



Southern Links 1, Industrial Development Hydrogeological Effects Assessment

ASSESSMENT OF EFFECTS

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1 EXECUTIVE SUMMARY

This hydrogeological assessment concludes that the proposed Industrial Development works are expected to result in temporary and localised groundwater effects that are no more than minor and can be appropriately managed with the implementation of the proposed conditioned management measures. Construction-related effects are expected to be temporary and localised, with the greatest effects associated with dewatering for wastewater pumping station excavations and deeper trenching. Operational effects are expected to comprise minor localised groundwater redistribution rather than any material long-term reduction in catchment groundwater resources.

Site investigations and groundwater monitoring confirm that the Industrial Area contains a shallow groundwater system within the peat that is overlying deeper Hinuera Formation sediments. Groundwater conditions are controlled by rainfall recharge, artificial drainage, and the low permeability of peat, resulting in a shallow phreatic surface above a deeper piezometric surface.

The assessment is based on site-specific investigations, monitoring data, and conservative analytical methods, and assumes existing pre-compaction peat conditions. This provides a conservative basis for evaluating groundwater drawdown and inflows with the proposed works.

The assessment indicates that groundwater effects during construction will be temporary, localised, and within the range of natural seasonal fluctuations, with no effects on nearby bores, surface water features, and surrounding land. During operation, the artificial wetlands will act as persistent hydraulic features within the catchment. Predicted groundwater interactions are minor and include small ongoing inflows and localised event mounding beneath Artificial Wetland 3, with mounding largely confined to the immediate vicinity of the wetland and reducing rapidly with distance.

Overall, our assessment indicates that effects are likely to be less than minor using conservative assumptions of a shallow phreatic surface during wet conditions and pre-compaction hydraulic parameters. Potential risks associated with groundwater drawdown and mounding can be appropriately managed through the standard construction practices that are proposed, the proposed erosion and sediment controls (as detailed in the ESCP), and the proposed targeted monitoring.

2 INTRODUCTION

2.1 Background

Wallbridge Gilbert Aztec (WGA) has been engaged to provide 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. This assessment of hydrogeological effects focuses on the industrial portion of the Project (the “Industrial Area” and the “Industrial Development”). (Figure 2).

2.2 Site Description

The entire SL1 Project site 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 (Figure 1). The SL1 site is currently zoned rural under the Waipa District Plan and consists of 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, grass lined swales and constructed wetlands. The SL1 site has been divided into two areas, known as the ‘Industrial Area’ and the ‘Residential Area’. This assessment of effects focuses only on the Industrial Development (Figure 2).

2.2.1 Industrial Development

The Industrial Area covers an area of approximately 60 ha, currently utilised for light industrial and farming purposes. Farmland areas occur over extensively drained peat soils (Figure 3). Industrial areas are predominantly underlain by gravel hardstand over peat. The site topography is generally flat, with localised areas of uncontrolled fill forming elevated features. A significant accumulation of heavily vegetated building and demolition waste and vehicle bodies known locally as “Latham’s Mountain” is present at the north-eastern end of the site (Figure 3).

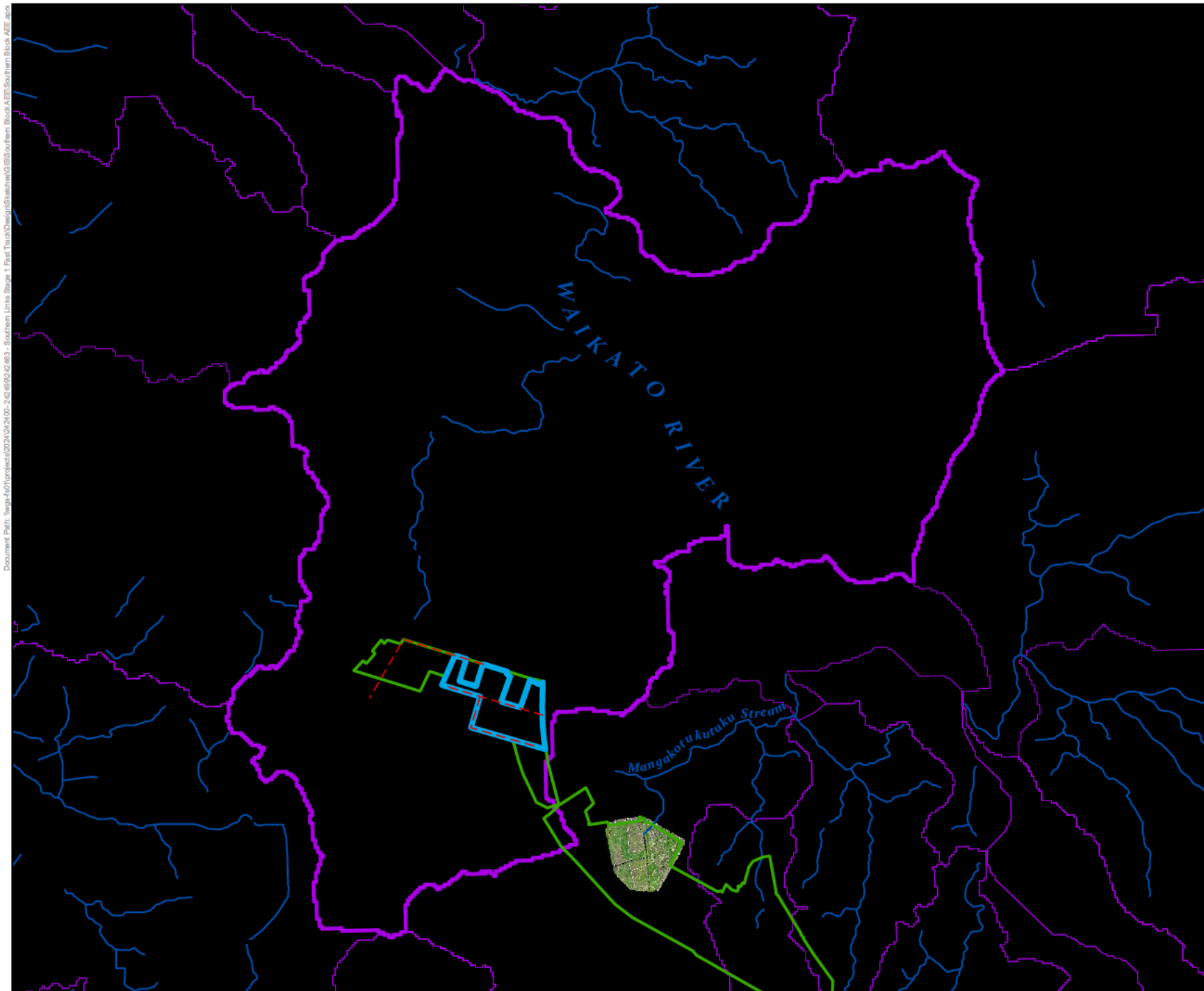
2.3 Proposed Activity

For the Industrial Development, the proposed activity comprises:

- Cut and fill to re-contour the site.
- Construction of three artificial wetlands (Figure 3).
- Installation of wastewater pipelines and two wastewater pump stations (Figure 3).
- Installation of drains and swales.
- Localised pre-construction preloading (or “surcharging”) of peat soils to accelerate peat compaction and stabilisation, thereby supporting site development. This is addressed in detail in the CMW geotechnical report for the Industrial Development (CMW 2026).

From a hydrogeological perspective, the proposed activity has four principal components with potential to influence the local groundwater system:

1. Removal of existing in-paddock farm drainage and reconfiguration of the site drainage network, which is proposed to be replaced by a combination of engineered fill and roadside swales integrated within the proposed roading network.
2. Short-term dewatering during construction earthworks. 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.
3. Potential groundwater mounding due to increased recharge from the Artificial Wetlands, which are to be permanent features within the developed catchment.
4. Pre-construction preloading on peat soils, resulting in compaction of these soils. This process, which is intended to improve ground stability prior to construction on site, may influence the hydraulic characteristics of shallow or sensitive soils. Changes in the hydraulic characteristics of shallow lithologies may have an effect on groundwater levels, groundwater flow directions and flow rates.



LEGEND

- River
- - - Site Specific Drains
- Catchment Outlines
- Waikato River at CMA Catchment
- Southern Links Boundary
- Industrial Development



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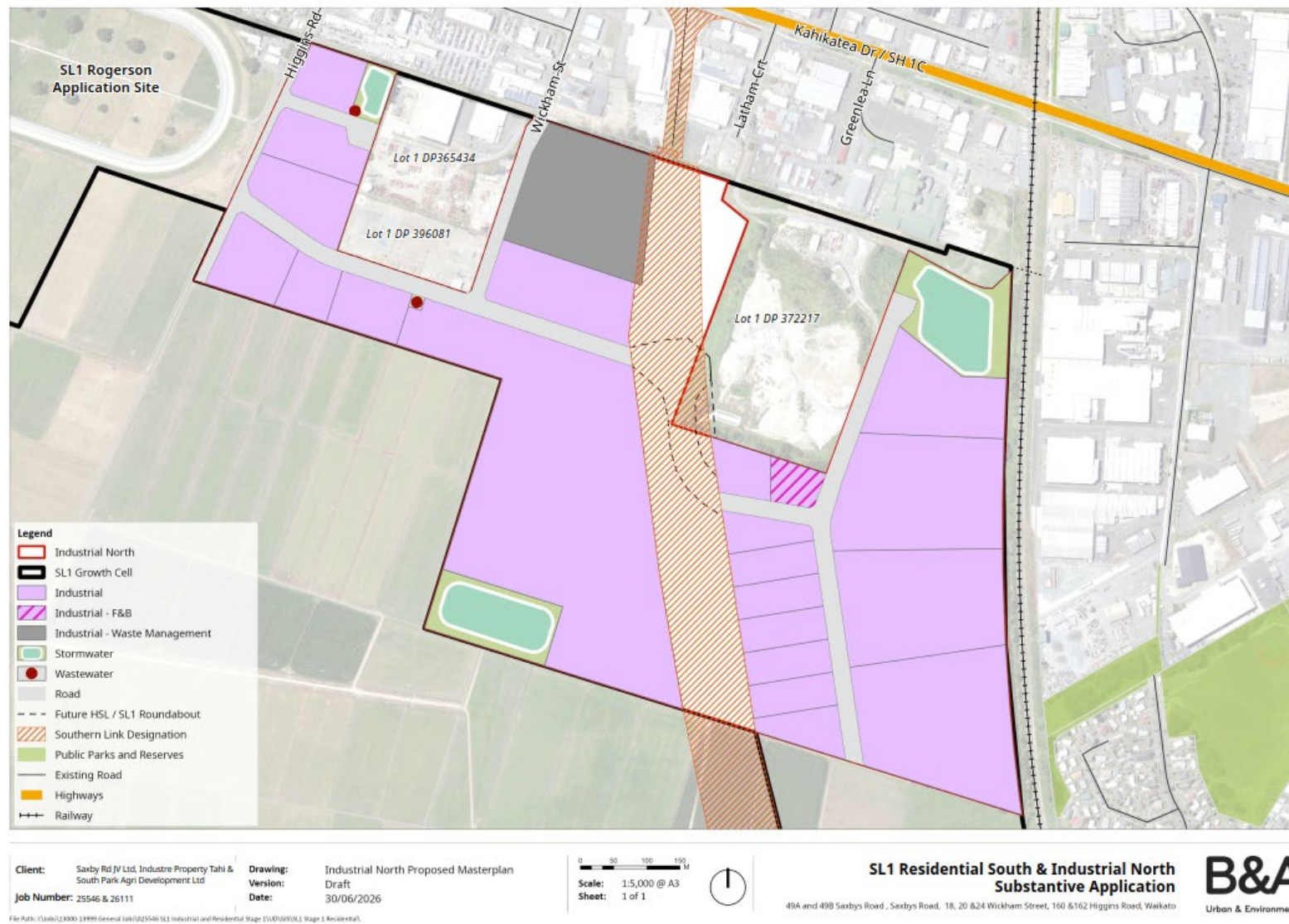
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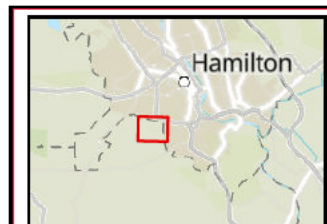
WGA

Figure 1

Southern Links Stage 1
Site Location and
Catchment Map

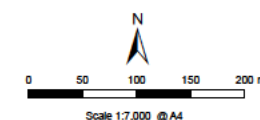
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LEGEND

- Proposed Wastewater Pipelines
- Southern Links Boundary
- Industrial Development
- Proposed Wastewater Pump Stations
- Proposed Artificial Wetlands
- Contour 1m



Coordinate System: NZGD 2000 Mount Eden Circuit

WGA

Figure 3

Southern Links Stage 1 Proposed Artificial Wetlands and Wastewater Infrastructure

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3 EXISTING ENVIRONMENT

3.1 Topography

Ground levels within the Industrial Area range in elevation from 36 mRL at the northwestern corner to 61 mRL in the “Latham’s Mountain” area in the northeastern quadrant of the Site. Prior to historical development, the Site would have been near level, grading gently downward from 41 mRL in the south to 39 mRL in the northwest. This profile is generally still reflected across the southern half of the site, which has been drained and recontoured for use as pasture (CMW 2026).

The northern half of the site has been extensively developed, recontoured and resurfaced to meet the demands of the industrial uses. The key feature of this area is “Latham’s Mountain”, with stored wastes producing an uneven top surface with heights of up to 21 m (between 40 mRL and 61 mRL) and slope angles up to 60 degrees to the horizontal (CMW 2026).

