



Southern Links 1 Residential Stage 1 Southern Block Hydrogeological Effects Assessment

ASSESSMENT OF EFFECTS

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1 EXECUTIVE SUMMARY

This Hydrogeological Effects Assessment has evaluated the potential groundwater effects associated with the Southern Links 1 Residential Development. The assessment has focused on temporary stormwater and wastewater infrastructure construction dewatering and longer-term groundwater diversion associated with the proposed Artificial Wetlands.

Site specific investigations and groundwater monitoring have confirmed the presence of a shallow unconfined aquifer overlying a deeper confined aquifer, separated by a thin, laterally persistent low permeability aquitard. Confined groundwater levels exhibit seasonal variability and indicate the deeper aquifer is hydraulically connected to local drains and the headwaters of the Mangakōtukutuku Stream, which act as key discharge features within the catchment.

Analytical modelling indicates that groundwater drawdown effects associated with the construction of artificial wetlands and wastewater infrastructure will be localised, temporary, and minor with the implementation of proposed management measures. Conservative assumptions have been adopted throughout the assessment, including the appropriate use of seasonal groundwater levels to support the effects assessment and worst-case construction scenarios. The accompanying Geotechnical Assessments (CMW 2026c) has used these results to undertake ground settlement assessments.

Where dewatering temporarily intercepts groundwater that would otherwise contribute to local drains or stream baseflow, effects can be effectively managed through controlled abstraction and return of flows upstream of works (as proposed in consent conditions), maintaining surface water continuity during construction. With the Residential Development's proposed management measures, surface water effects have been assessed as minor and temporary.

In the operational phase, the proposed artificial wetlands will function as permanent hydraulic features that intercept and redistribute groundwater discharge within the catchment. This represents a modification of groundwater flow paths rather than a net loss of groundwater from the system. Long-term effects on groundwater levels, nearby bores, natural wetlands, and connected surface water bodies are assessed as less than minor.

Overall, based on the investigations undertaken, modelling completed, and mitigation measures proposed, WGA considers that the groundwater effects associated with the construction and operation of the Residential Development can be appropriately managed and will be no more than minor. No additional permanent groundwater or surface water monitoring is considered necessary, with standard construction-phase environmental controls, as proposed, providing sufficient verification of predicted effects.

2 INTRODUCTION

2.1 Background

Wallbridge Gilbert Aztec (WGA) has been engaged for technical services in relation to the substantive application for the Southern Links 1 (“SL1”) development (the “Project”) under the Fast-track Approvals Act 2024 (“FTAA”). The Project is to provide a range of housing typologies and land tenure opportunities to provide affordable housing, and to boost the industrial capability of the region (Figure 1). This assessment of hydrogeological effects focuses on the residential portion of the Project (the “Residential Area” and the “Residential Development”).

2.2 Site Description

The entire SL1 Project area is approximately 440 ha in size and is situated between the established Hamilton suburbs of Dinsdale, Frankton, Deanwell and Glenview to the northeast, and rural land within Waipa to the southwest. The SL1 area is currently zoned rural under the Waipa District Plan and consists of lots in pasture, rural lifestyle, equine industry, and peri-industrial use.

The wider SL1 Project will consist of both residential and industrial areas with green spaces throughout the development. Stormwater across the SL1 area will be managed by naturalised streams, rain gardens and constructed wetlands. The SL1 site has been divided into two areas, known as the Residential Area and the Industrial Area. This assessment of effects focuses only on the Residential Development (Figure 1).

The Residential Area is a 46 Ha block of rural dairy farmland to the south of Saxbys Road. The proposed development is for subdivision for residential development. This includes 938 lots, and a 1,000 m² commercial lot, 3,200 m² neighbourhood park and associated infrastructure and riparian planting.

2.3 Proposed Activity

For the Residential Development, six wetlands are proposed to be constructed to manage stormwater in the development. Additionally, two wastewater pump stations (WWPS) and site-wide wastewater pipeline infrastructure need to be constructed to support the development (Figure 2).

The Residential Development has two main groundwater-related components that have the potential to influence catchment water balances:

1. **Short-term temporary dewatering during construction earthworks**, for proposed Artificial Wetlands and wastewater infrastructure (site-wide pipelines and two 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 ESCP) and relevant consent condition requirements.
2. **Long-term groundwater interception and diversion to the proposed Artificial Wetlands**, which will act as persistent hydraulic features within the developed catchment.

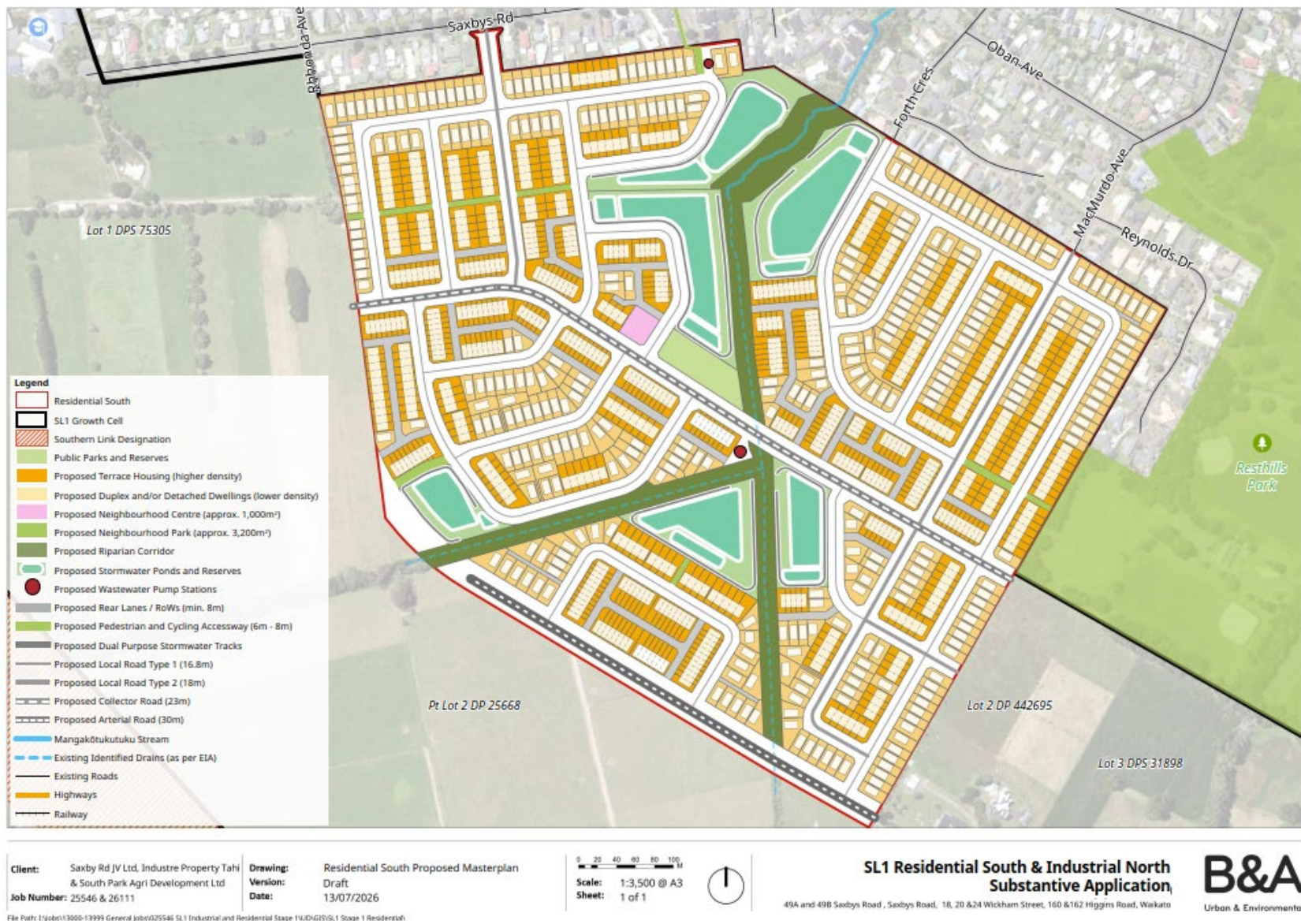
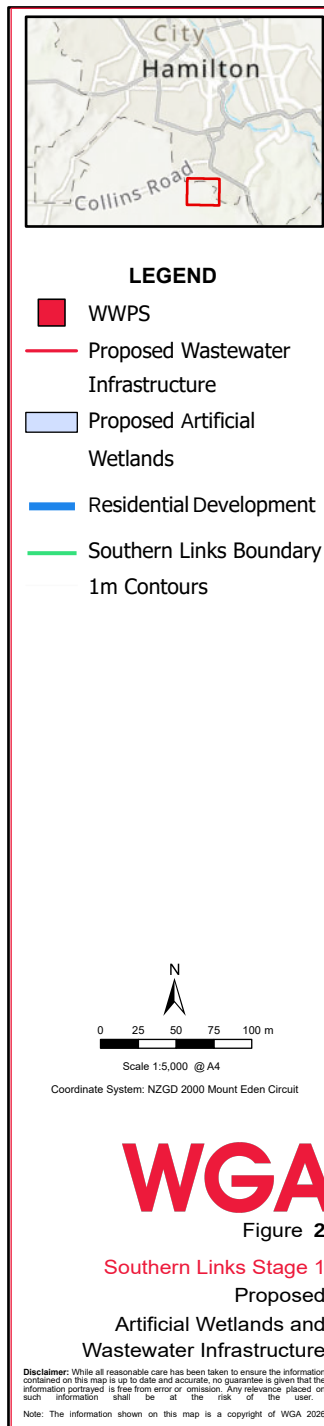


Figure 1: Residential Development Master Plan



3 EXISTING ENVIRONMENT

3.1 Rainfall

Rainfall data are recorded at the Ruakura Weather Station, which is located approximately 6.2 km northeast 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.

During the period of groundwater level monitoring undertaken at the Residential Area for this assessment (July 2025 to January 2026; see Section 3.5.3), the peak daily rainfall recorded at Ruakura was 42.4 mm. Total rainfall over this monitoring period was approximately 620 mm. When annualised, this equates to an annual rainfall of approximately 1,048 mm/year, which is similar to the long-term mean annual rainfall. Overall, rainfall conditions during the monitoring period are considered to be broadly representative of average long-term conditions.

