North Harbour 2 Watermain

WATERCARE SERVICES LIMITED

Technical Report C - Groundwater

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Contents

Executive Summary

This report assesses the potential groundwater effects related to the construction, operation and maintenance of Watercare Services Limited (Watercare)'s proposed North Harbour 2 Watermain (NH2) project between Titirangi and Albany. The groundwater effects are not required to be assessed in relation to the designation of that part of the Northern Interceptor (NI) Project between Westgate and Hobsonville, where a shared corridor is proposed for wastewater infrastructure.

The majority of the NH2 alignment (e.g. through Titirangi – Hobsonville, the SH18 corridor, and Greenhithe Bridge to Albany) traverses through sediments of the Tauranga Group (Alluvium and Puketoka Formation) as well as the East Coast Bays Formation (ECBF). The Tauranga Group contains materials that have, generally, a higher permeability than sediments of the ECBF. Along these areas, the NH2 alignment will be installed using an open trenching method with the water main pipe above the local groundwater levels. For the majority of the time, the water level will be below the trench excavation invert and no groundwater will seep into the open trench. In situations where the water level is higher (e.g. above the average water level of 3.8mBGL), groundwater may seep into the open trench excavation as the pipe is being laid, and will need to be pumped out; the expected drawdown due to trench excavation will be less than 1m at a distance of 30m from the excavation face, for a trench which will remain open for 8 days. Groundwater levels are expected to recover within 20 days of backfilling operations. An expected seepage inflow of less than 2m^3/d per m of excavated trench is anticipated at locations where sediments of the Tauranga Group lay on top of the ECBF. At locations where the trench is excavated directly into the ECBF, an expected seepage inflow of less than 0.2m^3/d per m of excavated trench is expected.

In cases where the water main is expected to cross or traverse major roads, trenchless methods are proposed. In these situations, the presence of temporary jacking or receiving pits can potentially induce drawdown during construction as groundwater may enter the excavation pit. For the deepest pit (depth = 12.8m), located at the Upper Harbour Motorway crossing (DWG 2010673.521 and DWG 2010674.316, NOR3) drawdown at the temporary pit will be about 1.7m but only 0.5m at a distance of 30m from the excavation. After the pit is backfilled, the water level is expected to recover within 90days. If Sheet Pile Walls (SPW) driven to 1m past the excavation invert are used, the expected drawdown will be reduced to 0.18m at the excavation and to 0.08m at a distance of 30m away from the pit. Seepage inflow into this pit will be about 12 m^3/d. Groundwater seepage into the pit will stop once the tunnel is completed and the pit is backfilled. The remaining pits are relatively shallow in comparison to this deep pit (average depth =6m) and are not expected to result in noticeable drawdown values during construction.

Groundwater seepage into the tunnel section between Manuka Rd and Shetland St will take place as this tunnel is being built. The maximum discharge rate will be attained at tunnel completion and this will be about 23.4 m^3/d. Seepage will be reduced straight after tunnel completion, and will stop once the tunnel annulus is grouted. At shallow crossings elsewhere along the alignment, seepage will not be significant because the unconsolidated sediments will close the annulus as the tunnel is being drilled and these tunnel sections are typically shallow. The tunnel section at Bush Rd, under Bushlands Reserve (DWG 2010674.331-.332) is about one third the length of the ManukaRd/Shetland St tunnel and not as deep (less than half the depth of the former) so it is anticipated that the seepage rates into this tunnel's annulus will be lower.

As the NH2 alignment traverses significant streams via pipe bridges, no effects on stream flow are expected at these locations. Shallow structures used to support the pipe above ground will only divert groundwater locally and in the direction of natural flow paths so the overall groundwater flow regime will not be affected by these structures. Other water main components, like valve and scour chambers, will be permanent but generally above the groundwater level therefore there is not expected to be any groundwater diversion around these structures. Groundwater in the area traversed by the NH2 alignment is not being extensively used. Only four groundwater bores that abstracted groundwater for domestic and stock purposes, were identified within 1.5km of the proposed NOR3 area. There will be no effect on these abstractions as a result of the proposed works given the minimal drawdown anticipated during the construction work, as well as the fact that all of the bores abstract water from the deep aquifer. Mitigation measures during construction include the use of Sheet Pile Walls (SPW) and monitoring of water levels at key locations.

In summary, as the expected drawdown resulting from open trench and pit excavations will be low, the effects associated with the construction of the NH2 water main are expected to be less than minor. The groundwater seepage rates into excavation areas and as the tunnelled sections are being completed will be reasonably low and can be managed with SPW on open face excavations or with adequate collection systems at the tunnels. Monitoring needs to be completed before the actual construction of the works and during construction, to ensure no environmental effects are taking place. This includes monitoring of project piezometers (Figure 4, Figure 5, and Figure 6) and regular water level monitoring. Groundwater users in these areas will not be affected by these activities. Consequently, no effects are expected from temporarily taking

groundwater during excavation and construction. After construction, discharge will stop and the water main pipe will be above the groundwater level (or completely isolated in case of the tunnels) so no adverse environmental effects are expected.

1. Introduction

Jacobs has been commissioned by Watercare Services Limited (Watercare) to assess the potential groundwater effects related to the construction, operation and maintenance of Watercare's proposed North Harbour 2 Watermain (NH2) project between Titirangi and Albany. Groundwater contamination is not considered in this report. For further details on groundwater quality and contamination see Volume 2 Technical Report B – Soil and Groundwater Contamination Assessment (Jacobs, 2015).

The NH2 will convey potable water from storage reservoirs in Titirangi, via west Auckland and North Shore to storage reservoirs in Albany (a length of approximately 33kms). Its purpose will be to increase capacity and resilience of the water supply network to western and northern Auckland.

The NH2 project incorporates:

- Pipeline installation, operation and maintenance of a new watermain of 1200 mm (west of Greenhithe Bridge) and 900mm (east of Greenhithe Bridge) nominal diameters (DN);
- Pipeline length of approximately 33 kilometres mostly within public road reserve; and
- Other features including valve chambers, scour valves, air valves, line valves, bulk supply points, pipe bridges, and associated works.

Most of the watermain will be constructed by open trenching, micro tunnelling or bored tunnel (the latter two referred to as "trenchless technology") within a typical construction corridor of approximately 12 – 22 metres width with additional areas required for erosion and sediment control devices, traffic management, construction yards and storage areas at intervals along the route for construction purposes.

The NI project comprises of a new wastewater pipeline and associated activities to convey flows from north-west Auckland to the Hobsonville Pump Station, and then to the Rosedale Wastewater Treatment Plant (WWTP).

The proposed NI project in the shared corridor begins in the vicinity of Hobsonville Road (West Harbour), near the intersection of the Upper Harbour and North Western Motorways (SH18 and SH16). From this location, the alignment follows the southern side of the SH18, continuing northeast to the Hobsonville Pump Station. Future phases of the NI project will also include new pipelines between the Hobsonville Pump Station and the SH18 causeway.

Within the shared corridor, the NI project incorporates the following:

- A new 5km wastewater pipeline of 2100mm DN;
- 16 pits / shafts for trenchless technology construction purposes. 5 of these will be permanent manholes (MT Pits 2, 7, 11, 13 & 17) while the others (MT Pits 3, 4, 5, 6, 8, 9, 10, 12, 14, 15 and 16) will be temporary only until construction / testing is completed;
- MT Pit 7 will be a drop structure with permanent access, to allow for a future wastewater pipeline connection across SH18;
- A new 50m long wastewater pipeline and manholes connecting the 2100mm ND pipeline to the existing pump station;
- A new 1750 l/s Pump Station with future capacity across the site of 3,500l/s;
- Wastewater storage (within pipeline);
- Two 800m 1500mm DN rising mains (length to the causeway); and
- A 2100mm DN pipe installed by trenchless technology at SH18.

The proposed alignment of NH2 and the location of the NI project are shown in Figure 1 below.

A full description of the proposed works and construction methodology is included in in the North Harbour 2 Watermain and Northern Interceptor Shared Corridor Assessment of Effects on the Environment (the AEE report) prepared by AECOM Consulting Services (NZ) Ltd (AECOM) and Jacobs New Zealand Limited (Jacobs).

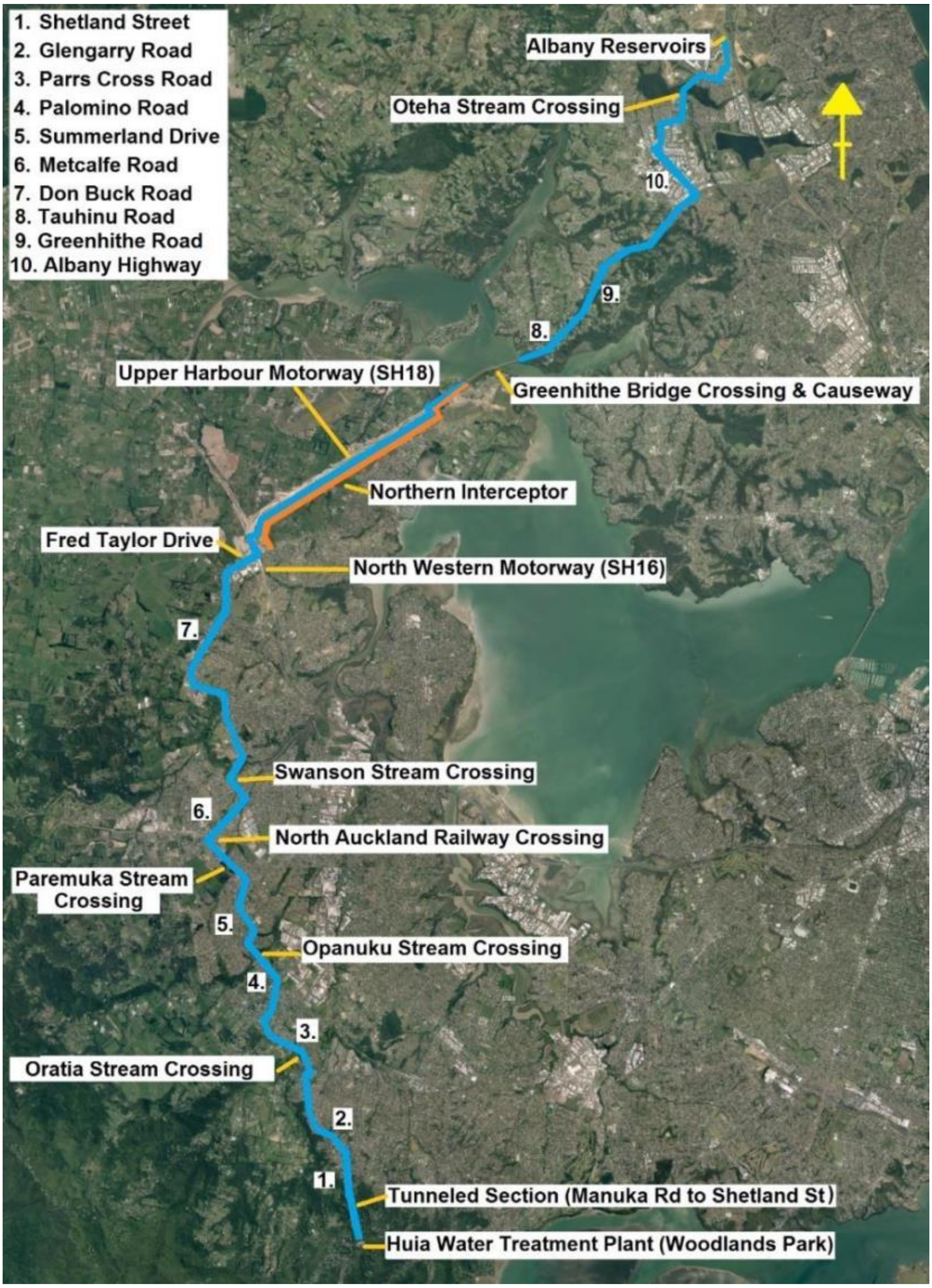

Figure 1– Blue line is the proposed NH2 route and Orange line is NI section within shared corridor

Watercare is proposing to designate land for the NH2 project between Titirangi and Albany and the NI project between Westgate and Hobsonville, and will also be seeking various resource consents for NH2 under the Resource Management Act 1991 (RMA). Resource consents for NI in the shared corridor are not part of this assessment. This technical report provides specialist input for the AEE which supports the resource consent application for NH2 only. The groundwater effects are not required to be assessed in relation to the designation of the NI Project in the shared corridor. Resource consents required for works associated with the NI project will be sought by Watercare at a later date, nearer to the proposed date of construction. The alignment drawings referred to in this report are contained within Volume 3 of the AEE.

This report provides the following in relation to NH2:

- A description of the environmental baseline for the particular receiving environment(s) potentially affected by the projects;
- Description of specific aspects of the projects in relation to the subject area being investigated;
- Description of the investigations undertaken to assess potential groundwater effects and assessment of effects of the proposed works within the existing hydrogeological framework (without mitigation);
- An assessment of the actual or potential effects on the environment (construction, operation and maintenance). This includes the identification of activities that could result in potential adverse effects and, in turn, identifying design refinements or construction methodologies that could avoid, remedy or mitigate potential adverse effects;
- Conclusions.

2. Existing Environmental Baseline

The existing environmental baseline relevant to hydrogeology is outlined below.

2.1 Geology

The Geology of the Auckland Area geological map (Institute of Geological and Nuclear Sciences, 2001) indicates the geological deposits likely be encountered along the proposed route from oldest to youngest include:

- **Puketoka Formation**. Pleistocene age fluvially deposited pumiceous deposits of light grey to orange brown pumiceous mud, sand and gravel with black muddy peat and lignite. This formation forms part of the Tauranga Group.
- **Alluvium.** Holocene age clays, silts and sands, muddy peat and unconsolidated organic-rich sediments. These sediments also form part of the Tauranga Group.
- **East Coast Bays Formation (ECBF)**. Early Miocene age flysch, a greenish grey, alternating muddy sandstone and mudstone, with occasional interbedded harder grit lenses (Parnell Grit). The weathered rocks of the ECBF weathers at the surface to brown and grey colour variations of soft to stiff, low to moderate plasticity clayey silt; soft to firm, non-plastic to high plasticity sandy silt; and very loose to very dense fine to medium sand.
- **Cornwallis Formation.** Early Miocene age volcanogenic flysch (alternating layers mudstone and sandstone) of the Waitemata Group, comprising grey brown, alternating, thick bedded sandstone and thin bedded mudstone.
- **Albany Conglomerate.** Early Miocene age well rounded pebbles and boulders in a medium to very coarse grained sandy matrix.
- **Piha Formation.** Coarse volcanoclastics, dominated by stratified, andesitic boulder-bearing, cobble-pebble breccia and conglomerate, locally interbedded with volcanoclastic granular sandstone
- **Nihotupu Formation.** Early Miocene age fine grained volcanoclastic sandstone which can include beds of reworked tuffaceous and pumiceous material and tuff breccia debris flows.

Figure 2 shows the surface geology for the three NH2 geographical areas (also used for notices of requirements, NOR, along the tunnel alignment). The main geological units occurring within these geographical areas are:

1. NOR 1: Titirangi to Hobsonville: The proposed alignment geology is dominated mainly by the ECBF and the Puketoka Formation with some alluvial sediments in the northern part of this area. However, the south of this area (e.g. through the tunnelled section from Manuka Rd to Shetland St) is dominated by the Cornwallis Formation, the Piha Formation and the Nihotupu Formation.

2. NOR 2: Greenhithe Bridge to Albany Reservoir: The proposed alignment goes through the ECBF and the Puketoka Formation within this area. There are some localised areas of Albany Conglomerate occurring about 1km west of the proposed alignment.
3. NOR 3: SH18 corridor: Within this area the proposed alignment geology is largely dominated by the Puketoka Formation but the ECBF is also present beneath the surface. Some alluvial sediments may be associated to the Puketoka Formation and this is evident in some patches towards the north of this area.

2.2 Regional hydrogeology

The hydrogeology throughout the project consists of a highly stratified geological sequence (see section 2.1) including faulting and fracturing, which compartmentalise parts of the groundwater system.

The ECBF is used as a water supply aquifer in the wider Auckland Region to varying degrees of production. The ECBF typical comprises a poor aquifer, while in some places (e.g. Kumeu) the ECBF sustains a reasonably large number of moderately productive horticulture and domestic bores. The variability in productivity is largely related to hydraulic conductivity, which in the ECBF is largely governed by the degree of open fractures and open bedding planes that are often associated with folding and faulting.

Typical ECBF matrix is of low hydraulic conductivity and low storage characteristics (**Table 2-1** and **Table 2-2**). Average hydraulic conductivity is approximately 2.3×10^{-7} metres per second (m/s), with a default horizontal hydraulic conductivity of about 6.0×10^{-7} m/s (used for modelling purposes). The strongly bedded sequence of thin (typically 0.1 to 0.5 m) alternating siltstone and fine sandstone give rise to vertical anisotropy in hydraulic conductivity, with horizontal hydraulic conductivity typically 40 to 250 times greater than vertical hydraulic conductivity (Tuhono Consortium, 2011).

The wECBF (weathered ECBF) comprise residual soils and weathered silts, sands, and clay from the underlying ECBF. Soils comprise of orange-brown to grey-brown mottled, stiff to very stiff, silty clay and clayey silt of intermediate plasticity and loose to medium dense, fine to medium silty sand. With depth, the relict structure of the original rock mass is evident. The ability of the wECBF to transmit and store groundwater is generally limited, and typically have lower permeability than the parent material, due to the loss of secondary permeability (primarily fractures and jointing). These units would typically have the lowest permeability in the project area, with average hydraulic conductivity for the wECBF of about 2.0×10^{-7} m/s. Storage characteristics are also typically low (**Table 2-1** and **Table 2-2**).

Hydraulic conductivity of the weathered deposits appears to decrease with the degree of weathering, hence the stiffer materials nearer the base of the unit have relatively higher permeability compared to the softer materials near the top of the soil/weathering profile. This has important implications for consolidation, in terms of timing for depressurisation to be transmitted from bottom to top of profile, and in terms of the ability of the softer materials to physically dewater.

The Tauranga Group consists of late Miocene to Holocene (mainly pumiceous) terrestrial and minor estuarine sediments which are present mainly in extensive lowland areas west and south of Auckland City. Tauranga Group sediments are heterogeneous, including gravels, sands, silts, muds, and peats of fluvial, lacustrine, and distal ignimbritic origin. The Puketoka Formation consists of non-marine sediments which form part of the Tauranga Group. Both recent alluvium and Puketoka Formation materials are derived from erosion of the underlying East Coast Bays Formation. Additionally, the alluvium contains some air fall ash deposits and organic materials and peat. Hydraulic conductivity is typically low to moderate, ranging from 10^{-8} to 10^{-5} m/s, while storativity is also low at around 1×10^{-3} (**Table 2-1** and **Table 2-2**). A horizontal hydraulic conductivity of 3.0×10^{-5} m/s for the Puketoka Formation is used for modelling purposes (e.g. maximum value derived from rising head tests). Three bores (BH201, BH202, and BH204) locate at the east and the west end of the Greenhithe Bridge, were slug tested during this study. These bores were completed in the Tauranga Group (silt, fine sand, and gravel sediments) and their hydraulic conductivities, presented in Table 10-1 (calculations in Figure 21, Figure 22, Figure 23, and Figure 24), are generally in the 10-5 m/s order of magnitude which coincides with what has been estimated during previous investigations.

