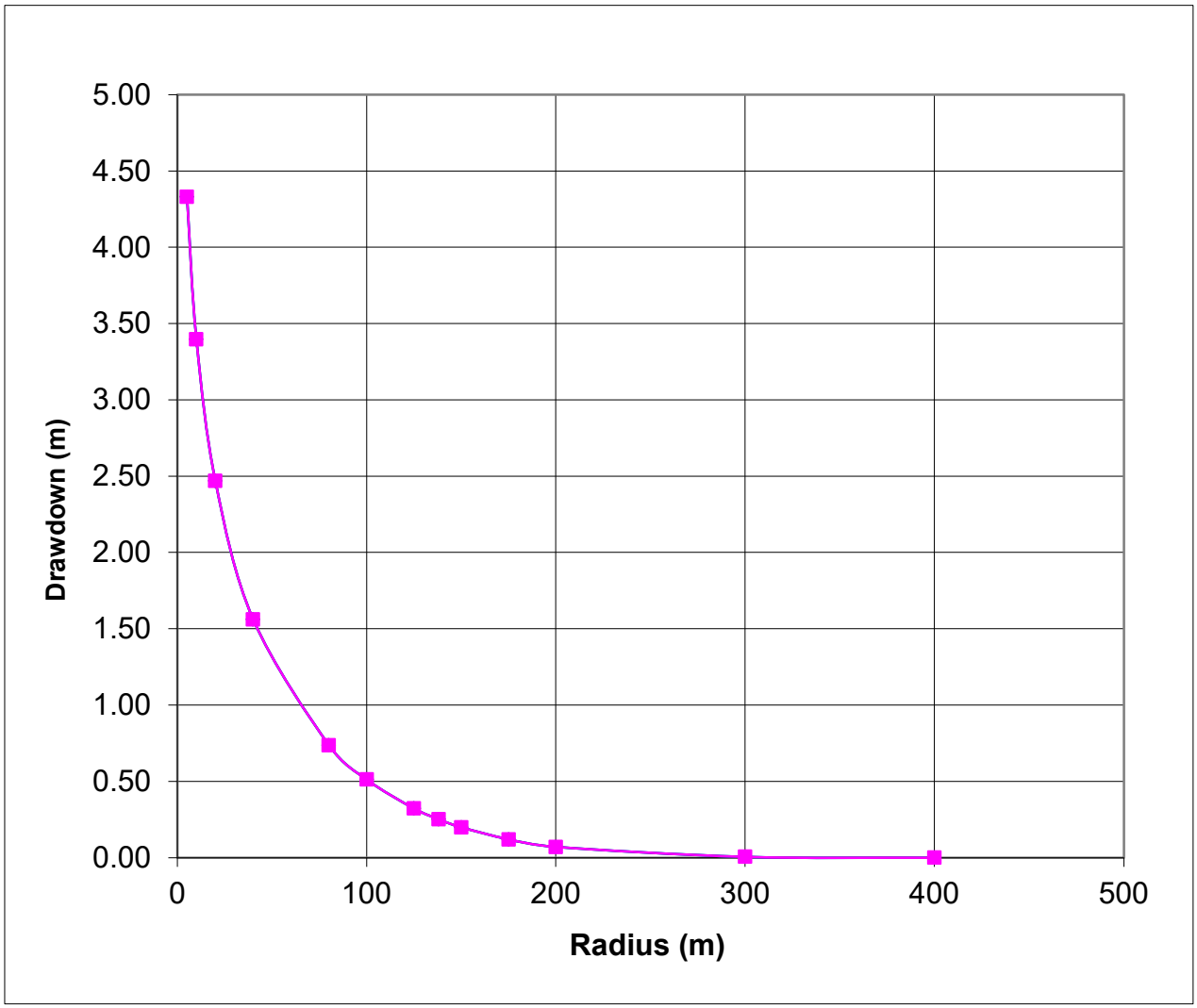
Radius (m)	5	5	5
Time (days)	Drawdown (m)	Drawdown (m)	Drawdown (m)
1	2.512	2.512	2.512
3	3.248	3.248	3.248
7	3.818	3.818	3.818
10	4.058	4.058	4.058
20	4.526	4.526	4.526
30	4.800	4.800	4.800
40	4.994	4.994	4.994
50	5.145	5.145	5.145
60	5.268	5.268	5.268
70	5.372	5.372	5.372
80	5.462	5.462	5.462
90	5.541	5.541	5.541
100	5.612	5.612	5.612