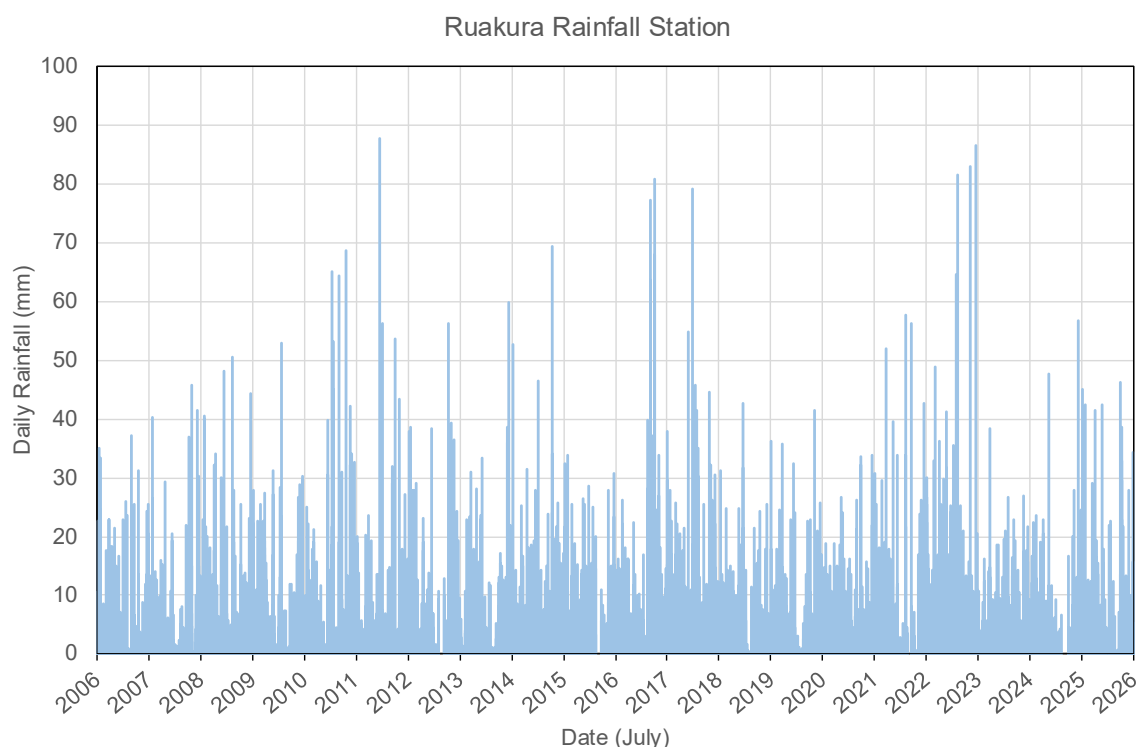
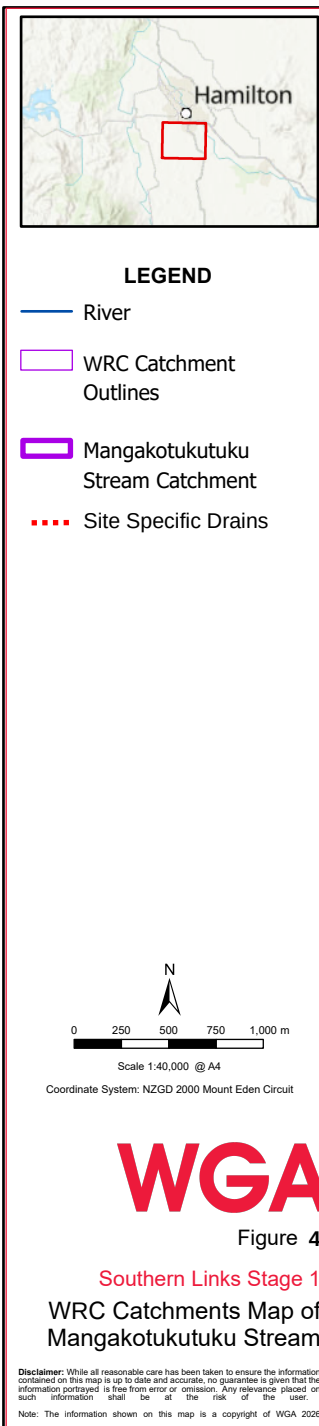
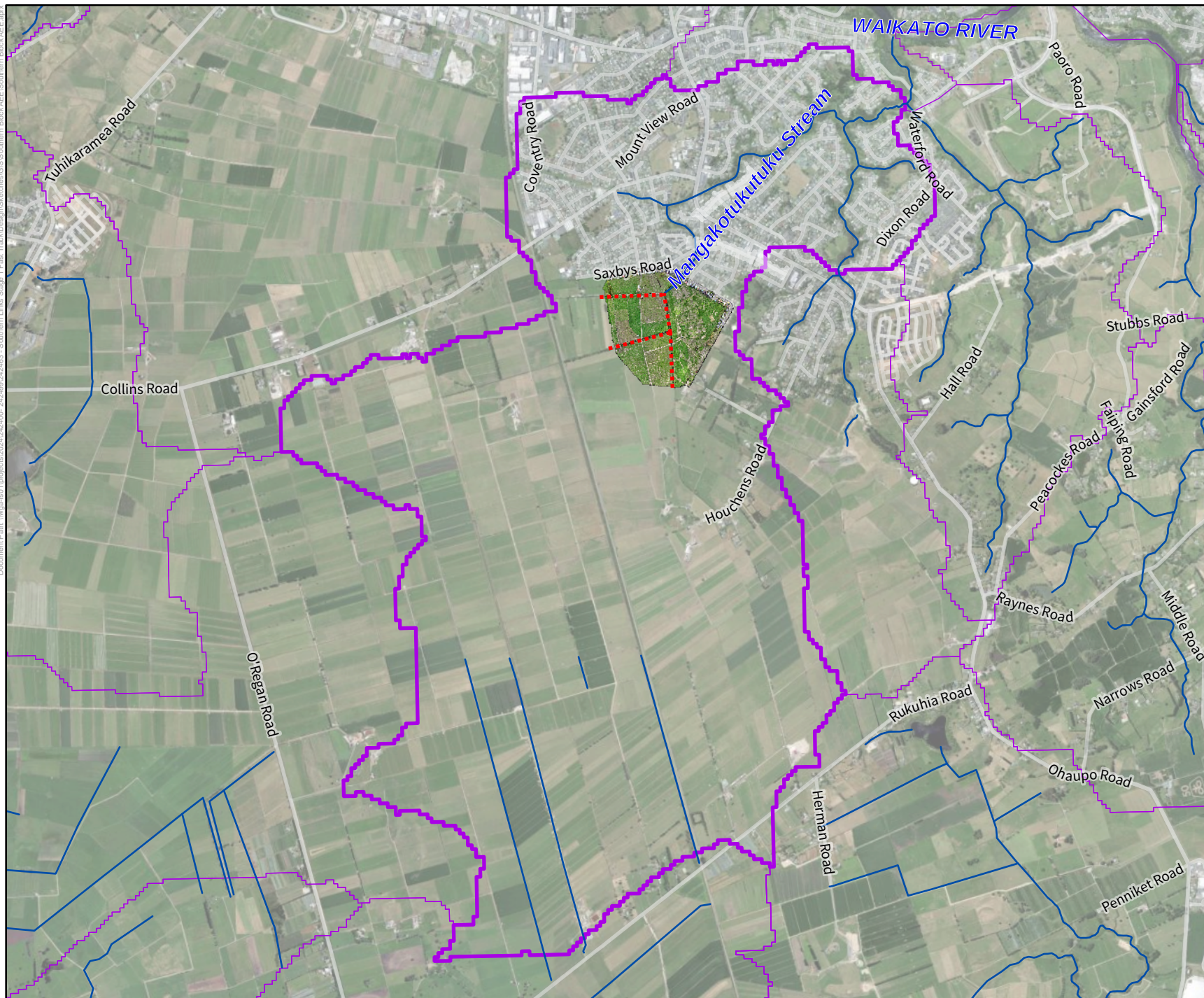


Figure 3: Rainfall Record at Ruakura (6.2 km northeast)

3.2 Surface Water Bodies and Catchment Boundary

The site is characterised by two converging drains (receiving water from a network of surrounding farmland drains). These two drains are approximately 1 to 2 m deep and have “moderate” ecological habitat (EcoLogical Solutions, 2025). The drains are flanked by riparian plantings, which are to be maintained during the construction period and as part of the proposed development. The drains flow into the headwaters of the Mangakōtukutuku Stream, which extends 150 m into the north-eastern end of the site (Figure 4). During project-specific field investigations, EcoLogical Solutions (2025) surveyed stream widths of between 2.6 to 4.2 m, with water depths ranging from 0.45 to 0.87 m and velocities of 0.1 to 0.9 m/s.



The site exists within the Mangakōtukutuku catchment, which is approximately 2,677 ha in area (AECOM, 2020). The catchment is bound to the north and east by the Peacocke Riverside catchment and to the west by the Waitawhiriwhiri catchment (AECOM, 2020). The Mangakōtukutuku stream discharges into the Waikato River approximately 2.5 km to the northeast of the site (Figure 4). Stormwater from the site will be discharged into planned artificial wetlands that will connect to the existing drains and the Mangakōtukutuku Stream.

Two existing natural wetlands were also identified within the site by EcoLogical Solutions Ltd (2025). One of these wetlands is located on the northeastern boundary of the site (approximately 285 m² in area) and the other is located in the upper reaches of the Mangakōtukutuku Stream (approximately 50 m²).

3.3 Geology and Hydrogeology

3.3.1 Regional Geology and Hydrogeology

The Hamilton Basin, in which the Residential Development is located, is a large tectonic basin centred on Hamilton City with an area of approximately 2,000 km² 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 6). 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.

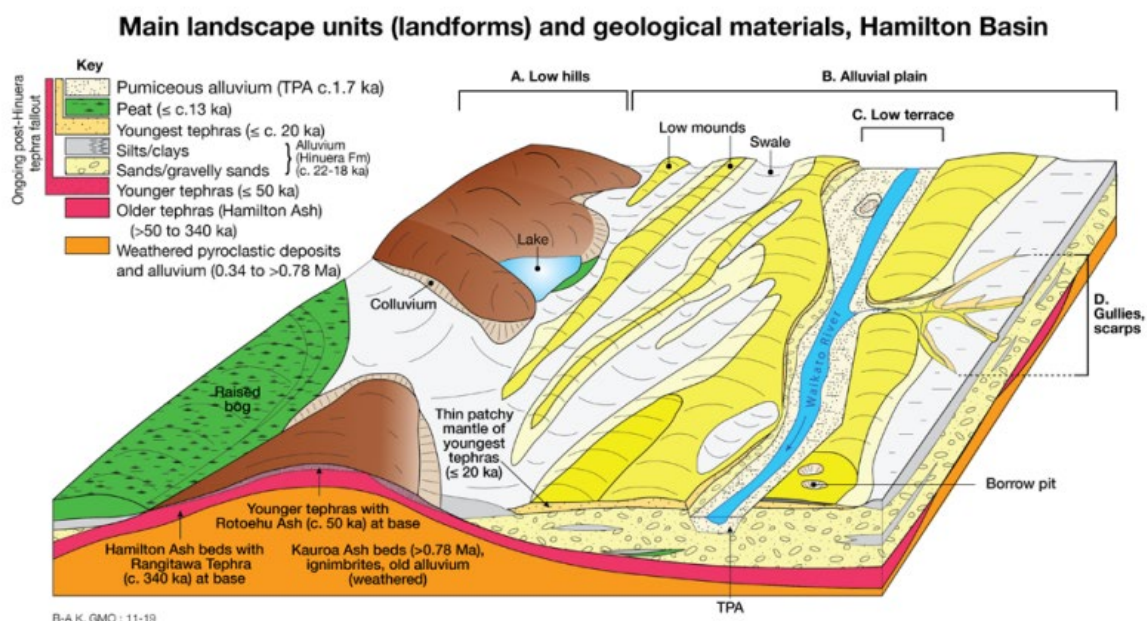
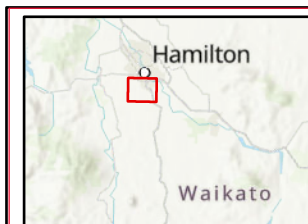
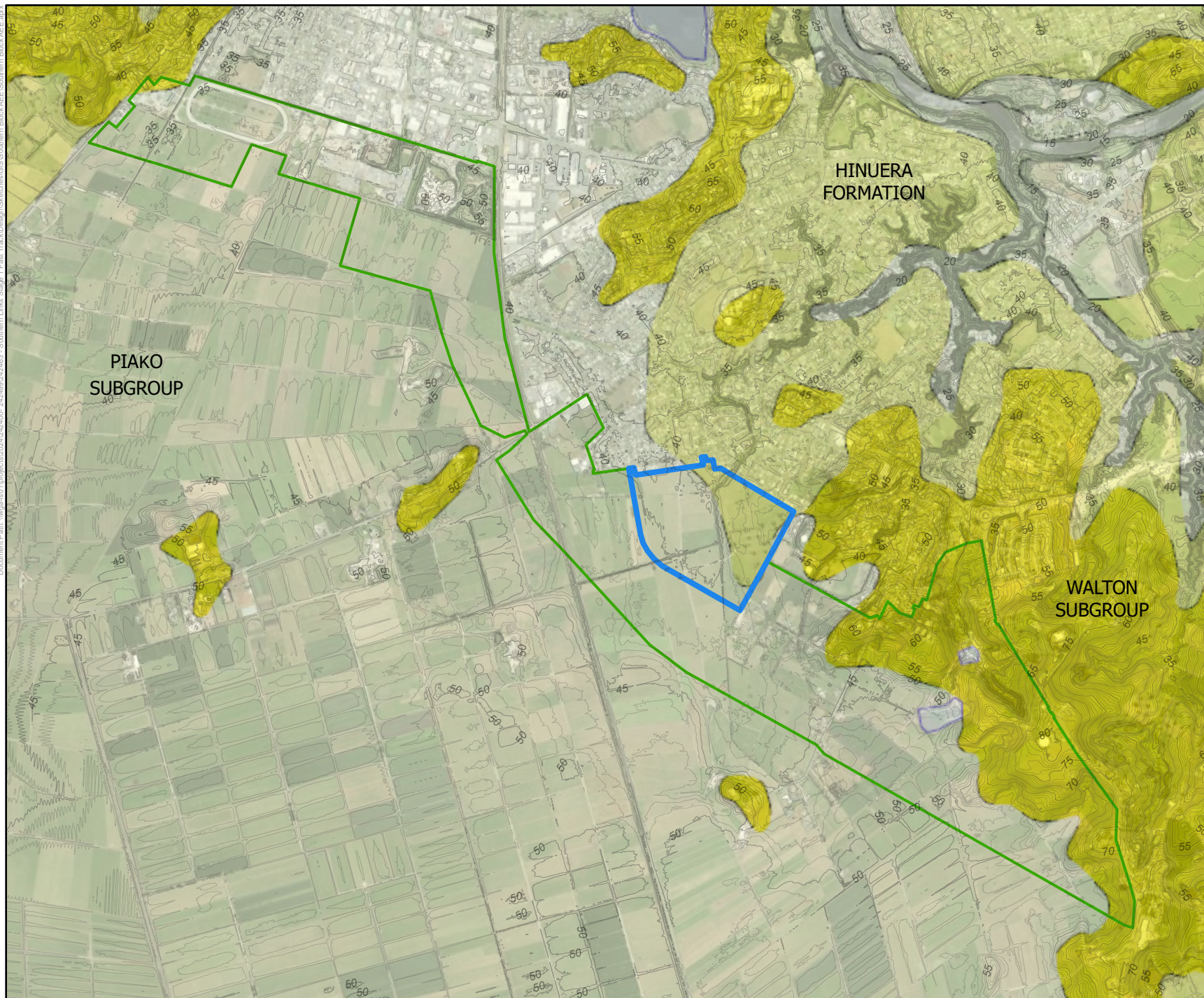


Figure 5: Geological Setting of the Hamilton Basin (Lowe, 2021)



LEGEND

- Residential Development
- Southern Links Growth Cell
- 1m Contours



Scale 1:30,000 @ A4

Coordinate System: NZGD 2000 Mount Eden Circuit

WGA

Figure 6

Southern Links Stage 1
Underlying Geological Units

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3.3.2 Local Shallow Lithology

The site is predominantly underlain by sediments of the Hinuera Formation. The Hinuera Formation consists of Late Pleistocene river deposits, including cross-bedded pumice sand, silt and gravel with interbedded peat. GNS Science Web Geology mapping (Figure 6) shows Piako Subgroup Holocene sediments are overlying the Hinuera Formation sediments at the site. However, based on the onsite observations of CMW (2026), the site is completely underlain by Hinuera Formation soils and the Piako Subgroup Holocene sediments (peat swamp deposits) are located further to the south-west, outside the Residential Area (CMW, 2026). A summary of the underlying geology encountered from the investigation is presented in Table 1.