Table 2-1. Summary of Aquifer Hydraulic Conductivity Characteristics

Unit	K_h (m/s) range	K_h:K_v ratio	Qualitative Comparison
Puketoka Formation / Alluvium	10^{-8} to 10^{-5}	>10	Low to High
wECBF	10^{-9} to 10^{-7}	>10	Very Low
ECBF	10^{-8} to 10^{-7}	40 to 250	Low

Table 2-2. Summary of Aquifer Storage Characteristics

Unit	S_s (m^{-1}) range	S_y	Qualitative Comparison
Puketoka Formation / Alluvium	$1x10^{-3}$	0.01	Low
wECBF	$1x10^{-3}$	0.01	Low
ECBF	$9x10^{-6}$	0.01	Very Low

The ECBF, Puketoka Formation, and Alluvium units are present throughout the 3 NORs throughout the project alignment. However, there are some additional geological units present in certain areas of the project. One of these formations is the Albany Conglomerate, which is present in patches at the surface to the west of the alignment, within the NOR 2 project area. This unit is present within the upper part of the ECBF and its relatively large grains (pebbles and boulders in a medium-to very coarse-grained matrix) can result in a relatively high hydraulic conductivity, if not cemented by calcite. Within the NOR 1 project area, between Manuka Rd and Shetland St, the tunnelled section goes through the Nihotupu Formation and the Cornwallis Formation. The Nihotupu Formation mainly consists of volcanoclastic sandstones whereas the Cornwallis Formation consists of volcanogenic flysch. Consequently, the Nihotupu Formation will have a higher permeability than the Cornwallis Formation. The Cornwallis Formation is also present at about 1km west of the proposed alignment at the northern part of NOR 1. In addition, the Piha Formation is present about 1km to the south west of the alignment at the tunnel entrance (Figure 2). This formation could have a similar permeability than the one for the Nihotupu and Cornwallis formations.

2.3 Site specific groundwater levels

Preliminary geotechnical investigations resulted in the installation of 8 piezometers between May and July 2013 (Opus, 2014). These piezometers were installed at stream crossings throughout the 3 NORs (Titirangi to Hobsonville, Greenhithe Bridge to Albany Reservoir, and SH18 shared corridor) along the project alignment. In addition, this investigation included 38 shallow hand auger excavations (up to 4m in depth) to assess soil and groundwater conditions. Groundwater levels were measured in bores during drilling and subsequently during monitoring. Similarly, when groundwater was encountered during hand auger excavations, the groundwater levels were recorded. In general, groundwater levels recorded at the time of the drilling and during hand auger excavations are not as accurate as levels monitored in piezometers or monitoring bores. Figure 3 shows the location of project bores and hand auger excavations in which groundwater levels have been measured. In addition to this information, a list of existing information was compiled to compliment the groundwater level data gathered at the project bores. This information was compiled from the following reports:

- SH16/18 Scheme Assessment Geotechnical Report, Beca Carter Hollings and Ferner Ltd, 1999
- SH18 Hobsonville Deviation and SH16 Brigham Creek Road Extension Geotechnical Factual Report, Maunsell Ltd, 2007
- Greenhithe Rd Interchange Geotechnical Foundation Parameters, Meritec, 2002
- Greenhithe Section Preliminary Geotechnical Report, Opus and Meritec, 2002
- Upper Harbour Corridor Geotechnical Investigations, Meritec, 2002
- UHP Duplication and Causeway Widening, Design Report Package B, Geotechnical Interpretive Report, Beca Carter Hollings and Ferner Ltd.

- Upper Harbour Bridge Duplication Geotechnical Investigation Report, Connell Wagner, 2001
- Woodlands Park Road Reservoirs, Geotechnical Factual Reports, OPUS, 2013
- NH2 Advanced Works, Geotechnical Factual Reports, OPUS, 2014
- Upper Harbour Corridor Hobsonville Section, Maunsell Ltd, February 2006

Groundwater levels for the whole project are 3.8mBGL (31.6mRL) on average, with a median of 3.2mBGL (27.0 mRL). The standard deviation for groundwater levels is 2.74mBGL (19.9mRL) which means that shallow groundwater levels would typically occur below 1mBGL. Specific groundwater levels throughout the 3 NOR areas are described in the following sections.

2.3.1 Groundwater levels in the Titirangi to Hobsonville area (NOR 1)

Project bores installed within this project area (Figure 4) show that the groundwater level near the streams is generally between 2.3 to 5.7mBGL. Hydrographs for project bores are presented in Figure 10 to Figure 13. Because these bores are completed in topographic lows, groundwater levels for these bores are closer to the surface than levels in bores away from the streams. In addition to groundwater levels from project bores, Figure 4 shows groundwater levels for piezometers constructed through the course of previous investigations (see list in previous section). According to these investigations, groundwater levels at the start of the tunnelled section near Woodlands Park ranges between 4 and 9.2mBGL. In general, other piezometers along this area of the alignment show groundwater levels in the 1.5-5mBGL range (Figure 4).

2.3.2 Groundwater levels in the area from Greenhithe Bridge to Albany Reservoir (NOR 2)

Project specific bores for this area include BH204 towards the south and BH265 to the north. Hydrographs for these two bores are presented in Figure 9 and Figure 14. The groundwater level in BH204 ranges between 3.79mBGL and 4.13mBGL, and the groundwater level in BH265 ranges between 1.71-2.15mBGL. Other non-specific piezometers in this area indicate, for the most part, groundwater levels in the 1.5-5mBGL range but there are some bores with shallow (0-1mBGL) water levels (Figure 5). In any case, many of these water level measurements may not be completely accurate as they do not come from monitoring bores but are recorded at the time of drilling, and may indicate unsaturated soils (e.g. moisture) or “perched” water conditions.

2.3.3 Groundwater levels in the SH18 corridor area (NOR 3)

Bores BH201 and BH202 show that the groundwater level towards the northern part of this area ranges between 2.71mBGL and 3.48mBGL. Hydrographs for these two bores are presented in Figure 7 and Figure 8. In addition, HA201 indicated a groundwater level of 4.3mBGL. Previous investigations carried out through other projects show that the majority of the groundwater levels are in the 1.5-5mBGL range but some locations have experienced higher groundwater levels (e.g. <1mBGL) and lower ones (e.g. >5mBGL) (Figure 6).

2.4 Groundwater use

Groundwater is generally not used in the project area (Titirangi to Hobsonville, Greenhithe Bridge to Albany Reservoir, and SH18 shared corridor). Most of the alignment goes through urban areas which do not use groundwater for domestic use or crosses unpopulated estuarine areas. However, some groundwater usage has been identified within the following project areas:

- Titirangi to Hobsonville (NOR 1): At the start of the alignment (tunnelled section within NOR 1) the proposed crossing is through a high ridge (e.g. Nihotupu Formation). This ridge would act as a recharge area for ephemeral streams in the vicinity of the ridge (Sharp Stream and Whakarino Stream). The exact water levels in the Nihotupu Formation are unknown and the capacity of this formation to transmit water is highly dependent on whether it is sufficiently fractured at depth. No groundwater takes were identified within 1.5 km of the proposed alignment in this area.

- Greenhithe Bridge to Albany Reservoir (NOR 2): The new watermain will be constructed in the Kumeu Waitemata Aquifer zone. A bore search enquiry from Auckland Council’s bore database was undertaken to identify any boreholes located within the vicinity of the proposed works. This search identified four deep groundwater take bores within 1.5 km of the proposed alignment (Table 2-3).

- SH18 corridor (NOR 3): The search only identified two site investigation bores within the area (Table 2-4). Although these bores are not used for actual groundwater use, they are used to assess the quantity and quality of groundwater in this area.

Table 2-3. Groundwater usage within area from Greenhithe Bridge to Albany Reservoir

Consent no.	Depth (m)	Casing depth (m)	Purpose	Address	Distance from proposed works (km)
13844	200	65	Stock and domestic supply	124 Hobsonville Road	1.3
23230	200	65	Stock and domestic supply	5 Upper Harbour Drive	0.9
21320	200	65	Domestic supply	74 Upper Harbour Drive	1.0
27736	200	70	Domestic supply	124 Upper Harbour Drive	1.5

Table 2-4. Groundwater monitoring within SH18 area

Consent no.	Depth (m)	Casing depth (m)	Purpose	Address	Distance from proposed works (km)
-	-	-	Groundwater and contaminated site investigation	12 Clark Road (BP Oil NZ Limited)	1.2
28653	5	2.6	Monitoring (3 bores)	1 Buckley Avenue	0.5

2.5 Surface water connections

Surface connections are possible at topographic lows with the possibility of groundwater discharging into streams when the groundwater levels are high (e.g. after heavy rainfall). Near the coast, groundwater discharges into the sea. Throughout the project alignment water levels have been monitored (Table 2-5) in piezometers adjacent to the most significant streams crossing the alignment (Figure 1).

Table 2-5. Stream crossings along the alignment

Area	Location / Monitoring Bore	Average depth to Groundwater (mBGL)[1]
NOR 1: Titirangi to Hobsonville	Oratia Stream Crossing (BH252)	4.0
	Opanuku Stream Crossing (BH253)	4.9
	Parekuma Stream Crossing (BH257)	3.8
	Swanson Stream Crossing (BH263)	2.3
NOR 2: Greenhithe Bridge to Albany Reservoir	East end near GBWD works (BH204)	4.0
	Oteha Stream Crossing (BH265)	2.0
NOR 3: SH18 corridor	West end near GBWD works (BH201, BH202)	3.0
[1] Water levels monitored between May 2014 and November 2014		

3. Scheme assessment

This report addresses hydrogeological issues in respect of the proposed alignment. A summary of the proposed structures and potential effects on groundwater along the alignment is given in Table 3-1.

Table 3-1. Evaluation of Potential Groundwater Issues

<table>
<tr><th>Project area</th><th>Structures</th><th>Location</th><th>Geology</th><th>Potential Groundwater Issues</th></tr>
<tr><td>NOR 1:

Titirangi to Hobsonville</td><td>901.76m tunnelled section with a 1200mm NB CLS pipe at depths of up to 58m.</td><td>Between Manuka Rd to Shetland St (DWG 2010674.301 and 2010674.302)</td><td>Nihotupu Formation and Cornwallis Formation</td><td>• Groundwater seepage into tunnel annulus during construction
• Localised groundwater drawdown due to seepage</td></tr>
<tr><td>NOR 1:

Titirangi to Hobsonville</td><td>Watermain pipe.[1]</td><td>Various locations (DWG 2010673.510-520)</td><td>Puketoka Formation, Alluvium, and ECBF</td><td>• Localised groundwater drawdown during construction, due to dewatering of cuts
• Local drawdown induced ground settlement
• Contaminant migration</td></tr>
<tr><td>NOR 1:

Titirangi to Hobsonville</td><td>62m tunnelled section with a 1200mm NB CLS pipe at depths of 4-5m (Option 2)</td><td>Metcalfe Rd crossing (DWG 2010674.312)</td><td>Puketoka Formation</td><td>• Temporary jacking and receiving pits may cause some localised groundwater drawdown during construction
• Local drawdown induced ground settlement
• Contaminant migration</td></tr>
<tr><td>NOR 2: Greenhithe Bridge to Albany Reservoir</td><td>Watermain pipe[1]</td><td>Various locations (DWG 2010673.526-533)</td><td>Puketoka Formation and ECBF</td><td>• Localised groundwater drawdown during construction, due to dewatering of cuts
• Local drawdown induced ground settlement
• Contaminant migration</td></tr>
<tr><td>NOR 2: Greenhithe Bridge to Albany Reservoir</td><td>194m tunnelled section having with a 1200mm NB CLS pipe depths of 5.2-10.4m</td><td>Upper Harbour Motorway, Tauhnu Rd access (DWG 2010674.322)</td><td>ECBF</td><td rowspan="3">• Temporary jacking and receiving pits may cause some localised groundwater drawdown during construction
• Local drawdown induced ground settlement
• Contaminant migration</td></tr>
<tr><td>NOR 2: Greenhithe Bridge to Albany Reservoir</td><td>170m tunnelled section with a 910mm NB CLS pipe at depths of 4.88-12.55m (Trenchless option)</td><td>Upper Harbour Motorway (Greenhithe Rd crossing) (DWG2010674.323)</td><td>ECBF</td></tr>
<tr><td>NOR 2: Greenhithe Bridge to Albany Reservoir</td><td>283m tunnelled section with a 910mm NB CLS pipe at depths of 3.9m-6.57m</td><td>Upper Harbour Motorway (DWG2010674.324-325</td><td>ECBF</td></tr>
</table>

Project area	Structures	Location	Geology	Potential Groundwater Issues
Greenhithe NOR 2: Greenhithe Bridge to Albany Reservoir	508m tunnelled section with a 910mm NB CLS pipe at depths of 5.1m-8.31m	Upper Harbour Motorway from Albany Highway to William Pickering Dr (DWG 2010674.326-327)	ECBF	
NOR 2: Greenhithe Bridge to Albany Reservoir	354m tunnelled section with a 910mm NB CLS pipe at depths of 4.4m-24.4m	Bush Rd, tunnel under Bushlands Reserve), DWG 2010674.331-.332	ECBF	• Groundwater seepage into tunnel annulus during construction • Localised groundwater drawdown due to seepage • Tunnel seepage may affect stream flow during tunnel construction as the tunnel goes under the stream
NOR 3: SH18 corridor	139.4m tunnelled section with a 1200mm NB CLS pipe at depths of 5.29-12.82m	North Western Motorway Crossing (DWG 2010674.316)	Puketoka Formation and ECBF	• Temporary jacking and receiving pits may cause some localised groundwater drawdown during construction • Local drawdown induced ground settlement may affect the motorway • Seepage of groundwater into the pits may cause contaminant migration.
NOR 3: SH18 corridor	Watermain pipe[1]	Various locations (DWG 2010673.521-525)	Puketoka Formation	• Localised groundwater drawdown during construction, due to dewatering of open section • Local drawdown induced ground settlement • Contaminant migration
NOR 3: SH18 corridor	Watermain pipe[1] through stream crossings	Various locations (DWG 2010673.522-524)	Puketoka Formation	• Localised groundwater drawdown during construction, due to dewatering of open section • Stream flow may be temporarily affected
NOR 3: SH18 corridor	91.5m tunnelled section with a 1200mm NB CLS pipe at depths of 4.84-7.93m	Upper Harbour Motorway crossing (DWG 2010674.321)	Puketoka Formation	• Temporary jacking and receiving pits may cause some localised groundwater drawdown during construction • Local drawdown induced ground settlement may affect the motorway • Seepage of groundwater into the pits may cause contaminant migration

Notes:

[1] The Watermain will have a nominal diameter of 1200mm and its construction will be in open trench sections (typically 2-3m wide and 3-5m below grade) which will be backfilled after the pipe is laid.

4. Proposed construction methodology in relation to hydrogeology

The proposed construction methodology for NH2 is outlined in section 2 of the main AEE report. Key assumptions made in development of models to assess potential environmental effects relating to groundwater are:

- A 4m to 5m deep trench excavated in sections of 12m to 24m in length (the open trench may be open for a maximum length of 90m at given time);
- A 910mm/1200mm NB concrete lined steel (CLS) pipe will be laid in the trench;
- A 1200mm NB CSL pipe in the trenchless sections;
- 1200mm – 1350mm NB reinforced concrete jacking pipe for trenchless sections;
- Flowable or granular fill will be placed around the pipe (300mm above and below the pipe);
- Compacted hard fill material will be used to backfill the trench up to ground level; and
- The final surface will be reinstated with asphalt (along road corridors).

NOR 1: Titirangi to Westgate

At the Woodlands Park Reservoir end, the water main tunnel will be constructed by a trenchless method through a ridge descending through to the Glen Eden side of the Waitakere Ranges. The tunnel will have a length of approximately 900m. The 1200mm NB CLS watermain pipe will be installed within the tunnel from Manuka Road to Shetland St. A 7.7m deep permanent access shaft will be constructed at the Woodland Park Rd end. The shaft will be excavated after sheet piling installation to limit water ingress. After construction, the shaft will be backfilled and a permanent concrete box will be constructed to allow access to the pipeline and valves for maintenance purposes. The tunnel will be fitted with a 2100mm NB reinforced jacking pipe. The annulus (450mm gap around the 1200mm pipe) will be grouted after the watermain has been placed. As a result, groundwater may be able to seep into this annulus during construction but any seepage will stop once the tunnel is completed.

The trenchless road crossing at Metcalfe Rd (Option 2, DWG 2010674.312) is a shallow crossing through unconsolidated material (Puketoka Formation). In this case, the soil material around the jacking pipe will seal the gap between the tunnel wall and the jacking pipe.

NOR 2: Greenhithe Bridge to Albany Reservoir

The Upper Harbour Motorway Crossings (2010674.322-327) in this area consist of shallow sections going through unconsolidated material of the ECBF and the Puketoka Formation. The tunnelled section under the Upper Harbour Motorway from Albany Highway to William Pickering Dr (DWG 2010674.331-332) also goes through these soil materials and is approximately 8 m deep. In these cases, the soil material around the jacking pipe will seal the annulus as the tunnel is being drilled. These sections are relatively short in length (less than 140m). The tunnelled section under Bushlands Reserve (DWG 2010674.331-332) is about 350m long. This tunnel will be constructed via a temporary receiving pit shaft with a depth of 2.5mBGL to the southern end of the Reserve. The shaft will be excavated after sheet piling installation to limit water ingress. After construction, the shaft will be backfilled and a permanent concrete box will be constructed to allow access to the pipeline and valves for maintenance purposes

NOR 3: SH18 corridor

The tunnelled sections under the North Western Motorway Crossing (DWG 2010674.316) and the Upper Harbour Motorway crossing (DWG 2010674.321) are shallow sections (less than13m) through unconsolidated material of the Puketoka Formation or weathered ECBF. In these situations the soil material around the jacking pipe will seal the gap formed by pipe and the tunnel wall. Consequently, seepage into the annulus will be limited. The deepest of these sections is the North Western Motorway crossing (NOR 3, DWG 2010674.316) which will be constructed through a temporary access shaft of about 12.8mBGL. The actual pit could be excavated to a depth of 13.8mBGL and could remain open for a period of up to 3 months. After this time, the pit will be sealed with compacted backfill material. To construct this pit, sheet piles may be used to control the ingress of groundwater during pipe jacking operations.

5. Groundwater models

Groundwater modelling has been undertaken to assess the likely effects on groundwater due to the construction and long-term operation of the NH2. Open Trench and Pit models were put together as 2D groundwater seepage models using SEEP/w 2007. These numerical models were used to assess the effects of pit and open trench excavations throughout the different project areas (e.g. Titirangi to Hobsonville, Greenhithe Bridge to Albany Reservoir, and SH18 corridor) where Tauranga Group sediments and the ECBF are present. To this effect, the section resulting in worst case effects corresponds to the Upper Harbour Motorway crossing (DWG 2010673.521 and DWG 2010674.316).

For the trenchless sections, an analytical model was utilised to quantify the potential seepage from the surrounding groundwater system as the tunnel is being built. Since this issue is important with long deep tunnels through consolidated rock materials, the tunnelled section from Manuka Rd to Shetland St was used as a worst case scenario.

5.1 Open Trench Models

Three numerical models were put together to provide an estimate of the volume of water that might enter the trench during construction, changes in groundwater level associated with the construction of the water main, as well as its performance after construction (e.g. water level recovery):

1) Open Trench Fill/ECBF model: open trench model considering Fill material (e.g. Alluvium or Puketoka Formation) to a depth of approximately 4.8 mBGL, on top of the ECBF.