Distance-drawdown calculations using Theis equation

Aquifer parameters		
T	45.31	m2/d
S	0.1	
B		
Pumping rate		
Q	4.45	l/s

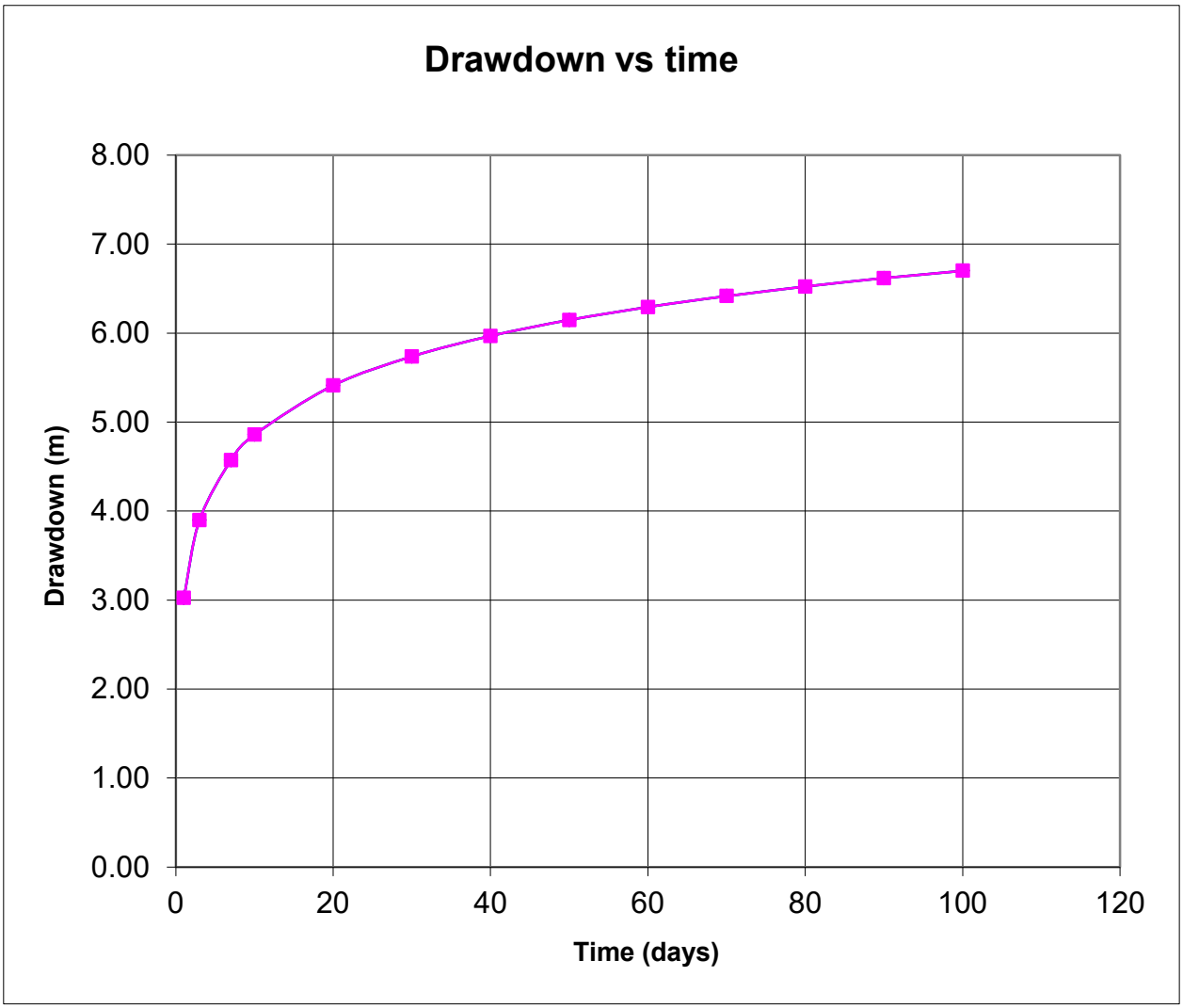
Time (days)	15	15	15
Radius (m)	Drawdown (m)	Drawdown (m)	Drawdown (m)
5	4.332	4.332	4.332
10	3.398	3.398	3.398
20	2.469	2.469	2.469
40	1.562	1.562	1.562
80	0.737	0.737	0.737
100	0.513	0.513	0.513
125	0.323	0.323	0.323
138	0.252	0.252	0.252
150	0.200	0.200	0.200
175	0.120	0.120	0.120
200	0.070	0.070	0.070
300	0.006	0.006	0.006
400	0.000	0.000	0.000