3.2 Rainfall

Rainfall data are recorded at the Ruakura weather station, which is located approximately 5.7 km northeast of the Site. The available record spans from November 2005 to the present. The long-term mean annual rainfall recorded at the station is approximately 1,121 mm, with a peak daily rainfall of 87.8 mm recorded during this period (Figure 4).

During the period of groundwater level monitoring undertaken at the Industrial Area for this assessment (28 January 2026 to 22 June 2026; see Section 3.6.3), the peak daily rainfall recorded at Ruakura was 46 mm. Total rainfall over the groundwater monitoring period was approximately 352 mm. When annualised, this equates to an annual rainfall of approximately 880 mm/year. This value reflects a relatively dry period, and therefore, field data may not be representative of conditions during long-term mean annual rainfall. This difference is considered further in Section 3.6.4.

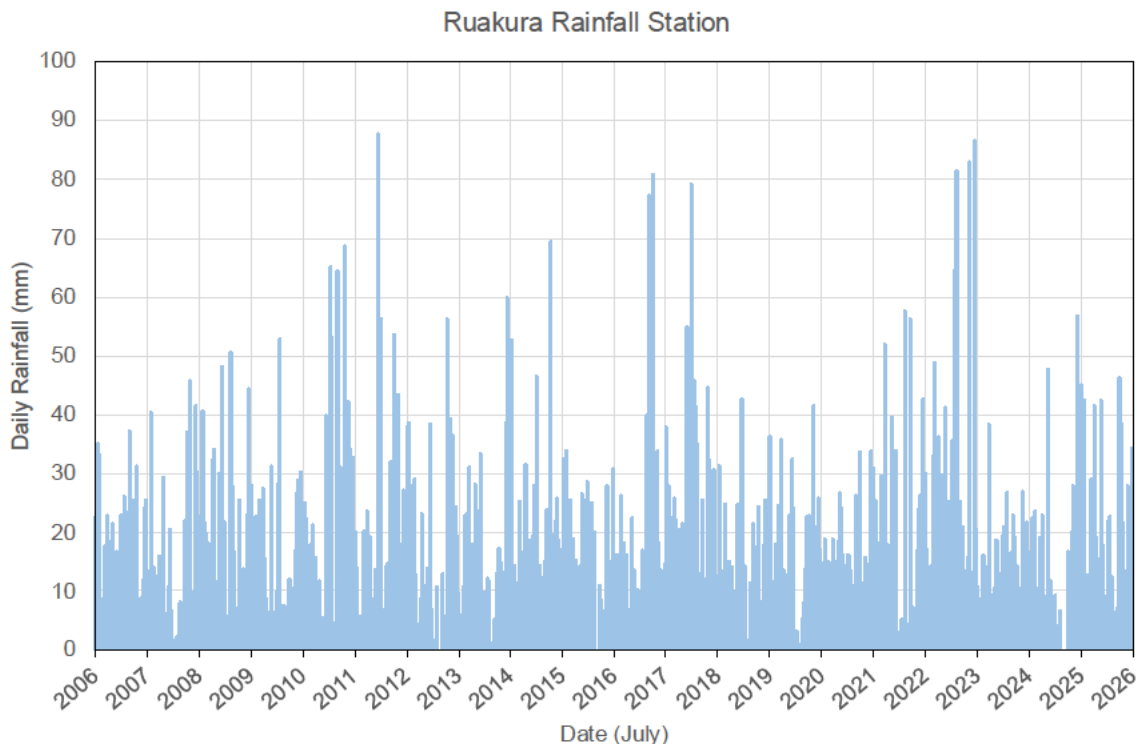


Figure 4: Rainfall Record at Ruakura (5.7 km Northeast)

3.3 Surface Water Bodies and Catchment Boundary

EcoLogical Solutions Ltd (EcoLogical) undertook freshwater ecology investigations at the Industrial Area during 2025. Outcomes from these investigations included the conclusions that there are no RMA-qualifying wetlands on the Site (EcoLogical 2026).

EcoLogical (2026) assessed the ecological value of the Site's artificial drains as low, reflecting poor habitat quality, uncertain flow regimes, and limited fish diversity.

Existing farm drainage channels are estimated to be 0.5 to 1 m deep and occur along the northern and southern boundaries of the Site, as well as within the Site (Figure 5 and Figure 6). Recorded drain water levels are relative to ground levels immediately adjacent to the drains. Ground levels across the paddocks are typically higher than those beside the drains, reflecting intentional or incidental shaping of the peat surface to direct runoff toward the drainage network (Figure 4).

Water levels recorded in farm drains during November 2025 were approximately 0.5 m bgl (below ground level). EcoLogical (2026) also noted that some drainage canals observed to be dry during the November site visit showed evidence that they may only hold water during wetter periods of the year.

Figure 1 shows the catchment according to Waikato Regional Council (WRC) database. The Site is identified by WRC within the "Waikato River at CMA" catchment boundary which is being developed for an Integrated Catchment Management Plan (ICMP) for Waitawhiriwhiri area. However, the estimated catchment outline provided by EcoLogical (2025) places the Site within the Mangakotukutuku catchment. Groundwater and surface water observations from the following assessment acknowledge both catchments as drainage and flow can appear to go in both directions (refer also to Section 3.6.4).



Figure 5: Central Farmland Drain Within the Industrial Area (WGA Field Visit, 11 November 2025)



Figure 6: Artificial Watercourses and Locations of Eels reported by EcoLogical (2026)

3.4 Regional Geology

The Hamilton Basin, in which the Industrial 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 7).

The younger Hinuera Formation of the Tauranga Group underlies much of the Hamilton basin (Figure 7). 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.

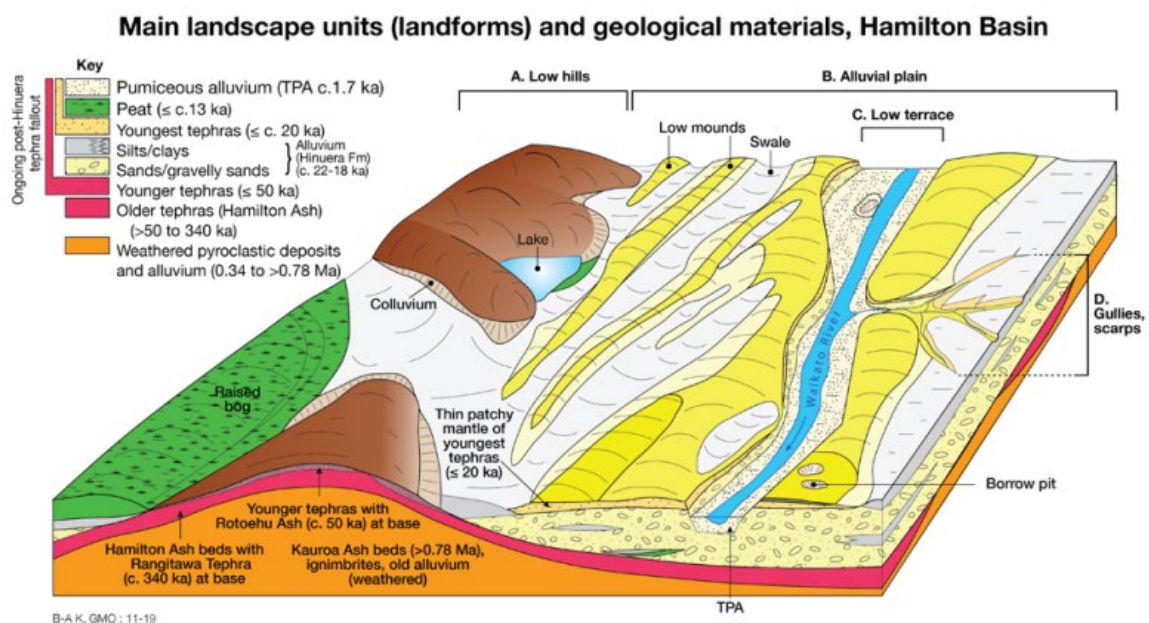
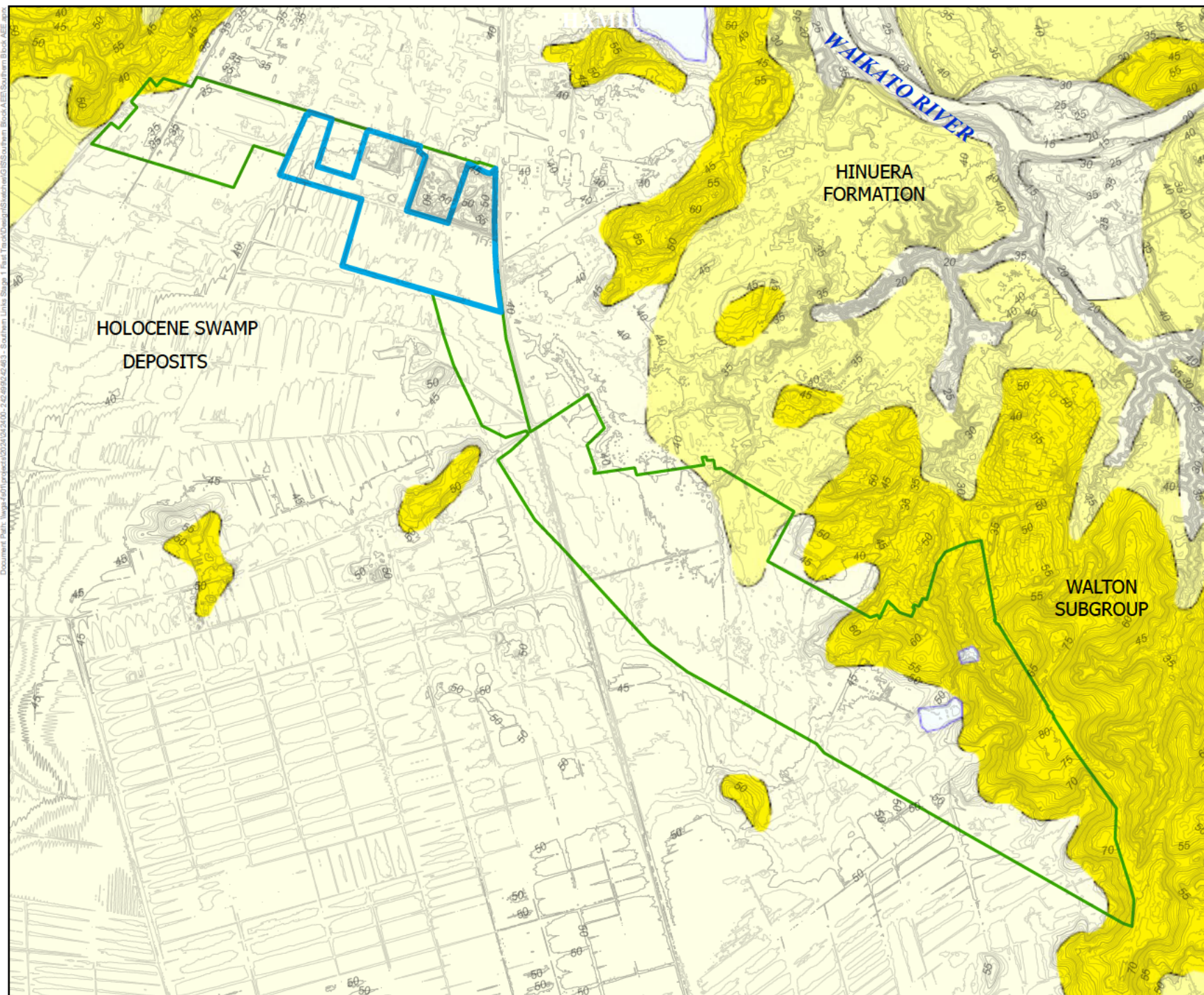


Figure 7: Geological Setting of the Hamilton Basin (Lowe 2021)

3.5 Local Geology

The Site is characterised by peat derived from Holocene Swamp Deposits (Figure 8). The peat 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.



Geotechnical investigations to determine near surface lithologies across the site have been conducted by CMW Geosciences (CMW) and documented in a separate report (CMW, 2026). Extracts from the CMW report are provided in Appendix A to support the hydrogeological assessment for the Site. The CMW report identifies the presence of imported gravel fill in the yard areas and the elevated areas of uncontrolled fill, specifically a waste storage site in the north-eastern corner of the site (Latham's Mountain). A map and cross sections are provided in Appendix A. A summary of the underlying geology interpreted from the CMW site investigations is presented in Table 1.

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

UNIT	TOP OF UNIT (mbgl)		THICKNESS (m)	
	Min	Max	Min	Max
Stiff to very stiff clayey silt and medium dense sand. Construction and landfill waste in the area of "Latham's Mountain" (Uncontrolled Fill)	0.0	0.2	0.5	3.2 ⁽¹⁾
Firm to very stiff fibrous organic peat (Holocene Swamp Deposits)	0.1	1.0	0.3	12.0
Loose to medium dense sand with interbedded stiff to very stiff silt, and lenses of organic silt (Hinuera Formation)	0.1	7.0	3.3	17.5
Stiff to hard clay/silt and dense to very dense sand (Walton Subgroup)	5.2	19.8	0.2	>14.8

Note:

(1) Investigation holes were unable to penetrate full depth of fill. Based on interpreted natural ground contours, the maximum thickness of uncontrolled fill in the construction waste storage area is likely to be in the order of 19 m.

3.6 Local Hydrogeology

3.6.1 Recharge

The shallow groundwater and surface waterbodies 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 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 rate of 350 mm per year was adopted for groundwater modelling purposes, to assess potential effects under elevated groundwater levels within the peat.