2) Open Trench ECBF model: an open trench model considering only the presence of ECBF material

3) Open Trench model using sheet piles: the open trench model for the Fill/ECBF materials was fitted with sheet piles at 1m and 3m below the invert level

5.1.1 Open Trench Fill/ECBF model

Model results are presented in Table 5-1 and Figure 15. The modelling predicts a drawdown of 2.3m at the excavation which corresponds to the excavation invert (e.g. 5mBGL). However, at 30m from the excavation, the drawdown is 0.3m. At 100m from the excavation, the drawdown is negligible. The discharge into the open trench is predicted to be approximately 2m^3/day/m of trench. Therefore, if the open trench can be up to 90m in length and could be open for up to 8 days, the total volume of water discharging into the open trench would be 1440m^3. After the trench is backfilled, the groundwater level would recover in approximately 100 days.

Table 5-1. Open Trench Fill/ECBF model results

Base case	Hydraulic conductivity (m/s)	Q[1] (m^3/d/m)	Drawdown at 0 m from excavation (m)	Drawdown at 30 m from excavation (m)	Lateral zone of influence[1] (m)	Recovery time[2] (days)
Fill	3×10^{-5}	2.0	2.3	0.30	100	100
ECBF	6×10^{-7}					

1. After 8 days of drainage.
2. 90% recovery of drawdown.
3. Q = groundwater discharge (m^3/d) per m of trench

5.1.2 Open Trench ECBF model

Model results are presented in Table 5-2 and Figure 16. The modelling predicts a drawdown of 2.3m at the excavation (e.g. 5mBGL) which is the excavation invert. However, at 30m from the excavation, the drawdown is 0.04m. At 70m from the excavation, the drawdown is negligible (zone of influence). The discharge into the open trench is approximately 0.2 m^3/day/m of trench. Therefore, if the open trench is up to 90m in length and could be open for up to 8 days, the total volume of water discharging into the open trench would be about 144m^3. After the trench is backfilled, the groundwater level would recover in approximately 20 days.

Table 5-2. Open Trench ECBF model results

Base case	Hydraulic conductivity (m/s)	Q[1] (m^3/d/m)	Drawdown at 0 m from excavation (m)	Drawdown at 30 m from excavation (m)	Lateral zone of influence[1] (m)	Recovery time[2] (days)
ECBF (weathered)	3×10^{-7}	0.2	2.3	0.04	70	20

4. After 8 days of drainage.
5. 90% recovery of drawdown.
6. Q = groundwater discharge (m^3/d) per m of trench

5.1.3 Open Trench model using sheet piles

Model results are presented in Table 5-3. The modelling predicts the drawdown to be less than 1cm at the excavation edge. For both alternatives (1m and 3m below the excavation invert), the drawdown is basically nil 30m away from the trench excavation. With the sheet pile driven 1m passed the invert, the discharge into the open trench would be approximately 0.02 m^3/day/m of trench. Therefore, if the open trench can be up to 90m in length and could be open for up to 8 days, the total volume of water discharging into the open trench would be 14.4m^3. After the trench is backfilled, the groundwater level would recover almost instantaneously.

Table 5-3. . Results for model of Open Trench Fill/ECBF with sheet pile

Sheet pile depth past excavation invert	Unit	Hydraulic conductivity (m/s)	Q[1] (m^3/d/m)	Drawdown at 0 m from excavation (m)	Drawdown at 30 m from excavation (m)	Lateral zone of influence[1] (m)	Recovery time[2] (days)
1m	Fill	**3×10^{-4}**	0.02	0.01	0.0	0.0	0.0
	ECBF	6×10^{-7}					
3m	Fill	**3×10^{-4}**	0.01	0.0	0.0	0.0	0.0
	ECBF	6×10^{-7}					

1. After 8 days of drainage.
2. 90% recovery of drawdown.
3. Q = groundwater discharge (m^3/d) per m of trench

5.2 Shaft model for receiving/jacking pit

5.2.1 Open pit model with no mitigation measures

Model results are presented in Table 5-4 and Figure 18. The modelling predicts a drawdown of 1.72m at zero metres from the excavation. However, at 30m from the excavation, the drawdown is 0.46m. At 110m from the excavation, the drawdown is negligible. The discharge into the open pit is 12m^3/day. Therefore, if the open pit will remain open for 90 days, the total volume of water discharging into the open trench would be 1080m^3. After the trench is backfilled, the groundwater level would recover in approximately 90 days.

Table 5-4. Receiving pit model results

Base case	Hydraulic conductivity (m/s)	Q[1] (m^3/d)	Drawdown at 0 m from excavation (m)	Drawdown at 30 m from excavation (m)	Lateral zone of influence[1] (m)	Recovery time[2] (days)
Fill	3×10^{-5}	12	1.72	0.46	110	90
ECBF	6×10^{-7}					

1. After 90 days of drainage.
2. 90% recovery of drawdown.
3. Q = groundwater discharge (m^3/d) for the whole pit

5.2.2 Open pit model with Sheet Pile Walls (SPW) as a mitigation measure

Model results are presented in Table 5-5 and Figure 19. The modelling predicts the drawdown is less than 0.18m at the excavation edge. Also, the drawdown is about 8cm away from the trench excavation. With the sheet pile driven 1m past the invert, the discharge into the open trench would be 2.37m^3/day. Therefore, if the open pit could remain open for up to 90 days, the total volume of water discharging into it would be about 213m^3. After the pit is backfilled, the groundwater level would recover in 45 days.

Table 5-5. Open pit with sheet pile walls results

SPW	Hydraulic conductivity (m/s)	Q[1] (m^3/d)	Drawdown at 0 m from excavation (m)	Drawdown at 30 m from excavation (m)	Lateral zone of influence[1] (m)	Recovery time[2] (days)
Fill	3×10^{-5}	2.37	0.18	0.08	100	45
ECBF	6×10^{-7}					

1. After 90 days of drainage.
2. 90% recovery of drawdown.
3. Q = groundwater discharge (m^3/d) for the whole pit

5.3 Tunnel Seepage Estimates

According to the base case model, the maximum discharge rate is predicted to be about 23m^3/day at the tunnel completion time (75 days). This rate will significantly decrease after this time and will reach a value lower than 5m^3/day after 500 days. However, if the annulus is grouted immediately after completion, the discharge rate would be nil. A plot of the discharge rate over time is presented in Figure 20. This figure shows that the discharge rate increases as the tunnel is being drilled, reaching a maximum at day 75 (completion time).

6. Assessment of Potential or Actual Effects

Groundwater modelling has been used to assess the effects on the groundwater regime resulting from the construction of NH2 through the 3 project areas (Titirangi to Hobsonville, Greenhithe Bridge to Albany Reservoir, and SH18 corridor).

6.1 NOR 1: Titirangi to Hobsonville

Potential Groundwater Drawdown

In this area the NH2 alignment mainly goes through Alluvium and Puketoka Formation material as well as the ECBF. In addition, some locations may contain fill material (e.g. Puketoka Formation soils). Drawdown in the open trench sections (e.g. DWG 2010673.510-521) can be in the order of 2.3m at the excavation, and about 1m at a distance of 30m from the excavation face. However, as groundwater levels recover to pre-construction conditions within a matter of days (e.g. 20 days with ECBF material), the risk of ground settlements occurring in response to groundwater drawdown is low.

The tunnel section between Manuka Rd and Shetland St will go through sandstones of the Cornwallis and Nihotupu formations. Drawdown in these units will be localised and temporary as the tunnel will be sealed with grout upon completion. Therefore, any potential settlement due to tunnelling will be negligible (AECOM, 2015).

Groundwater elevation

Analyses of groundwater levels shows that groundwater levels are generally deeper than (or nearly at the same level) the pipe invert throughout this area. In general, groundwater levels are 1.5–5mBGL at stream crossings (Figure 4) and the pipe invert will be at 2.7mBGL (1.5m clearance plus 1.2m diameter pipe). Groundwater levels can be deeper (5-16mBGL) near Woodlands Park Road so it is unlikely the pipe will cause an elevation or damming of groundwater. No areas of unusually low permeability have been identified throughout this project area.

Groundwater inflow

As the tunnel section between Manuka Rd and Shetland St is being built, groundwater discharge into the tunnel will take place. The maximum discharge rate will be attained at tunnel completion and this will be about 23.4m^3/d. Seepage will reduce immediately after tunnel completion and will stop once the tunnel annulus is grouted. At shallow crossings along the rest of the alignment, seepage will not be significant because the unconsolidated sediments will close the annulus as the tunnel is being drilled and these tunnel sections are typically shallow.

Groundwater inflow into the open trench sections and pit excavations into soft soils (e.g. Alluvium and Puketoka Formation) are expected to be of similar magnitude than the inflow estimated for trenches in the SH18 corridor (see section 6.3).

Groundwater diversion

There is a permanent access shaft between Woodlands Park Road and Scenic Drive in the Nihotupu Formation and at the bottom of a steep ridge. Groundwater flow in this area is expected to be closely dependant on rainfall and discharging seasonally in the form of springs. The pit occupies a small area which is expected to be dry during dry periods. Its presence is not expected to influence groundwater flow paths when precipitation occurs.

As the pipe will be generally above the groundwater level in this area, there will be basically no groundwater diversions taking place. In the case of seasonal highs where the groundwater level increases, the groundwater level is not expected be higher than the pipe crown level for longer periods of time. Temporary jacking and receiving pits are not expected to affect groundwater flow paths in the long term because their effect will cease after they are backfilled. Other structures like valve and scour chambers, will be permanent but generally above the groundwater level therefore it is not expected to be any groundwater diversion around these structures. In the case of seasonally high groundwater levels, any potential diversions will be in the direction of natural flow paths and will not change the overall regime of groundwater flow. The watermain pipe will cross over the Oratia, Opankuku, Paremuka, and Swanson streams in this area (Table 2-5) so there will be no effects on stream flow due to pipe emplacement.

6.2 NOR 2: Greenhithe Bridge to Albany Reservoir

Potential Groundwater Drawdown

The NH2 alignment goes mainly through the ECBF and the Puketoka Formation in this area. In addition, some locations may contain fill material (e.g. Puketoka Formation soils). Drawdown in the open trench sections (e.g. DWG 2010673.526-533) can be in the order of 2.3m at the excavation, and about 1m at a distance of 30m from the excavation face. However, as groundwater levels recover to pre-construction conditions within a matter of days (e.g. 20 days with ECBF material and 100 days with fill material), the risk of ground settlements occurring in response to groundwater drawdown is low.

Groundwater elevation

On average, groundwater levels are between 1.5mBGL and 5.0mBGL in this area, but both shallower and deeper groundwater levels have been recorded at some locations (Figure 5). The watermain pipe will sit at about 2.7mBGL so it is unlikely it will cause an elevation or damming of groundwater and, in localised areas where the pipe will intersect groundwater, the water level above the pipe will not be significantly high. No areas of unusually low permeability have been identified throughout the extent of this project.

Groundwater inflow

Groundwater inflows into open trenches and temporary pits are not expected to be higher than the inflows estimated for these excavations in the SH18 shared corridor area (see section 6.3). The tunnel section at Bush Rd, under Bushlands Reserve (NOR 2, DWG 2010674.331-.332) is about one third the length of the ManukaRd/Shetland St tunnel and not as deep (less than half the depth of the former) so it is anticipated that the seepage rates into this tunnel's annulus will be lower than 23.4m^3/d.

Groundwater diversion

As in the previous area, temporary jacking and receiving pits are not expected to affect groundwater flow paths in the long term. Other structures (e.g. valve and scour chambers) will be permanent but generally above the groundwater level.

6.3 NOR 3: SH18 shared corridor

Potential Groundwater Drawdown and Induced Settlements

In this area, drawdown is predicted within loose sediments of the Tauranga Group (e.g. Puketoka and Alluvium). In addition, some locations may contain fill material (e.g. Puketoka Formation soils). Drawdown may potentially occur for open trench sections (e.g. DWG 2010673.521-525) along the route, as well as through jacking and receiving pits along tunnelled sections. Drawdown in the open trench sections (e.g. DWG 2010673.521-525) can be in the order of 2.3m at the excavation, and about 1m at a distance of 30m from the excavation face. However, as groundwater levels recover to pre-construction conditions within a matter of days (e.g. 20 days with ECBF material and 100 days with fill material), the risk of ground settlements occurring in response to groundwater drawdown is low. An independent settlement technical report (AECOM, 2015) provides further details regarding calculated settlement from the different drawdown scenarios presented in this report.

Drawdown at the deepest receiving pit (approximately 13 mBGL, DWG 2010674.316) will be about 1.7m at the excavation, and less than 0.5m at a distance of about 30m from the excavation face. At 110m from the excavation, the drawdown will be zero. After the pit is backfilled, it will take about 90 days for the water level to recover. If SPW are used, the expected drawdown will be significantly lower (see section 7). Please refer to the independent settlement report (AECOM, 2015) for further details on how this drawdown would affect settlement.

Groundwater elevation

In general, groundwater levels are 1.5–5mBGL in this area, and the pipe invert will be at 2.7mBGL (1.5m clearance plus 1.2m diameter pipe) (Figure 6). Therefore, the groundwater level is not expected to be significantly higher than the pipe invert level in this area. No areas of unusually low permeability have been identified throughout this project area.

Groundwater inflow

An expected seepage inflow of less than 2m^3/d per metre of excavated trench is anticipated in locations where sediments of the Tauranga Group (e.g. Alluvium and Puketoka Formation) lay on top of the ECBF. In localised areas where the permeability of these materials could be unusually high (e.g. clean sand) this value could increase to up to 9m^3/d/m. However this is unlikely because sand material within the Tauranga Group will always contain an important proportion of fine material (e.g. silt) which will result in lower permeabilities. At locations where the trench is excavated directly into the ECBF, an expected seepage inflow of less than 0.2m^3/d per m of excavated trench is expected. At locations where the ECBF is more permeable this value could increase to up to 0.8m^3/d. In both these cases, discharge will stop as soon as the trench is backfilled.

Seepage inflow will occur at the excavations of pits used for pipe jacking during micro tunnelling. The deepest pit which has the potential to cause this effect is located at the Northern Western Motorway (SH18) crossing (2010674.316). Seepage inflow into this pit will be about 12 m^3/d. Discharge into the pit will stop once the tunnel is completed and the pit is backfilled.

Groundwater diversion

As in the previous areas, the final trench and temporary jacking and receiving pits are not expected to affect groundwater flow paths in the long term. Other structures (e.g. valve and scour chambers) will be permanent but generally above the groundwater level.

6.4 Potential impact on neighbouring groundwater users

Section 2.4 outlined the groundwater users that were identified from the Auckland Council bore database as being located within 1.5 km of the proposed works. In total, four groundwater bores that abstracted groundwater for domestic and stock purposes, were identified. There will be no effect on these abstractions as a result of the proposed works given the minimal drawdown anticipated during the construction work, as well as the fact that all of the bores abstract water from the deep aquifer.

7. Mitigation and management measures

Mitigation options to limit groundwater drawdown (and transport of contaminants) along the proposed NH2 alignment will be implemented depending on the type of construction methodology being used. In general, open trench excavations will induce low drawdown values at distances greater than 30m away from the excavations. Should drawdown close to open trenches be a concern (e.g. trench in close proximity to houses and infrastructure) the use of Sheet Pile Walls (SPW) during trench construction can effectively reduce drawdown to negligible levels (section 5.1.3). The use of SPW will effectively control groundwater inflow to negligible levels (<0.02 m^3/d per m of excavated trench). The Gull petrol station, located at 1-3 Forest Hill Road in Henderson, has been categorised as low to medium risk in the Soil and Groundwater Contamination Technical Report (Jacobs, 2015). At this location, clay cut-offs will be constructed to prevent potential contaminants from entering the trench and to inhibit groundwater flow along the trench where necessary, to maintain the existing groundwater regime.

Excavation of temporary jacking pits is likely to induce drawdown through unconsolidated soil materials as groundwater may seep through the excavation walls. This can be controlled by driving SPW past the excavation invert level so that there is an impermeable barrier between shallow groundwater and the excavation. The effectiveness of this mitigation measure has been assessed in section 5.2.2.

7.1 NOR 1: Titirangi to Hobsonville

The tunnelled sections along this section (Between Manuka Rd to Shetland St and Metcalfe Rd crossing) will be constructed via temporary jacking and receiving pits. As groundwater may seep into these pits during construction, the use of SPW will effectively reduce groundwater seepage. SPW will be driven to 1m below the excavation invert thus reducing drawdown to 0.08m at a distance of 30m from the excavation. Since groundwater drawdown due to pit excavation with SPW will be minimal, no monitoring will be required at these locations.

For the open trench sections through the Puketoka Formation, Alluvium, and ECBF, drawdown will be low (<1m) at 30m from the excavation. These trenches are going to be open for a relatively short period of time (e.g. not exceeding 3 weeks) so effects to nearby buildings will be minor as groundwater levels will recover after trench completion.

7.2 NOR 2: Greenhithe Bridge to Albany Reservoir

Open trenches through the Puketoka Formation and ECBF (DWG 2010673.526-533) will induce low drawdown values (<1m) at 30m from the trench excavation. The tunnelled sections along the Upper Harbour Motorway (DWG 2010674.322-327) will be excavated through ECBF material. As the temporary jacking and receiving pits may cause some localised drawdown during construction, SPW will be used to minimise induced drawdown.

The tunnelled section under the Bushlands reserve (DWG 2010674.331-.332) goes through the ECBF and groundwater may seep into the annulus as the tunnel is being constructed. The use of SPW will minimize drawdown due to pit excavation.

7.3 NOR 3: SH18 shared corridor

The tunnelled section under the North Western Motorway (DWG 2010674.316) goes through the Puketoka and ECBF. Drawdown due to the construction of temporary jacking and receiving pits will be controlled by using SPW to reduce groundwater seepage. The use of SPW driven to 1m below the excavation invert reduces drawdown to 0.18m at the excavation and to 0.08m at 30m from it; the water level will recover after the tunnel is completed.

Open trenches will be excavated at various locations (DWG 2010673.521-525) through the Puketoka Formation in this area. However, due to the short duration of the works, only localised groundwater drawdown is expected and this will be lower than 1m at 30m away from the trench excavation.

8. Monitoring Requirements

A monitoring programme will be developed to record groundwater effects and trigger appropriate responses. This program consists of a combination of flow monitoring (e.g. through tunnel annulus, pit excavations, and trench excavations) and water level monitoring in project piezometers. Flow monitoring in excavations fitted with SPW will take place regardless of minimum flows after these mitigation measures are installed, to test their effectiveness. Baseline piezometer monitoring data taken in advance of works will be used to obtain seasonal and annual variations, and these will be used to define alert and alarm trigger levels based on average water levels.

8.1 NOR 1: Titirangi to Hobsonville

Groundwater seepage through the annulus of the Manuka Rd to Shetland St tunnel (DWG 2010674.301-302) will be collected as the tunnel is being drilled. Thus, the discharge rate will be monitored on a daily basis (collected at the start/end sections). Similarly, seepage rates into the jacking and receiving pits for both this tunnel and the Metcalfe Rd crossing tunnel (DWG 2010674.312) will be monitored. Seepage rates into the open trenches will be monitored to assess potential issues due to open trench excavations. Once the tunnels and the trenches are completed no monitoring will be necessary.