Time-drawdown calculations using Theis equation

Aquifer parameters		
T	48.09	m2/d
S	0.1	
B		
Pumping rate		
Q	5.6	l/s

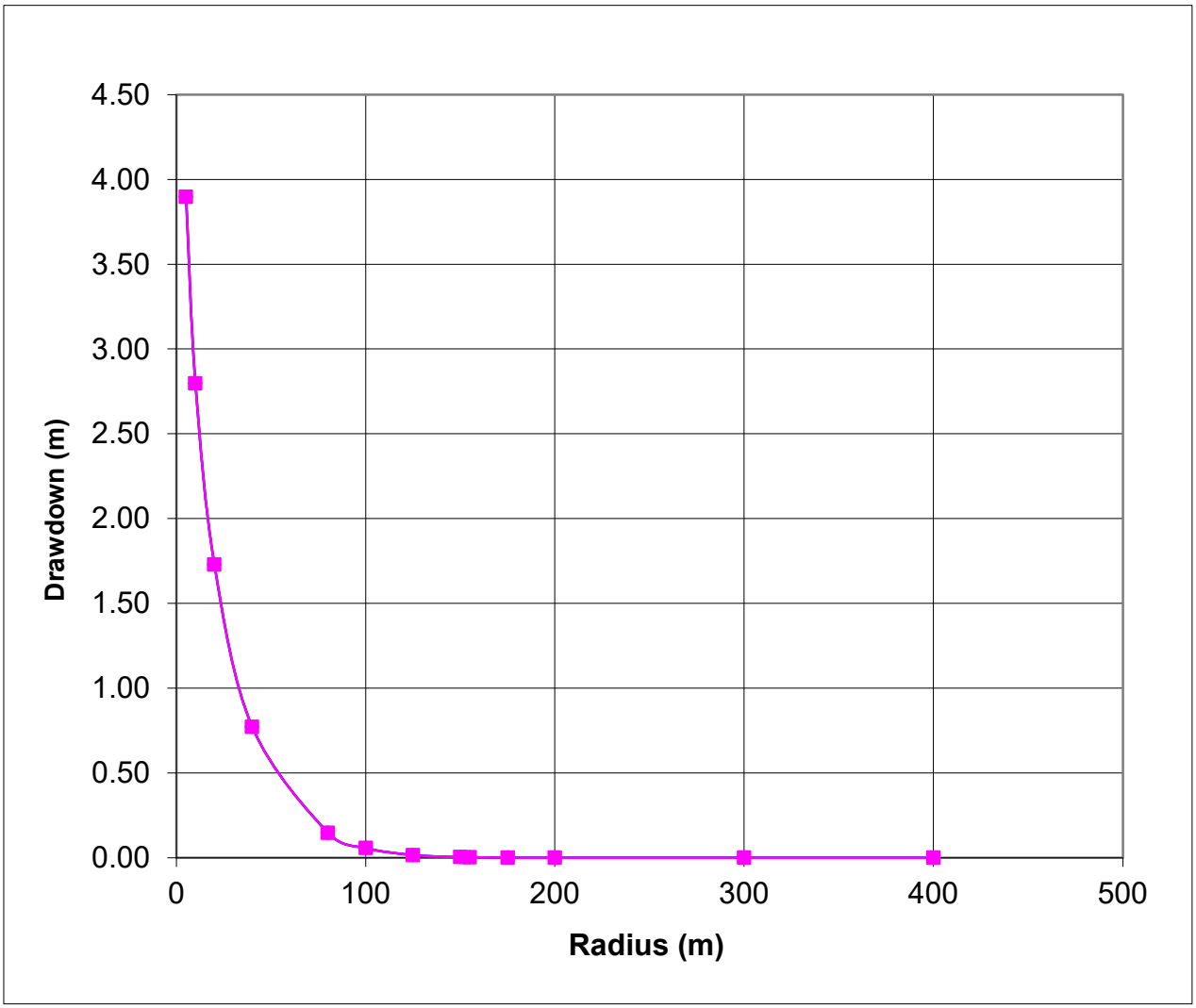
Radius (m)	5	5	5
Time (days)	Drawdown (m)	Drawdown (m)	Drawdown (m)
1	3.025	3.025	3.025
3	3.898	3.898	3.898
7	4.575	4.575	4.575
10	4.860	4.860	4.860
20	5.414	5.414	5.414
30	5.739	5.739	5.739
40	5.969	5.969	5.969
50	6.147	6.147	6.147
60	6.293	6.293	6.293
70	6.417	6.417	6.417
80	6.524	6.524	6.524
90	6.618	6.618	6.618
100	6.702	6.702	6.702