Table 1: Stratigraphic Units of the Residential Area (CMW, 2026)

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 firm to very stiff Silts (Hinuera Formation)	0.1	0.5	1.9	18.0
Stiff to very stiff Clayey Silt (Hamilton Ash/Walton Subgroup) ⁽¹⁾	1.9	18.0	5.0 to 10.0 +	

Note:

⁽¹⁾ Base of unit not encountered.

3.3.3 Local Hydrogeology

In July 2025, WGA oversaw the installation of five piezometers at four locations at the site (Figure 7). An overview of piezometer as-built layouts is provided in Table 2. Piezometers P1, P3 and P4 were installed at a target depth of 6 m. Two piezometers were installed at the P2 location (Figure 8), the deeper one screened from 8.5 to 10 m below ground level (bgl; P2D) and the shallow piezometer was screened from 3.5 to 5 m bgl (P2S, Figure 7).

Drilling provided sediment core samples for each drilling location. Samples were logged on-site by CMW in order to have consistent results with other geological tests undertaken at the site. Comprehensive drill logs and core photos are provided in Appendix A.

Table 2: Piezometer Construction Details

PIEZOMETER	DEPTH (m bgl)	CASING DIAMETER (mm)	SCREEN LENGTH (m)	EASTING (NZTM)	NORTHING (NZTM)
P1	6.14	40	1.5	1800593	5811145
P2S	5.14	40	1.5	1800673	5811072
P2D	10.06	40	1.5	1800670	5811073
P3	6.02	40	1.5	1800589	5811047
P4	5.92	40	1.5	1800595	5810750

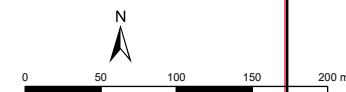


LEGEND

- Piezometric Surface
- Contour 1m
- Residential Development
- Southern Links Boundary

Subsurface Testing

- Cone Penetration Testing
- Hand Auger
- Piezometer
- Test Pit



Scale 1:5,000 @ A4

Coordinate System: NZGD 2000 Mount Eden Circuit

WGA

Figure 7

Southern Links Stage 1 Investigation Locations Piezometric Surface

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Figure 8: Twin Bore Headworks for P2S (Shallow) and P2D (Deep), Screened Above and Below the Low Permeability Aquitard Layer

Sites P1, P2 and P3 were all characterized by coarse sand, typical of the Hinuera Formation, with a thin clayey silt layer (referred to as a low permeability aquitard layer) appearing between 36 and 36.5 m RL.

Piezometer P4 was characterized by approximately 1 m of silt toward the top of the bore in addition to the low permeability aquitard layer at 37.4 m RL (metres Relative Level, or metres above mean sea level).

Across the four sites, the low permeability aquitard layer varied in thickness from 0.13 m to 0.45 m but appeared to be relatively persistent and to have an effect on the groundwater characteristics of the site. During drilling, after penetrating the low permeability aquitard layer, sand tended to rapidly flow into the borehole from below (called flowing sands by the drillers). This is evidence that the low permeability aquitard layer has a confining effect on the underlying deep aquifer, which is under some degree of pressure. The layer appears to be continuous across the site, appearing clearly in the majority of the geotechnical investigation sites. CMW have prepared a summary map illustrating this and presented in Appendix A.

WGA observed that sand layers above the aquitard had an orange-brown coloration while below the layer they tended to be entirely grey (Figure 9). This indicates that water levels above the low permeability aquitard layer fluctuate seasonally, while sediment below the low permeability aquitard layer is saturated year-round, providing further evidence to the confining effect of the aquitard.

Groundwater across the site is also likely strongly influenced by the two major drains (tributaries of Mangakōtukutuku Stream) running through the farm. From the observations taken during site visits these drains appear to have significant flow for the catchment size, indicating that they are at least partially groundwater fed. This is also supported by the brown discolouration of the drain, known locally to be caused by groundwater discharge and iron oxidation. In general, groundwater levels are likely drawn down to the water level in the incised drains.

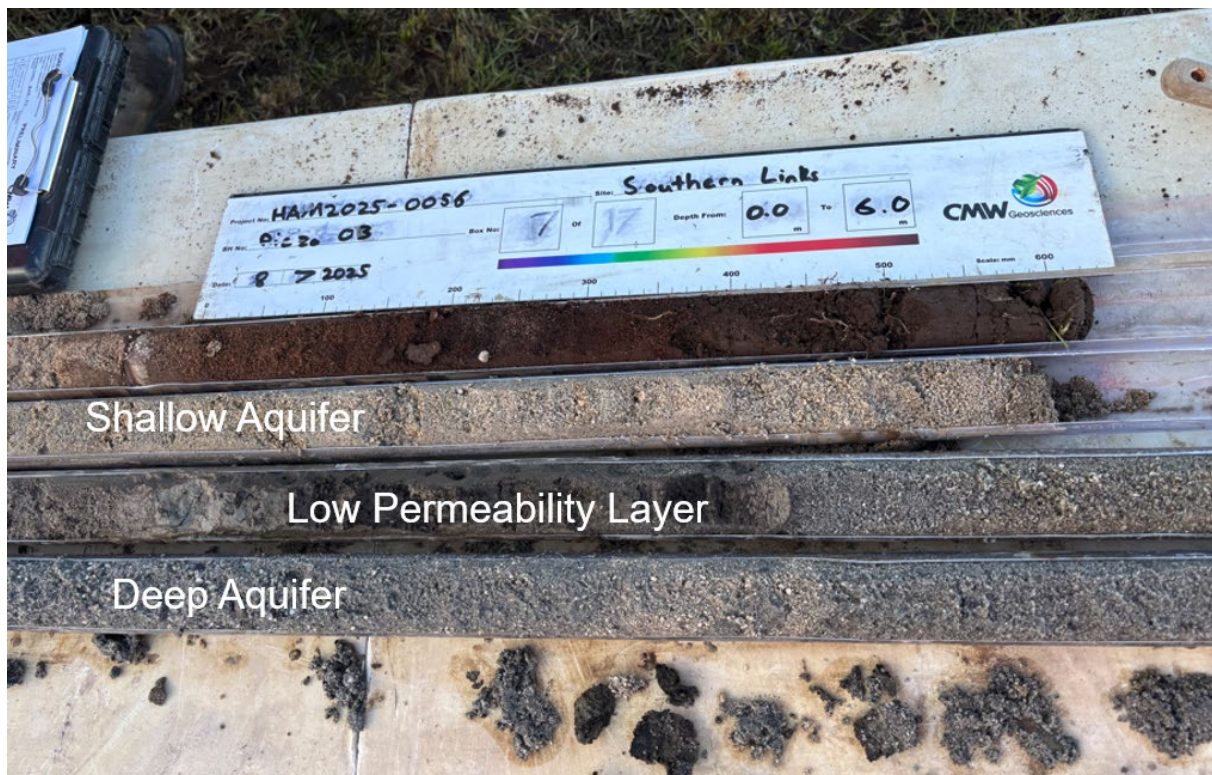


Figure 9: Field Identification of Lithological Units Present at Bore P3

3.4 Hydrogeological Parameters

The hydraulic conductivity of the geological layers found at the site has been determined by WGA. Hydraulic conductivities were determined by the analysis of hydraulic slug tests performed on-site. These analysis results were compared to the results from soakage testing undertaken by CMW, and decisions were made on appropriate hydraulic conductivity values for each of the sedimentary units listed in Table 3. For lithological units with no data available for hydraulic test analysis, appropriate literature values have been applied.

The hydraulic conductivity for the Hinuera Formation sediments found at the site has been determined by hydraulic slug tests performed in each of the five piezometers. Both falling head and rising head slug tests were analysed where data was available. Methods and results of each test are presented in Appendix B. The groundwater at each site reacted very rapidly, typically recovering within a few seconds of each test. For the Hinuera Formation screened beneath the low permeability aquitard layer, resulting hydraulic conductivities were on average 4×10^{-4} m/sec, which is considered a representative value for the site. This value is consistent with the hydraulic conductivity determined from pumping tests in the same aquifer, completed as part of hydrogeological investigations at a site 800 m north-west at Collins Road (Earthtech, 2025).

One of the five piezometers tested by WGA was screened above the low permeability aquitard layer. To consider the hydraulic conductivity of the Hinuera Formation above the low permeability aquitard layer, the hydraulic slug test analysis results were compared to the results from soakage tests undertaken by CMW. A decision was then made on an appropriate hydraulic conductivity value for above the low permeability aquitard layer of 2.5×10^{-5} .

WGA notes that due to the rapid recovery of the slug tests, resulting hydraulic conductivities should be treated as minimum values as they may represent the screen and filter pack characteristics rather than those of the target aquifer. Similarly, CMW notes that during soakage testing, water emptied from four soakage test locations faster than they could be filled, indicating the resulting permeabilities may be higher than recorded values (CMW, 2026a). However, the consistency of the results with recent hydrogeological studies conducted by Earthtech (2025) offers confidence in the validity of the findings.

A hydraulic conductivity range of $7.5 \times 10^{-6} \text{ m/s}$ to $2 \times 10^{-8} \text{ m/s}$ was determined for the low permeability aquitard layer characterised as organic silt with low plasticity. This range was produced from calibrated hydrogeological modelling at the nearby Collins Road investigation (Earthtech, 2025) and testing completed by WGA at the nearby Rotokauri Project (Table 3; WGA 2023). An assumed vertical hydraulic conductivity $4 \times 10^{-9} \text{ m/s}$ was adopted for modelling purposes.

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. Differences in the observed responses between the shallow and deep groundwater levels indicate that the deep aquifer has a direct connection with the rainfall influenced fluctuations in the nearby drain

WRC estimates that annual rainfall recharge (soil drainage) is 22% to 40% of rainfall in the Waikato region¹. 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.

Table 3: Hydraulic Conductivities of the Residential Area

MATERIAL NAME	HYDRAULIC CONDUCTIVITY RANGE (m/s)	HYDRAULIC CONDUCTIVITY ADOPTED FOR MODELLING (m/s)
Unsaturated zone Hinuera Formation ⁽¹⁾	$1.27 \times 10^{-6} - 1.43 \times 10^{-4}$	2.5×10^{-5}
Upper Interbedded loose to dense sands and firm to very stiff silts Hinuera Formation ⁽²⁾	$1.25 \times 10^{-4} - 1.83 \times 10^{-4}$	
Low permeability aquitard layer. Organic silt with low plasticity. Hinuera Formation ^(3,4,5)	$7.5 \times 10^{-6} \text{ (4)} - 2 \times 10^{-8} \text{ (3)}$	$K_h = 2 \times 10^{-8}$ $K_v = 4 \times 10^{-9} \text{ (5)}$
Lower Interbedded loose to dense sands and firm to very stiff silts Hinuera Formation ⁽²⁾	$8.75 \times 10^{-5} - 1.03 \times 10^{-3}$	4×10^{-4}

Notes:

(1) Range derived from CMW soakage tests (1 - 1.7 m deep) considered to underestimate the hydraulic conductivity of the unit (CMW, 2026a).

(2) Average derived from WGA aquifer hydraulic slug testing is considered to underestimate the hydraulic conductivity of the unit (Appendix B).

(3) Derived through separate investigations by WGA (2023). Assessed from the testing of unit 2b Hinuera Formation: Soft to firm SILT, some organics, low plasticity.

(4) Earthtech (2025) value used for hydrogeological modelling of the local confining unit.

(5) K_v (vertical hydraulic conductivity) = K_h (horizontal hydraulic conductivity)/5 = $2 \times 10^{-8}/5$.¹

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

3.5.2 Groundwater and Surface Water Flow Paths

Groundwater within the Mangakōtukutuku catchment is closely connected to surface water bodies, providing sustained flow to nearby drains, wetlands, and streams. In the upper catchment, which is characterised by rural land use and peat-rich soils, infiltrating rainfall that then flows to drains 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.

Groundwater levels have been measured and monitored by WGA and CMW at the Residential Development site. Site observations and an array of geotechnical investigations undertaken in the area since 2011 were used to estimate a preliminary winter piezometric surface (Figure 7). WGA notes that much 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's hydrogeological setting. Interpreted groundwater flow directions discharge to local drains and the headwaters of the Mangakōtukutuku Stream.

3.5.3 Groundwater Levels

Groundwater levels were recorded by WGA manually from the installed piezometers on 18 July 2025 (Table 4). Measurements from the deeper piezometers indicate the aquifer underlying the thin organic silt layer is confined, with recorded groundwater levels being above the low permeability aquitard described in Section 3.3.3. Water level measurements were made by CMW at numerous investigation sites in April 2025. Recorded groundwater level measurements ranged from 1.7 to 5.5 m bgl (refer to Table A1 in Appendix A).