Baseline monitoring of project piezometers will continue to be carried out to obtain seasonal and annual variations; this has been underway for bores installed at selected stream crossings (BH251, BH252, BH253, BH253, BH256, BH257, BH258, BH261, BH263, and BH268 (Figure 4)) and it is recommended this should continue throughout the construction period.

8.2 NOR 2: Greenhithe Bridge to Albany Reservoir

Groundwater seepage into the annulus of the 354m section trenchless section under Bush Rd (DWG 2010674.331-.332) will be monitored on a daily basis. Similarly, seepage from the annulus of the Upper Harbour Motorway tunnels (DWG 2010674.322-327) will be monitored daily as construction takes place. Seepage rates into temporary jacking and receiving pits will be monitored on a daily basis during construction.

Seepage rates into the open trenches (DWG 2010673.526-533) will be monitored to assess potential issues due to open trench excavations. Baseline monitoring of project piezometers will continue to be carried out to obtain seasonal and annual variations. This has been underway for bores installed at selected stream crossings (BH204 and BH265 (Figure 5)) and is recommended this should continue throughout the construction period.

8.3 NOR 3: SH18 shared corridor

Seepage of groundwater into the temporary jacking and receiving pits for the tunnelled section under the North Western Motorway (DWG 2010674.316) will be monitored on a daily basis. Similarly, seepage will be monitored on a daily basis in open trenches (DWG 2010673.521-525) as the water main is constructed. Project bores BH201, BH202, and BH203 will continue to be monitored throughout the duration of the project (Figure 6).

9. Conclusions

The majority of the NH2 alignment traverses through Alluvium, the Puketoka Formation, and the ECBF. In addition, some locations close to pre-existing infrastructure projects contain fill material (e.g. Puketoka Formation soils). The NH2 alignment will be completed via open trenches with the water main pipe above the local groundwater levels. For the majority of these cases, the water level will be below the trench excavation invert and no groundwater will seep into the open trench. In situations where the water level is higher (e.g. above the average water level of 3.8mBGL), groundwater may seep into the open trench excavation as the pipe is being laid. In cases where the water main is expected to cross or traverse major roads, trenchless methods will be employed. In these situations, the temporary jacking or receiving pits can potentially induce drawdown during construction. In general, the completed NH2 watermain pipe will be above the groundwater level so there will be no direct interaction with groundwater. In the case of seasonally high groundwater levels affecting deeper features (e.g. valves and scour chambers), any potential diversions will be in the direction of natural flow paths and will not change the overall regime of groundwater flow.

Groundwater seepage rates into excavation areas will be reasonably low (0.2-9 m^3/d per m of trench) and, if necessary, can be managed with SPW on open face excavations. Groundwater levels are expected to recover within days after the excavations are completed (e.g. 20-100 days for the trenches and 90 days for the temporary pits). Similarly, seepage will stop at the deep tunnelled sections (Woodlands Park Reservoir at NOR1 and under the Bushlands Reserve at NOR2), after the tunnels are completed and the excavation annuli are grouted.

Groundwater is not being extensively used in the areas through which the NH2 alignment traverses, and the only bores identified throughout this study are located in deep aquifers which will not be affected by construction activities. Mitigation measures, which include the use of SPW during construction, will be effective at controlling groundwater drawdown and discharge. Monitoring of project bores and inflow rates (e.g. seepage into excavations and tunnel annulus during construction) will be used to produce alert and alarm levels to ensure groundwater drawdown in the surrounding areas stay within negligible values.

Groundwater effects associated with the construction and emplacement of the NH2 watermain are expected to be no more than minor. The following areas highlight significant project features which have been covered in this assessment.

9.1 NOR 1: Titirangi to Hobsonville

The water main in this area is going to go through a tunnelled section through sandstones and mudstones (Nihotupu and Cornwallis formations) between Manuka Rd and Shetland St. During construction, seepage of groundwater into the tunnel annulus is expected not to exceed 23.4m^3/d. Groundwater seepage rates will be monitored at the tunnel end sections on a daily basis. This section of the project includes the Metcalfe road crossings which may be completed by an optional trenchless method. This option will cause minimum drawdown in the surrounding area. The remainder of the NH2 alignment will be built via open trench sections going through Alluvium, Puketoka Formation material, and the ECBF. Drawdown of groundwater due to the presence of the open trench is predicted to be minimal (e.g. less than 1m at a distance of about 30m from the open trench). Any drop in water level due to trench excavation is expected to recover to normal levels after trench completion (e.g. 20 days).

9.2 NOR 2: Greenhithe Bridge to Albany Reservoir

This section of the alignment mainly goes through the ECBF and the Puketoka Formation. The watermain pipe will be primarily constructed via an open trench method but will also include trenchless excavations (e.g. tunnels) through and along the Upper Harbour Motorway as well as a tunnelled section under the Bushlands reserve. Potential drawdown due to open trench excavation is expected to be minimal away from the excavation face (e.g. <1m at 30m from the trench and tapering off to zero beyond this distance). The temporary jacking and receiving pits associated with tunnelled sections are expected to cause minimum drawdown as these will be fitted with SPW which will prevent significant groundwater seepage during construction. The tunnel section under Bushlands Reserve (NOR2, DWG 2010674.331-.332) will induce minimum seepage rates into this tunnel annulus (< 23.4m^3/d) as the tunnel is being constructed.

9.3 NOR 3: SH18 shared corridor

In this area, drawdown is predicted within loose sediments of the Tauranga Group (e.g. Puketoka and Alluvium) but some locations may include fill material which will most likely consist of loose or compacted Puketoka Formation soils. This section of the alignment will be completed via open trench excavations. Temporary drawdown caused by these excavations is expected to be negligible (lower than 1m at 30m away from the excavation face and tapering off to zero beyond this distance). Groundwater levels will recover to pre-construction conditions within a matter of days (e.g. 20 days with ECBF material and 100 days with fill material) so the risk of ground settlements occurring in response to groundwater drawdown is low. This section also includes a tunnelled section constructed under the North Western Motorway (DWG 2010674.316). A temporary jacking pit, excavated to a depth about 13.8mBGL, will go through a 4.8m thick layer of Alluvium and Puketoka Formation material which may also include fill material, followed by the ECBF. This pit will be fitted with SPW driven to 1m past the excavation invert, to prevent drawdown issues. Drawdown at the pit will be about 0.18m at the excavation, and less than 0.08m at a distance of about 30m from the excavation face; the water level will recover after the tunnel is completed.

10. References

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Appendix A. Figures

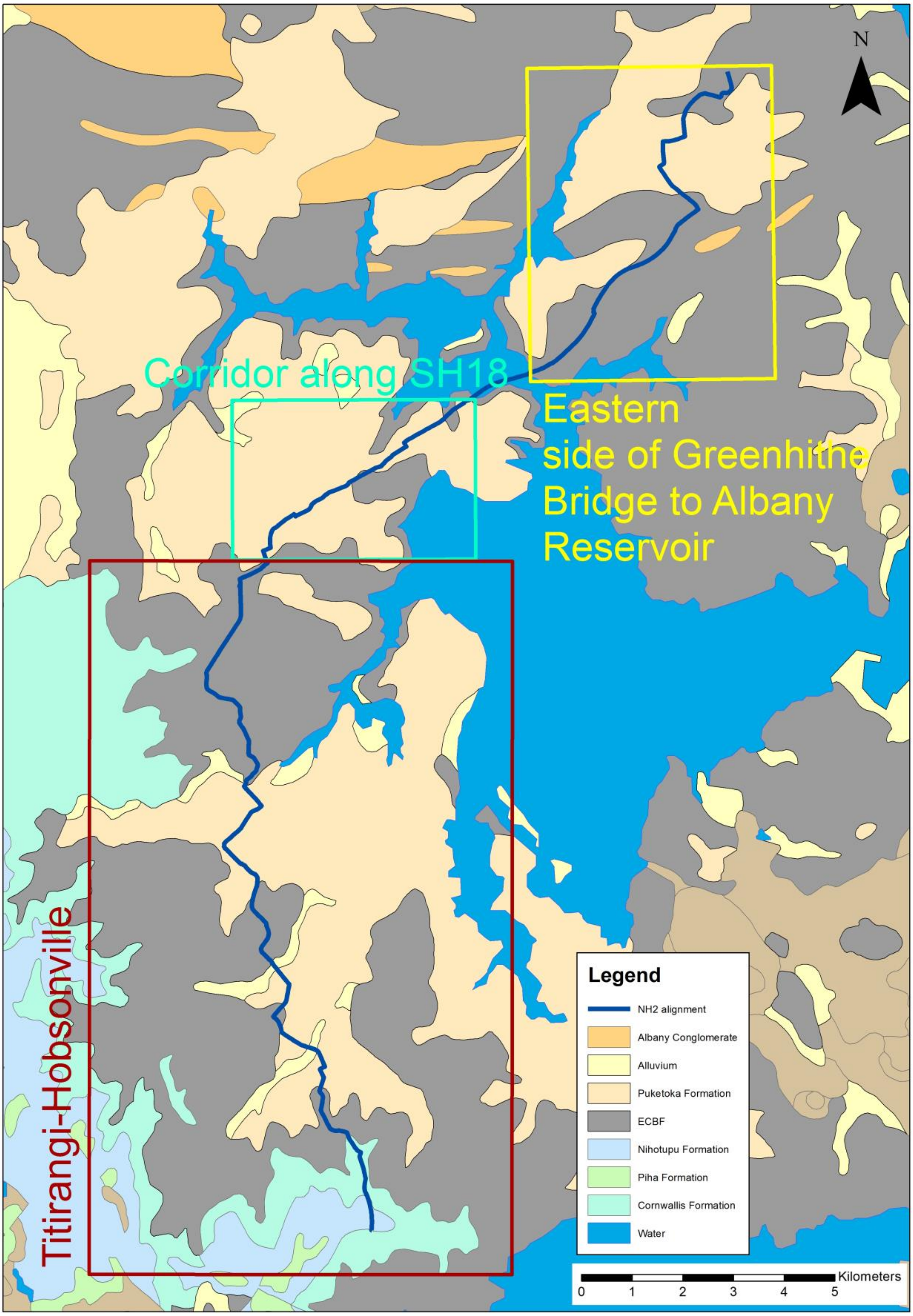


Figure 2. Geology along the route of the proposed alignment

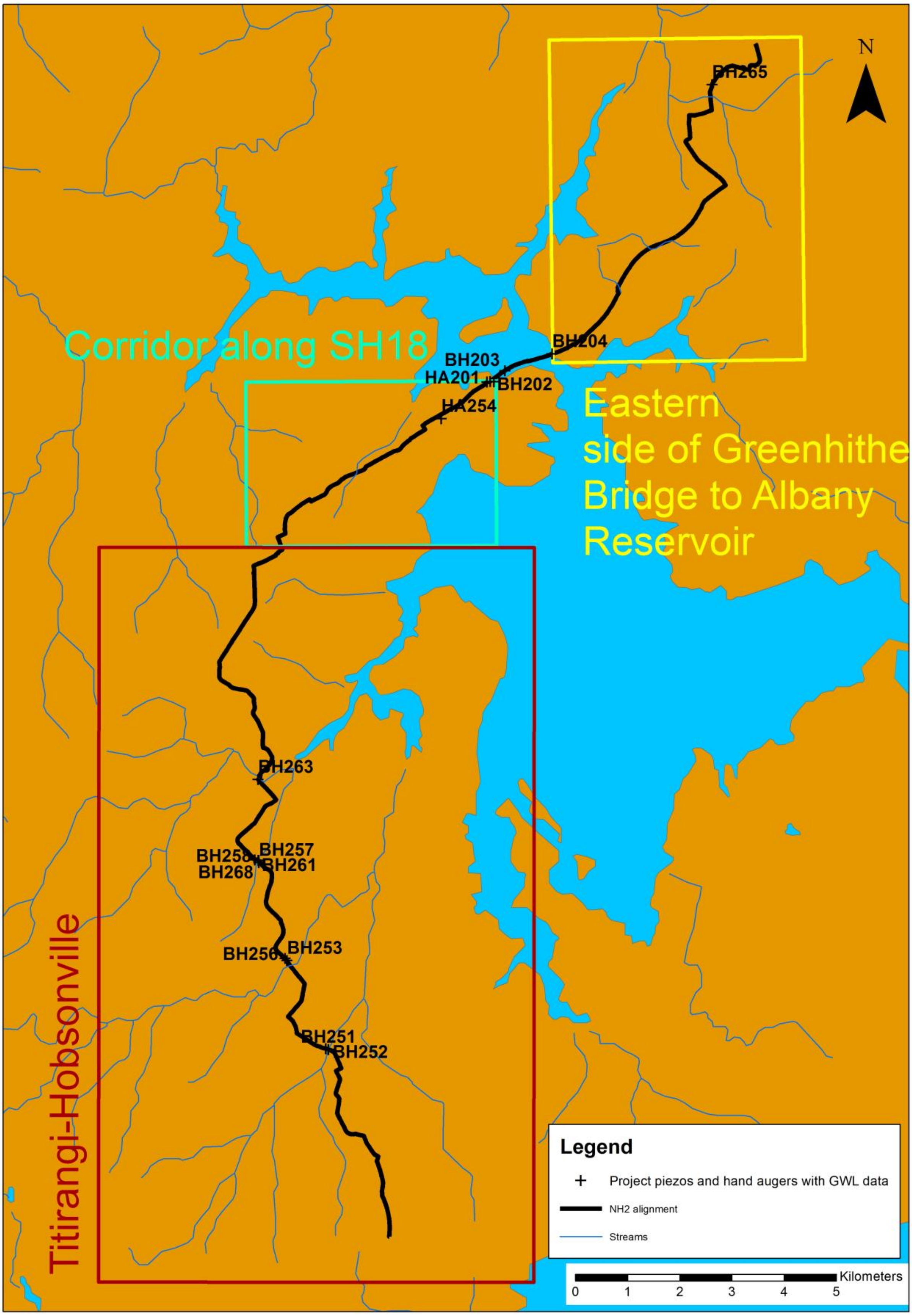


Figure 3. Location of project bores and hand auger excavations. Bores IDs start with BH and hand auger excavation IDs start with HA.

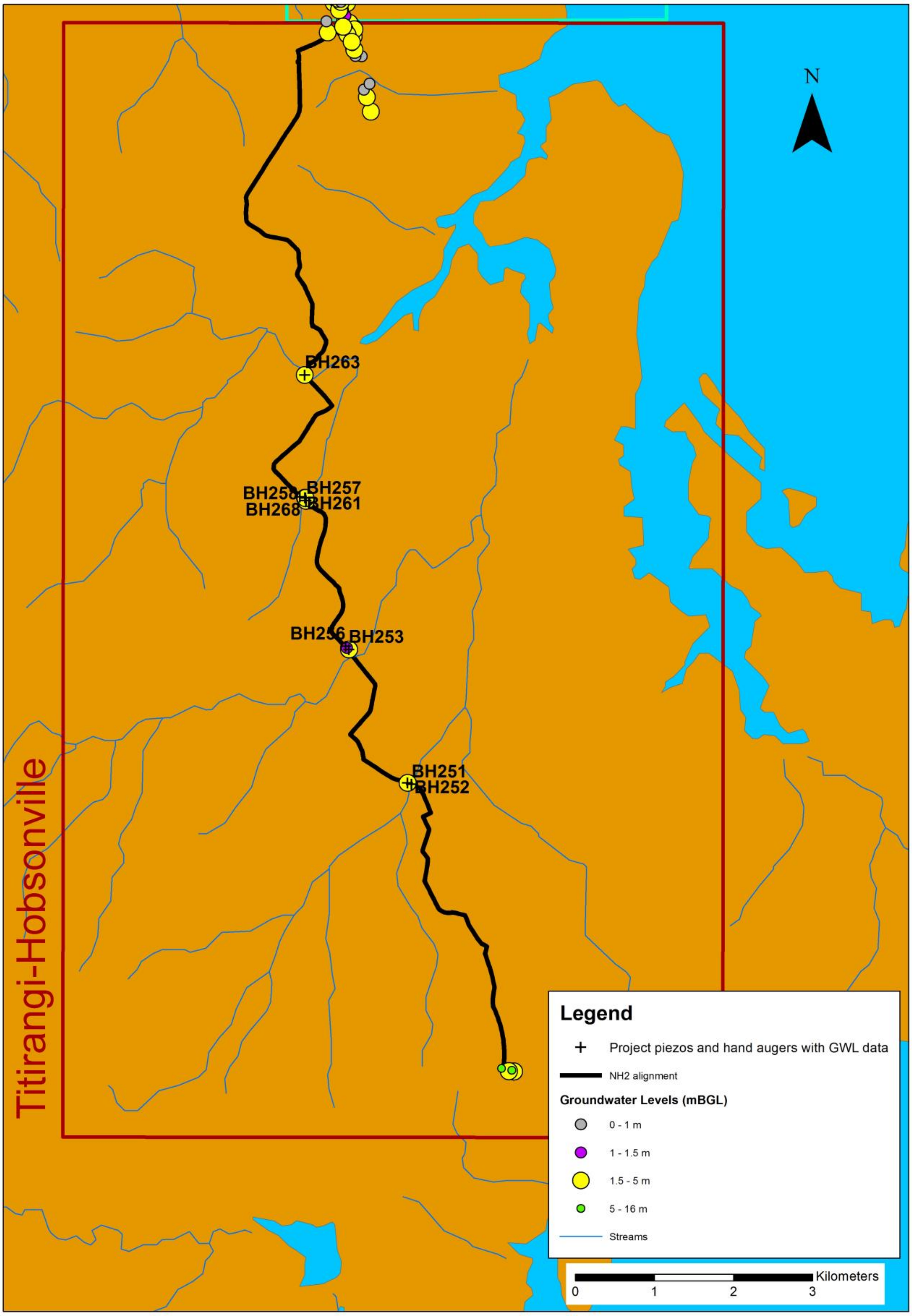


Figure 4. NOR 1 groundwater levels for project bores and bores from previous investigations

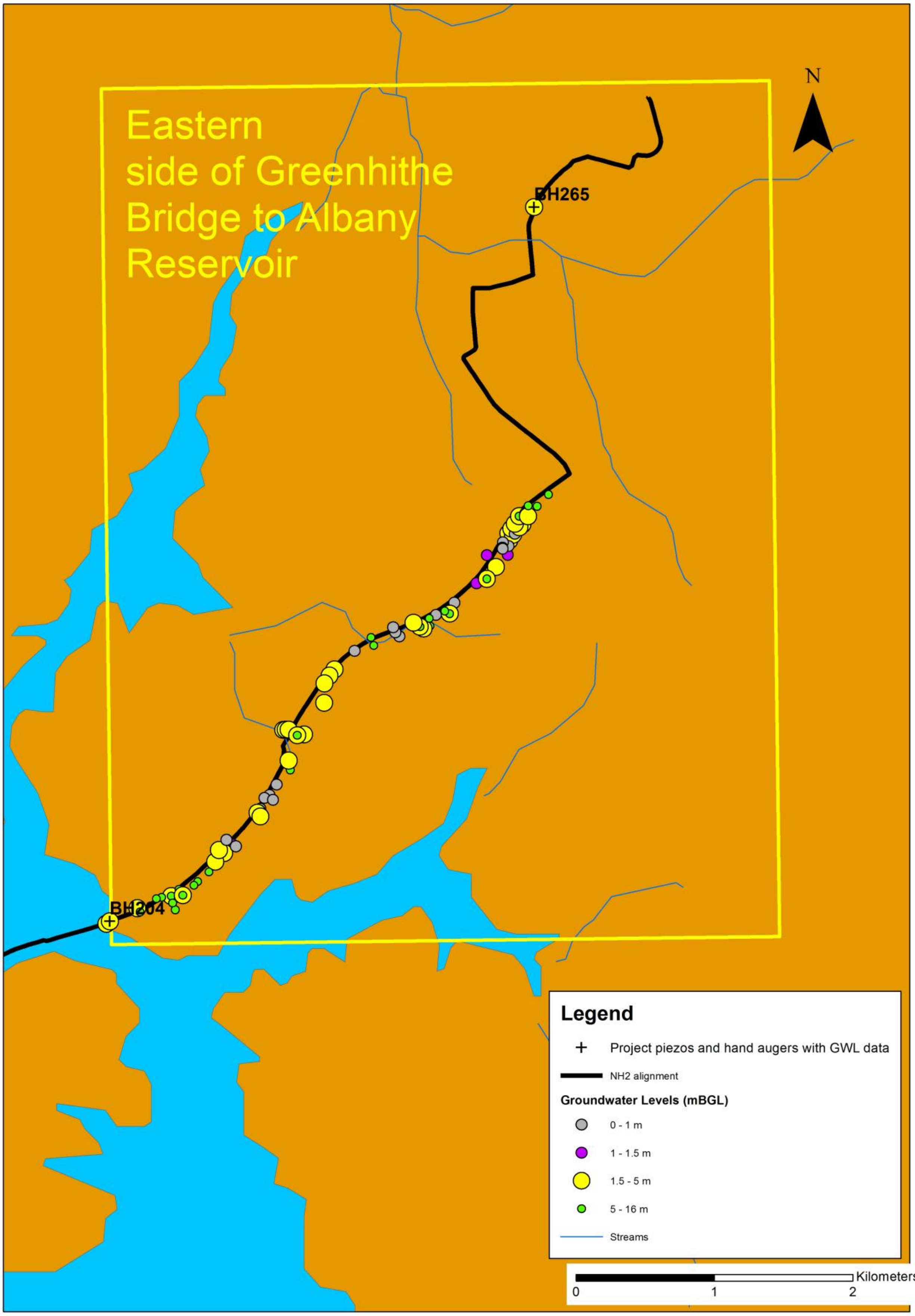


Figure 5. Groundwater levels to the eastern side of Greenhithe Bridge to Albany Reservoir (NOR 2).