3.6.2 Hydrogeological Parameters

In December 2025, WGA oversaw the installation of four piezometers (Figure 9). Piezometers P1, P2, P3 and P4 were installed to a target depth of 6 m. Drilling samples were logged on-site by CMW in order to have consistent results with other geological tests undertaken at the site. Drill core photos and drillhole logs are provided in Appendix A.

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.)

The surficial geological unit encountered across the site was a peat layer of Holocene Swamp Deposits. Beneath the peat, each piezometer intersected fine silt and sand, also interpreted to be of Holocene Swamp Deposits. Piezometers P1 and P2 were screened in the Holocene Swamp Deposits. Piezometers P3 and P4 were screened in the clays, silts and sands of the upper Hinuera Formation beneath the Holocene Swamp Deposits.

Hydraulic conductivity values for the Holocene Swamp Deposits and Hinuera Formation were estimated from slug tests completed in the screened intervals of the four piezometers. Both falling-head and rising-head tests were performed on each piezometer and the data from each analysed. The average hydraulic conductivity for each lithology and location is summarised in Table 2, with the full analyses provided in Appendix C.

Table 2: Hydraulic Conductivity Values from Testing of Industrial Area Sediments

MATERIAL NAME	LOCATION	HYDRAULIC CONDUCTIVITY RESULT/RANGE (m/s)	ADOPTED HYDRAULIC CONDUCTIVITY FOR MODELLING (m/s) ⁽³⁾
Unsaturated zone ⁽¹⁾ (Peat)	CMW Soakage Test Sites	SO1 = 5.95×10^{-7} SO2 = 2.44×10^{-6} SO3 = 1.36×10^{-6}	4.48×10^{-7}
Peat, low plasticity. (Holocene Swamp Deposits) ⁽²⁾	P2	2.04×10^{-8}	
Peat, low plasticity and fine to coarse sands. (Holocene Swamp Deposits) ⁽²⁾	P1	1.02×10^{-5}	N/A
Interbedded fine to coarse sands and silt, low plasticity. (Hinuera Formation) ⁽²⁾	P3	1.26×10^{-4}	2.79×10^{-5}
Interbedded clayey organic silt, fine to coarse sands and minor gravel. (Hinuera Formation) ⁽²⁾	P4	6.17×10^{-6}	

Notes:

(1) CMW soakage tests (CMW 2026). At 1 m depth it is assumed this could be considered for unsaturated zone in peat or potential shallow groundwater in peat.

(2) Average result derived from WGA aquifer hydraulic slug testing.

(3) Geometric mean of all Kh results available.

Soakage tests in the unsaturated zone undertaken by CMW also recorded low soakage rates (Table 4). Full documentation of these tests is provided in the CMW Geotechnical Investigation Report (2026).

Preloading and compaction of peat within roading and infrastructure corridors (addressed in detail in the Geotechnical Investigation Report (CMW 2026) is expected to reduce peat porosity and hydraulic conductivity relative to current conditions in the Holocene Swamp Deposits (Table 4). Because the modelling in this hydrogeology report uses pre-compaction hydraulic conductivity values, groundwater inflows and drawdown effects for construction dewatering in preloaded areas are likely to be overestimated. No reduction factors have been applied at this stage because the extent of peat compaction and the resulting change in permeability have not yet been confirmed. This approach is therefore conservative for assessing dewatering effects.

3.6.3 Groundwater Levels

WGA installed pressure transducers in all four piezometers to continuously record groundwater level fluctuations between 28 January 2026 and 22 June 2026 (Appendix B). At the time of reporting, the transducers remain installed and monitoring has continued beyond the period assessed in this report. The monitoring data highlight contrasting groundwater level responses to rainfall recharge between piezometers P3 and P4, located east of the Site and screened within the Hinuera Sands, and piezometers P1 and P2, located west of the Site and screened within the peat deposits (Figure 10).

Groundwater level measurements were also collected by CMW from numerous investigation sites in November 2025 (Appendix B) and were taken into consideration by WGA staff during the hydrogeological assessments.

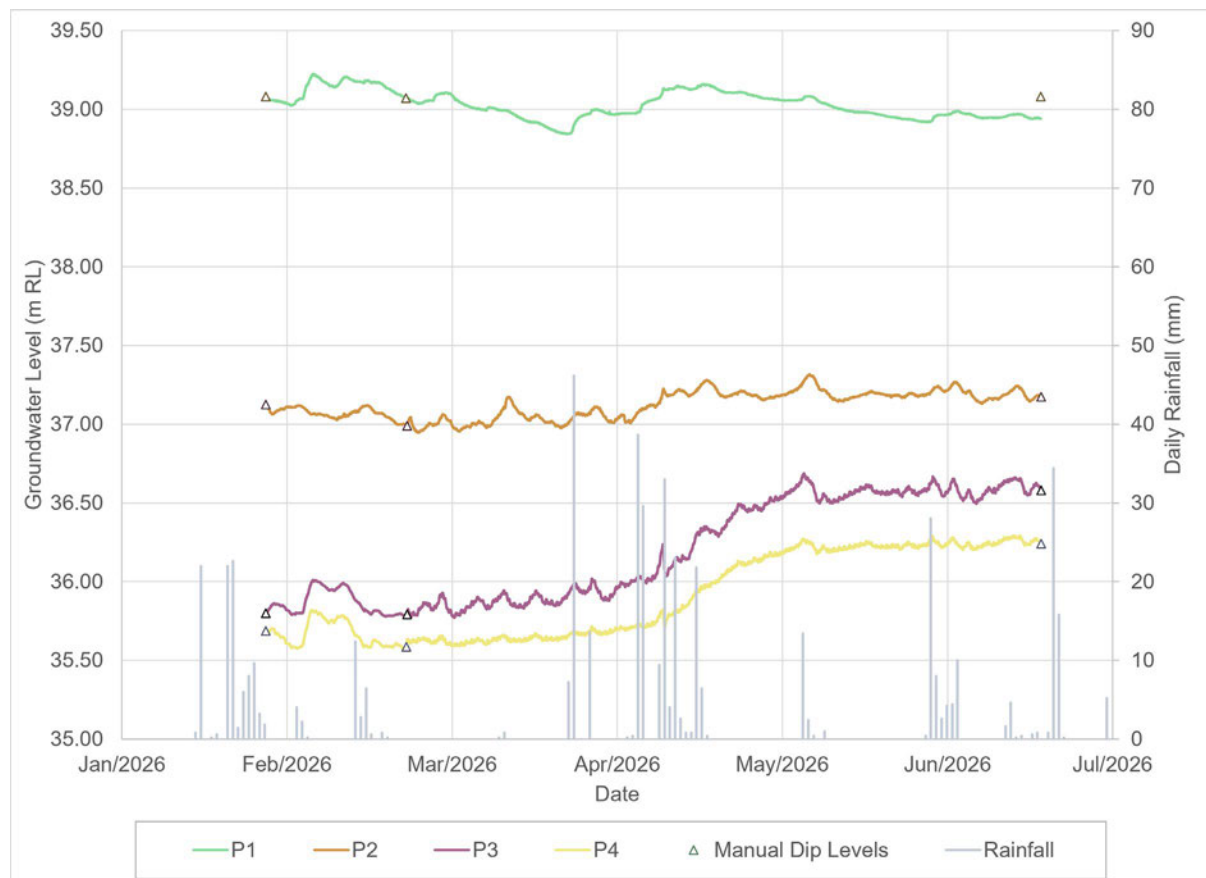


Figure 10: Groundwater Level Responses in the Industrial Area Piezometers

Due to the nature of the uncontrolled fill placed in the “Latham’s Mountain” area of the Industrial Area, it was not possible to obtain groundwater level measurements in this area. Therefore, piezometric maps for the Industrial Area either exclude this area or use extrapolations to provide an indication of groundwater levels (refer to Section 3.6.4).

3.6.4 Ground and Surface Water Flow Paths

Groundwater flow across the Industrial Area has been interpreted using a combination of subsurface piezometric data and surface water observations. Due to the effects of extensive peat drainage across the Site, two piezometric maps have been produced for the purposes of this assessment, as described below.

The first piezometric surface (Figure 11) represents winter groundwater levels measured by WGA and CMW within the Industrial Area. Inclusion of an interpreted winter piezometric surface is important for characterising the seasonal upper bound of groundwater levels and provides a conservative basis for the assessment.

The second piezometric surface, which represents the free phreatic surface (Figure 12), is derived from water level observations in long-standing artificial drains. These observations indicate that the phreatic surface within the peat is shallower than the piezometric surface described above. Water levels recorded in farm drains during November were approximately 0.5 m bgl, which is shallower than the groundwater levels measured in the piezometers and therefore not represented by that monitoring network. Together, these observations indicate that a shallow phreatic surface occurs within the peat deposits and reflects near-surface groundwater conditions across the Site. Figure 12 therefore represents a conceptual phreatic surface occurring at approximately 0.5 m below existing ground level under the current drainage regime.

Both groundwater surfaces are therefore considered in subsequent assessments:

- The deeper piezometric surface (Figure 11), representing sub-peat groundwater flow conditions.
- The shallow phreatic surface (Figure 12), representing local, drainage-controlled groundwater conditions within the peat.

The difference in water levels between the shallow phreatic surface and the deeper piezometric surface represents a vertical hydraulic gradient across the site. This gradient reflects downward leakage from the shallow, drainage-controlled groundwater into the underlying aquifer, with the magnitude of the gradient controlled by the low hydraulic conductivity of the peat.

3.6.5 Shallow Groundwater Assumptions

For the purposes of this report, shallow groundwater levels (phreatic surface) have been utilised as the basis for most of the following site dewatering assessments. This approach is conservative for the assessment of construction-phase effects, as most infrastructure within pre-loaded areas will be installed within this shallow groundwater zone. It is possible the shallow (phreatic surface) groundwater will not be intersected by excavations during dry construction conditions.

Peat compaction associated with pre-loading may locally modify shallow groundwater conditions by reducing storage capacity and vertical permeability within the peat sequence. In some locations, this process may result in a relative raising of the shallow phreatic surface with respect to ground level once pre-loading is completed. However, the subsequent placement of engineered fill will permanently increase site elevations above the groundwater table and is expected to offset any localised increase in shallow groundwater levels resulting from peat compression. Therefore, no material adverse groundwater effects are anticipated.



Hamilton

LEGEND

- Contour 1m
- Southern Links Boundary
- Industrial Development
- Piezometric Surface
- - - Conceptual Piezometric Surface

Subsurface Testing

- CPT
- Piezometer (Conceptual Winter SWL)
- Test Pit

Scale 1:8,000 @ A4

Coordinate System: NZGD 2000 New Zealand Transverse Mercator

WGA

Figure 11

Southern Links Stage 1

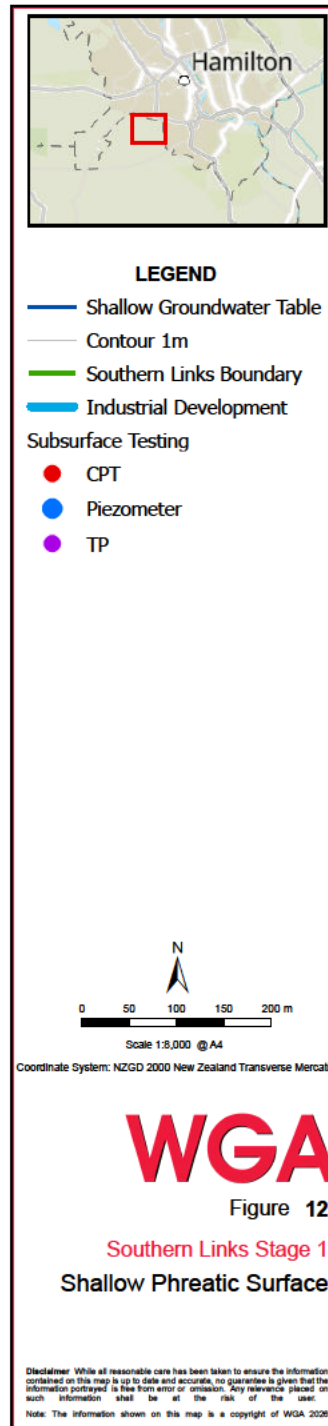
Investigation Locations

Piezometric Surface

Groundwater Monitoring

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 2028



3.7 Surrounding Groundwater Users

Water supply bores (as listed in Appendix B) and groundwater abstraction consents within one kilometre of the Site are shown in Figure 13. According to the WRC database, there are four groundwater abstraction consents linked to bores located within a 1 km radius of the Industrial Development site boundary (Figure 13). Of these, one is related to agriculture, one is related to potable water supply and the remaining two are related to a site dewatering programme that is considered to be a 'zero' net take (Table 3).

Table 3: Nearby Groundwater Takes

CONSENT NUMBER	USER	PERMITTED VOLUME (m ³ /day)	PURPOSE
AUTH125831.01.01	J B Ford Limited	31.5	Agricultural farming - dairy
AUTH136670.01.01	Enviro NZ Services Limited	85	Water supply - municipal/community
AUTH145510.01.01	Hamilton City Council AECOM New Zealand Limited (Auckland) (Holders' Representative)	0	Land development (Dewatering)
AUTH147036.01.01	Hamilton City Council Potentialis Limited (Holders' Representative)	0	Land development (Dewatering)

4 GROUNDWATER DRAWDOWN AROUND ARTIFICIAL WETLANDS

4.1 Introduction

Three artificial wetlands are proposed within the Industrial Development (Figure 3 and Appendix D). To assess their potential hydrogeological effects, WGA considered two groundwater conditions and evaluated both construction and operational scenarios.