Table 4: Manual Static Water Level Measurements in Each Piezometer (18 July 2025)

PIEZOMETER	STATIC WATER LEVEL (m bgl)	STATIC WATER LEVEL (m RL)	TOP OF AQUITARD LAYER (m RL)
P1	2.38	37.80	36.39
P2S	3.43	38.10	36.56
P2D	3.88	37.70	36.56
P3	2.07	37.90	36.32
P4	1.42	38.40	37.53

Downhole continuous groundwater level monitoring was installed in all five piezometers to record fluctuations between dry and wet periods from July 2025 to January 2026. The continuous monitoring also confirmed the confined nature of the aquifer beneath the low permeability aquitard layer (Figure 10; P1, P2D, P3 and P4), compared to the aquifer responses observed in the shallower P2S.

The shallower P2S aquifer shows only small water table fluctuations in response to rainfall. The strong groundwater level fluctuations exhibited in the deeper bores (Figure 10; P1, P2D, P3 and P4) indicate the confined aquifer is directly connected to the nearby drains. Water level responses in these drains to storm events cause similar responses in the confined aquifer, as recorded in the deep piezometers. This interpretation aligns with the observation that the drain is groundwater fed. Figure 11 shows the discolouration of the drain water due to oxidation of dissolved iron carried in groundwater discharges.

The maximum groundwater level height experienced during monitoring at Residential Area was 38.73 mRL in the southernmost piezometer, P4. During the summertime, levels dropped as low as 36.96 mRL in the northernmost piezometer, P1.

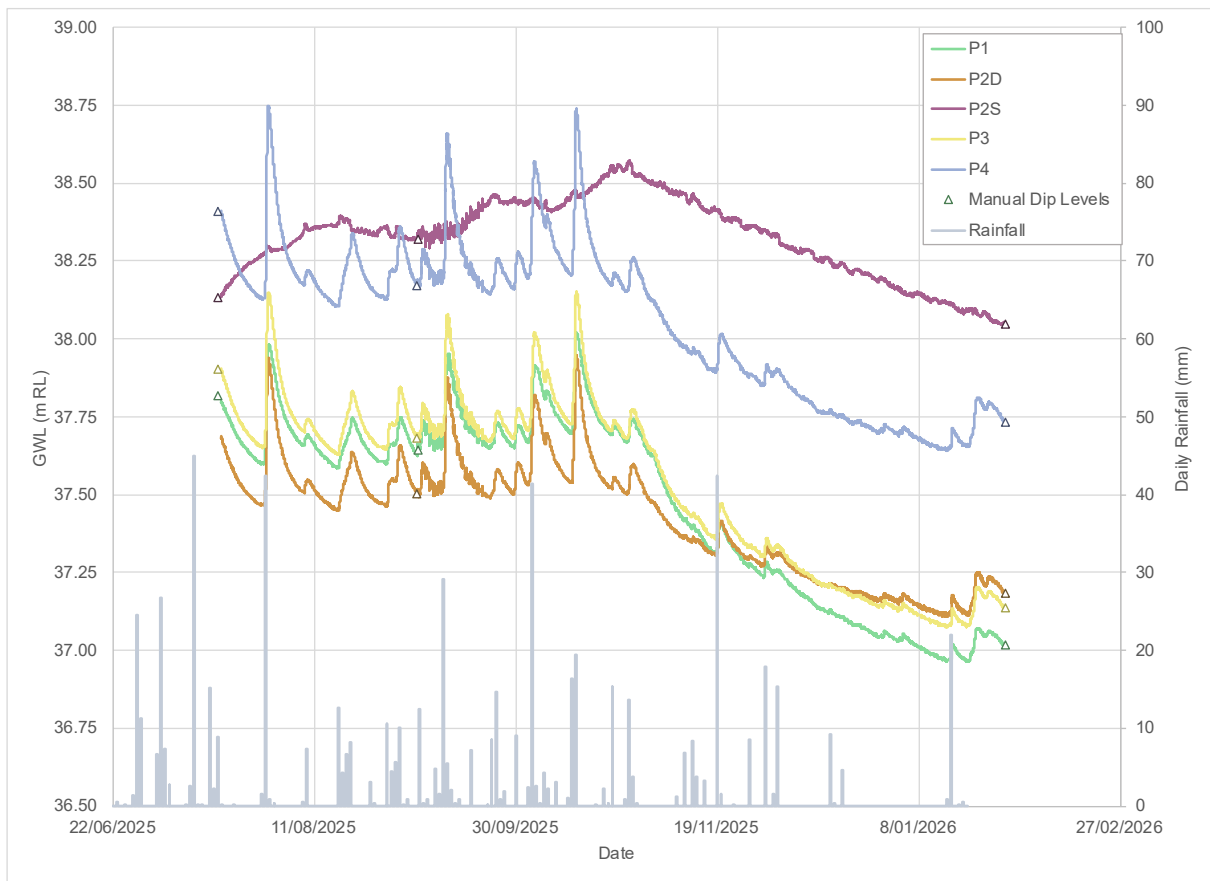


Figure 10: Residential Area Piezometer Groundwater Level Responses to Rainfall



Figure 11: Discolouration in Drain Due to Iron Oxidation from Groundwater Discharge

3.6 Surrounding Groundwater Users

Water supply bores (as listed in Appendix A) and groundwater abstraction consents within one kilometre of the Residential Development site are shown in Figure 12, with the consents summarised in Table 5. At the time of writing, according to the WRC consents database, there are four groundwater abstraction consents within one kilometre of the site. Of these, two are related to agricultural supplies. The remaining two are related to dewatering activities that are considered to be 'zero' net takes. The two consents related to agriculture are upgradient of the proposed Residential Development activities (south of the Residential Area).

Table 5: Nearby Groundwater Takes

CONSENT NUMBER	USER	PERMITTED VOLUME (m ³ /day)	PURPOSE
AUTH125828.01.01	Golden Valley Farm Limited	28	Agricultural farming - dairy
AUTH140970.01.01	The land Tahi Farm Group Limited AgVice Limited (Holders' Representative)	66	Agricultural farming - dairy
AUTH145510.02.01	Hamilton City Council AECOM New Zealand Limited (Auckland) (Holders' Representative)	0 (Zero net take)	Land development (Groundwater Diversion/ Dewatering)
AUTH147036.01.01	Hamilton City Council Potentialis Limited (Holders' Representative)	0 (Zero net take)	Waste treatment/disposal (Dewatering)

4 DRAWDOWN EFFECTS OF ARTIFICIAL WETLANDS

4.1 Introduction

The proposed Artificial Wetlands (Section 2.3) are designed to intersect the existing groundwater levels. WGA has modelled the drawdown effects of the proposed Artificial Wetlands using analytical methods, drawing on parameters from on-site testing (Section 3.4) and the Residential Development design provided by Maven Limited (Maven; Drawing no. R1-C440 Rev C). The analytical pit modelling was used to assess any drawdown impacts of the Artificial Wetlands extending to neighbouring properties and the drains of the Mangakōtukutuku stream headwaters.

WGA has depicted monitored high and low groundwater levels on Maven Drawing no. R1-C440 Rev C, included in Appendix C. Maven has verified that stormwater soakage is not required as a component of the wetland function. Furthermore, the operational “permanent” water levels in the wetlands are below the existing surrounding groundwater levels. Therefore, groundwater mounding is not expected to occur as a result of the Artificial Wetland operations.

The analytical assessments presented in this report are intended to evaluate groundwater inflows, dewatering requirements and groundwater drawdown effects. Assessment of the implications of artesian pressures on wetland stability and associated geotechnical design requirements is addressed separately by CMW within the geotechnical assessment (CMW 2026b and CMW 2026c).

4.2 Analytical Pit Modelling Methodology for Artificial Wetlands

An assessment of potential groundwater inflows to each of the planned Artificial Wetlands within Residential Development 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 wetlands are assumed to be hydraulically connected to the underlying groundwater system. The analytical pit modelling does not rely on the presence of the low-permeability aquitard to restrict groundwater inflows or drawdown and instead applies the representative hydraulic conductivity of the Hinuera Formation (4×10^{-4} 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 Artificial Wetlands, 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 wetland, including forebays and rain gardens, assuming the area is circular and calculating the radius of the “equivalent” area. The effective radii for the wetlands derived using this “equivalent” calculation are presented in Table 6.

Table 6: Inputs for Pit Models Representing Each of the Artificial Wetlands

ARTIFICIAL WETLAND NUMBER	EXISTING GROUND ELEVATION (mRL) ⁽¹⁾	EXISTING WINTER GROUNDWATER LEVEL (mRL) ⁽²⁾	DESIGN PERMANENT WATER ELEVATION (mRL) ⁽³⁾	DESIGN INVERT ELEVATION (mRL) ⁽¹⁾	DESIGN EXCAVATION DEPTH (m)	DESIGN AREA AT RIM FLOW ELEVATION (m ²) ⁽¹⁾	EFFECTIVE RADIUS AT OVERFLOW ELEVATION (m) ⁽⁴⁾
A	41.74	38.57	37.50	37.00	4.74	8,700	53
B	39.95	38.57	37.40	36.90	2.69	4,200	37
C	39.74	38.57	37.40	37.35	2.39	6,700	46
D	39.74	38.75	37.95	37.48	2.26	5,000	40
E	41.02	38.75	37.90	37.46	3.56	4,350	37
F	40.24	39.20	38.80	38.40	1.84	3,400	33

Notes:

(1) Existing and deepest design levels provided by Maven (Appendix C).

(2) 38.57 mRL (P2S, 28/10/2025). 38.75 mRL (P4, 30/07/2025). 39.20 mRL (Level recorded in CMW CPT).

(3) As provided by Maven (Drawing: C440 SW Wetland).

(4) Assuming ponded area is treated as a circle.

For long-term scenarios, the Artificial Wetlands have been modelled with an operational 'pit lake' representing the permanent water level. To assess inflows during the construction phase, the base of the Artificial Wetland has been modelled as dry. Both models used surrounding winter groundwater levels as the starting conditions to identify peak inflow rates. For those wetlands near existing housing or buried infrastructure, an additional scenario has been run using summer groundwater levels as the starting conditions. The drawdowns derived from this summer drawdown assessment are intended to inform ground settlement calculations.

4.3 Results

4.3.1 Drawdown Effects of Artificial Wetlands on Groundwater

The projected drawdown effects have been modelled for each of the Artificial Wetlands, (as detailed in Appendix D) and the results summarised in Table 7. The groundwater inflows (groundwater abstraction during construction) at each of the Artificial Wetlands have been calculated assuming the drawdowns generated by each of the wetlands do not intersect during individual construction. The radius of groundwater drawdown influence and the groundwater inflows are presented separately for construction (dry pit) and for long-term (operational permanent water level) effects in Table 7.

Maps identifying long-term Artificial Wetland effects on groundwater drawdown contours of the operational Artificial Wetlands are provided in Appendix E, with the 'de-minimis' contour being defined as 0.25 m drawdown. Drawdown below that magnitude is considered to be effectively indistinguishable from natural fluctuations. Drawdown assessments were conducted using winter groundwater levels to provide a conservative estimate of inflows. Drawdown contours do not intersect hydraulic boundaries such as the farm drains and streams, with the exception of the drain currently between proposed Artificial Wetland B and C, which WGA understands will be removed. This drain will be filled and existing flows will be diverted via a pipe (Maven Drawing no. C400-1 for plan view and C420-37 for long sections).

In addition, summer groundwater level scenarios were simulated for Artificial Wetlands A and B to demonstrate how seasonality affects drawdown results. These Artificial Wetlands were chosen for the summer scenarios because they are located closest to existing residences. For Artificial Wetland A, drawdown impacts at the nearest building reduce from 1.06 m in winter to 0.45 m in summer. Wetland B shows a similar pattern, with drawdown impacts at the nearest building reducing from 0.91 m in winter to 0.31 m in summer. The accompanying drawdown contour figures (Appendix E) illustrate this difference. The larger drawdown beneath housing in winter includes approximately 0.6 m attributed to the existing seasonal variance in natural groundwater levels. The accompanying Geotechnical Assessment (CMW 2026c) has used these results to undertake ground settlement assessments.

4.3.2 Drawdown Effects of Artificial Wetlands on Surface Water

Artificial Wetlands D and E are expected to temporarily cause stream depletion effects during construction due to their design base levels being lower than the surveyed bed level of the adjacent farm drain (Table 8). Groundwater that would otherwise discharge to the drain will be captured by construction dewatering. To manage this effect, groundwater intercepted during construction dewatering shall be discharged to the receiving drain upstream of the works, as proposed through the Residential Development consent conditions, to maintain surface water flows and mitigate temporary stream depletion effects.

Interception of shallow groundwater contributions to the neighbouring drains/stream baseflow is also anticipated during construction of the remaining Artificial Wetlands; however, to a lesser extent, because the other Artificial Wetlands do not extend deeper than the drains/stream (Table 8). Any intercepted groundwater from these works can likewise be returned to the local drainage network or stream.