Distance-drawdown calculations using Theis equation

Aquifer parameters		
T	48.09	m2/d
S	0.1	
B		
Pumping rate		
Q	5.6	l/s

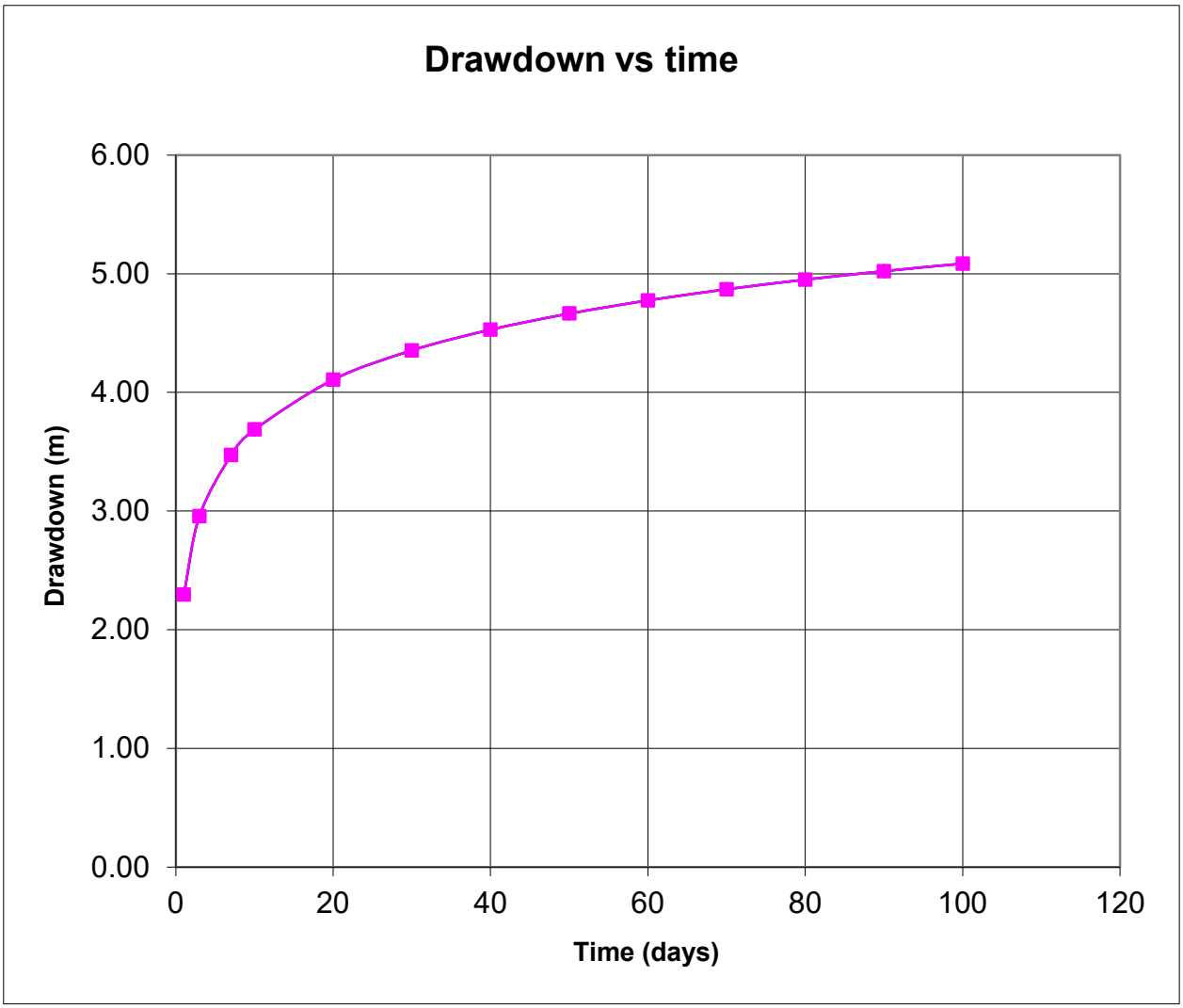
Time (days)	3	3	3
Radius (m)	Drawdown (m)	Drawdown (m)	Drawdown (m)
5	3.898	3.898	3.898
10	2.799	2.799	2.799
20	1.729	1.729	1.729
40	0.772	0.772	0.772
80	0.147	0.147	0.147
100	0.057	0.057	0.057
125	0.015	0.015	0.015
150	0.003	0.003	0.003
155	0.002	0.002	0.002
175	0.001	0.001	0.001
200	0.000	0.000	0.000
300	-	-	-
400	-	-	-