Groundwater conditions considered for the following assessment are:

- A shallow phreatic surface for dewatering impacts
- A deeper piezometric surface for mounding impacts

Wetland-specific considerations:

- **Artificial Wetland 2** is to be constructed into engineered fill, which will be emplaced to a height of approximately 2 m in the wetland area. The wetland basin is to be excavated to an elevation of 40.20 m RL, which is approximately 2 m below the current ground surface (Figure 12). The basin floor is below the interpreted local winter phreatic groundwater surface (Figure 12). However, the phreatic surface is currently maintained at an artificially low level by intensive drainage. Many of these drains will be infilled during placement of the engineered fill. Reduction in the intensity of drainage may therefore result in the winter phreatic surface rising to be within the engineered fill.

This wetland basin may intercept groundwater and receive small groundwater inflows from within the engineered fill during operation. However, the flow rates are expected to be small due to the compacted nature of the fill. Any drawdown of the phreatic table within the fill would not result in a drawdown of groundwater levels below the current phreatic surface.

Construction of the wetland may require temporary localised dewatering of the peat to maintain a dry working depth of 1 m below the basin invert (as requested by Maven, pers comm 12 May 2026).
- **Artificial Wetlands 1 and 3** are proposed to intersect peat and, under winter conditions, may intercept shallow groundwater within the peat. Construction may also require dewatering to maintain a dry working depth of 1 m below the basin invert.

WGA undertook analytical pit modelling to estimate anticipated groundwater inflows and the extent of temporary drawdown associated with the proposed artificial wetlands, including potential effects on neighbouring properties. The assessment conservatively considered:

- Operational groundwater inflow to Artificial Wetlands 1 and 3 from a potential shallow groundwater table within the peat.
- Temporary groundwater drawdown associated with construction dewatering to approximately 1 m below the basin inverts for Artificial Wetlands 1, 2 and 3.

The groundwater modelling is based on site-specific hydrogeological parameters derived from on-site testing (Section 3.4) and wetland design information provided by Maven (Appendix D).

4.2 Analytical Pit Modelling Methodology for Artificial Wetlands

The assessment 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 E).

The equations applied in this method assume laterally radial groundwater flows toward a circular pit. In applying this methodology to the proposed artificial wetlands, each wetland layout is simplified by converting to an equivalent circular pond. This has been done by taking the full footprint area of the wetland, including forebays and rain gardens, assuming the area is circular and calculating the radius of the “equivalent” area (Table 4). To assess inflows during the construction phase, the base of each artificial wetland has been modelled as dry, with active dewatering to 1 m below the invert. Each model used surrounding conservative shallow phreatic groundwater levels as the starting conditions to identify peak potential inflow rates. The methodology applied to assess the construction dewatering rates and extent of effects is generic and takes no account to the field techniques used to achieve the required dewatering.

For long-term scenarios, Artificial Wetlands 1 and 3 have been modelled with an operational ‘pit lake’ representing the potential intersecting shallow groundwater in the peat (Table 5). For Artificial Wetland 3, the proposed pit depth has been revised to 38.10 mRL, which extends the excavation below the interpreted base of the peat. WGA has nonetheless retained the existing pit model for this assessment because it accounts for the saturated peat sequence, while under-draining of the Hinuera Formation occurs, until the piezometric groundwater level is intersected at approximately 36 m RL. On that basis, the existing model is considered appropriate for estimating drawdown effects associated with the revised depth.

Table 4: Inputs for Construction Dewatering Pit Models

ARTIFICIAL WETLAND	WINTER GROUNDWATER LEVEL (mRL) ⁽¹⁾	DESIGN INVERT ELEVATION (mRL) ⁽²⁾	DEWATERED GROUNDWATER ELEVATION FOR DRY CONSTRUCTION (mRL) ⁽³⁾	DESIGN AREA AT RIM (m ²) ⁽²⁾	EFFECTIVE PIT RADIUS (m) ⁽²⁾
1	35.50	33.10	32.10	2,289	26.9
2	41.5	40.20	39.20	11,008	59.2
3	40.7 ⁽⁴⁾	38.10	39.2 (base of peat)	10,455	57.7

Notes:

(1) Based on conservative shallow phreatic surface (Figure 12).

(2) As provided by Maven (Drawing C440).

(3) As provided by Maven (Drawing C440).

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

(5) Extrapolation of phreatic surface in Peat in BH_152683, due to lack of data from “Latham’s Mountain” area of the site.

(6) Assumption all 3 artificial wetlands are unlined.

Table 5: Input for Long-Term Groundwater Inflow Pit Models

ARTIFICIAL WETLAND	WINTER GROUNDWATER LEVEL (mRL) ⁽¹⁾	DESIGN INVERT ELEVATION (mRL) ⁽²⁾	MAVEN DESIGNED PERMANENT OPERATIONAL WATER LEVEL (mRL) ⁽²⁾	DESIGN AREA AT RIM (m ²) ⁽²⁾	EFFECTIVE PIT RADIUS (m) ⁽²⁾
1	35.50	33.10	34.80	2,289	26.9
2	41.50	40.20	41.70 ⁽³⁾	11,008	59.2
3	40.70 ⁽⁴⁾	39.2 (saturated base of peat) occurs above invert 38.10	39.60	10,455	57.7

Notes:

(1) Conservative shallow phreatic surface (Figure 12).

(2) As provided by Maven (Drawing C440).

(3) Not assessed as only small inflows may occur from within engineered fill, due to PWL height above winter groundwater level.

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

(5) Extrapolation of phreatic surface in Peat in BH_152683, due to lack of data from "Latham's Mountain" area of the site.

(6) Assumption all 3 artificial wetlands are unlined and will act as recharge points to the peat.

4.3 Results

The projected groundwater inflow and drawdown effects have been modelled for Artificial Wetlands 1, 2 and 3, (as detailed in Appendix E) and the results summarised in Table 6. 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 basin construction. The radius of groundwater drawdown influence and the groundwater inflows are presented separately for construction (dry pit) and for long-term (operational) effects in Table 6.

The key points arising out of this drawdown assessment are:

1. The calculated drawdown relating to both the construction and the long-term operation of each wetland does not extend outward far enough to impact the nearest buildings (Table 6).
2. The calculated groundwater inflow for each of the scenarios considered is less than 12 m³/day (0.14 L/s). This flow rate is not expected to make a material difference to flows in any receiving drain, unless the drain was initially dry during the construction period.
3. The drawdown assessments were conducted using groundwater levels from the interpreted shallow phreatic surface as a starting condition to provide estimates of inflows and drawdown extents. Therefore, the calculated results are considered to represent conservatively large outcomes.
4. It is likely wetland construction would occur during a summer period when earthmoving operations and water management would be easier. The groundwater levels are reasonably expected to be lower during such periods, with correspondingly reduced requirements for groundwater drawdown and abstraction rates.

Table 6: Projected Effects of Proposed Artificial Wetlands

ARTIFICIAL WETLAND	SCENARIO	INITIAL SATURATED THICKNESS ABOVE BASIN FLOOR (m)	DEWATERING REQUIRED (m) ⁽³⁾	RADIUS OF DRAWDOWN INFLUENCE FROM PIT CENTRE (m) ^(1,2)	CALCULATED GROUNDWATER INFLOW (m ³ /day) ⁽²⁾	DRAWDOWN AT NEAREST BUILDING (m)	DISTANCE TO NEAREST BUILDING FROM PIT EDGE (m)
Wetland 1	Winter Construction Dry	2.4	3.4	47	9	0	28
	Winter Operational Groundwater Inflow	2.4	0.7	37	3	0	28
Wetland 2	Winter Construction Dry	1.3	2.3	73	12	N/A	N/A
	Winter Operational Groundwater Inflow	1.3	0	N/A	Not assessed. Only small inflows may occur below the permanent operational water level.	N/A	N/A
Wetland 3	Winter Construction Dry	1.5 (peat thickness)	1.5	67	8	0	37
	Winter Operational Groundwater Inflow	1.5 (peat thickness)	1.1	67	6.5	0	37

Notes:

(1) Radius of influence is considered to be defined by the distance that 0.25 m drawdown occurs from the pit centre. Effective pit radius from pit centre is shown in Table 5. Therefore, drawdown from pit edge can be calculated by the effective pit radius subtracted from the radius of drawdown influence.

(2) Drawdown extents and inflows do not take account of changes in permeability such as Walton Subgroup sediments underlying the Hinuera Formation. Assumption all 3 artificial wetlands are unlined and will act as recharge points to the peat.

(3) Winter Construction for dry conditions, includes 1 m dewatering below invert.

5 GROUNDWATER MOUNDING FROM ARTIFICIAL WETLANDS

5.1 Introduction

Stormwater runoff from much of the Industrial Development is to be directed to the three proposed artificial wetlands. Of these, Artificial Wetland 3 is proposed to be excavated through the peat such that the invert is underlain by the Hinuera Formation. In the scenario where a phreatic surface within the peat does not exist, Wetland 3 has the potential to increase groundwater levels to approximately 3.6 m above the deeper piezometric surface (36 m RL Figure 11). Should this occur, there is potential for groundwater mounding beneath the basin invert. Due to the proximity of proposed Artificial Wetland 3 to neighbouring buildings and the railway, groundwater mounding simulations have been undertaken to evaluate potential groundwater table mounding in the surrounding area. These simulations are intended to reflect wetland operation during a storm event or continuous wet period. The analysis methodology and results are summarised in this section, with the detailed analyses presented in Appendix F.

WGA considers that Proposed Artificial Wetlands 1 and 2 do not require similar assessments as:

- Proposed Artificial Wetland 2 has no current infrastructure of concern in the close vicinity.
- Proposed Artificial Wetland 1 is unlikely to have an unsaturated zone beneath it that would exceed natural groundwater fluctuations. Additionally, Artificial Wetland 1 directly neighbours a drain that reduces any risk of mounding impacts.

Artificial Wetland designs have been provided by Maven (2026). A copy of the design drawings for the proposed basins is presented in Appendix D.

5.2 Methodology for Mounding Assessment on Artificial Wetland 3

Mounding assessments documented in this report have been performed using MOUNDSOLV software package, Version v3.0, developed by HydroSOLVE, Inc. MOUNDSOLV calculates the transient response of an unconfined groundwater table beneath a rectangular recharge source to a defined recharge event. The package applies a simulation methodology published by Zlotnik et al. (2017) for this purpose. The use of the MOUNDSOLV package is widely accepted by professional hydrogeologists for the assessment of groundwater mounding.

For the mounding assessments undertaken at proposed Artificial Wetland 3, two scenarios were modelled:

1. Simulation of an extreme continuous recharge period of one day.
2. Simulation of a prolonged continuous recharge event of 30 days with a 335-day recovery period.

Assumptions applied in each simulation with respect to the aquifer characteristics for the transient solutions are:

- Aquifer properties (horizontal hydraulic conductivity, specific yield, and dip) are homogeneous and isotropic and remain constant over time.
- Saturated thickness of the aquifer is uniform and constant prior to the start of the recharge event.
- There is no potential confining layer above the receiving aquifer.
- The infiltration rate over a rectangular recharge source is uniform and constant.
- The aquifer base has a constant slope.

For the purposes of modelling both short term and long-term scenarios, it has been assumed that water levels within the basin do not exceed the permanent water level (39.60 m RL Maven Drawing C440).

The average number of rain days through the winter months of a year recorded by the long-term climate station at Hamilton (NIWA Hamilton, AWS rainfall station 2122) is approximately 30. The 30 day model incorporates an assumption that all the average winter rainfall/inflow occurs within one continuous period. This is a conservative basis for modelling, as it assumes a high frequency of rain events and does not allow time for ponded water levels to drop between rainfall events. In reality, water levels will fluctuate, and the ponded water will often drain down between rainfall events, resulting in less mounding.

The interpreted winter phreatic surface (Figure 12) is at or above each of the floor elevations for the planned wetlands. This feature implies that it is likely the wetlands will contain water continuously through the winter months, even if there are extended periods of no rain.

The infiltration rate applied to each model is the rate that results in the mounding reaching the permanent water level (overflow invert elevation provided by Maven drawing C440) by the end of the storm event/wet period. In considering the scenarios, the infiltration rates would initially be high. However, these rates will drop off substantially as the groundwater mounding starts to reach the floor of the basin. Consequently, the Scenario 2 infiltration rates are much lower than the Scenario 1 (one-day event) infiltration rates.

Artificial Wetland 3 is proposed to have an invert level of 38.10 m RL. On that basis, this wetland would intersect the Hinuera Formation, as indicated by the Hinuera Formation intersected in the nearby borehole (BH_152683). Therefore, the hydraulic conductivity value for the Hinuera Formation (Table 2) has been used for the Artificial Wetland 3 mounding simulations. This focus on the soakage acceptance potential for the saturated aquifer results in a conservatively low projection of recharge rate.

5.3 Mounding Assessment Results

Results for the two modelling scenarios are reported in Appendix F and summarised in Table 7. Overall, the modelling predicts that mounding induced by Artificial Wetland 3 reduces with increasing distance from the centre of the wetland, with substantially reduced effects predicted at nearby infrastructure:

- The highest modelled mounding occurs beneath Artificial Wetland 3, at 3.6 m for Scenario 1 and 3.6 m for Scenario 2.
- During the 335-day recovery period modelled for Scenario 2, mounding at Artificial Wetland 3 reduces to 0.23 m.
- Figure 14 shows that peak mounding occurs at about day 50, following the 30-day wet period. Maximum modelled mounding at nearby infrastructure remains below 1 m in all reported cases for the 30-day wet period and recovery scenario. While localised groundwater level increases may occur, the predicted mounding is comparable to naturally occurring groundwater level fluctuations within the observed groundwater system and is not anticipated to materially change groundwater conditions at neighbouring infrastructure.