Table 7: Projected Effects of Proposed Artificial Wetlands

ARTIFICIAL WETLAND	SCENARIO	INITIAL SATURATED THICKNESS ABOVE BASIN BASE (m)	RADIUS OF DRAWDOWN INFLUENCE (m) ^(1,2)	CALCULATED GROUNDWATER INFLOW (m ³ /day) ⁽²⁾	DRAWDOWN AT NEAREST EXISTING BUILDING (m)	DISTANCE TO NEAREST EXISTING BUILDING (m)
Wetland A	Winter Construction Dry	1.57	90	5,361	1.06	8
	Summer Construction Dry	1.04	15	3,434	0.45	
	Winter Permanent Level	1.57	80	3,703	0.87	
	Summer Permanent Level	1.04	10	1,789	0.30	
Wetland B	Winter Construction Dry	1.67	89	4,000	0.91	15
	Summer Construction Dry	1.14	17	2,617	0.31	
	Winter Permanent Level	1.67	80	2,876	0.78	
	Summer Permanent Level	1.14	12	1,474	0.22	
Wetland C	Winter Construction Dry	1.22	60	3,648	0.05	133
	Winter Permanent Level		59	3,504	0.05	
Wetland D	Winter Construction Dry	1.27	60	3,284	0.00	200
	Winter Permanent Level		49	2,100	0.00	
Wetland E	Winter Construction Dry	1.25	57	3,018	0.00	250
	Winter Permanent Level		49	2,080	0.00	
Wetland F	Winter Construction Dry	0.80	25	1,702	0.00	420
	Winter Permanent Level		12	869	0.00	

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 8: Existing and Design Elevations of Artificial Wetlands Compared to Neighbouring Stream/Drain Heights

ARTIFICIAL WETLAND	EXISTING GROUND ELEVATION (mRL) ⁽¹⁾	DESIGN INVERT ELEVATION (m RL) ⁽¹⁾	DESIGN EXCAVATION DEPTH (m)	DESIGN PERMANENT WATER ELEVATION (mRL) ⁽²⁾	STREAM/DRAIN INVERT (mRL)	DEPTH FROM DESIGN INVERT TO STREAM/DRAIN INVERT (m)
A	41.74	37.00	4.74	37.50	36.67	0.33
B	39.95	36.90	2.69	37.40	35.85	1.05
C	39.74	37.35	2.39	37.40	36.67	0.68
D	39.74	37.48	2.26	37.95	37.70	-0.22
E	41.02	37.46	3.56	37.90	37.70	-0.20
F	40.24	38.40	1.84	38.80	37.83	0.57

5 DRAWDOWN EFFECTS OF WASTEWATER INFRASTRUCTURE

5.1 Introduction

WGA has modelled the drawdown effect of the dewatering required for the construction of two proposed wastewater pump stations and the associated wastewater pipeline network. The modelling used hydrogeological parameters from on-site testing (Section 3.4) and the design provided by Maven (Drawing no. R1-C440 Rev C).

Dewatering is not anticipated to be required in association with the installation of the proposed water supply pipeline, due to its depth being above the groundwater level (Maven Drawing no. C340 TYP RD Section). If extraordinary wet conditions require dewatering, any dewatering effects will be less than those required and assessed in Section 5.3 for wastewater pipelines.

5.2 Drawdown Effects of Wastewater Pump Station Installation

5.2.1 Construction

Two wastewater pumping stations (WWPS) are planned for construction: “WWPS A” located on the northern boundary and “WWPS B” positioned centrally within the development (Figure 2). The engineering design team has provided an outline of the proposed WWPS designs (Appendix F).

Earthworks associated with the wastewater pumping stations involve excavations below the groundwater table and beneath the confining aquifer's low permeability aquitard layer. The wet wells' depth necessitates specialised construction techniques to minimize excavation size and the groundwater dewatering required. In accordance with best practice, the contractor will implement erosion and sediment control measures as stipulated by the Waikato Regional Standard TR2009/02, as required by the proposed consent conditions.

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.

Due to the position of the low permeability aquitard layer within the wet well excavation, this determines that the disconnected shallow aquifer and deep aquifer are both required to be dewatered. To achieve this, a combination of top-down and bottom-up dewatering methods is anticipated. Assessments assume the use of a sheet pile wall to support each excavation. A dewatering bore situated outside the sheet pile wall is to be employed to facilitate dewatering beneath the low permeability aquitard layer. Additionally, a surface water pump may be utilized during earthworks to temporarily lower shallow groundwater levels, where necessary, ensuring safe and efficient excavation above the low permeability aquitard layer. The sheet pile wall is expected to reduce horizontal groundwater inflows from the shallower aquifer.

Groundwater dewatering is not required for earthworks associated with the pump station pipework, as these excavations are above the groundwater table.

5.2.2 Dewatering for Wet Well Installation

As described above, excavation for the installation of each wet well is expected with the following dimensions:

- **WWPS A** will require a 2.5 m width and 6.6 m depth for wet well installation beneath the aquitard.
- **WWPS B** will require a 2.5 m width and 6.2 m depth for wet well installation beneath the aquitard.
- Any further excavations required for WWPS infrastructure are not anticipated to extend beneath the aquitard with the design information provided (Appendix F). Excavations for WWPS infrastructure can be dewatered with similar volumes as the trenching dewatering excavations described in Section 5.3.

It is assumed that the excavation will be completed and the walls of the wet well sealed within a maximum of 10 days. The wet well excavation will intersect the confined aquifer. An assessment applying the Hunt and Scott (2007) solution for pumped drawdown in a two-aquifer system has been used to assess the drawdown effects of the excavation.

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. A number of hydraulic tests and drillhole logs are available in the area, and aquifer transmissivities have been derived from hydraulic conductivity values from on-site testing (Section 3.4). Table 9 presents the hydraulic parameters applied to each drawdown assessment. A period of 15 days of dewatering was modelled to allow for delays over the planned duration of the works.

Table 9: Input Parameters for Hunt and Scott Assessment of Dewatering WWPS A and WWPS B

INPUT PARAMETER	WWPS A	WWPS B
Transmissivity in pumped confined aquifer	192 m ² /day Based on an aquifer thickness of 5.55 m at WWPS A and K value of 34.56 m/day (Table 3)	501 m ² /day Based on an aquifer thickness of 14.49 m at WWPS B and K value of 34.56 m/day (Table 3)
Storativity of pumped aquifer	0.001 Typical of a semi-confined aquifer in the Hinuera Formation	
Transmissivity in overlying aquifer	4.71 m ² /day Based on an aquifer thickness of 2.18 m and a K value of 2.16 m/day (Table 3)	1.0152 m ² /day Based on an aquifer thickness of 0.47 m and a K value of 2.16 m/day (Table 3)
Specific yield in overlying aquifer	0.2 Value for sandy material as described in the Castlegrace Farms bore log (Heath, 1983).	
Aquitard thickness	0.2 m (thickness of layer in piezometer P1, Appendix A).	0.2 m (thickness of layer in piezometer P4, Appendix A).
Vertical hydraulic conductivity for the aquitard	3.46 x 10 ⁻⁴ m/day (Table 3)	
Starting groundwater level	3.82 m bgl (38.02 mRL from P1)	1.05 m bgl (38.74 mRL from P4)
Target drawdown required to dewater 0.5 m below excavation	3.28 m	5.65 m

5.2.2.1 Deep Dewatering Results

Results of the Hunt and Scott analysis are shown in Figure 13 and Figure 14.

The WWPS A analysis indicates that pumping from a bore at a rate of 10.5 L/s (907 m³/day) will achieve a drawdown in the confined aquifer of 3.28 m at a distance of 5 m from the pumped bore. Provided the bore is positioned alongside the wet well, this drawdown would enable the excavation to be undertaken while maintaining a dry floor (Table 9). After 15 days pumping at this rate, the area of influence (drawdown of 0.25 m) extends 412 m from the pumping bore within the confined aquifer. Drawdown in the overlying unconfined aquifer reaches 0.25 m at 19 m from the pumped bore.

The WWPS B analysis indicates that pumping from a bore at a rate of 42.4 L/s (3,661 m³/day) will achieve a drawdown in the confined aquifer of 5.65 m at a distance of 5 m from the pumped bore. Provided the bore is positioned alongside the wet well, this drawdown would enable the excavation to be undertaken while maintaining a dry floor (Table 9). After 15 days pumping at this rate the area of influence (drawdown of 0.25 m) extends 854 m from the pumping bore in the confined aquifer and 107 m in the overlying unconfined aquifer.

WGA notes that the aquifer described as 'confined' is technically 'semi-confined' due to the confining layer being leaky. However, for simplicity, the lower aquifer is described in this report as confined.

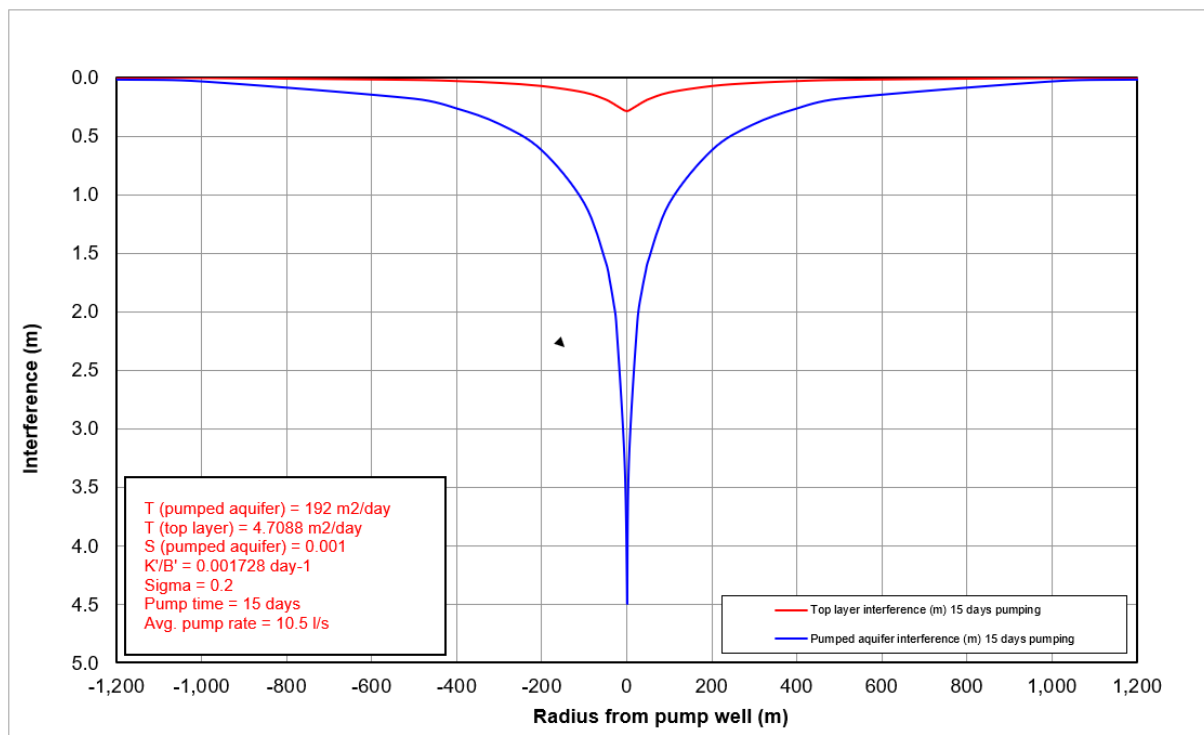


Figure 13: Projected Drawdown in confined and unconfined Aquifers Following Dewatering of Wet Well Excavation WWPS A

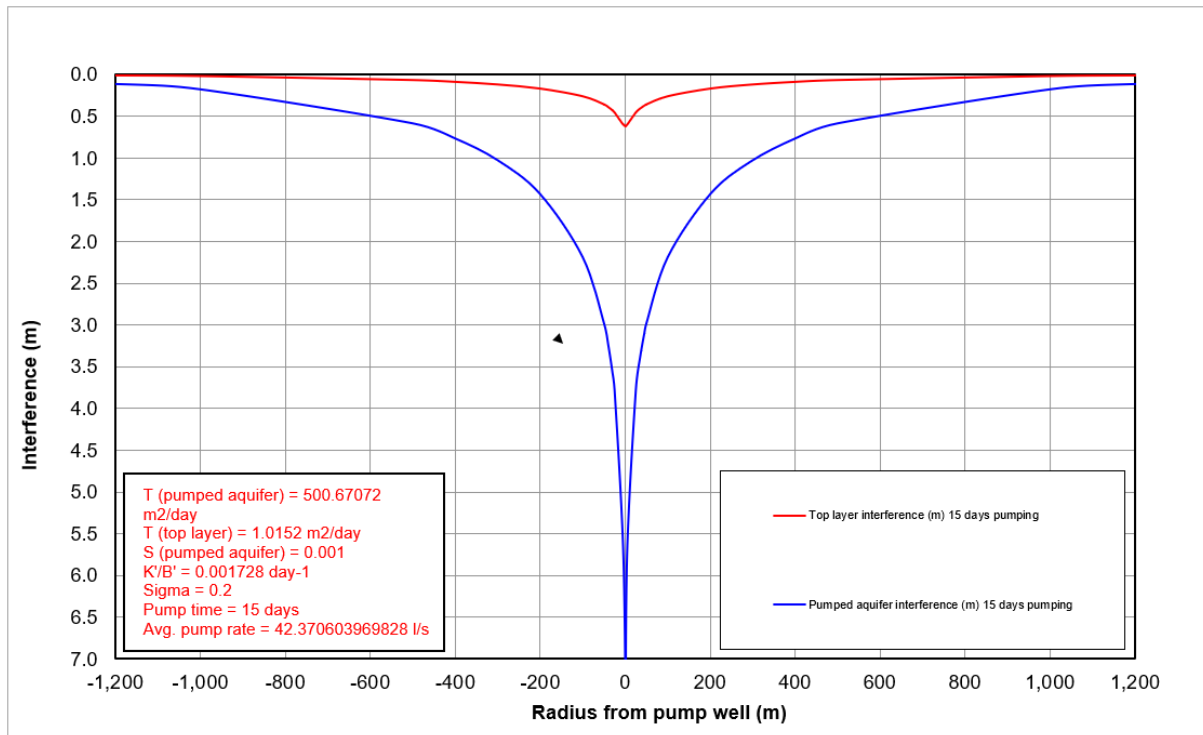


Figure 14: Projected Drawdown in confined and unconfined Aquifers Following Dewatering of Wet Well Excavation WWPS B

5.2.3 Shallow Aquifer Dewatering During Wet Well Excavation

If construction of the wet well is undertaken using a technique that requires dewatering of the unconfined aquifer until the excavation reaches the confining layer, temporary dewatering of the aquifer may be required to support the excavation process. This dewatering would result in a direct drawdown of the groundwater table. For completeness, an assessment has been undertaken to estimate how much water would flow into this excavation and therefore, require pumping. The extent of the consequent drawdown in the unconfined aquifer has been estimated from these assessments. The modelling has a conservative assumption that the dewatering method leads to floor heave within the excavation and consequent unhindered flows upward from the underlying confined aquifer.