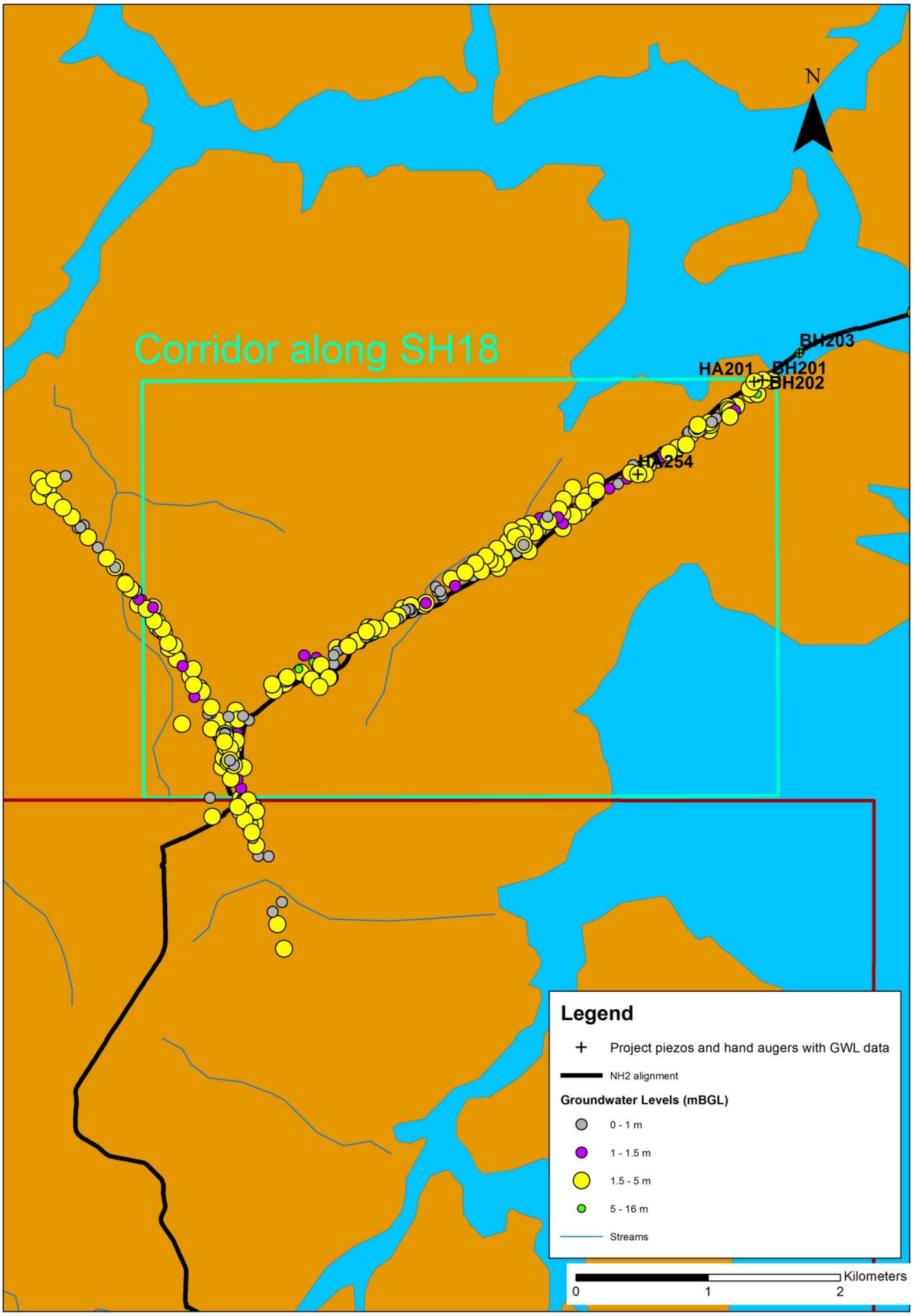


Figure 6. Groundwater levels for NOR 3

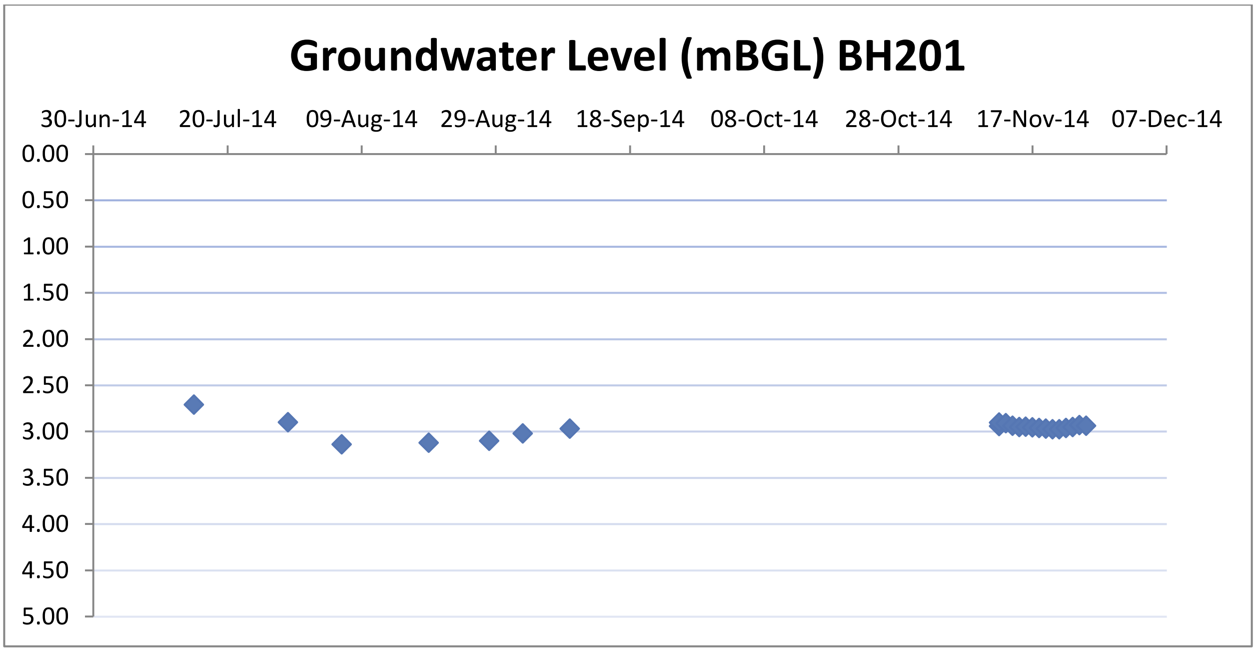


Figure 7. Hydrograph for BH201

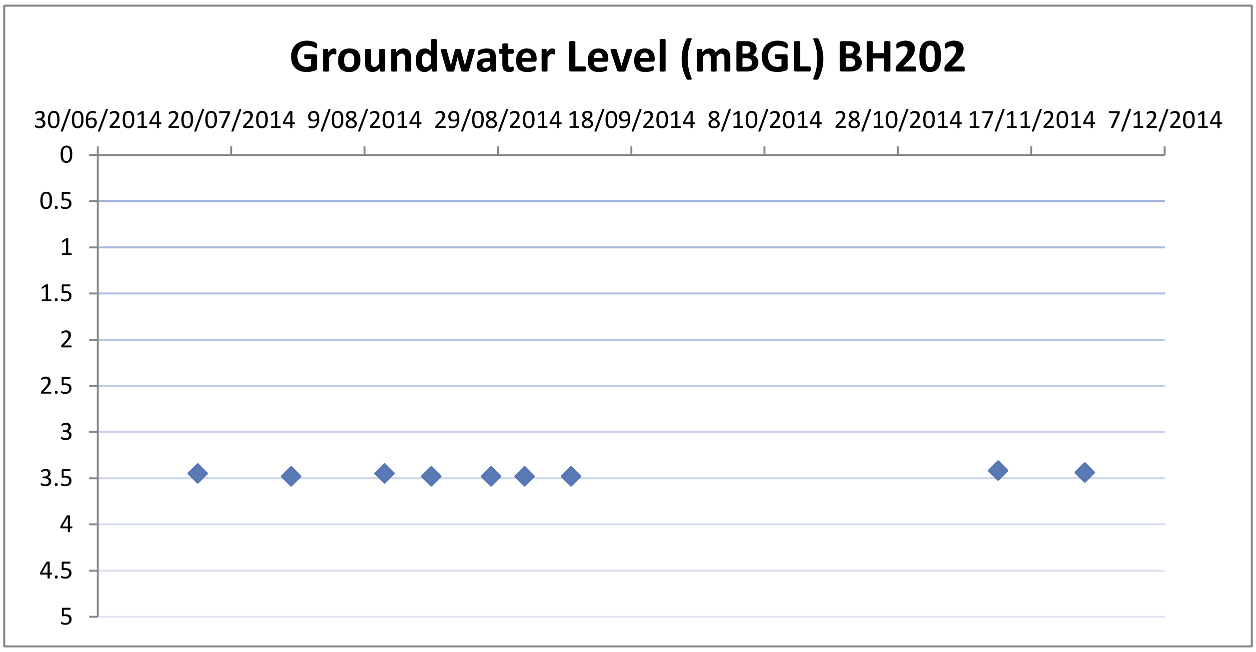


Figure 8. Hydrograph for BH202

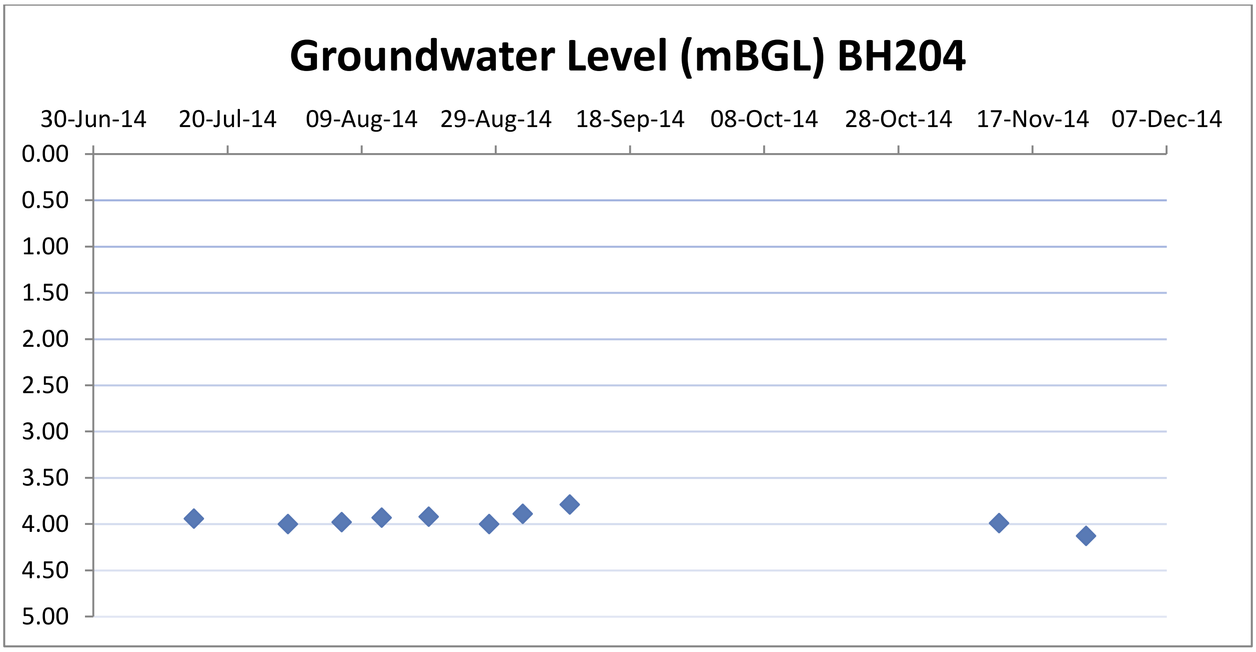


Figure 9. Hydrograph for BH204

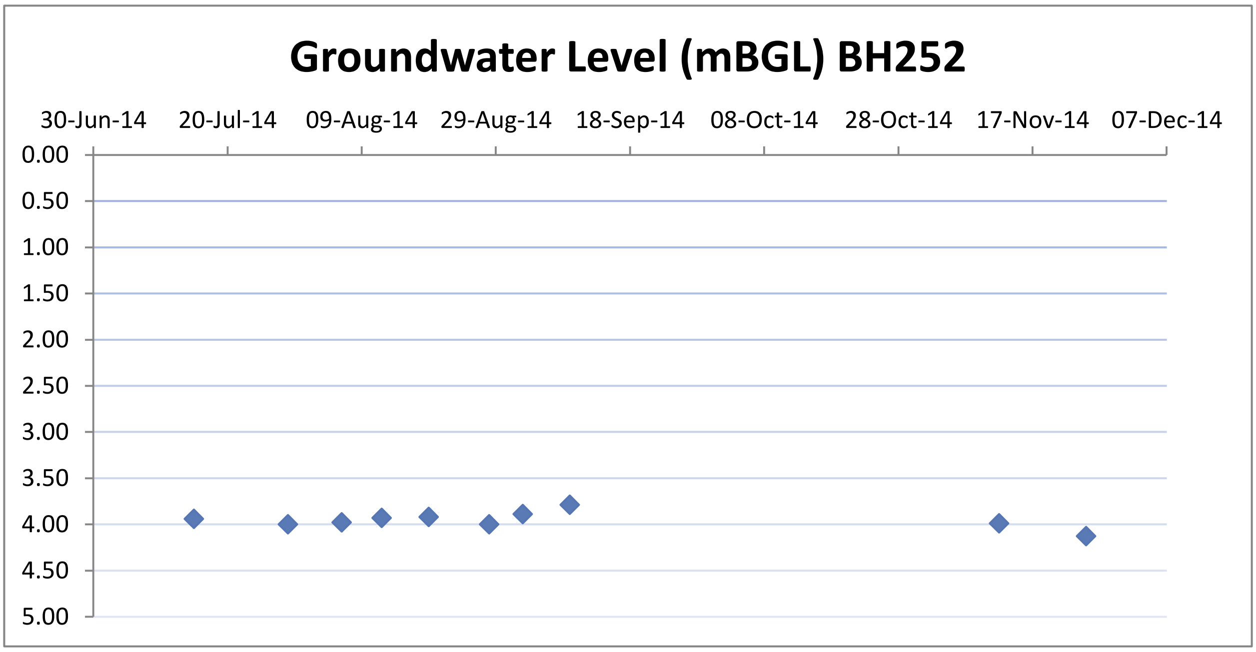


Figure 10. Hydrograph for BH252

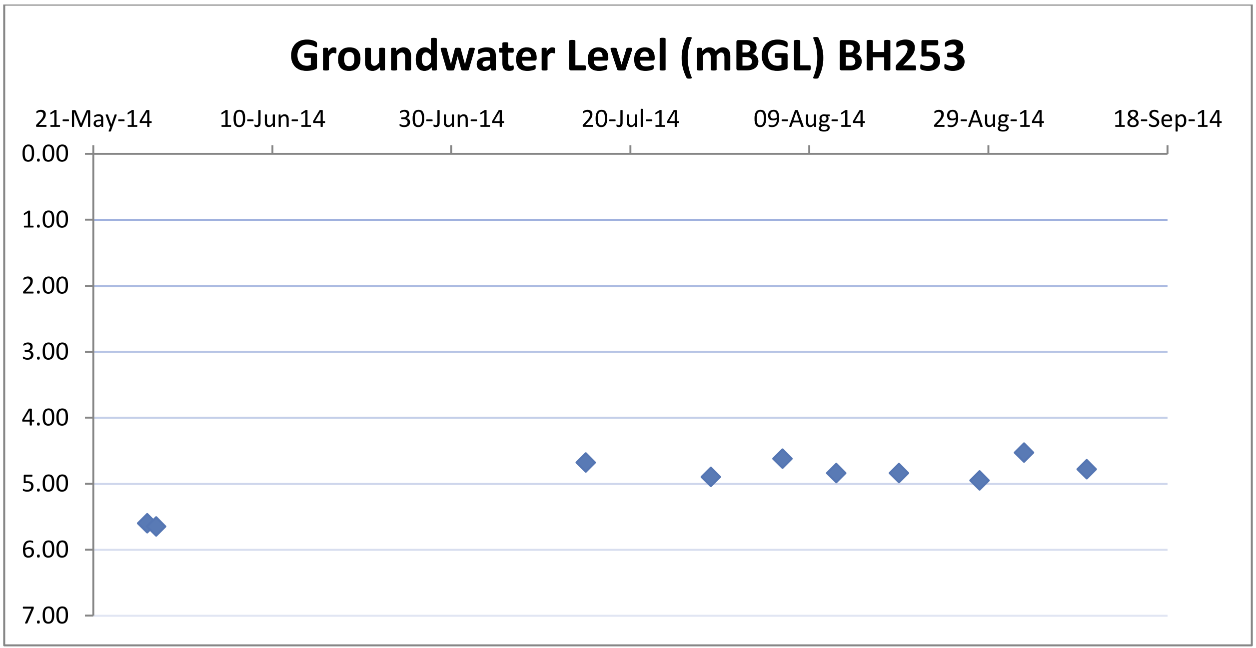


Figure 11. Hydrograph for BH253

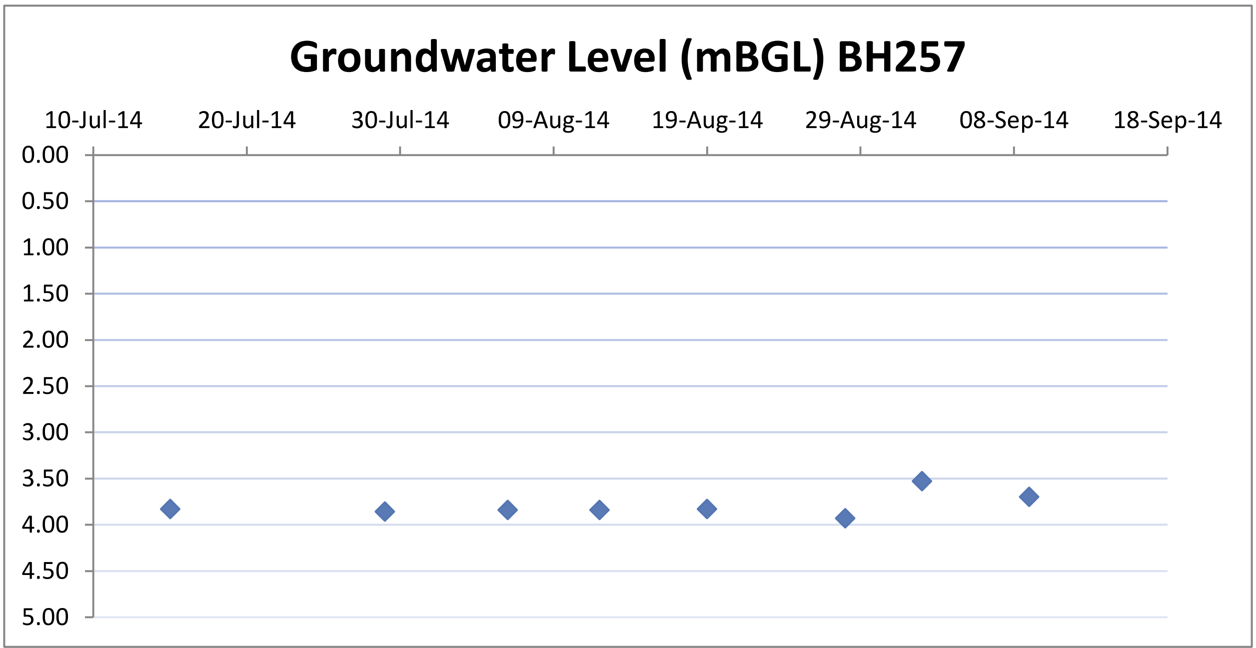