Time-drawdown calculations using Theis equation

Aquifer parameters		
T	48.09	m2/d
S	0.1	
B		
Pumping rate		
Q	4.25	l/s

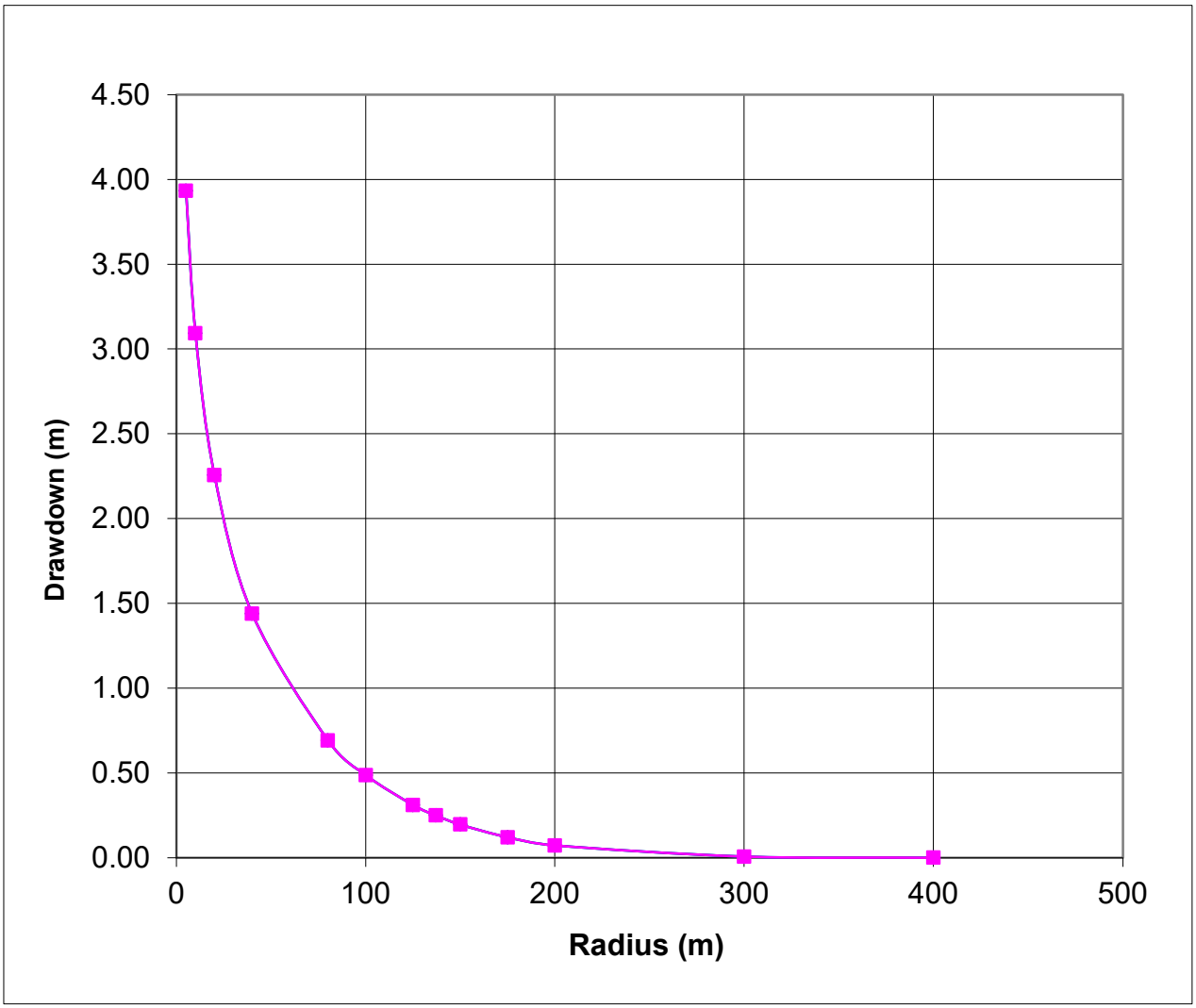
Radius (m)	5	5	5
Time (days)	Drawdown (m)	Drawdown (m)	Drawdown (m)
1	2.296	2.296	2.296
3	2.958	2.958	2.958
7	3.472	3.472	3.472
10	3.688	3.688	3.688
20	4.109	4.109	4.109
30	4.355	4.355	4.355
40	4.530	4.530	4.530
50	4.665	4.665	4.665
60	4.776	4.776	4.776
70	4.870	4.870	4.870
80	4.951	4.951	4.951
90	5.023	5.023	5.023
100	5.087	5.087	5.087