The shallower excavation dewatering has been assessed using the published method of estimating groundwater inflows to open-cut pits (Marinelli & Niccoli 2000), as was applied to the assessment of the Artificial Wetland dewatering in Section 4. The pit model accounts for two zones of inflow, the upper zone being Zone 1 and the lower being Zone 2, with a no flow boundary between them at a level equivalent to the position of the confining layer. The hydraulic conductivity for the Upper Hinuera Formation has been applied to Zone 1 and Lower Hinuera Formation to Zone 2 (as described with modelling parameters in Table 3). The input parameters for the pit models are shown in Figure 15 and Figure 16. The highest recorded localised groundwater level was used for the assessment.

The pit dewatering model applied in this assessment functions on a steady-state basis, which means the equations used apply for permanent dewatering. As the wet well construction process is expected to require only a few days, the extent of the zone of influence calculated using this methodology is conservatively large.

The pit model assumes Zone 2 is infinite in depth. However, according to lithological logs from local bores, this aquifer is between 5.5 m and 14.5 m thick (for WWPS A and WWPS B, respectively). Therefore, inflows from Zone 2 may be overestimated in the modelling.

The results of the pit dewatering modelling are summarised in Table 10.

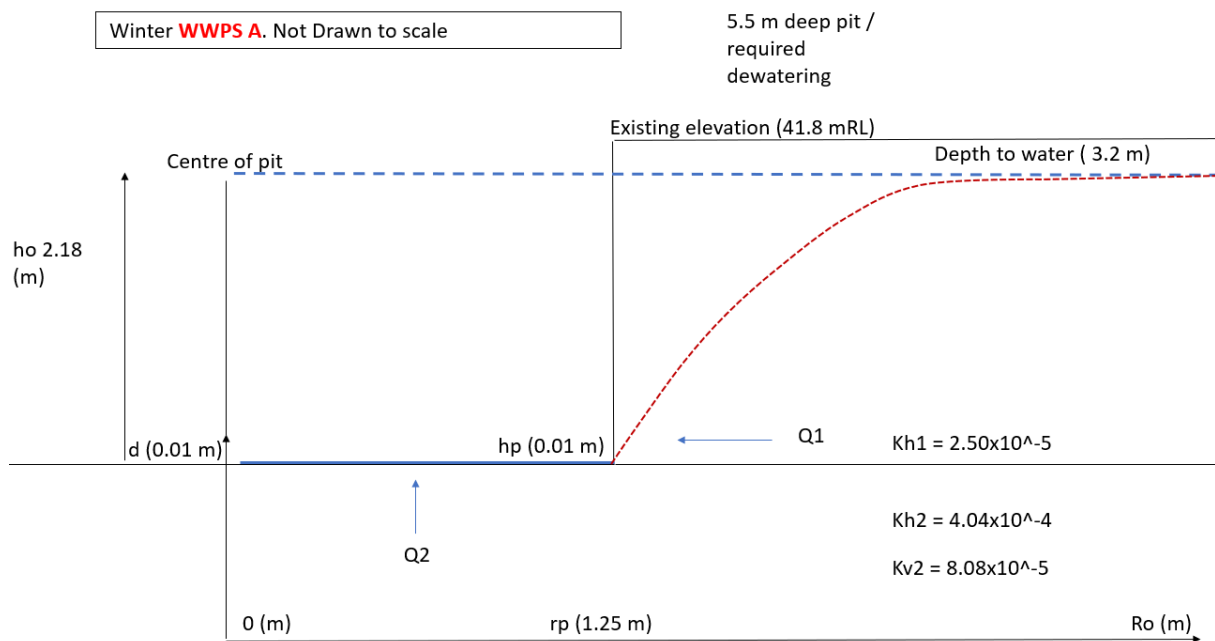


Figure 15: Inputs for Pit Model Assessing Potential Dewatering of Unconfined Aquifer of Planned WWPS A Installation

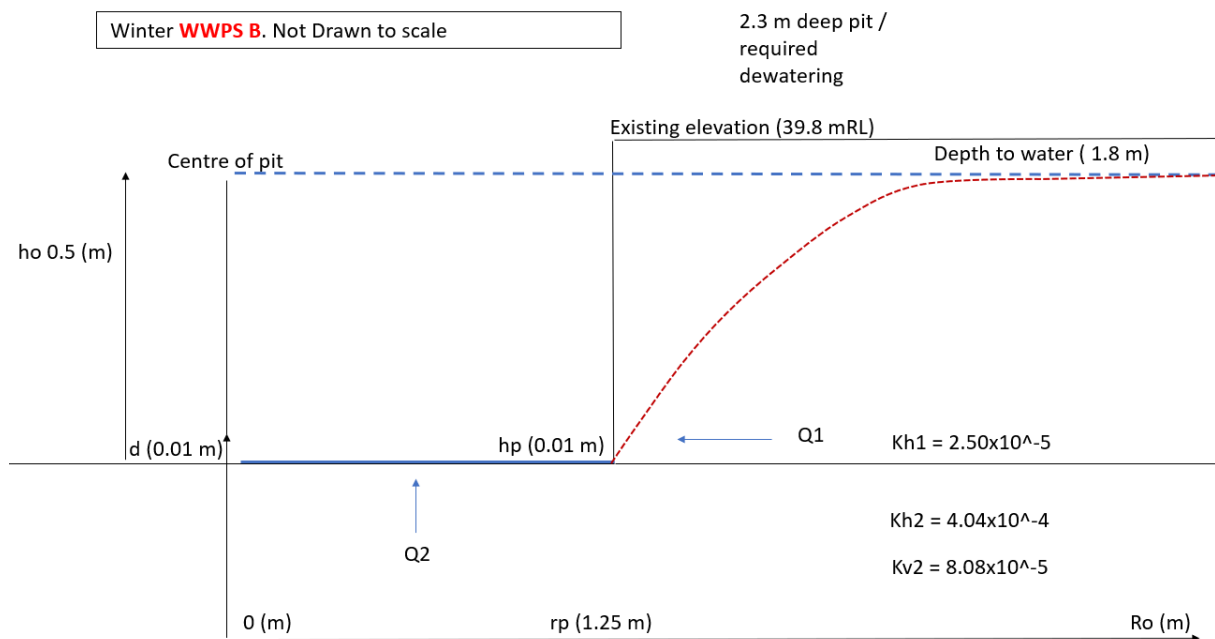


Figure 16: Inputs for Pit Model Assessing Potential Dewatering of Unconfined Aquifer of Planned WWPS B Installation

Table 10: Projected Effects of Wet Well Excavation Through Upper Aquifer Only

SITE	DEPTH TO GROUNDWATER (m)	INITIAL SATURATED THICKNESS ABOVE BASIN BASE (m)	RADIUS OF INFLUENCE (m) ⁽¹⁾	CALCULATED GROUNDWATER INFLOW (m ³ /day)	DISTANCE TO NEAREST BUILDING (m)	DRAWDOWN AT NEAREST BUILDING (m)
WWPS A	3.2	2.18	20	179	8	0.59
WWPS B	1.8	0.51	2.2	40	15	0

Notes:

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

5.3 Drawdown Effects of Wastewater Pipeline Trenching

5.3.1 Wastewater Pipeline Modelling Methodology

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 completed within a period of 10 days. This has been used as the basis of the groundwater drawdown and inflow modelling. The trench excavation depths have been provided by Maven to WGA (version 16 March 2026).

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

WGA understands that the wastewater pipelines will be constructed at varying depths throughout the Residential Development. As hydraulic conductivity varies with depth at the Residential Development site, WGA has completed two worst case scenarios for consideration of drawdown effects, in order to provide a conservative assessment:

1. A section of 20 m pipeline trench on the northern residential boundary of the site, requiring temporary dewatering in the upper Hinuera formation, above the low permeability aquitard layer.
2. A section of 20 m pipeline in one of the deepest pipeline trenches located centrally in the site, requiring temporary dewatering in the lower Hinuera formation, below the low permeability aquitard layer.

The differing hydraulic conductivities for the upper and lower Hinuera Formations (Section 3.4) are applied to reflect the different dewatering aquifer depths of the wastewater pipeline trench (Table 11).

For these models, WGA has assumed that the trenching for the wastewater pipeline will occur during the summer earthworks season. WGA understands that settlement effects occur when groundwater draws down below normal seasonal fluctuation ranges. Therefore, summer drawdown is considered the most relevant for the geotechnical assessment. Tabulated drawdown scenarios and dewatering rates for winter have been provided for comparison.

Table 11: Input Parameters for Wastewater Pipeline Analytical Modelling

WASTEWATER PIPELINE	HYDRAULIC CONDUCTIVITY (m/s)	AQUIFER SPECIFIC YIELD	NUMBER OF DAYS TRENCH OPEN	RECHARGE (mm/year)	TRENCH DRAWDOWN REQUIRED (m)
Wastewater Pipelines on the Edges of the Residential Development Summer	2.5×10^{-5}	0.25	10	250	1.44
Wastewater Pipelines at the Centre of the Residential Development Summer	4.04×10^{-4}	0.25	10	250	1.27
Wastewater Pipelines on the Edges of the Residential Development Winter	2.5×10^{-5}	0.25	10	250	1.97
Wastewater Pipelines at the Centre of the Residential Development Winter	4.04×10^{-4}	0.25	10	250	2.38

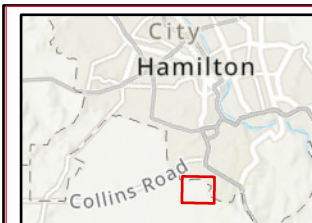
5.3.2 Wastewater Pipeline Analytical Modelling Results

Projected drawdown from the wastewater pipeline trenches is summarised in Table 12, Figure 18 and Figure 19. Projected groundwater inflows are shown in Figure 20 and Figure 21.

Table 12: Projected Distance of Influence

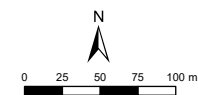
WASTEWATER PIPELINE	DISTANCE OF INFLUENCE (m)
Wastewater Pipelines on the Edges of the Residential Development Summer	73
Wastewater Pipelines at the Centre of the Residential Development Summer	250
Wastewater Pipelines on the Edges of the Residential Development Winter	83
Wastewater Pipelines at the Centre of the Residential Development Winter	330

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



LEGEND

- WWPS
- Proposed Wastewater Infrastructure
- Residential Development
- Southern Links Boundary
- 20m Trench Sections
- Summer Drawdown Contours



Scale 1:5,000 @ A4

Coordinate System: NZGD 2000 Mount Eden Circuit

WGA

Figure 17

Southern Links Stage 1
Summer Drawdown
Contours for Wastewater
Pipeline 20m Trenches

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 reliance placed on such information shall be at the risk of the user.