Table 7: Summary of Mounding Assessment Results for Artificial Wetland 3

MODEL OUTPUT	SCENARIO 1: 1 DAY EVENT	SCENARIO 2: 30 DAY WET PERIOD	SCENARIO 2: 335 DAY RECOVERY ⁽¹⁾	NOTES
Modelled Infiltration Rate (m/day)	0.72	0.04 (total of 1.2 m over 30 days)		Modelled with water levels in the basin reaching the permanent water level (outflow point).
Modelled Mounding at Artificial Wetland 3 (m)	3.6	3.6	0.23	
Maximum Modelled Mounding at Neighbouring Infrastructure (m)	< 0.01	0.49	0.21	Northern Building
	< 0.01	0.39	0.21	Railway Line
	< 0.01	0.14	0.19	Eastern Building

Notes:

(1) This residual mound should not be interpreted as cumulative between seasons (i.e. successive increases of ~0.2 m), but rather as a conservative representation using MOUNDSOLV software. Refer to Appendix F for a more detailed explanation.

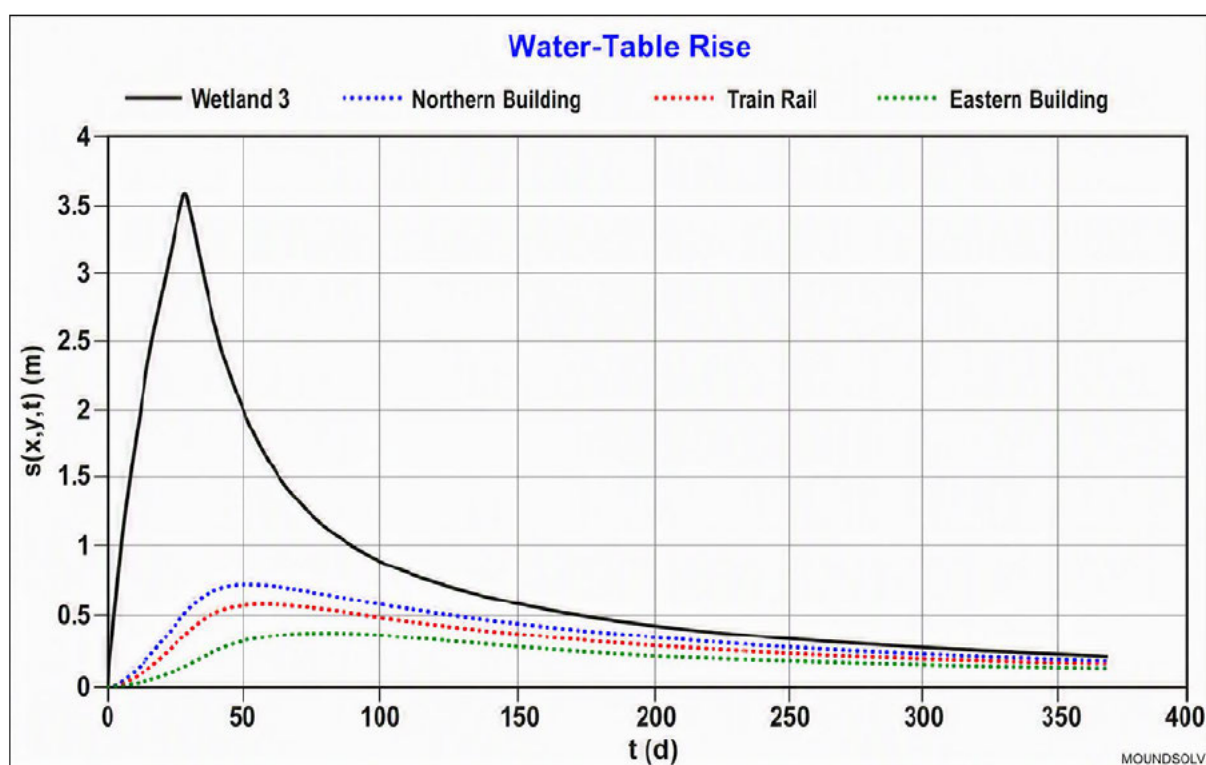


Figure 14: Groundwater Hydrographs from Artificial Wetland 3 in Response to 30 Day Event

6 GROUNDWATER DRAWDOWN FROM WASTEWATER AND DRAIN INFRASTRUCTURE

6.1 Introduction

WGA has modelled the construction dewatering, drawdown and inflow assessments for proposed wastewater pipelines, wastewater pump stations and drains or swales. The modelling used hydrogeological parameters from on-site testing (Section 3.4) and the design provided by Maven for wastewater pipeline design locations and depths and wastewater pump station locations (Appendix D and Appendix G).

The assessments have been undertaken assuming existing hydraulic properties of the peat. Where infrastructure is located within areas subject to peat pre-loading, these assessments are considered conservative, as peat compaction is expected to reduce hydraulic conductivity and limit groundwater inflows during excavation. While peat compaction may result in localised changes to shallow groundwater levels, including potential increases in groundwater level relative to ground surface, these changes are not expected to result in materially greater adverse effects than those assessed, given the temporary nature of dewatering activities. Wetland areas (assessed in Section 3) are not currently proposed to be pre-loaded and have therefore been assessed assuming existing peat conditions.

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 or within proposed fill (Maven drawing no. C600 WS and C680 WS). If extraordinary wet conditions require dewatering, any dewatering effects will be less than those required and assessed in Section 5.3 for wastewater pipelines. Drawdown Effects of Wastewater Pump Station Installation

6.1.1 Construction

Two wastewater pumping stations (WWPS) are planned for construction at the Industrial Development. Engineering designs are provided in Appendix G. Consequently, WGA has made the following assumptions to support an assessment of the dewatering required to support construction of each WWPS and the associated drawdown:

- The **Northern WWPS B**, planned near Artificial Wetland 1 (Figure 3), requires excavation to a depth of **6 m** below proposed ground level and a width of approximately **2.5 m**.
- The **Central WWPS A** (Figure 3), requires excavation to a depth of **6.5 m** below proposed ground level and a width of approximately **3.1 m**.
- Both excavations assume:
 - An additional 1 m dewatering radius for bore offset distance from excavation width. Hence, Northern WWPS B requires dewatering to a radius of 3.5 m and Central WWPS A requires dewatering to radius of 4.1 m.
 - 0.5 m of dewatering will be achieved below the excavation invert for construction purposes.
 - Respective depths should ensure excavation of the peat beneath the wet well. Should peat be encountered at greater depths, further excavation and installation of engineered fill or a drainage blanket may be required to stabilise the peat beneath the wet well floor for construction purposes (Maven pers comm 26 May 2026).
 - Any additional excavations required for WWPS infrastructure have been designed to have dewatering requirements similar to, or less than, the trench dewatering requirements assessed in Section 6.2 and Table 9, based on the design information provided in Appendix G.

Earthworks associated with both wastewater pumping stations will involve excavation below the groundwater table. Therefore, it has been assumed that a shield pile wall will be used to support the walls of each excavation, to minimize the excavation size and the associated groundwater dewatering required. The shield pile wall is expected to minimise horizontal groundwater inflows to the excavation from the shallowest lithologies. As the shield pile wall is expected to cut off the shallow phreatic surface within the peat, dewatering for the wet well excavation has focused on the Hinuera Formation. Therefore, the required drawdown is assessed with reference to the deeper piezometric surface beneath the excavation.

WGA has assumed that the dewatering at the base of the shield pile wall would be achieved using a single dewatering bore outside the pile wall. This bore would induce drawdown primarily from the underlying Hinuera Formation, drawing groundwater from beneath the shield piles and excavation base, although a delayed reduction in pressure within the shallow peat groundwater should also be expected.

For the purposes of this assessment, it is assumed that installation of the wet well will require 10 days until the well is fully sealed and active dewatering can cease. For groundwater drawdown assessment purposes, dewatering has been modelled to be achieved within three days. The targeted dewatering depth is assumed to increase through the 10 day dewatering and construction period, consistent with the progressive increase in the excavation depth.

6.1.2 Shallow Aquifer Dewatering During Wet Well Excavation

The excavation dewatering has been assessed using the published Hantush-Jacob function to calculate drawdown from a pumping well under leaky aquifer conditions. The assessment accounts for a leakage coefficient using the site specific vertical hydraulic conductivity of the peat ($K_v 4.48 \times 10^{-8}$ m/s, using an anisotropy factor of 10) and the horizontal hydraulic conductivity of the silty Hinuera Formation ($K_h 2.79 \times 10^{-5}$ m/s). The geometric input parameters for the assessments are summarised in Figure 15 and Figure 16.

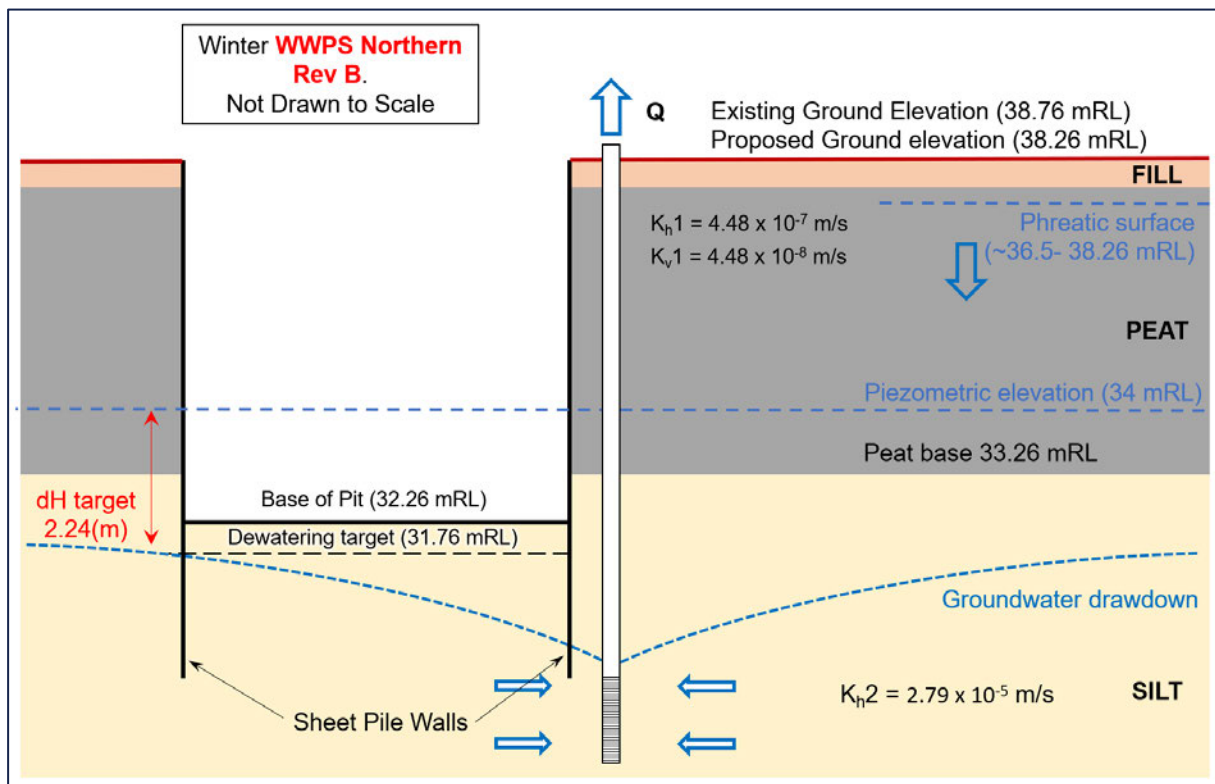


Figure 15: Conceptual Pit Dewatering Model for Proposed Northern WWPS B

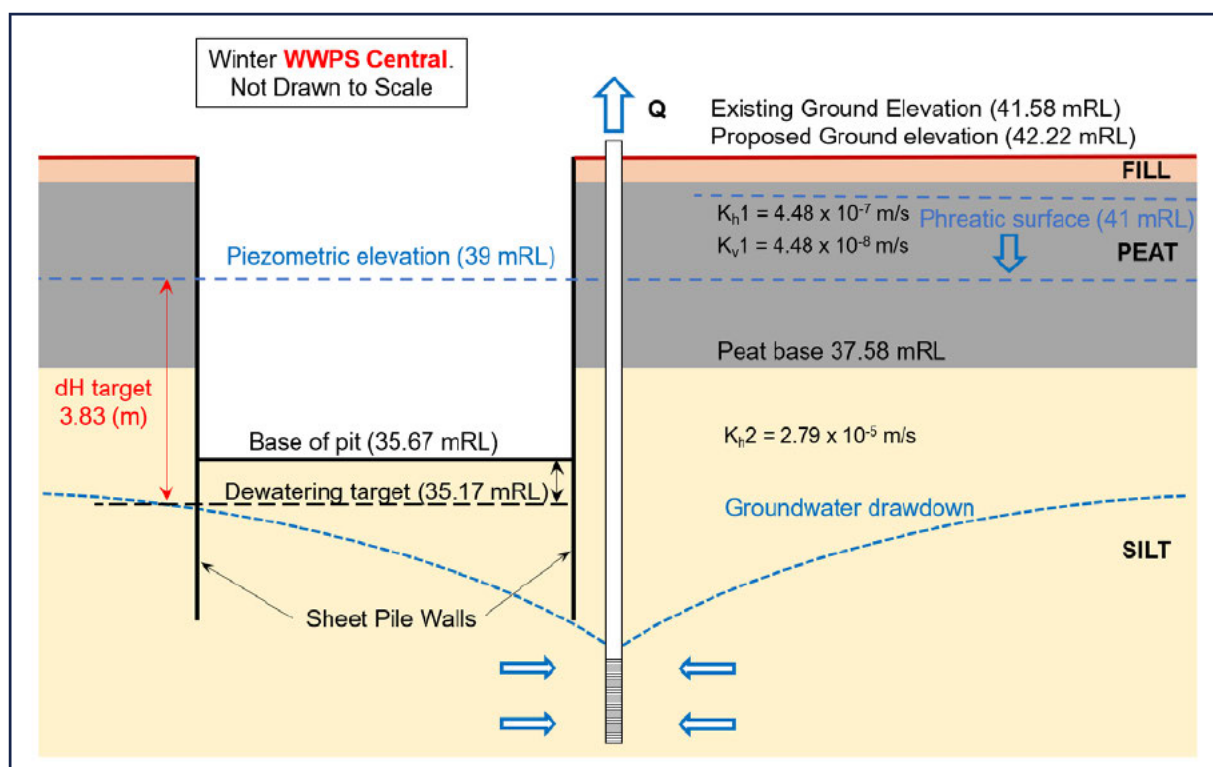


Figure 16: Conceptual Pit Dewatering Model for Proposed Central WWPS A

The dewatering model applied in this assessment functions on a transient basis. The pumping rate identified in Table 8 is defined to achieve the dewatering target within three days. After that time, the pumping rate would be reduced to a rate sufficient to simply maintain the level of drawdown through the wet well construction period. On that basis:

1. The pumping rate required has been calculated based on achieving the drawdown target within three days.
2. The calculated drawdown effects in Table 8 have been assessed based on an assumed constant flow rate that would achieve the targeted drawdown over 10 days.