Figure 12. Hydrograph for BH257

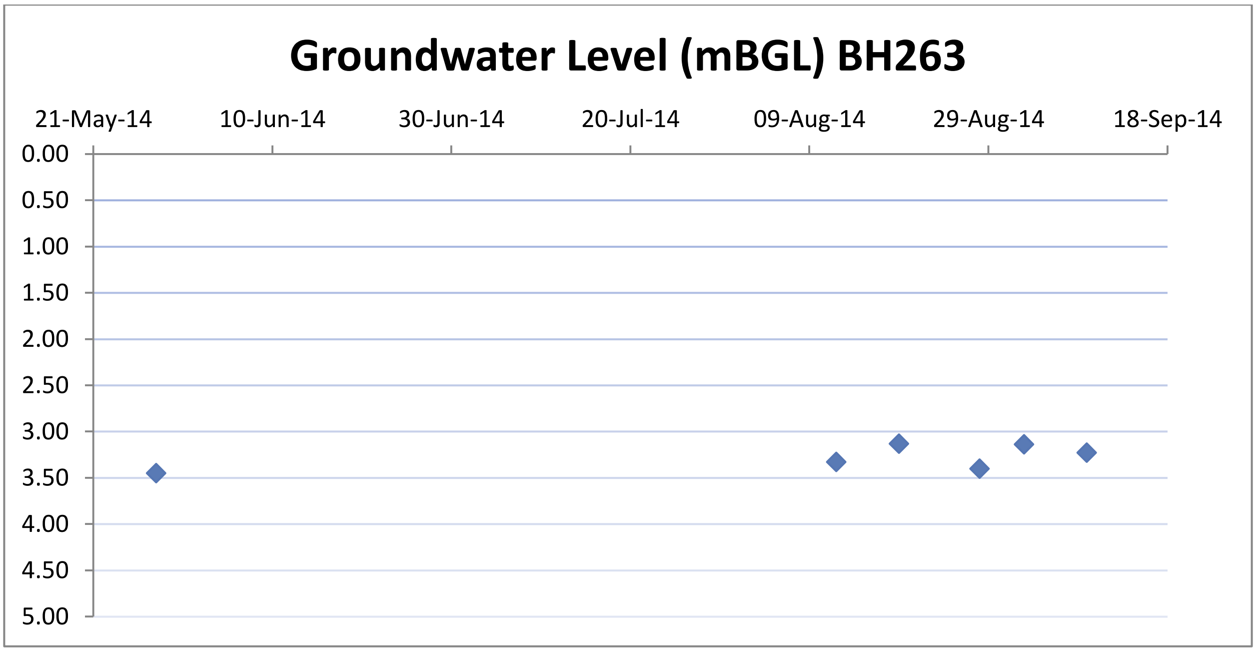


Figure 13. Hydrograph for BH263

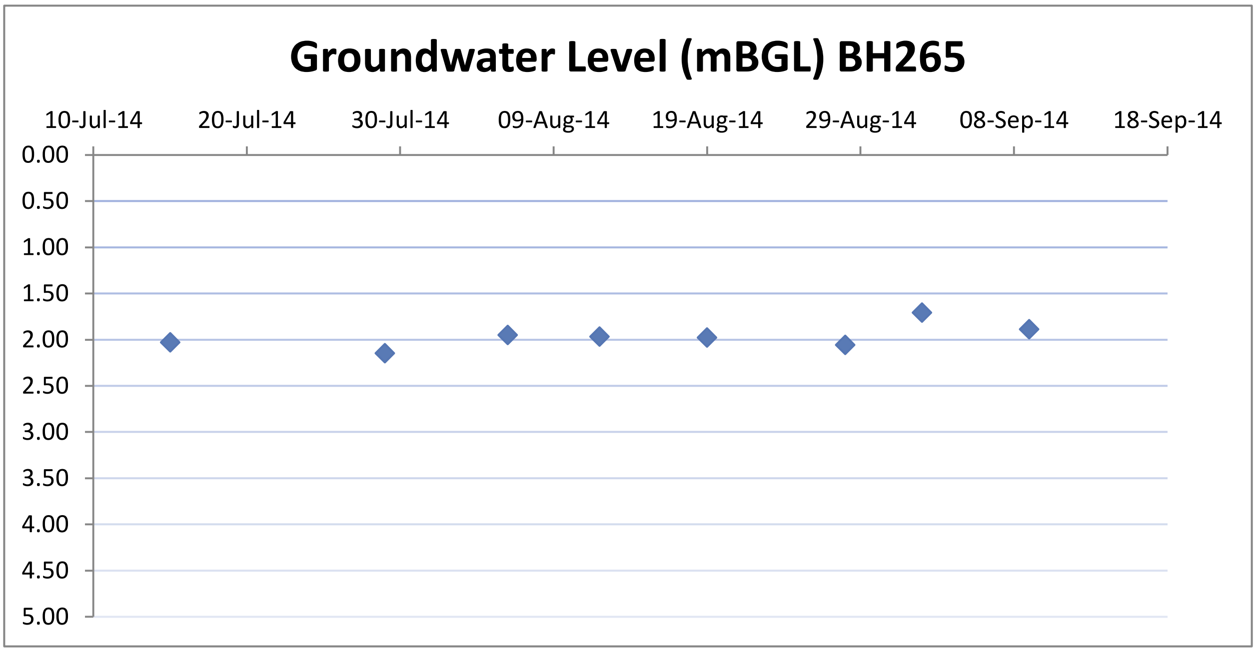


Figure 14. Hydrograph for BH265

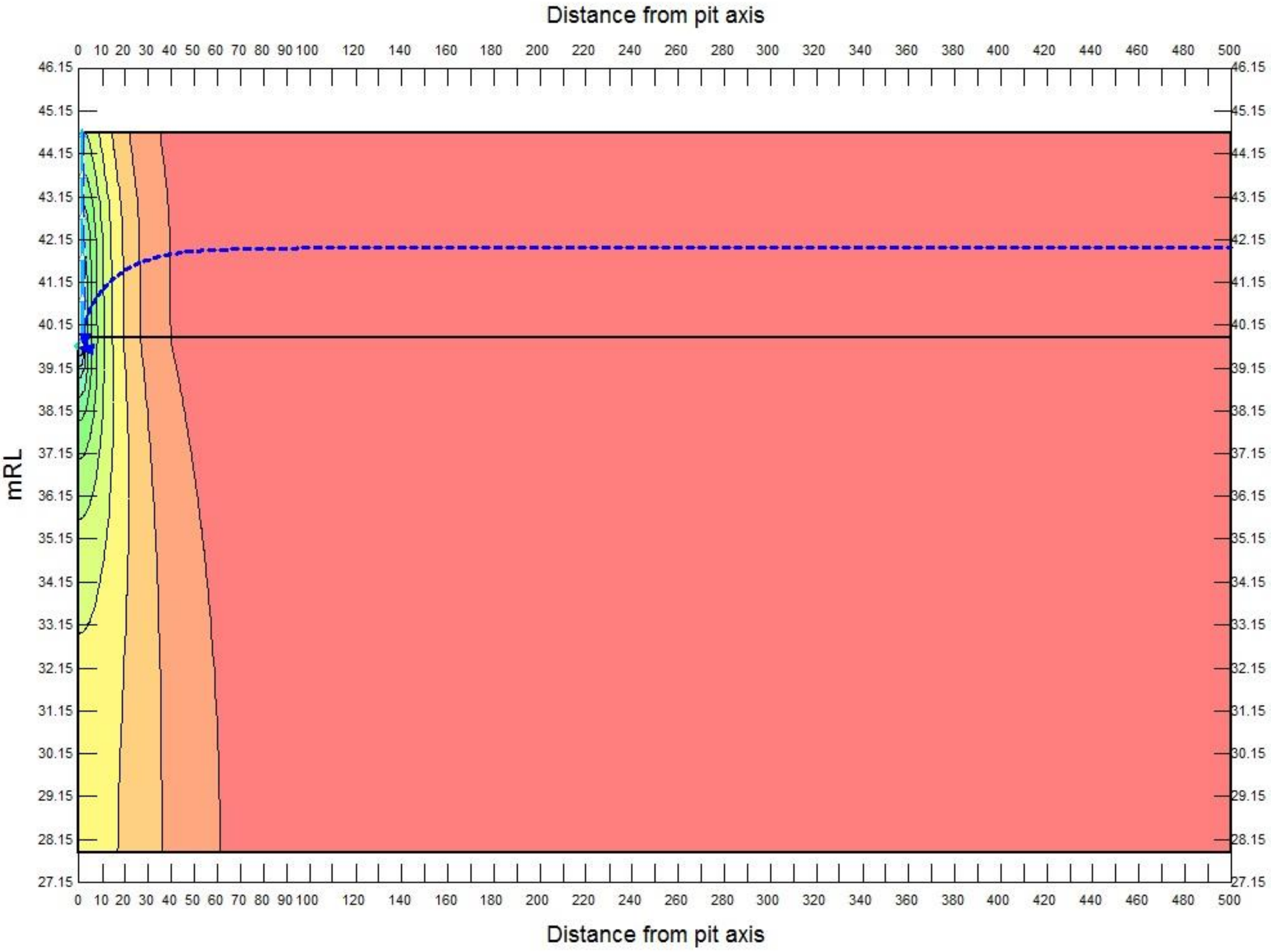


Figure 15. Open Trench model of Fill/ECBF framework

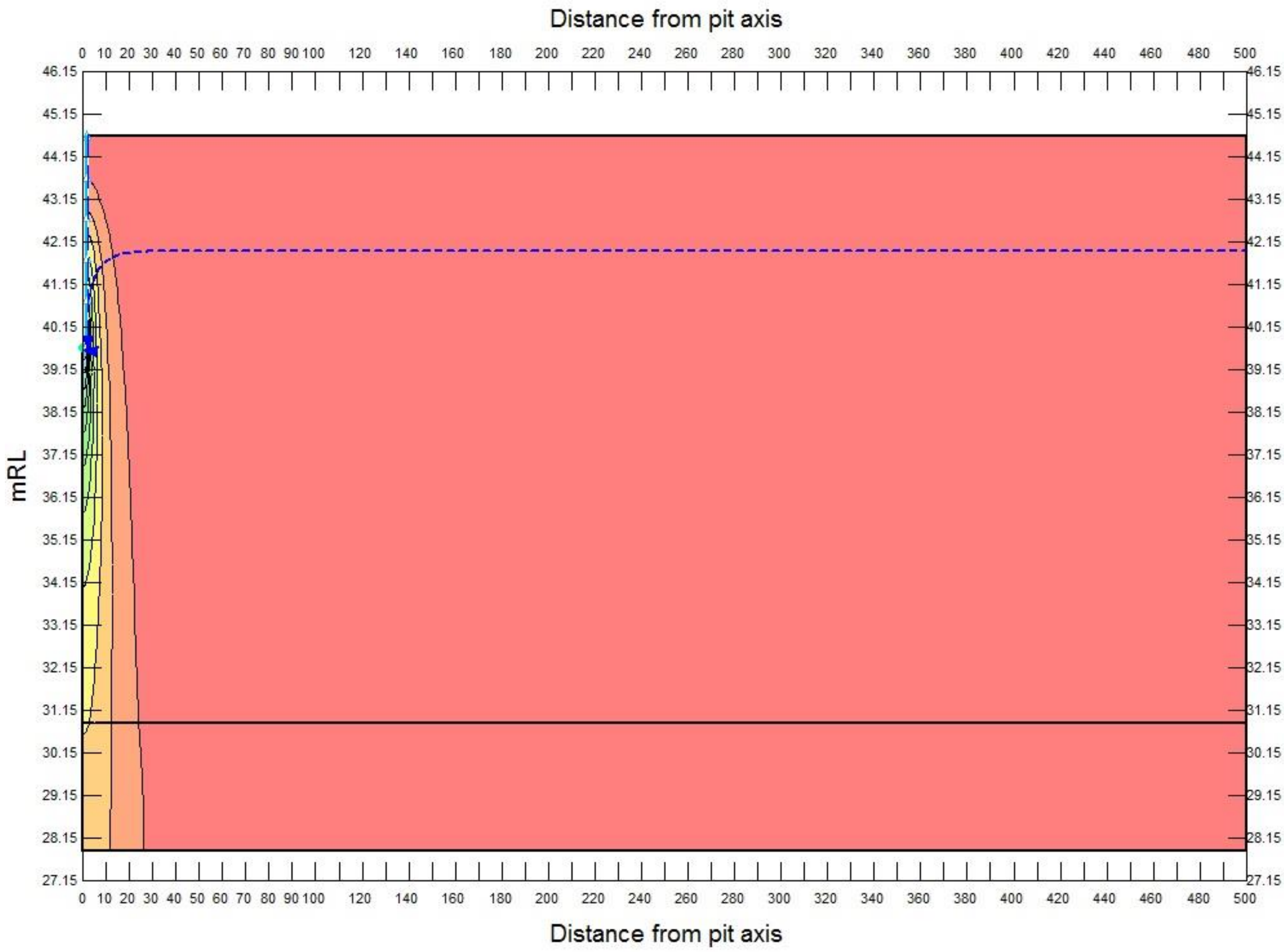


Figure 16. Open trench model of ECBF framework

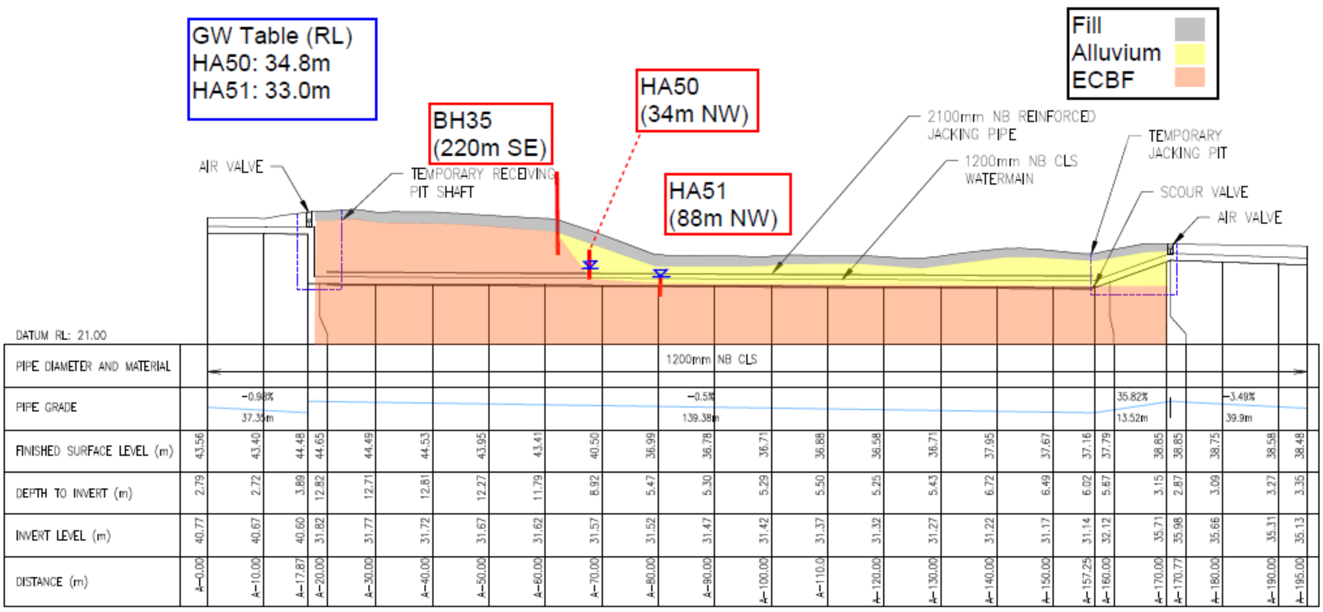


Figure 17. Model cross section for SH18 crossing (DWG 20210674.316)