Note: The information shown on this map is a copyright of WGA 2025

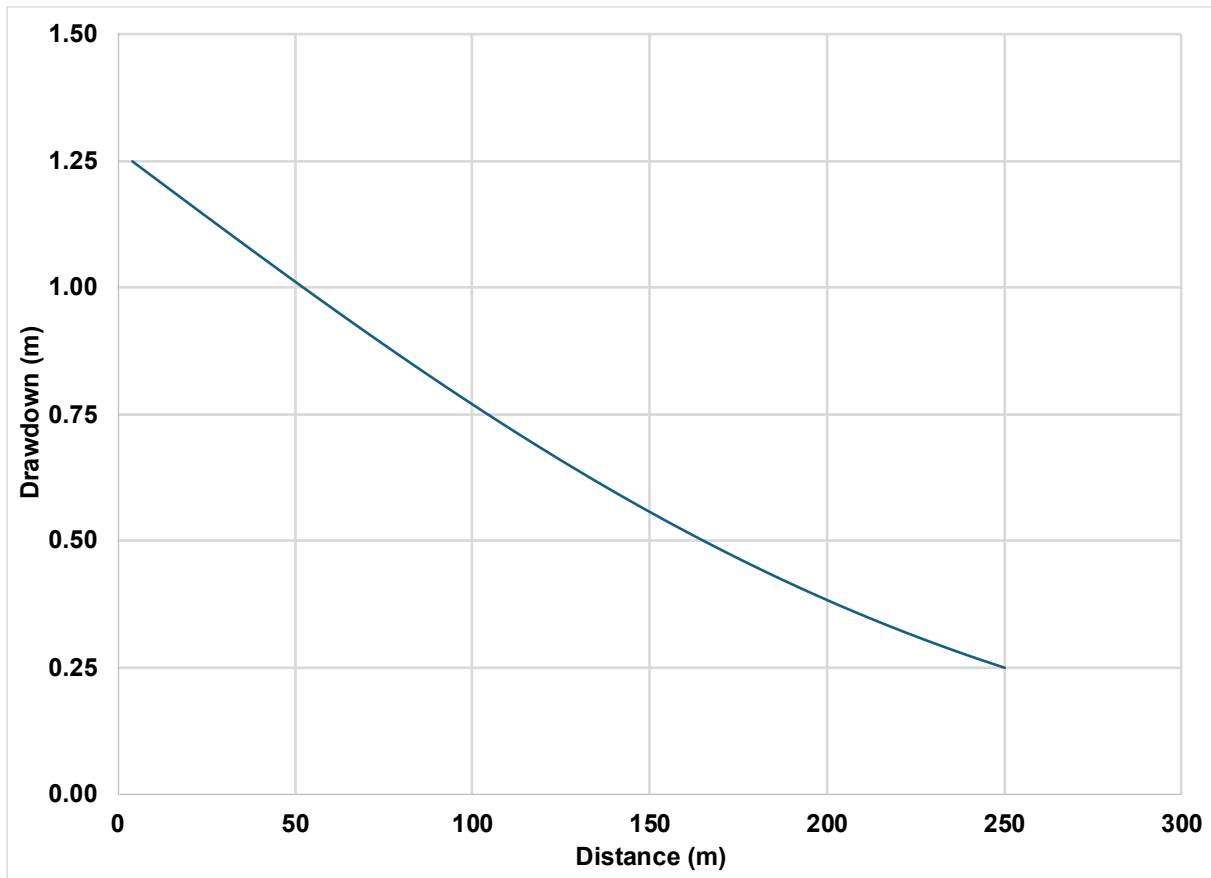


Figure 18: Projected Drawdown from Wastewater Pipelines at the Centre of the Residential Development During Summer

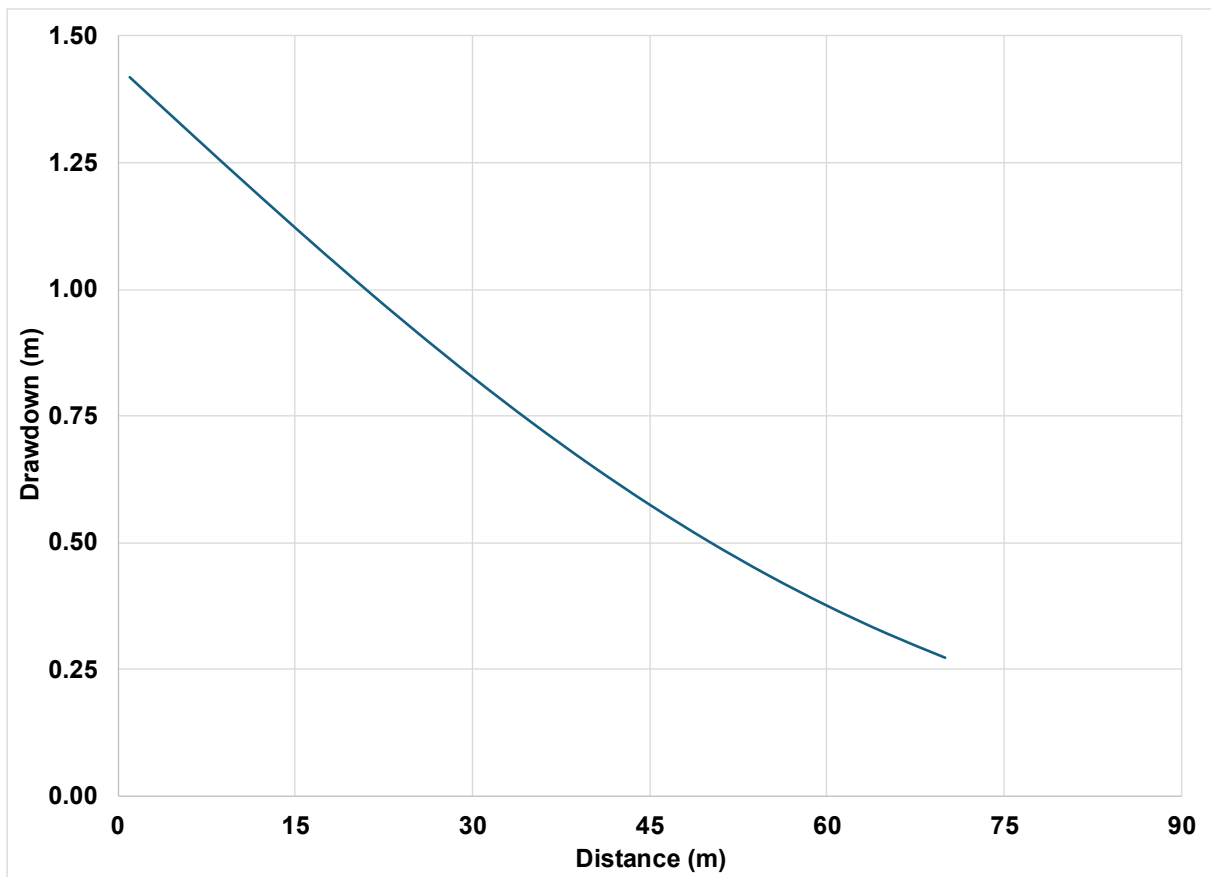


Figure 19: Projected Drawdown from Wastewater Pipelines at the Edges of the Residential Development During Summer

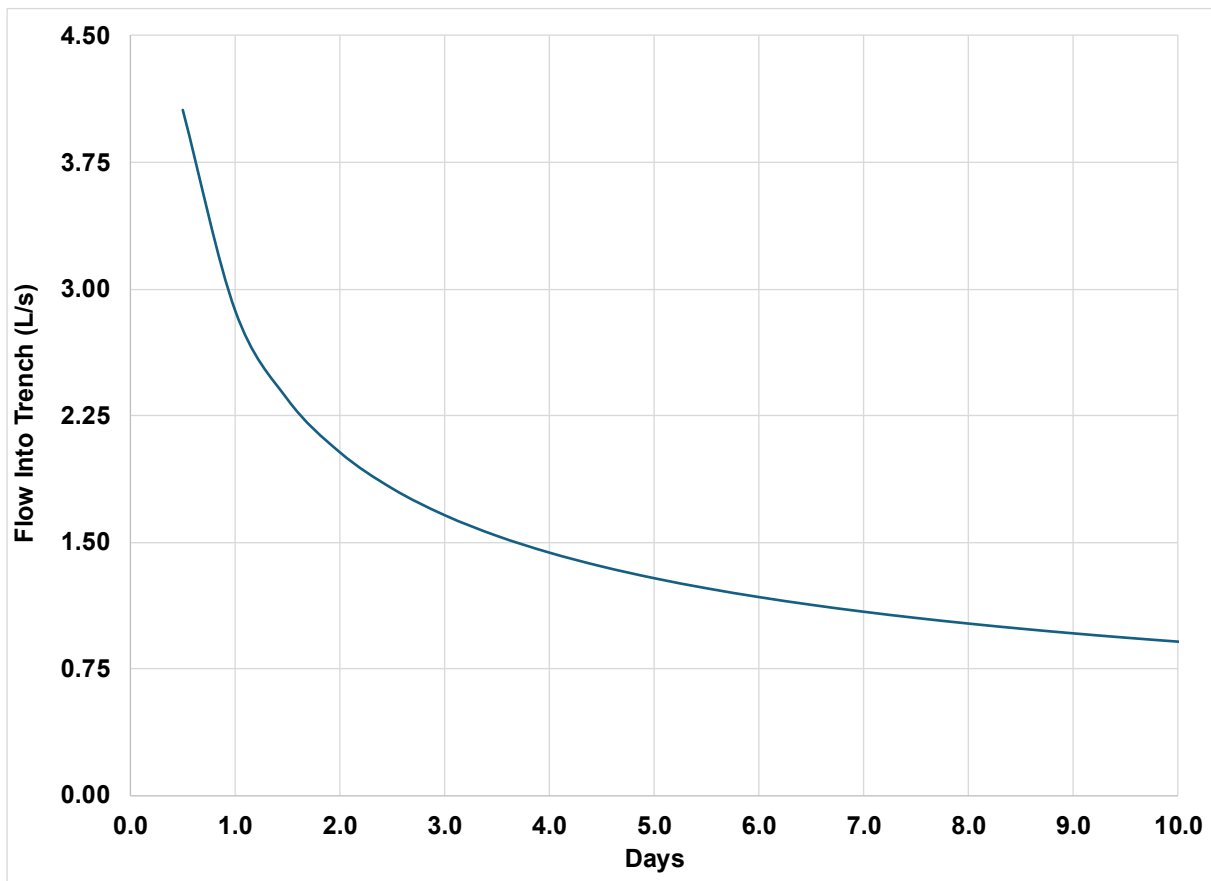


Figure 20: Projected Groundwater Inflows Over Time into a 20 m Trench at the Centre of the Residential Development During Summer

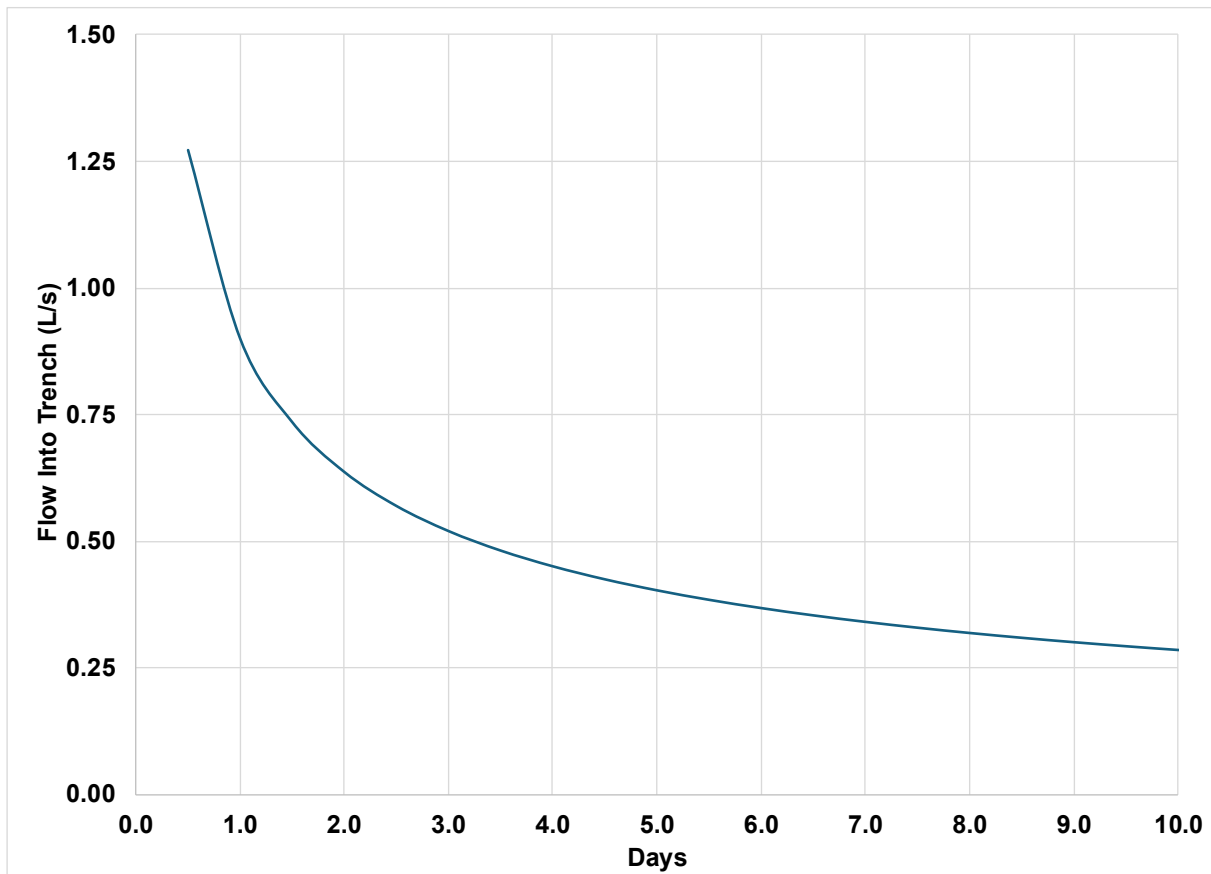


Figure 21: Projected Groundwater Inflows over Time into a 20 m Trench at the Edges of the Residential Development During Summer

6 GROUNDWATER EFFECTS ASSESSMENT

6.1 Effects Overview on Catchment Water Balance

The proposed development is located within the upper Mangakōtukutuku catchment, where the catchment's hydrology is characterised by strong interaction between groundwater and surface water features. The groundwater contributes flows to drains; natural wetlands, and headwater stream reaches (as described in Section 3.5). This connectivity influences local groundwater flow paths and supports the persistence of flow through the catchment.

The proposed Residential Development has two main groundwater-related components that have the potential to influence the catchment water balance (Figure 22):

1. Short-term temporary dewatering during construction of the Artificial Wetlands and wastewater infrastructure.
2. Long-term groundwater interception and diversion to the proposed Artificial Wetlands, which will act as persistent hydraulic features within the developed catchment.