Table 8: Projected Effects of Wet Well Dewatering

PARAMETER	NORTHERN WWPS B	CENTRAL WWPS A
Static groundwater elevation (mRL)	34	39
Targeted drawdown (m) ⁽¹⁾	2.24	3.83
Pumping rate to achieve dewatering within three days (L/s)	1.65	2.97
Pumping rate to achieve dewatering within 10 days (L/s)	1.39	2.42
Calculated radius of influence after 10 days pumping (m) ⁽²⁾	57	78
Distance to nearest building (m)	100	48
Calculated drawdown at nearest building after 10 days (m) ⁽²⁾	0.06	0.66

Note:

(1) Targeted drawdown required at a 1 m offset distance from excavation width. Hence, Northern WWPS B requires dewatering to a radius of 3.5 m and Central WWPS A requires dewatering to radius of 4.1 m.

(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.

6.2 Drawdown Effects of Wastewater Pipeline, Drain and Swale Construction.

6.2.1 Trench Modelling Methodology

Construction of the planned wastewater pipeline network will require trench excavations, some of which may extend below the shallow water table within the peat (Appendix D). There is a construction requirement that these trenches are maintained in a dry state, up to 0.5 m below the invert, until backfilling starts. WGA have assumed that the wastewater pipeline is likely to be constructed in 20 m sections, and that each section will be completed within a period of 10 days. This guidance 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).

WGA also understand there are proposed roadside drains to be constructed across the site, which are anticipated to have an invert depth of 1 m bgl. The same approach of trench dewatering for pipelines will be applied to consider the construction of roadside drains, and additionally, an assessment of the potential shallow groundwater through-flow anticipated during operation.

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

Drawdown with respect to time and distance: $s(x, t) = s_0 \cdot \operatorname{erfc} \left(\frac{x}{2\sqrt{Tt/S}} \right)$

Inflow rate with respect to time: $Q_{TOTAL}(t) = 2s_0 \sqrt{\frac{TS}{\pi t}}$

Where:

$s(x, t)$	=	Drawdown at a defined distance from the trench after a defined time
s_0	=	Drawdown at the trench
t	=	Time since start of drawdown
T	=	aquifer transmissivity
S	=	aquifer storativity (equivalent to specific yield)
erfc	=	complementary error function
Q	=	total inflow to a one metre length of trench at a defined time drainage initiated

WGA understands that due to planned placement of engineered fill covering some areas of the site, the trenches will be constructed at varying depths with respect to existing ground levels across the Site. This means the trenches will either be excavated wholly within engineered fill or through any emplaced fill into the shallow peat. WGA has completed the following conservative scenarios for the drawdown of the shallow groundwater table within the peat (Table 9):

1. A section of 20 m trench for a 4.55 m bgl pipeline at the design level indicated for manhole inlet 1 (MH Inlet 1 as shown on Maven Design long sections drawn April 2026, Appendix D), including dewatering to 0.5 m below the drain invert, impacting the shallow phreatic groundwater table within the peat.
2. A section of 20 m trench for a 1 m bgl roadside drain (or swale) at the design ground levels (below the phreatic surface) at manhole 15 (MH 15 as shown on Maven Design long sections drawn April 2026 Appendix D), including dewatering to 0.5 m below the invert, impacting the shallow phreatic groundwater table within the peat.
3. A section of 20 m trench for a 1 m bgl roadside drain (or swale) at the design ground levels (below the phreatic surface) at manhole 15 (MH 15 as shown on Maven Design long sections drawn April 2026 Appendix D), impacting the shallow phreatic groundwater table within the peat on a permanent basis.

These locations were chosen for assessment of potential groundwater drawdown as representing the largest potential interaction between the planned trenches and the shallow phreatic groundwater table within the peat.

Table 9: Input Parameters for Trench Analytical Modelling

TRENCH FOR PIPELINE OR DRAIN/SWALE	HYDRAULIC CONDUCTIVITY (m/s)	AQUIFER SPECIFIC YIELD	NUMBER OF DAYS TRENCH OPEN	TRENCH DRAWDOWN REQUIRED (m)
Scenario 1 – Pipeline Construction	4.48×10^{-7}	0.2	10	3.91
Scenario 2 – Drain/Swale Construction				1.5
Scenario 3 – Drain/Swale Operation				1

6.2.2 Wastewater Pipeline Analytical Modelling Results

The extent of project drawdowns arising from the trench scenarios described in Section 6.2.1 are summarised in Table 10. A minimum drawdown criterion of 0.25 m, indicating the distance of influence, has been applied. Drawdown less than this value is within the expected range of uncertainty of the analysis method and is within the expected variability of groundwater levels through the winter season.

Table 10: Projected Distance of Influence from Trench or Drain Construction

TRENCH FOR PIPELINE OR DRAIN/SWALE	0.25 M DRAWDOWN DISTANCE OF INFLUENCE (m)
Scenario 1 – Pipeline Construction	9
Scenario 2 – Roadside Drain/Swale Construction	6
Scenario 3 – Roadside Drain/Swale Operation	5

The drawdown curves relating to modelled scenarios (Figure 17, Figure 18 and Figure 19) show a relatively rapid decline in drawdown within the first 6 m to 9 m distance from the trench. Beyond that distance, the drawdown drops quickly to a level that would be effectively non-detectable. The shorter drawdown distances predicted for the Industrial Development reflect the lower hydraulic conductivity of the peat-dominated shallow groundwater system assessed in this area, compared with the more permeable Hinuera Formation sediments that control groundwater responses across much of the Residential Development.

The projected groundwater inflow rates to the simulated 20 m sections of trenches over time (Figure 20 to Figure 22) decline over time as the area of groundwater drawdown expands. The initial calculated inflows are likely to be overestimates as the equation applied assumes the 20 m trench is constructed instantaneously to its full depth.

At this stage, the number of trenching teams that may be operating concurrently has not been confirmed, and the cumulative discharge rate during construction therefore remains uncertain (see Section 7.2). The trench scenarios assessed have been selected from the Maven designs to represent locations with the greatest likely interaction with groundwater; while some overlap of drawdown effects may occur where multiple trenches are constructed at the same time, other sections of the alignment are expected to require less dewatering due to shallower excavation depths or reduced groundwater interaction. Concurrent trenching activities may increase the total groundwater abstraction rate during construction but are not expected to materially alter the nature or extent of the groundwater drawdown effects assessed in this report. Accordingly, cumulative groundwater effects associated with multiple trenching crews operating simultaneously are not considered to be a material issue and do not alter the conclusions of this assessment.