Distance-drawdown calculations using Theis equation

Aquifer parameters		
T	48.09	m2/d
S	0.1	
B		
Pumping rate		
Q	4.25	l/s

Time (days)	15	15	15
Radius (m)	**Drawdown (m)**	**Drawdown (m)**	**Drawdown (m)**
5	3.934	3.934	3.934
10	3.093	3.093	3.093
20	2.257	2.257	2.257
40	1.440	1.440	1.440
80	0.692	0.692	0.692
100	0.487	0.487	0.487
125	0.311	0.311	0.311
137	0.250	0.250	0.250
150	0.196	0.196	0.196
175	0.120	0.120	0.120
200	0.072	0.072	0.072
300	0.007	0.007	0.007
400	0.000	0.000	0.000



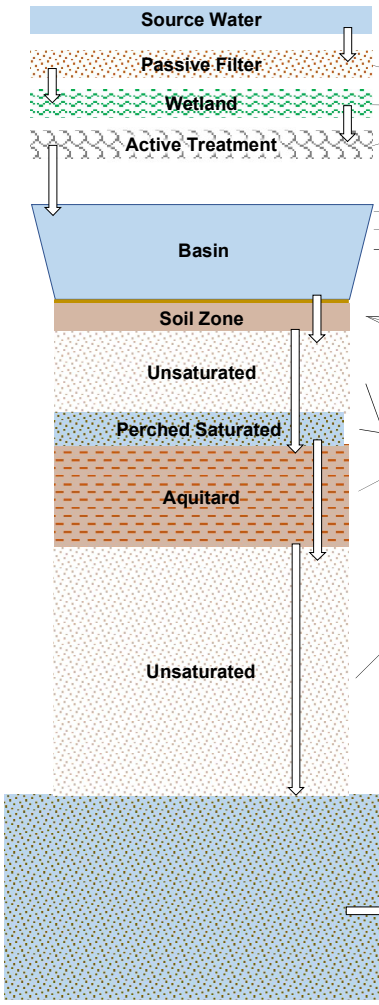
APPENDIX F
WASTEWATER LAYOUT AND ATTENUATION



INFILTRATION BASIN - PATHOGEN ATTENUATION ESTIMATION

PROJECT: Waikato Thoroughbred Racing
Job Number: WGA 253138
Date: 13/09/2026
Document #: WGA253138-RP-HG-0003

Notes: Main Pond Area



	Value	Units	Ref
Source water quality	**200**	MPN/100mL	
Pre-treatment attenuation			
Passive filter log attenuation	0	log reduction	
Pre-treatment wetland	0	log reduction	
Active treatment log attenuation	0	log reduction	
Treated water quality	**200**	MPN/100mL	
Basin contamination			
Introduced pathogen load	0	log increase/day	
UV / dieback attenuation	0	log reduction/day	
Water retention time in basin	0	days	
Water quality in basin	**200**		
Soil Aquifer Treatment			
Schmutzdecke attenuation factor	0	log reduction	
Soil horizon attenuation factor	5.48	log reduction/m	2
Soil horizon thickness	0.1	m	
Soil horizon attenuation	0.548	log reduction	
Total SAT attenuation	0.548	log reduction	
Seepage water quality beneath soil horizon	**56.63**	MPN/100mL	
Vertical saturated flow beneath soil horizon			
Infiltration rate	0	m/day	
Vertical flow distance	0	m	
Effective porosity	0	m3/m3	
Flow time	0	days	
Daily attenuation factor	0	log/day	
Vertical saturated flow attenuation	0	log reduction	
Vertical unsaturated flow (vadose zone)			
Vadose zone vertical thickness	1.2	m	
Vadose zone attenuation factor	0.05	log reduction/m	2
Vertical unsaturated flow attenuation	0.06	log reduction	
Seepage water quality entering saturated groundwater system	**49.32**	MPN/100mL	
Lateral groundwater flow within aquifer			
Receiver distance from MAR site	250	m	
Attenuation per metre	0.050	log/m	2
Aquifer saturated flow attenuation	12.50	log reduction	
Groundwater quality at receiving point	**1.56E-11**	MPN/100mL	

References:

1 Pfannes, K.R., Langenbach, K.M., Pilloni, G., Stührmann, T., Euringer, K., Lueders, T., Neu, T.R., Müller, J.A., Kästner, M. and Meckenstock, R.U., 2015. Selective elimination of bacterial faecal indicators in the Schmutzdecke of slow sand filtration columns. Applied microbiology and biotechnology, 99(23), pp.10323-10332.

2 Pang, L., 2009, Microbial removal rates in subsurface media estimated from published studies of field experiments and large intact soil cores. J. Environ. Qual., 38: 1531-1559.