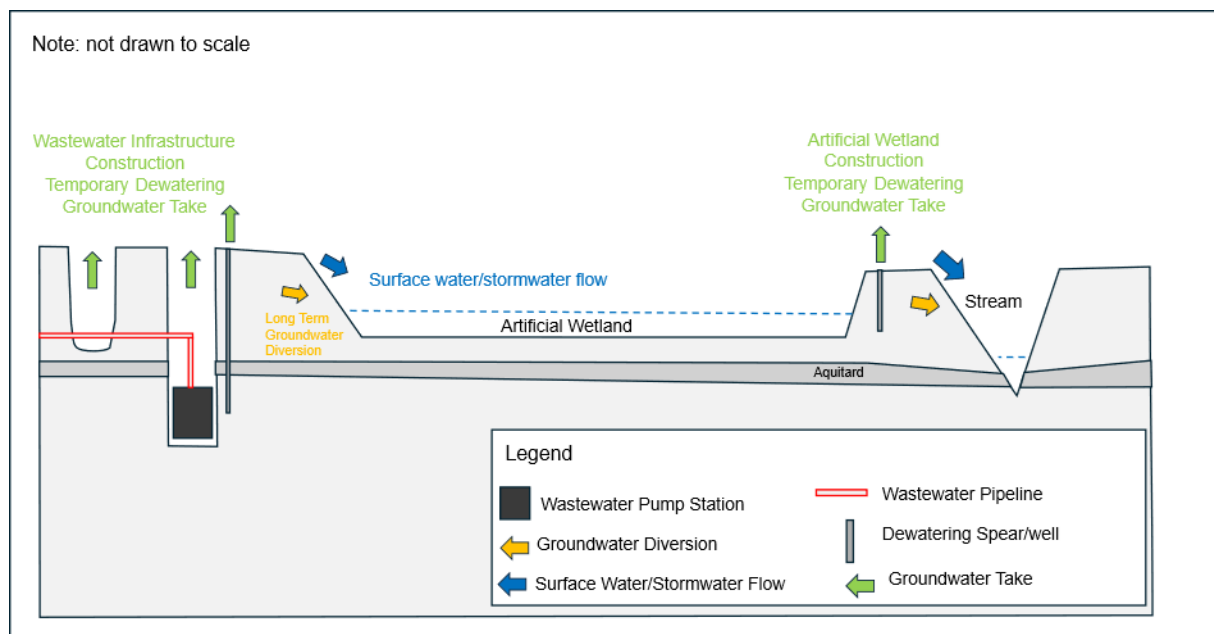


Figure 22: Conceptual Schematic of Water Flow Required and Induced by the Proposed Activity

The development represents a land use change from existing dairy farmland to residential. This land use change is expected to alter the way rainfall is divided between infiltration (recharge), evapotranspiration, and runoff. A typical residential development ratio for impervious/pervious land is approximately 70 % impervious and 30 % pervious. A similar ratio is anticipated across the Residential Development (Maven 2026). Rainfall recharge is expected to reduce beneath impervious surfaces relative to the existing rural state, with a corresponding increase in runoff captured by the proposed stormwater management system (Figure 22).

In the long term, the Residential Development site water balance is expected to shift from predominantly distributed recharge and surface runoff toward a more managed regime, where a larger proportion of the rainfall is routed to engineered Artificial Wetlands, stored within, and subsequently discharged to existing drains and the Mangakōtukutuku Stream. Because the catchment currently exhibits close groundwater–surface water connectivity, maintaining hydraulic continuity to headwater drains and wetlands is an important consideration in the design and staging of temporary dewatering activities.

6.2 Dewatering Effects Assessment

Artificial Wetland construction is planned to be staged. The combined rate of groundwater abstraction during the construction phase is expected to vary as each wetland is excavated and brought online (Table 13). Based on current design and staging, the largest construction groundwater abstraction is expected to occur for Wetland A (**5,361 m³/day** during construction). Pumped groundwater (where required during construction) and diverted inflows will be managed under a Water Management Plan (WMP) and an Erosion and Sediment Control Plan (ESCP) for the Residential Development.

Table 13: Summary of Construction Dewatering Flow Rates and Operational Cumulative Groundwater Diversions

EXCAVATION TYPE	TOTAL INFLOW (WINTER) CONSTRUCTION (m³/day)	TOTAL INFLOW (WINTER) OPERATIONAL GROUNDWATER DIVERSION (m³/day)	CUMULATIVE OPERATIONAL GROUNDWATER DIVERSION (m³/day)
See Section 4			
Artificial Wetland A	5,361	3,703	15,132
Artificial Wetland B	4,000	2,876	
Artificial Wetland C	3,648	3,504	
Artificial Wetland D	3,284	2,100	
Artificial Wetland E	3,018	2,080	
Artificial Wetland F	1,702	869	
See Section 5.2			
WWPS A	907 (Deep Aquifer) + 179 (Shallow Aquifer)	N/A	
WWPS B	3,661 (Deep Aquifer) + 40 (Shallow Aquifer)		
See Section 5.3			
Shallow 20 m Trench for Wastewater Pipeline	< 153	N/A	
Deep 20 m Trench for Wastewater Pipeline	< 690		

Notes: Table 11 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 4 and 5. Those listed with "<" are the starting rates for dewatering analyses.

Wastewater infrastructure is anticipated to be constructed progressively, based on the installation of consecutive 20 m lengths of pipeline in similar dewatered trench sections. The WWPSs will be constructed separately, with construction schedules that will probably overlap the construction schedule for the pipelines. Based on the current design, the largest individual groundwater abstraction to enable for wastewater infrastructure construction is expected to occur for WWPS B (**3,661 m³/day**).

Similarly, the largest abstraction for pipeline summer installation is expected to occur for the deepest section of pipe, at **690 m³/day**. While overlapping works may increase the overall rate of groundwater abstraction on any one day during construction, they do not alter the groundwater drawdown required for individual trench sections or the nature of the associated effects. Accordingly, the pipeline trenching assessments presented in Section 4.3 remain representative of the anticipated groundwater effects associated with construction of the wastewater network. During dewatering initial flow rates may need to be higher than the modelled flow rates to initiate local drawdown. Therefore, to accommodate initial higher flow rates and multiple pipelines being installed simultaneously a daily maximum abstraction rate of **1,500 m³/day** is proposed.

6.2.1 Effects on Nearby Bores

Of the surrounding groundwater users, there are four current groundwater-related consents within one kilometre of the Residential Development site (Table 5, Figure 12), however, only two authorise groundwater abstraction for consumptive use. The furthest radial extent of dewatering impacts from proposed site activities assessed in this report is the 854 m extent of drawdown from WWPS B in Winter conditions. Neither of the two groundwater takes exists within this radius from WWPS B.

Of the existing groundwater bores within one kilometre of the Residential Development site, one bore may be impacted by drawdown during construction of WWPS B, during winter conditions of high groundwater levels (Section 5.2.2.1). Bore 70_501 is located 770 m from WWPS B (Figure 12). At this distance, it may experience up to 0.30 m of drawdown after 15 days of dewatering. As this drawdown effect would only apply during otherwise high groundwater conditions, the impact on water availability at Bore 70_501 is expected to be less than minor and within the range of annual groundwater level fluctuations.

There are no other drawdown effects, assessed in this report, that extend to nearby bores.

6.2.2 Effects on Nearby Surface Water Features

For Artificial Wetlands D and E, where construction dewatering is expected to intercept groundwater that would otherwise contribute to flows in the adjacent drains and headwaters of the Mangakōtukutuku Stream, groundwater intercepted during construction will be discharged to the receiving drain upstream of the works, as proposed through the Residential Development consent conditions. This measure will maintain continuity of surface water flows and mitigate temporary stream depletion effects during construction. For the remaining Artificial Wetlands, intercepted groundwater may be discharged in accordance with the construction management framework, as stream depletion effects are not predicted to occur. With these measures in place, no sustained reduction in surface water flows or material changes to downstream hydrology are anticipated, and effects on stream flows are assessed to be minor and temporary.

6.2.3 Effects on Natural Wetlands

Two natural inland wetlands have been identified within the site (EcoLogical Solutions Ltd, 2025). These include a small pasture wetland and a floodplain wetland associated with the Mangakōtukutuku Stream. The pasture wetland will be unavoidably removed as part of the proposed development. The ecological effects of this wetland removal, and effects management measures proposed in relation to this wetland loss, are addressed in separate ecological assessment for the Residential Development. However, given the loss of this wetland and the assessment of effects being undertaken on the basis of complete loss, any effects of groundwater drawdown on this feature are not relevant to the construction or operational assessment.

The floodplain wetland is hydraulically connected to the Mangakōtukutuku Stream and is primarily supported by surface water runoff rather than local groundwater discharges. During construction at the Residential Development site, stream flows will be maintained (as outlined in Section 4.3.2). No sustained reduction in stream level is anticipated as a result of groundwater drawdown. Where dewatering discharge may be directed to or near the floodplain wetland, the discharge location and method will be managed through the construction management framework, including the Erosion and Sediment Control Plan (ESCP). Any sediment treatment or removal measures required to manage discharge water quality will be implemented in accordance with the ESCP prior to discharge to the wetland.

Groundwater drawdown arising from construction operations is therefore not expected to alter the hydrological regime, extent, or ecological function of the floodplain wetland. Overall effects on natural wetlands are assessed to be minor and temporary.

6.2.4 Effects on Ground Subsidence

The groundwater drawdown predictions presented in Sections 4 and 5 have been provided to CMW for assessment of potential settlement and other geotechnical effects associated with the proposed construction and operational dewatering activities. Assessment of ground settlement, including any implications for existing and proposed buildings and infrastructure, falls outside the scope of this hydrogeological assessment and is addressed in the accompanying Geotechnical Assessment (CMW, 2026c).

6.3 Anticipated Implications for Water Quality

The land use change from dairy farming to residential land use, together with the introduction of Artificial Wetlands, is expected to result in reduced nutrient inputs to shallow groundwater and the Mangakōtukutuku Stream over time (relative to the existing rural baseline). While construction activities can temporarily elevate sediment related risks, for the Residential Development this risk will be addressed through the proposed Erosion and Sediment Control Plan (ESCP).

As presented in Section 3.5.3, there is existing discolouration of drain water due to oxidation of dissolved iron carried in groundwater discharges. This discolouration is expected to continue in the existing streams during low flow conditions.

7 MONITORING AND MITIGATION

7.1 Groundwater Monitoring

Groundwater levels across the Residential Area have been characterised using a network of five piezometers installed and monitored between July 2025 and January 2026. This monitoring period captured both winter and summer groundwater conditions and confirmed the presence of a shallow aquifer overlying a confined deeper aquifer, separated by a thin, laterally persistent, low-permeability aquitard. The monitoring data were sufficient to define seasonal groundwater level variability, hydraulic connectivity with local drains, and baseline groundwater conditions for the purposes of the effects assessment.

The investigations and modelling completed for this assessment indicate that the predicted groundwater drawdown effects associated with construction dewatering and long-term operation of the proposed artificial wetlands are localised, temporary (during construction), and have been conservatively assessed using site-specific hydraulic parameters. Groundwater drawdown effects do not impact the availability of water at nearby groundwater abstraction bores. The long-term groundwater diversion to the artificial wetlands represents a redistribution of groundwater discharge within the Residential Development site catchment rather than a net loss.

Based on the investigations, monitoring and assessments completed for this report, no additional permanent groundwater monitoring sites are considered necessary to manage or verify effects, assessed in this report, from the proposed development.

WGA understands that targeted short-term groundwater level monitoring will be required to support the identification and mitigation of potential ground settlement effects during construction to confirm groundwater behaviour remains consistent with predictions (CMW 2026b).

The existing piezometer network may provide a useful opportunity for collection of long-term groundwater level information should the consent holder wish to retain the installations following completion of construction. However, such monitoring is not considered necessary to manage, mitigate or verify the groundwater effects assessed in this report.

Should the existing site piezometers be destroyed during construction, replacement piezometers may be required for the above reasons.

7.2 Surface Water Monitoring

The investigations completed for this assessment indicate that the surface water features within and downstream of the Residential Area comprise farm drains and the headwaters of the Mangakōtukutuku Stream. At least some of these features are hydraulically connected to the confined groundwater system identified within the Residential Area. Potential effects on surface waters are limited to localised interception of groundwater contributions during construction dewatering for artificial wetlands and wastewater infrastructure. With the proposed management measures (Section 6.2.2 and Section 6.2.3), these effects have been assessed as minor and temporary (i.e. they will be appropriately managed through the proposed construction methodology and the placement of temporary water discharge points).

In the above context, including 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 Residential Development's construction management framework (namely, the ESCP), 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 and incorporated into proposed consent conditions, are considered sufficient to confirm expected performance.

8 CONCLUSIONS

This assessment has considered the potential groundwater effects associated with the construction and operation of the Southern Links 1 Residential Development, including construction dewatering for artificial wetlands and wastewater infrastructure, and the long-term groundwater diversion associated with the proposed artificial wetlands. The investigations, monitoring and analytical assessments undertaken indicate that the predicted groundwater effects are localised and can be appropriately managed through the proposed development design and associated management measures. Accordingly, groundwater effects associated with the proposed Residential Development are assessed as no more than minor.

No additional permanent groundwater or surface water monitoring is considered necessary for hydrogeological assessment, with standard construction-phase environmental controls, as proposed, providing sufficient verification of predicted effects. Visual inspections of drains and stream reaches during construction, undertaken as part of standard environmental compliance checks and incorporated into proposed consent conditions, are considered sufficient to confirm expected performance.

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