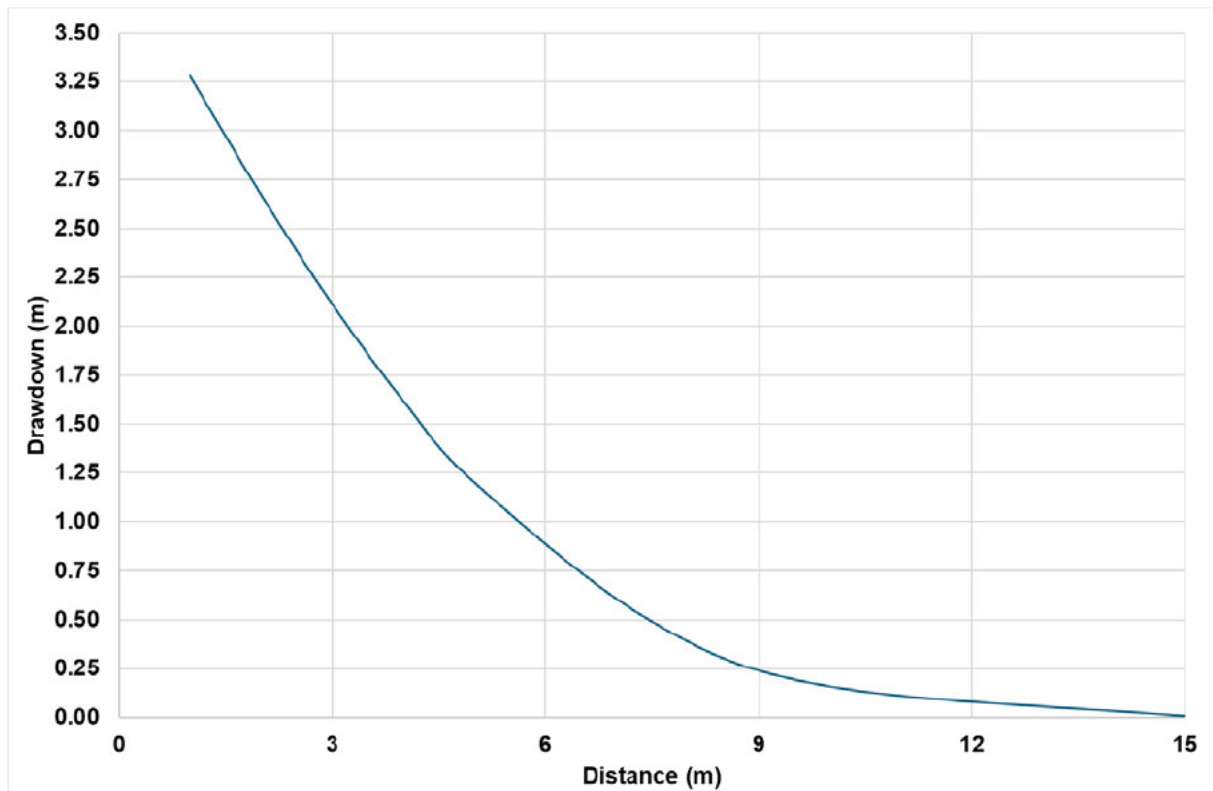


Figure 17: Scenario 1 Projected Drawdown from Wastewater Pipeline Installation

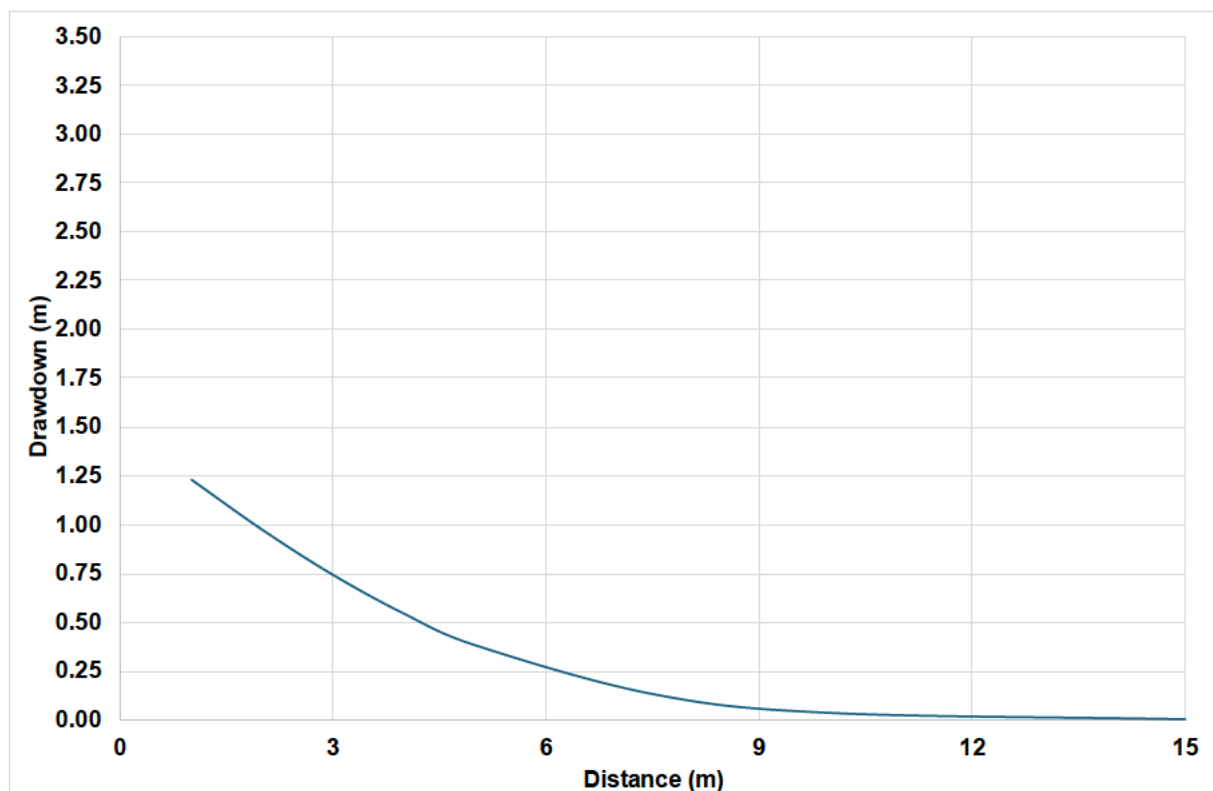


Figure 18: Scenario 2 Projected Drawdown from Drain/Swale Installation

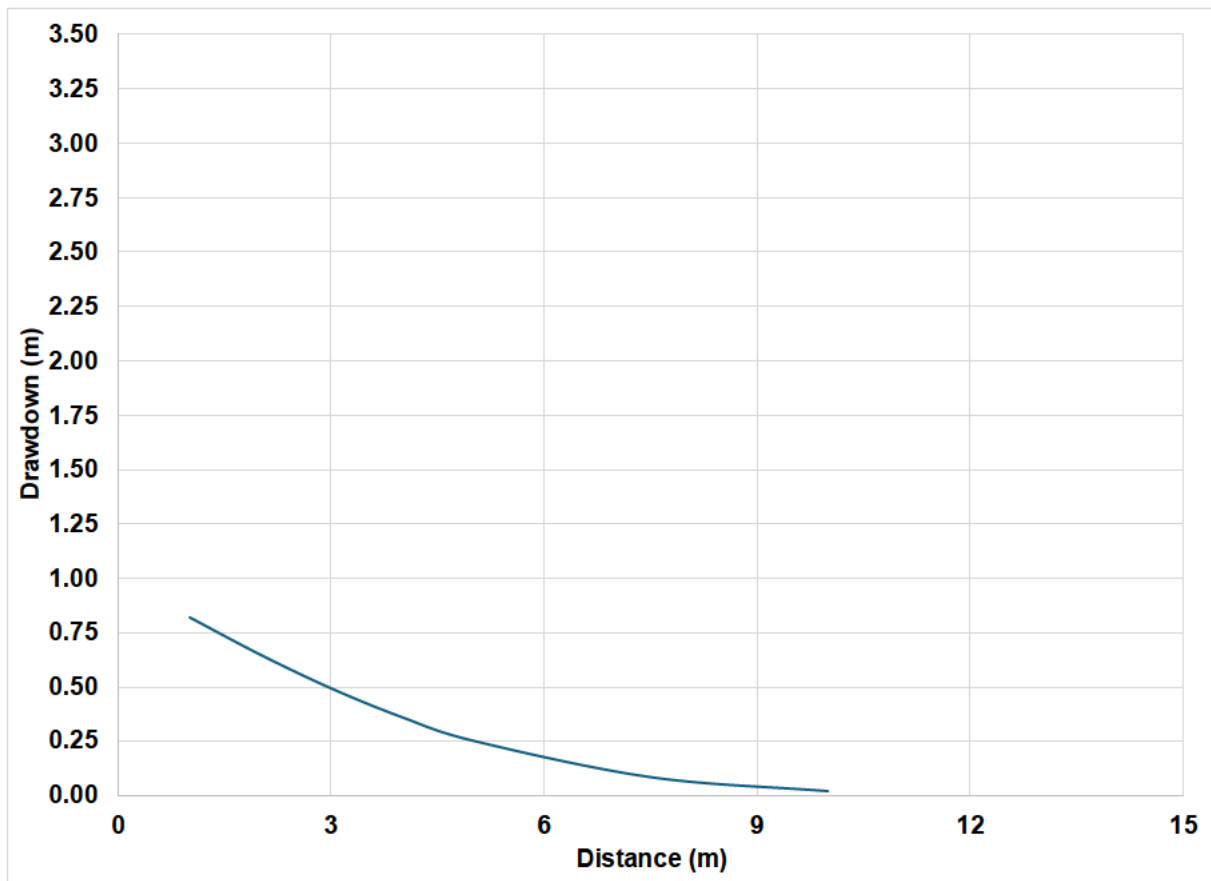


Figure 19: Scenario 3 Projected Long-Term Drawdown from Drain/Swale Installation

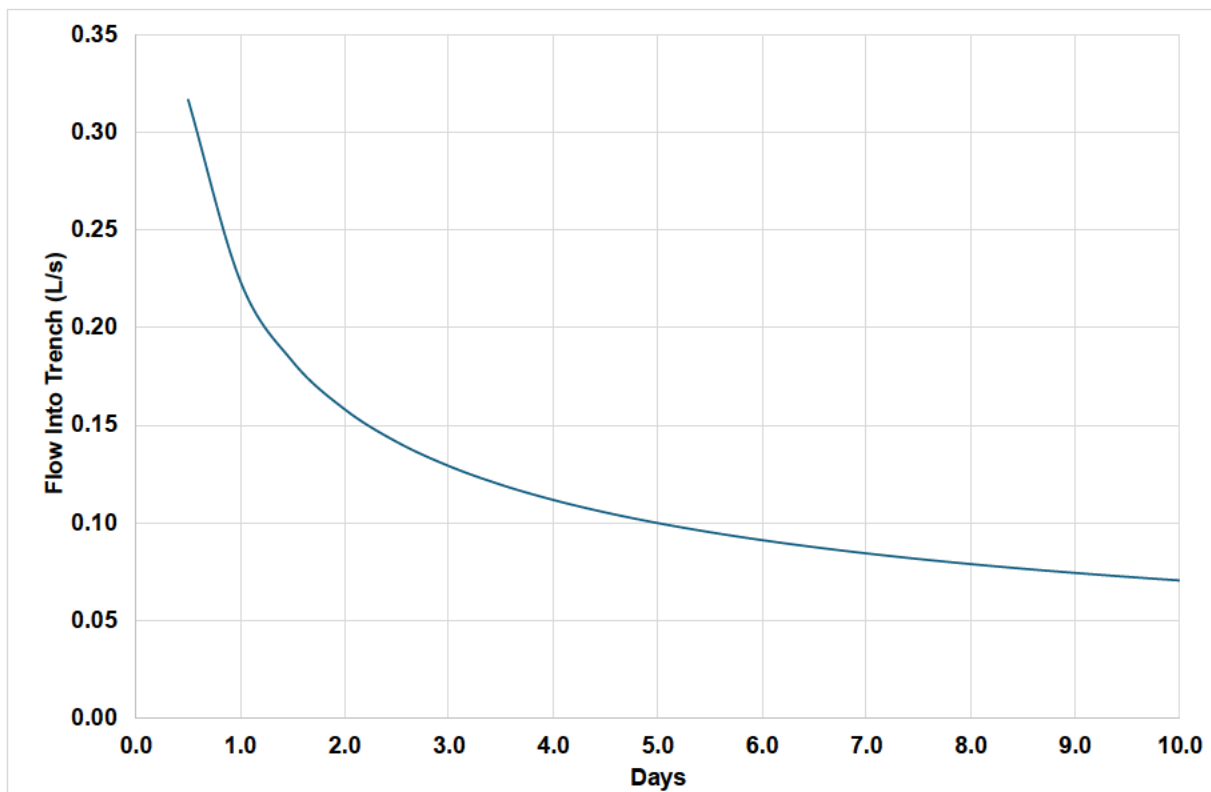


Figure 20: Scenario 1 Projected Groundwater Inflows Over Time into a 20 m Trench

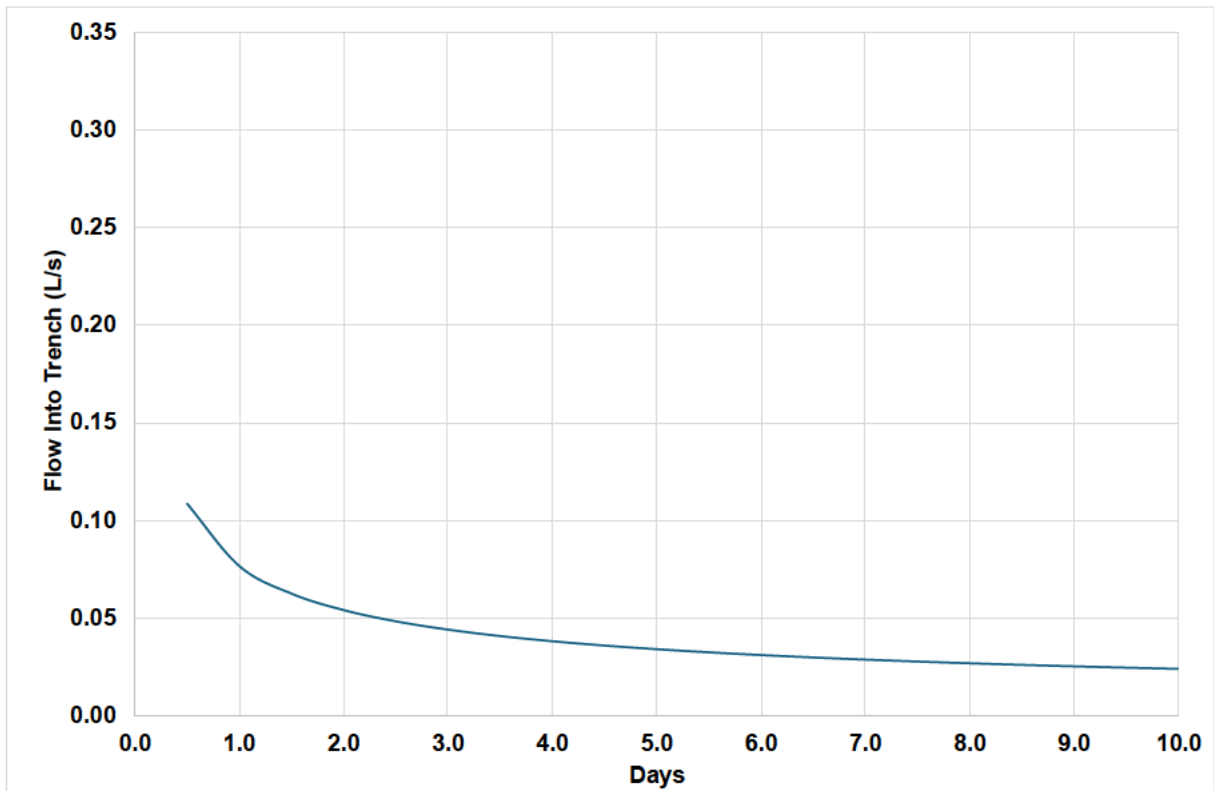


Figure 21: Scenario 2 Projected Groundwater Inflows Over Time into a 20 m Trench

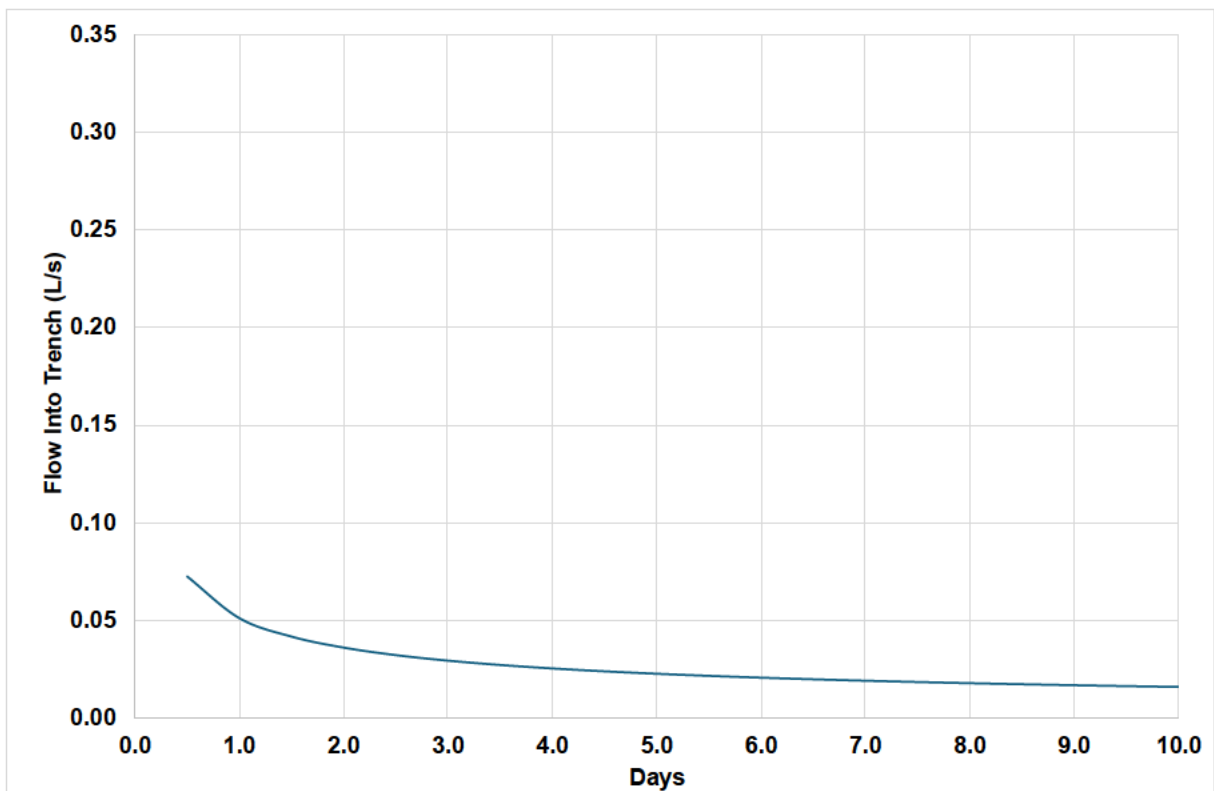


Figure 22: Scenario 3 Projected Groundwater Inflows Over Time into a 20 m Trench

7 GROUNDWATER EFFECTS ASSESSMENT

7.1 Effects Overview on Catchment Water Balance

The proposed Industrial Development includes the following features relevant to this hydrogeological assessment:

- Placement of engineered fill on top of peat across much of the site.
- Pre-construction pre-loading of peat in areas of planned roading and associated infrastructure to help stabilise the ground surface in these areas (as described in detail in the CMW geotechnical report for the Industrial Development (2026)).
- Construction of wastewater infrastructure and stormwater management features.