3 John, D. E. & Rose, J. B. 2005. Review of factors affecting microbial survival in groundwater. Environmental Science & Technology 39 (19): 7345 – 7356.

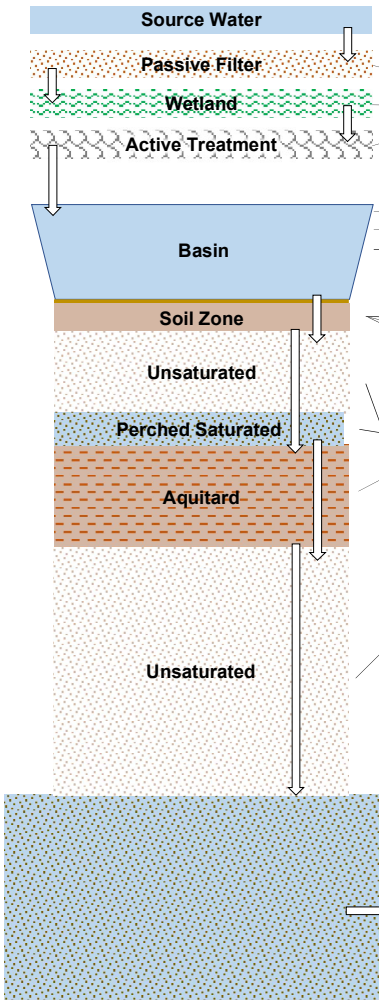
4 Sinton, L. W. 1986. Microbial contamination of alluvial gravel aquifers by septic tank effluent. Water Air Soil Pollution 28: 407 – 425.

5 Schijven, J., Pang, L. and Ying, G.G., 2017. Evaluation of subsurface microbial transport using microbial indicators, surrogates and tracers. Global water pathogen project.

INFILTRATION BASIN - PATHOGEN ATTENUATION ESTIMATION

PROJECT: Waikato Thoroughbred Racing
Job Number: WGA 253138
Date: 13/09/2026
Document #: WGA253138-RP-HG-0003

Notes: Senior Living Area



	Value	Units	Ref
Source water quality	**200**	MPN/100mL	
Pre-treatment attenuation			
Passive filter log attenuation	0	log reduction	
Pre-treatment wetland	0	log reduction	
Active treatment log attenuation	0	log reduction	
Treated water quality	**200**	MPN/100mL	
Basin contamination			
Introduced pathogen load	0	log increase/day	
UV / dieback attenuation	0	log reduction/day	
Water retention time in basin	0	days	
Water quality in basin	**200**		
Soil Aquifer Treatment			
Schmutzdecke attenuation factor	0	log reduction	
Soil horizon attenuation factor	5.48	log reduction/m	2
Soil horizon thickness	0.1	m	
Soil horizon attenuation	0.548	log reduction	
Total SAT attenuation	0.548	log reduction	
Seepage water quality beneath soil horizon	**56.63**	MPN/100mL	
Vertical saturated flow beneath soil horizon			
Infiltration rate	0	m/day	
Vertical flow distance	0	m	
Effective porosity	0	m3/m3	
Flow time	0	days	
Daily attenuation factor	0	log/day	
Vertical saturated flow attenuation	0	log reduction	
Vertical unsaturated flow (vadose zone)			
Vadose zone vertical thickness	1.9	m	
Vadose zone attenuation factor	0.05	log reduction/m	2
Vertical unsaturated flow attenuation	0.095	log reduction	
Seepage water quality entering saturated groundwater system	**45.50**	MPN/100mL	
Lateral groundwater flow within aquifer			
Receiver distance from MAR site	250	m	
Attenuation per metre	0.050	log/m	2
Aquifer saturated flow attenuation	12.50	log reduction	
Groundwater quality at receiving point	**1.44E-11**	MPN/100mL	

References:

1 Pfannes, K.R., Langenbach, K.M., Pilloni, G., Stührmann, T., Euringer, K., Lueders, T., Neu, T.R., Müller, J.A., Kästner, M. and Meckenstock, R.U., 2015. Selective elimination of bacterial faecal indicators in the Schmutzdecke of slow sand filtration columns. Applied microbiology and biotechnology, 99(23), pp.10323-10332.

2 Pang, L., 2009, Microbial removal rates in subsurface media estimated from published studies of field experiments and large intact soil cores. J. Environ. Qual., 38: 1531-1559.

3 John, D. E. & Rose, J. B. 2005. Review of factors affecting microbial survival in groundwater. Environmental Science & Technology 39 (19): 7345 – 7356.