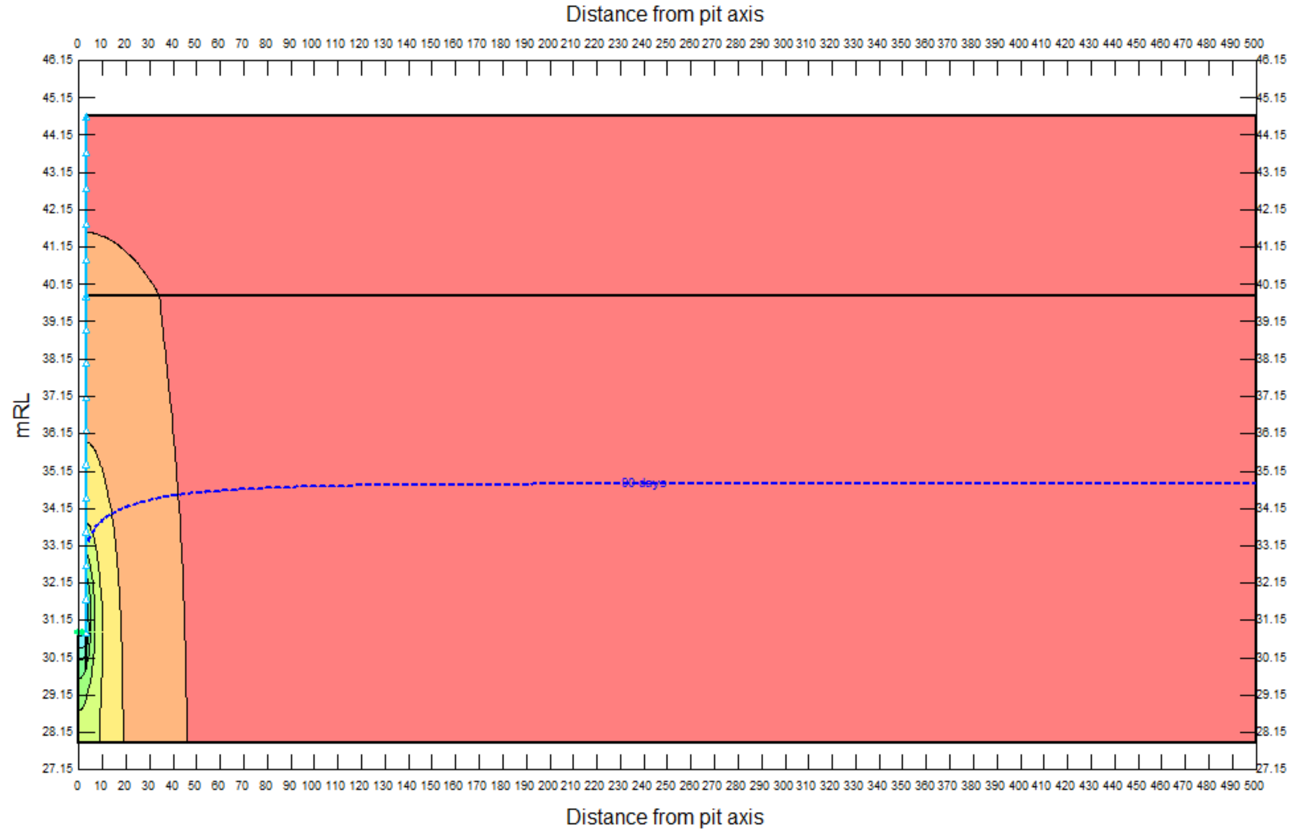


Figure 18. Open Pit Model Fill/ECBF with no mitigation measures in place

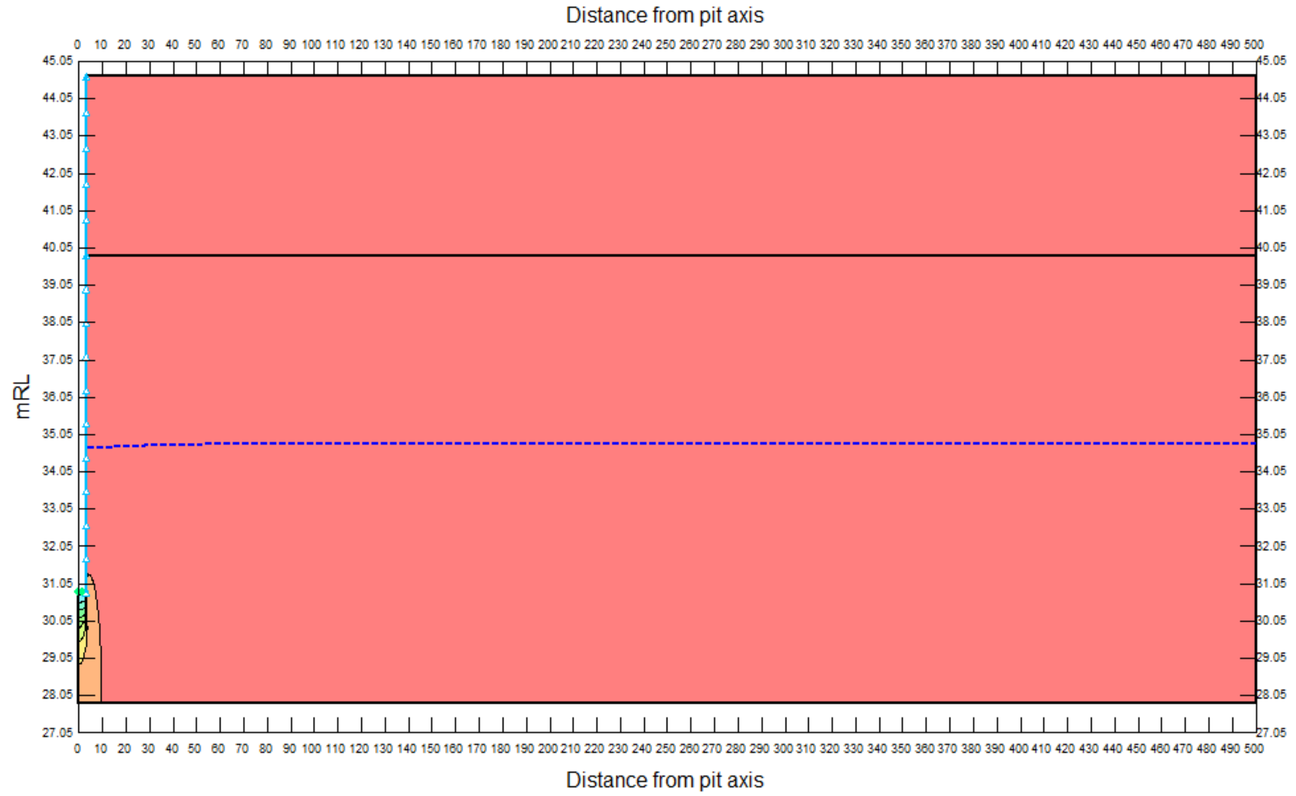


Figure 19. Open Pit Model of Fill/ECBF with Sheet Pile Walls as mitigation option

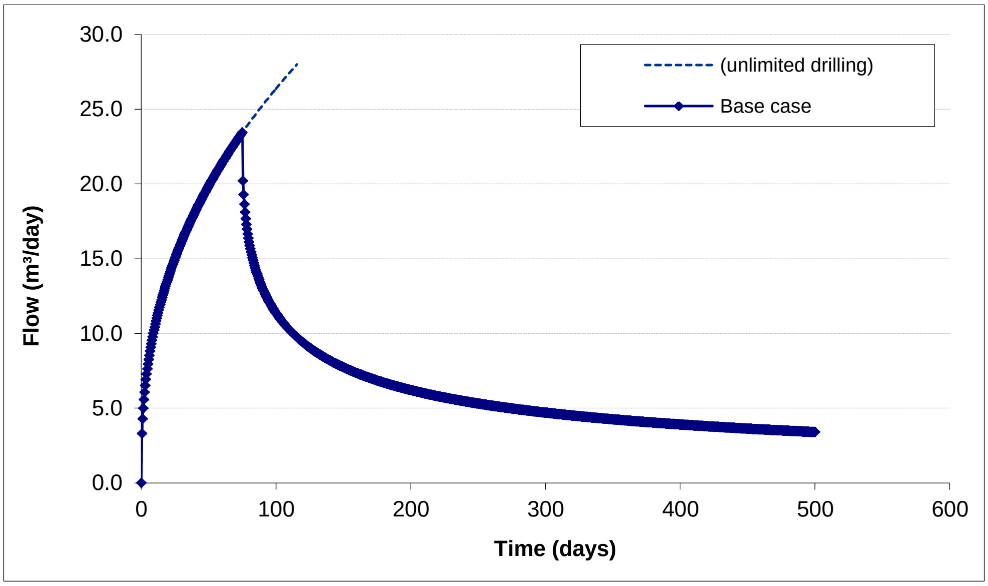


Figure 20. Discharge rate over time for tunnelled section.

Appendix B. Slug Test Results

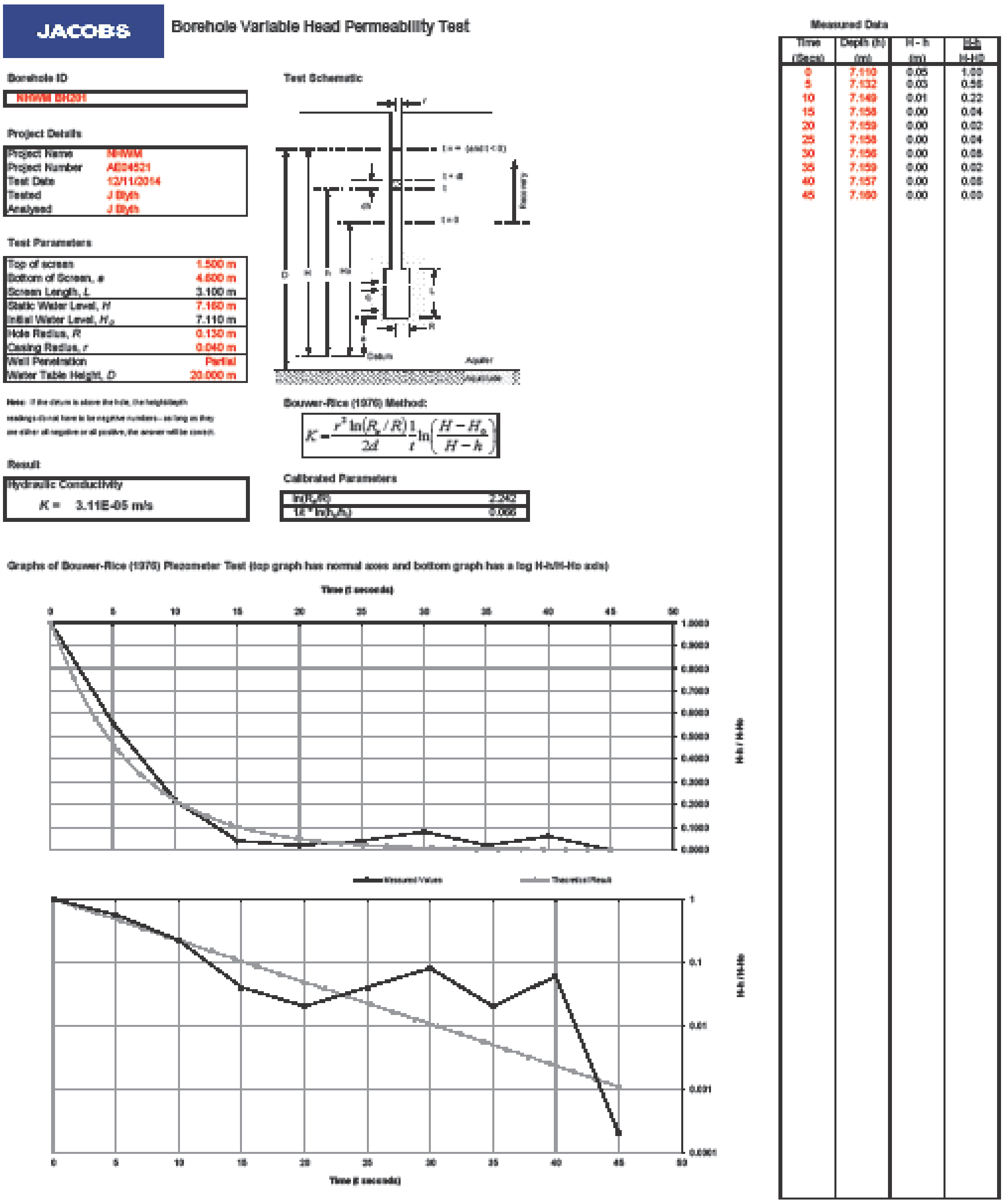
Table 10-1. Slug Test results from tests at selected bores

Bore ID	Date of test	Type of test	Hydraulic conductivity (m/s)	Geology	Accuracy of test
BH201	12/11/2014	RHT	3.11×10^{-5}	Tauranga Group	
BH202	25/11/2014	RHT	1.43×10^{-5}	Tauranga Group	
BH204	12/11/2015	RHT	8.42×10^{-6}	Tauranga Group	
	25/11/2014	RHT	2.21×10^{-5}	Tauranga Group	
BH257	12/11/2014	RHT	-		Logger at 5 s intervals however recovery was too rapid to analyse
BH265	10/11/2014	FHT	-		Logger at 10 s intervals however recovery was too rapid to analyse
	10/11/2014	FHT	-		Logger at 0.5 s intervals however recovery was too rapid to analyse
	10/11/2014	RHT	-		

Notes:

RHT = Rising Head Test

FHT = Falling Head Test



Borehole ID

NHWM BH201

Project Details

Project Name	NHWM
Project Number	AE04521
Test Date	12/11/2014
Tested	J Blyth
Analysed	J Blyth

Test Parameters

Top of screen	1.500 m
Bottom of Screen, e	4.600 m
Screen Length, L	3.100 m
Static Water Level, H	7.160 m
Initial Water Level, H_o	7.110 m
Hole Radius, R	0.130 m
Casing Radius, r	0.040 m
Well Penetration	Partial
Water Table Height, D	20.000 m

Bouwer-Rice (1976) Method:

$$K = \frac{r^2 \ln(R_e / R)}{2d} \frac{1}{t} \ln\left(\frac{H - H_o}{H - h}\right)$$

Result

Hydraulic Conductivity

$K =$ 3.11E-05 m/s

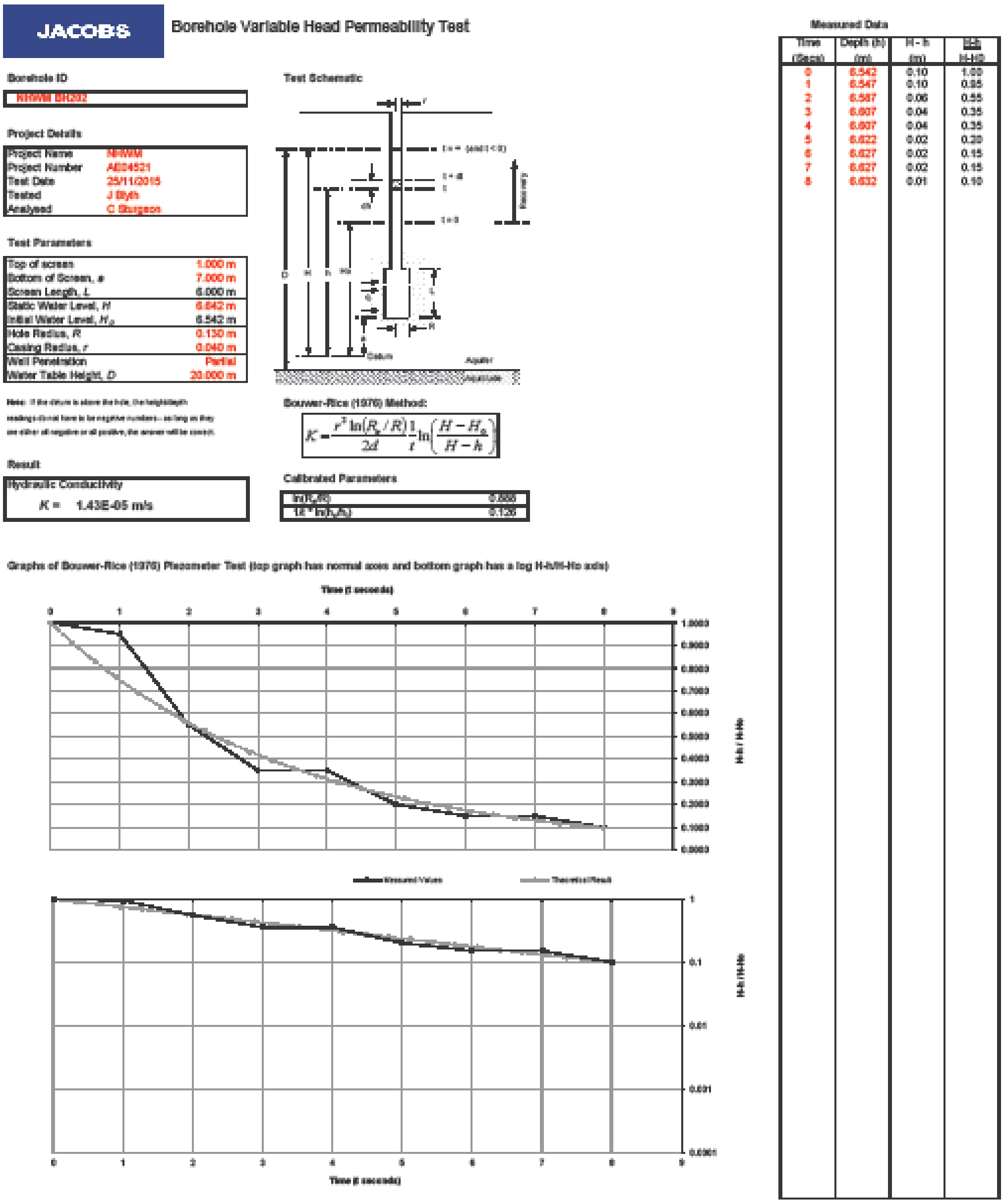
Calibrated Parameters

$\ln(R_e/R)$	2.242
$1/t * \ln(h_o/h_t)$	0.066

Measured Data

Time (Secs)	Depth (h) (m)	H - h (m)	H-h / H-H0
0	7.110	0.05	1.00
5	7.132	0.03	0.56
10	7.149	0.01	0.22
15	7.158	0.00	0.04
20	7.159	0.00	0.02
25	7.158	0.00	0.04
30	7.156	0.00	0.08
35	7.159	0.00	0.02
40	7.157	0.00	0.06
45	7.160	0.00	0.00

Figure 21. Slug Test for BH201



JACOBS Borehole Variable Head Permeability Test

Borehole ID

NHWM BH202

Project Details

Project Name	NHWM
Project Number	AE04521
Test Date	25/11/2015
Tested	J Blyth
Analysed	C Sturgeon

Test Parameters

Top of screen	1.000 m
Bottom of Screen, a	7.000 m
Screen Length, L	6.000 m
Static Water Level, H	6.642 m
Initial Water Level, H_o	6.542 m
Hole Radius, R	0.130 m
Casing Radius, r	0.040 m
Well Penetration	Partial
Water Table Height, D	20.000 m

Result

Hydraulic Conductivity

$K =$ 1.43E-05 m/s

Test Schematic

Bouwer-Rice (1976) Method:

$$K = \frac{r^2 \ln(R_e / R)}{2d} \frac{1}{t} \ln\left(\frac{H - H_o}{H - h}\right)$$

Calibrated Parameters

$\ln(R_e/R)$	0.888
$1/t \cdot \ln(h_o/h_t)$	0.126

Measured Data

Time (Secs)	Depth (h) (m)	H - h (m)	H-h / H-Ho
0	6.542	0.10	1.00
1	6.547	0.10	0.95
2	6.587	0.06	0.55
3	6.607	0.04	0.35
4	6.607	0.04	0.35
5	6.622	0.02	0.20
6	6.627	0.02	0.15
7	6.627	0.02	0.15
8	6.632	0.01	0.10

Graphs of Bouwer-Rice (1976) Piezometer Test (top graph has normal axes and bottom graph has a log H-h/H-Ho axis)

Figure 22. Slug Test for BH202

JACOBS **Borehole Variable Head Permeability Test**

Borehole ID

NHWM BH204

Project Details

Project Name	NHWM
Project Number	AE04521
Test Date	12/11/2014
Tested	J Blyth
Analysed	J Blyth

Test Parameters

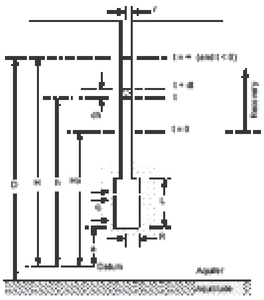
Top of screen	2.500 m
Bottom of Screen, *a*	7.500 m
Screen Length, *L*	5.000 m
Static Water Level, *H*	6.135 m
Initial Water Level, H_0	6.080 m
Hole Radius, *R*	0.130 m
Casing Radius, *r*	0.040 m
Well Penetration	Partial
Water Table Height, *D*	20.000 m

Note: If the datum is above the hole, the height/depth readings d (and h) have to be negative numbers – as long as they are either all negative or all positive, the answer will be correct.