These activities have the potential to influence shallow groundwater conditions locally during construction and over the long term. However, no long-term adverse effects on the catchment-scale groundwater balance are anticipated.

Groundwater conditions across the Site are characterised by two hydrogeologically distinct surfaces: A shallow phreatic surface within the peat and a deeper piezometric surface within the underlying Hinuera Formation. Recharge to the shallow groundwater system is primarily rainfall-driven and will continue following development, although redistributed spatially through engineered drainage, fill placement, and stormwater management features. Groundwater flow across the Site is currently generally directed toward bordering drains, which act as key discharge features, particularly during wetter conditions. The proposed development is not expected to alter this overall flow regime or materially affect discharge to these drains.

Peat compaction associated with pre-loading is expected to reduce the storage capacity and hydraulic conductivity of peat soils within treated areas. As a result, groundwater inflows to excavations and trenches during construction are likely to be less than those predicted using pre-compaction hydraulic parameters. The groundwater assessments presented in this report have been undertaken assuming existing (pre-compaction) peat conditions, providing a conservative representation of potential dewatering demands, drawdown effects, and groundwater interactions.

Localised groundwater mounding beneath Artificial Wetland 3 has also been assessed (Section 4). While mounding is predicted to occur immediately beneath the wetland during recharge events, the magnitude of mounding attenuates rapidly with distance and does not represent a net increase in groundwater storage at the catchment scale. Instead, this process reflects a redistribution of recharge within the shallow groundwater system.

Overall, the development is not expected to result in a measurable long-term change to the catchment water balance.

Note: not drawn to scale

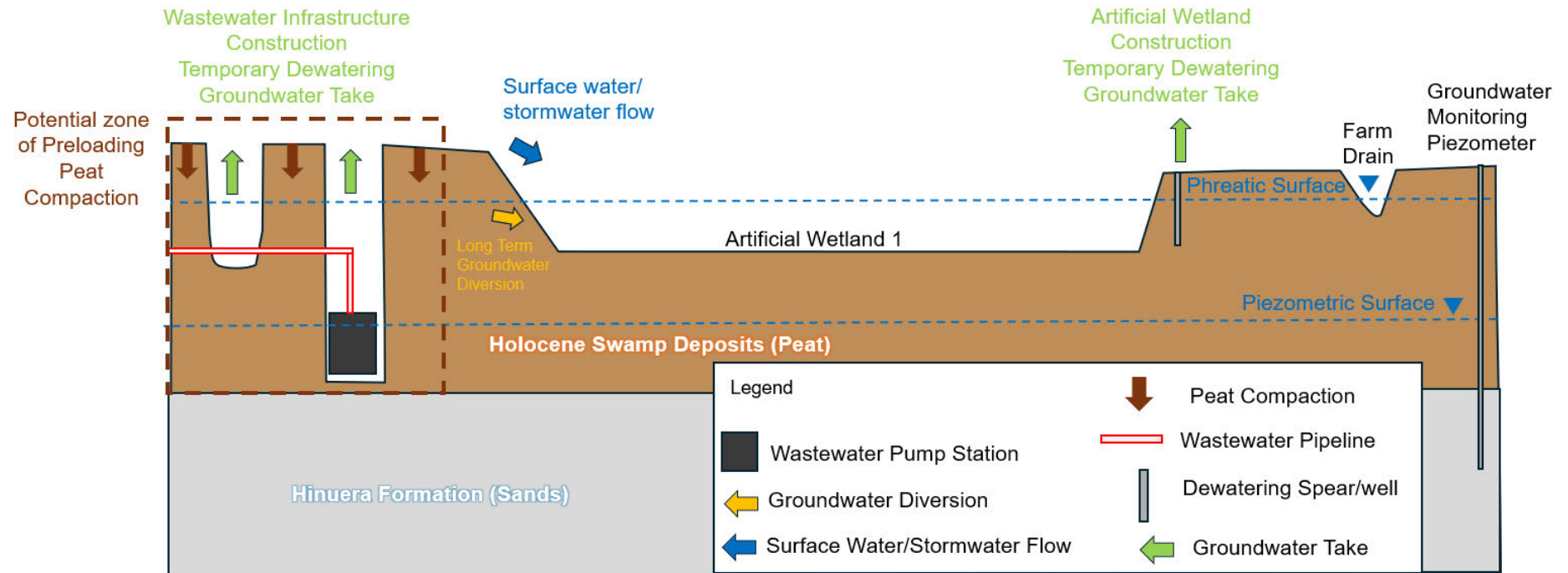


Figure 23: Conceptual Schematic of Water Interactions of the Combined Proposed Activities

7.2 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 11). Conservative dewatering assessments were undertaken by considering the groundwater table that exists at the phreatic surface (approximately 0.5 m bgl in some areas). Based on current design and staging, the largest construction groundwater abstraction is expected to occur between **8 m³/day to 12 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 for the Industrial Area. The smaller dewatering requirements and drawdown extents predicted for the Industrial Development reflect the lower-permeability peat-dominated groundwater system present across the site, compared with the more permeable Hinuera Formation aquifer assessed within much of the Residential Development.

Wastewater infrastructure may be planned to be constructed progressively or overlapping, based on the installation of consecutive 20 m lengths of pipeline in similar dewatered trench sections. The WWPS's will be constructed separately, with construction schedules that will probably overlap the construction schedule for the pipelines. Based on the proposed design, the largest individual groundwater abstraction to enable for wastewater infrastructure construction is expected to occur for Central WWPS A (**209 m³/day** Table 8).

Similarly, the largest abstraction for pipeline and drain installations is expected to occur for the deepest sections, at **< 0.5 L/s** (Figure 20 to Figure 22). 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 6.2 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 **100 m³/day** is proposed.

Table 11: Summary of Construction Dewatering Flow Rates and Operational Cumulative Groundwater Diversions

EXCAVATION TYPE	TOTAL INFLOW (WINTER) CONSTRUCTION (m ³ /day) ⁽¹⁾	TOTAL INFLOW (WINTER) OPERATIONAL GROUNDWATER DIVERSION (m ³ /day)	CUMULATIVE OPERATIONAL GROUNDWATER DIVERSION (m ³ /day)
See Section 4			
Artificial Wetland 1	9	3	9.5
Artificial Wetland 2	12	N/A	
Artificial Wetland 3	8	6.5	
See Section 6			
Northern WWPS B	119	N/A	
Central WWPS A	209		
See Section 6			
20 m Trench for Wastewater Pipeline	< 28	N/A	N/A
20 m Trench for Drain/Swale Construction	< 10	< 6	

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 3 to 6. Those listed with "<" are the max starting rates for dewatering analyses.

7.2.1 Effects on Nearby Bores

Of the surrounding groundwater users, there are only four current groundwater take consents within 1 km of the Site (Table 3 and Figure 13). The furthest radial extent of dewatering impacts from proposed Site activities assessed in this report did not extend to the distance of any of the existing groundwater takes.

Of the groundwater bores identified within the predicted drawdown area, bore 72_1349 is located within the Industrial Development footprint and is expected to be removed as part of the proposed works. Bore 72_13192 is WGA monitoring Piezometer P1 and may be utilised to verify groundwater drawdown predictions during construction. As neither bore represents a neighbouring third-party groundwater user, no adverse effects on existing groundwater takes are anticipated. The remaining groundwater bores and consented takes are located beyond the predicted extent of drawdown and are therefore not expected to be affected by the proposed works (Table 8 and Table 10). Effects on Nearby Surface Water Features and Wetlands.

There are no RMA-qualifying wetlands across the Site (EcoLogical 2025) and no valuable watercourses have been identified (EcoLogical 2026). No substantial changes to downstream hydrology are expected. Modelled groundwater mounding beneath Artificial Wetland 3 is not predicted to extend to existing surface water features or site drainage channels at levels that would alter surface water flow regimes or create new areas of groundwater emergence.

Overall effects are assessed to be temporary and minor.

7.2.2 Effects on Ground Subsidence

The effect of the conservative drawdown associated with construction dewatering has the potential to induce ground subsidence, particularly where drawdown occurs within compressible near surface soils. For the proposed construction of wastewater pipelines and construction and operation of proposed drains, it is not anticipated that there will be any seasonal groundwater levels temporarily lowered beyond naturally experienced fluctuations. For the construction dewatering required for Artificial Wetlands, the calculated radius of influence of 0.25 m drawdown occurs between 10 m to 20 m maximum distance from Artificial Wetland edges causing no impact to nearby buildings. For WWPS installation, the calculated radius of influence occurs at 57 m in the Northern WWPS B and 78 m in the Central WWPS A (Table 8). Localised groundwater drawdown results (0.06 m and 0.66 m respectively) are comparable to naturally occurring groundwater level fluctuations within the observed groundwater system.

It is likely construction would occur during a summer period when earthmoving operations and water management would be easier. The groundwater levels are reasonably expected to be lower during such periods, with correspondingly reduced requirements for groundwater drawdown and abstraction rates.

Modelled groundwater mounding associated with Artificial Wetland 3 is generally less than 1 m at nearby infrastructure and, given its temporary nature within the piezometric groundwater levels it is also comparable to naturally occurring groundwater level fluctuations within the observed groundwater system.

The groundwater drawdown and mounding predictions presented in Sections 4, 5 and 6 have been provided to CMW during their assessment of potential settlement and other geotechnical effects associated with the proposed construction of the Industrial Development. 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, 2026). Anticipated Implications for Water Quality.

The change in land use from farming (in part) to industrial activity, together with the introduction of artificial wetlands, is expected to reduce nutrient inputs to shallow groundwater. However, measures such as bunding and other site controls will be required to prevent industrial contaminants from entering drainage systems and surrounding water catchments. In summary, while construction activities may temporarily increase sediment-related risks, for the Industrial Development, these risks will be managed through the implementation of the proposed Erosion and Sediment Control Plan (ESCP).

A potential localised source of contamination is present at “Latham’s Mountain”, an area of historic uncontrolled fill comprising building and demolition waste located in the north-eastern portion of the Site (Section 3.5). Contaminated land investigations identified asbestos-containing material (ACM) fragments within the fill, with contamination considered heterogeneous and associated with discrete waste materials rather than widespread soil contamination. WGA understands that the base of the proposed Artificial Wetland 3 will be constructed following removal of existing fill materials and will extend into the underlying natural lithology, thereby reducing the potential for ongoing contaminant interaction with shallow groundwater. In addition, the Latham’s Mountain area is proposed to be capped in accordance with the geotechnical design (CMW 2026), which is expected to limit infiltration through residual fill and reduce the risk of contaminant mobilisation. On this basis, the potential for contaminated stormwater recharge to groundwater is considered low.²

² Any effects are expected to relate primarily to legacy infiltration that may have occurred prior to development rather than ongoing inputs.

8 MONITORING AND MITIGATION

8.1 Groundwater Monitoring

Groundwater levels across the Industrial Area have been characterised using a network of four piezometers installed and monitored during January 2026. This monitoring period captured both summer groundwater conditions and, combined with site observations, confirmed the presence of a shallow phreatic surface overlying a deeper piezometric surface. The monitoring data, combined with observations from Residential Area monitoring bores, were sufficient to define potential seasonal groundwater level variability 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 bores. The long-term groundwater diversion to the artificial wetlands represents a redistribution of groundwater discharge within the Industrial Development site catchment rather than a net loss.

Based on the investigations, monitoring and assessments completed for this report, permanent post-construction monitoring is not considered necessary.

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 2026).

Given the predicted localised nature of groundwater mounding beneath Artificial Wetland 3, no specific monitoring of mounding effects is considered necessary beyond the general groundwater monitoring approach (as detailed above).

8.2 Surface Water Monitoring

To recap, surface water features within and downstream of the Industrial Area consist of farm drains that, during wet periods, are likely to discharge to the boundary drains and the headwaters of the Mangakotukutuku catchment. This is consistent with the catchment boundary reported by EcoLogical (2025), which places the Site within the Mangakotukutuku Stream catchment.

Potential effects on surface water are limited, as no RMA-qualifying wetlands have been identified on the Site (EcoLogical 2025) and no valuable watercourses have been identified (EcoLogical 2026). No substantial changes to downstream hydrology are expected, and overall effects are assessed as temporary and minor.

Visual inspections of drains and streams during construction, as part of standard environmental compliance checks, are therefore considered sufficient to confirm expected performance.

Given the short duration of dewatering, the absence of predicted long-term surface water effects, and the use of erosion and sediment controls under the project's construction management framework, additional monitoring of surface water flow or levels is not considered necessary during operation.

9 CONCLUSIONS

This assessment has considered the potential groundwater effects associated with the construction and operation of the Southern Links 1 Industrial Development, including construction dewatering for artificial wetlands, wastewater infrastructure, drainage works, and the long-term groundwater diversion associated considered alongside impacts of peat pre-loading. 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 Industrial Development are assessed as no more than minor.

No additional permanent groundwater or surface water monitoring is considered necessary, for hydrogeological assessment purposes.

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