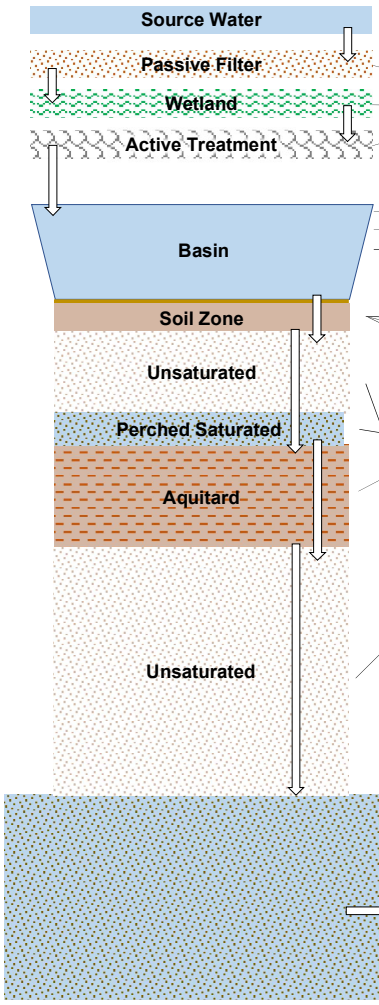
4 Sinton, L. W. 1986. Microbial contamination of alluvial gravel aquifers by septic tank effluent. Water Air Soil Pollution 28: 407 – 425.

5 Schijven, J., Pang, L. and Ying, G.G., 2017. Evaluation of subsurface microbial transport using microbial indicators, surrogates and tracers. Global water pathogen project.

INFILTRATION BASIN - PATHOGEN ATTENUATION ESTIMATION

PROJECT: Waikato Thoroughbred Racing
Job Number: WGA 253138
Date: 13/09/2026
Document #: WGA253138-RP-HG-0003

Notes: Trainer Area



	Value	Units	Ref
Source water quality	**200**	MPN/100mL	
Pre-treatment attenuation			
Passive filter log attenuation	0	log reduction	
Pre-treatment wetland	0	log reduction	
Active treatment log attenuation	0	log reduction	
Treated water quality	**200**	MPN/100mL	
Basin contamination			
Introduced pathogen load	0	log increase/day	
UV / dieback attenuation	0	log reduction/day	
Water retention time in basin	0	days	
Water quality in basin	**200**		
Soil Aquifer Treatment			
Schmutzdecke attenuation factor	0	log reduction	
Soil horizon attenuation factor	5.34	log reduction/m	2
Soil horizon thickness	0.1	m	
Soil horizon attenuation	0.534	log reduction	
Total SAT attenuation	0.534	log reduction	
Seepage water quality beneath soil horizon	**58.48**	MPN/100mL	
Vertical saturated flow beneath soil horizon			
Infiltration rate	0	m/day	
Vertical flow distance	0	m	
Effective porosity	0	m3/m3	
Flow time	0	days	
Daily attenuation factor	0	log/day	
Vertical saturated flow attenuation	0	log reduction	
Vertical unsaturated flow (vadose zone)			
Vadose zone vertical thickness	0.7	m	
Vadose zone attenuation factor	0.05	log reduction/m	2
Vertical unsaturated flow attenuation	0.035	log reduction	
Seepage water quality entering saturated groundwater system	**53.95**	MPN/100mL	
Lateral groundwater flow within aquifer			
Receiver distance from MAR site	250	m	
Attenuation per metre	0.050	log/m	2
Aquifer saturated flow attenuation	12.50	log reduction	
Groundwater quality at receiving point	**1.71E-11**	MPN/100mL	

References:

1 Pfannes, K.R., Langenbach, K.M., Pilloni, G., Stührmann, T., Euringer, K., Lueders, T., Neu, T.R., Müller, J.A., Kästner, M. and Meckenstock, R.U., 2015. Selective elimination of bacterial faecal indicators in the Schmutzdecke of slow sand filtration columns. Applied microbiology and biotechnology, 99(23), pp.10323-10332.

2 Pang, L., 2009, Microbial removal rates in subsurface media estimated from published studies of field experiments and large intact soil cores. J. Environ. Qual., 38: 1531-1559.

3 John, D. E. & Rose, J. B. 2005. Review of factors affecting microbial survival in groundwater. Environmental Science & Technology 39 (19): 7345 – 7356.

4 Sinton, L. W. 1986. Microbial contamination of alluvial gravel aquifers by septic tank effluent. Water Air Soil Pollution 28: 407 – 425.

5 Schijven, J., Pang, L. and Ying, G.G., 2017. Evaluation of subsurface microbial transport using microbial indicators, surrogates and tracers. Global water pathogen project.

WGA

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