Result

Hydraulic Conductivity
K = 8.42E-06 m/s

Test Schematic



Bouwer-Rice (1976) Method:

$$K = \frac{r^2 \ln(R_e / R)}{2d} \frac{1}{t} \ln\left(\frac{H - H_0}{H - h}\right)$$

Calibrated Parameters

$\ln(R_e/R)$	1.940
$1/t * \ln(h_0/h_t)$	0.030

Measured Data

Time (Secs)	Depth (h) (m)	H - h (m)	H-h / H-H0
0	6.080	0.05	1.00
5	6.095	0.04	0.73
10	6.119	0.02	0.29
15	6.126	0.01	0.16
20	6.125	0.01	0.18
25	6.129	0.01	0.11
30	6.133	0.00	0.04
35	6.127	0.01	0.15
40	6.129	0.01	0.11
45	6.131	0.00	0.07
50	6.130	0.00	0.09
55	6.128	0.01	0.13
60	6.130	0.00	0.09

Graphs of Bouwer-Rice (1976) Piezometer Test (top graph has normal axes and bottom graph has a log H-h/H-Ho axis)

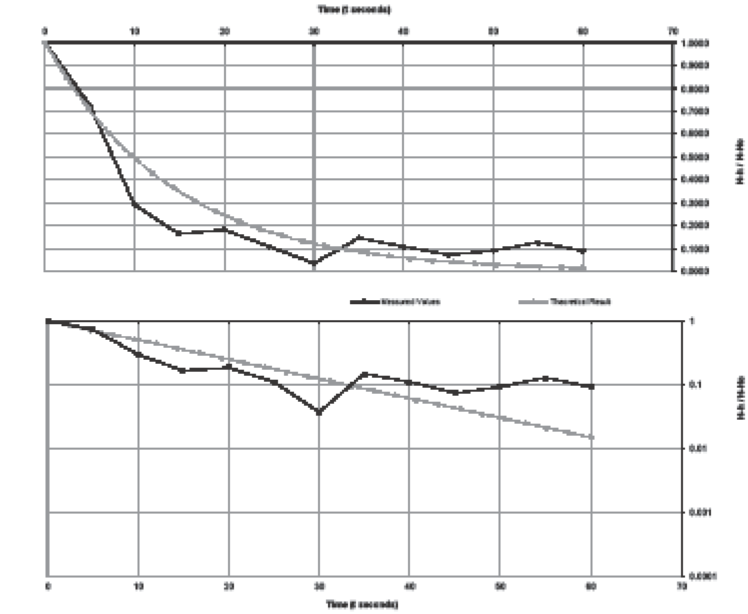


Figure 23. Slug Test 1/2 for BH 204

JACOBS Borehole Variable Head Permeability Test

Borehole ID

NHWM BH204

Project Details

Project Name	NHWM
Project Number	AE04521
Test Date	25/11/2014
Tested	J Blyth
Analysed	J Blyth

Test Parameters

Top of screen	2.500 m
Bottom of Screen, *a*	7.500 m
Screen Length, *L*	5.000 m
Static Water Level, *H*	5.975 m
Initial Water Level, H_0	5.913 m
Hole Radius, *R*	0.130 m
Casing Radius, *r*	0.040 m
Well Penetration	Partial
Water Table Height, *D*	20.000 m

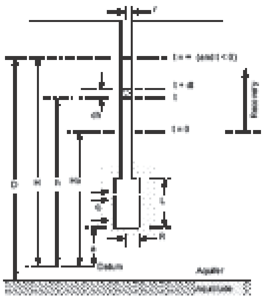
Note: If the datum is above the hole, the height/depth readings do not have to be negative numbers – as long as they are either all negative or all positive, the answer will be correct.

Result

Hydraulic Conductivity

$K =$ 2.21E-05 m/s

Test Schematic



Bouwer-Rice (1976) Method:

$$K = \frac{r^2 \ln(R_e / R)}{2d} \frac{1}{t} \ln\left(\frac{H - H_0}{H - h}\right)$$

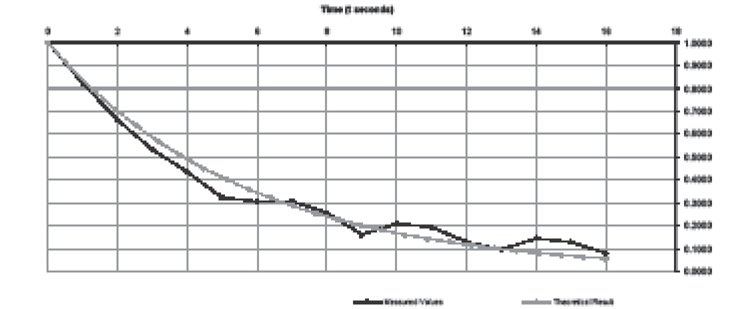
Calibrated Parameters

$\ln(R_e/R)$	2.022
$1/t * \ln(h_0/h_t)$	0.077

Measured Data

Time (Secs)	Depth (h) (m)	H - h (m)	H-h / H-H0
0	5.913	0.06	1.00
1	5.924	0.05	0.82
2	5.934	0.04	0.66
3	5.942	0.03	0.53
4	5.948	0.03	0.44
5	5.955	0.02	0.32
6	5.956	0.02	0.31
7	5.956	0.02	0.31
8	5.959	0.02	0.26
9	5.965	0.01	0.16
10	5.962	0.01	0.21
11	5.963	0.01	0.19
12	5.967	0.01	0.13
13	5.969	0.01	0.10
14	5.966	0.01	0.15
15	5.967	0.01	0.13
16	5.970	0.00	0.08

Graphs of Bouwer-Rice (1976) Piezometer Test (top graph has normal axes and bottom graph has a log H-h/H-Ho axis)



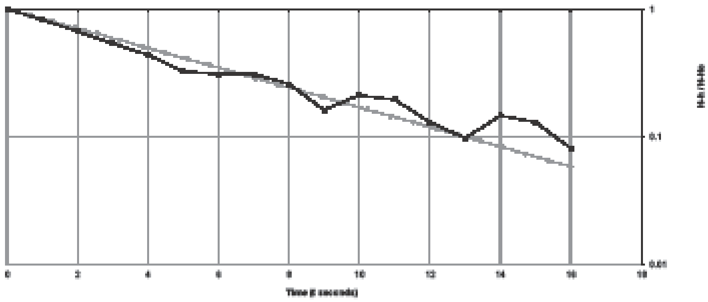


Figure 24. Slug Test 2/2 for BH204

Appendix C. Groundwater Model Development

SEEP/w was selected because this software is able to model unsaturated/saturated water flow which occurs when an excavation causes dewatering. The outputs were used to assess the likely effects on groundwater levels (e.g. drawdown) in the immediate vicinity of the open trench sections. Also, these models were used to assess recovery after backfilling, and an additional model was put together to assess the effectiveness of sheet piles during construction.

This section was selected as the worst case because the tunnelled section will be constructed with the deepest receiving shaft pit (12.87mBGL invert and the excavation could be up to 13.87mBGL). Also, since the water main pipes leading to the crossing are traversing through areas consisting of both the Puketoka Formation and the ECBF, the cross section at this intersection was used as the framework for the open trench models. Given the short construction time frame (short dewatering period) and limited extent of effects, broader scale 3D groundwater modelling has not been carried out.

To estimate total inflow into the drilled tunnels, an analytical model developed by Perrochet (2005), was used to predict transient discharge into the tunnel. This method assumes the tunnel goes through an homogeneous formation and at constant drilling speed. The method relies on the solution of analytical equations to define transient inflows, to provide total discharge estimates.

C.1 Open Trench Models

The open trenches in these models are 3m in width and 5m deep. The open trench would act as an open drain as long as it is open. Once the pipe is laid inside the trench, the open trench will be backfilled with compacted material. Since the length of open trench can be up to 90m and the rate of advance is anywhere between 12-24 m/day, then the maximum period of time during which the trench will be open is 8 days. Since the majority of the alignment goes through housing or road areas where rainwater will be collected via the storm-water system, rainfall recharge was not considered in this model. The models were built symmetrically for simplicity. Constant head boundaries representing the known groundwater level data were modelled at 500m from the excavations, but then removed during dewatering. Since there is no available long term water level data for bores, the models are uncalibrated but a sensitivity analysis was carried out in order to provide a range of possible outcomes.

C.1.1 Open Trench Fill/ECBF model

Model conceptualisation

Table 10-2 shows the hydrogeological framework adopted for the 2D model of the Fill/ECBF open trench section. This model assumes a 4.8m thick layer of fill. The fill material consists of Alluvium or Puketoka Formation material, which have similar hydraulic properties. However, the hydraulic properties of these materials may exhibit some variations, so a sensitivity analysis was carried out to assess a wide range of values. In any case, the contrast in hydraulic properties between ECBF and Fill material is taken into account by using a lower hydraulic conductivity (K) for the ECBF (about 2 orders of magnitude lower). These values were taken from known reference values in the area (see section 3.2).

Table 10-2. Hydrogeological framework for Fill/ECBF model

Unit	K (m/s)	Top (mRL)	Bottom (mRL)
Fill	3×10^{-5}	44.65	39.85
ECBF	6×10^{-7}	39.85	27.85

In this model, ground level is at 44.65mRL and the trench invert level is 39.65mRL (5m deep trench). The initial water level in these models was assumed to be just below the pipe invert, which is calculated by assuming 1.5m of clearance above the pipe and 1.2m of pipe diameter (WL=2.7 mBGL or 41.95mRL). This water level was chosen in order to assess a typical situation with high water level, bearing in mind that throughout the project the water level is, on average, 3.79mBGL (see section 2.3).

Model Results

Model results are presented in section 5.1.1.

Sensitivity analysis

The base-case model uses the hydraulic conductivities in Table 10-2. A sensitivity analysis which consisted in increasing and decreasing the hydraulic conductivities of the Fill and ECBF materials is presented in Table 10-3. This analysis indicates that the worst case scenario is when the hydraulic conductivity of the Fill increases in one order of magnitude in relation to the baseline condition. This results in a drawdown of 1m at 30m from the excavation. Also, at 300m from the excavation, the drawdown is negligible. The discharge rate into the open trench is 9.0m^3/day per m of trench, which corresponds to 6480m^3 of water discharging into the open trench for 90 days. After the trench is backfilled, the groundwater level would recover in approximately 160 days. Again, this situation is the worst case scenario and is unlikely due to the high standing water level (2.7mBGL) and the high K value chosen for the fill.

Table 10-3. Sensitivity analysis for Open Trench Fill/ECBF model

Scenario	Unit	Hydraulic conductivity (m/s)	Q[1] (m^3/d/m)	Drawdown at 0 m from excavation (m)	Drawdown at 30 m from excavation (m)	Lateral zone of influence[1] (m)	Recovery time[2] (days)
1 K Fill ↑	Fill	**3×10^{-4}**	9.0	2.3	1.00	300	160
	ECBF	6×10^{-7}					
2 K Fill↓	Fill	**3×10^{-6}**	0.4	2.3	0.04	80	40
	ECBF	6×10^{-7}					
3 K ECBF ↑	Fill	3×10^{-5}	2.2	2.3	0.40	150	100
	ECBF	**6×10^{-6}**					
4 K ECBF ↓	Fill	3×10^{-5}	2.0	2.3	0.30	100	100
	ECBF	**6×10^{-8}**					

1. After 8 days of drainage.
2. 90% recovery of drawdown.

C.1.2 Open Trench ECBF model

Model conceptualisation

Table 10-4 shows the hydrogeological framework adopted for the 2D model of the ECBF open trench section. This model assumes the open trench will be excavated on weathered ECBF material which has a hydraulic conductivity of about 3×10^{-7} m/s. ECBF hydraulic conductivity variations (due to various degrees of weathering) are taken into account by conducting a sensitivity analysis. Initial base case values were taken from known reference values in the area (see section 3.2).

Table 10-4. Hydrogeological framework for ECBF model

Unit	K (m/s)	Top (mRL)	Bottom (mRL)
ECBF (weathered)	3×10^{-7}	44.65	30.83

In this model, ground level is at 44.65mRL and the trench invert level is 39.65mRL (5m deep trench). The initial water level in these models was assumed to be just below the pipe invert, which is calculated by assuming 1.5m of clearance above the pipe and 1.2m of pipe diameter (WL=2.7 mBGL or 41.95mRL). This water level was chosen in order to assess a typical situation with high water level, bearing in mind that throughout the project the water level is, on average, 3.79mBGL (see section 3.3).

Model Results

Model results are presented in section 5.1.2.

Sensitivity analysis

The base-case model uses the hydraulic conductivities in Table 5-2. A sensitivity analysis which consisted of systematically varying the hydraulic conductivity of the weathered ECBF is presented in Table 10-5. This analysis indicates that the worst case scenario is when the hydraulic conductivity of the weathered ECBF increases in one order of magnitude in relation to the baseline condition. This results in a drawdown of 0.3m at 30m from the excavation. Also, at 120m from the excavation, the drawdown is negligible. The discharge rate into the open trench is $0.8m^3$/day per metre of trench, which corresponds to $576m^3$ of water discharging into the open trench for 90 days. After the trench is backfilled, the groundwater level would recover in approximately 100 days. Again, this situation is represents an unlikely worst case scenario.

Table 10-5. Sensitivity analysis for Open Trench ECBF model

Scenario	Unit	K (m/s)	Q[1] (m^3/d/m)	Drawdown at 0m from excavation (m)	Drawdown at 30 m from excavation (m)	Lateral zone of influence[1] (m)	Recovery time[2] (days)
1 K wECBF ↑	wECBF	3×10^{-6}	0.8	2.3	0.30	120	100
2 K wECBF ↓	wECBF	3×10^{-8}	0.02	2.3	0.0	20	20

1. After 8 days of drainage.
2. 90% recovery of drawdown.

C.1.3 Open Trench model using sheet piles

Model conceptualisation

An open trench model using sheet piles was put together to assess the mitigation option for the worst case scenario (scenario 1 for the Fill/ECBF model). The sheet piles are assumed to be placed at the excavation face and will have a very low hydraulic conductivity (3×10^{-12} m/s). The rest of the conceptualisation is identical to the one used for Scenario 1 of the Open Trench Fill/ECBF model. Two different sheet pile depths were modelled: 1m and 3m below the excavation invert.

Model results are presented in section 5.1.3.

Sensitivity analysis

No sensitivity analyses were carried out with the sheet pile models. However, it is worth noting that the depth at which the sheet pile is placed will only have a less than minor effect on the discharge rate as long as this is deeper than the excavation invert. A reduction of about 50% of discharge will take place with a 2m increase in sheet pile depth (e.g. down to 3m below the excavation invert).

C.1.4 Shaft model for receiving/jacking pit

Open pit model with no mitigation measures

Model conceptualisation

The Northern Western Motorway (SH18) crossing was investigated through a desktop study which included a few existing geotechnical logs of bores previously drilled in the area (NOR 3). This enabled formulating the cross section presented in Figure 17. This section shows the presence of a 1.6m thick fill unit near the temporary receiving pit. In addition, a layer of Alluvium material, about 3.2m thick, was identified under the actual motorway crossing. Two bores near the area showed a water level of 34.8mRL (HA50) and 33.0mRL (HA51). The receiving pit has its invert at 12.82mBGL (31.83mRL) and the jacking pit has its excavation invert at 3.15mBGL. The receiving pit was modelled because this pit is deeper and would thus induce more drawdown as it is excavated. The receiving pit would be typically excavated to 1m past its invert (e.g. to 30.83mRL). The receiving pit is 6m in diametre and its excavation would be about 1m below the actual pit invert.

The receiving pit was modelled as an axisymmetric model with constant head boundaries at 500m away from the pit to represent initial water levels. The initial water level selected was the highest one registered at nearby bores (34.8 mRL). The actual hydrogeological framework consisted on 2 layers with distinct hydraulic properties. The first layer has a K of 3 x 10^{-5} m/s and represents sediments of the Tauranga Group (Alluvium, Puketoka Formation, and Fill material). Under this layer, the model includes the ECBF with a K of 6 x 10^{-7}m/s. Actual unit elevations are presented in Table 10-6.

Table 10-6. Hydrogeological framework for receiving pit

Unit	K (m/s)	Top (mRL)	Bottom (mRL)
Fill/Alluvium	3 x 10^{-5}	44.65	39.85
ECBF	6 x 10^{-7}	39.85	27.85

Model results are presented in section 5.2.1.

Sensitivity analysis

The base-case model uses the hydraulic conductivities in Table 10-6. A sensitivity analysis which consisted in increasing and decreasing the hydraulic conductivities of the Fill and ECBF materials is presented in Table 10-7. This analysis indicates that the worst case scenario is when the hydraulic conductivity of the wECBF increases in one order of magnitude in relation to the baseline condition. This results in a drawdown of 0.8m at 30m from the excavation. Also, at 300m from the excavation, the drawdown is negligible. The discharge rate into the open pit is about 104 m^3/day which corresponds to 9360m^3 of water discharging into the open pit for a total period of 90 days. After the trench is backfilled, the groundwater level would recover in approximately 90 days. Again, this situation is the worst case scenario and is unlikely that the K value for the ECBF will be this high.

Table 10-7. Sensitivity analysis for open pit model

Scenario	Unit	Hydraulic conductivity (m/s)	Q (m^3/d)	Drawdown at 30 m from excavation (m)	Lateral zone of influence[1] (m)	Recovery time[2] (days)
1 K Fill ↑	Fill	**3×10^{-4}**	12.6	0.38	100	90
	ECBF	6×10^{-7}				
2 K Fill↓	Fill	**3×10^{-6}**	11.3	0.3	140	90
	ECBF	6×10^{-7}				
3 K ECBF ↑	Fill	3×10^{-5}	103.8	0.8	300	90
	ECBF	**6×10^{-6}**				
4 K ECBF ↓	Fill	3×10^{-5}	1.32	0.26	140	34
	ECBF	**6×10^{-8}**				

Open pit model with Sheet Pile Walls (SPW) as a mitigation measure

Model conceptualisation

An open pit model using sheet piles was put together to assess the mitigation option for the worst case scenario (scenario 3 for the Open pit model). The sheet piles are assumed to be placed at the excavation face and will have a very low hydraulic conductivity (3×10^{-12} m/s). The rest of the conceptualisation is identical to the one used for Scenario 3 of Open pit model. Two different sheet pile depths were modelled: 1m and 3m below the excavation invert. In this model, the sheet pile was placed at 1m below the excavation invert level.

Model results are presented in section 5.2.2.

C.2 Tunnel Seepage Estimates

The Perrochet (2005) method assumes perfect 1D radial flow, aquifer boundaries at infinity, confined conditions everywhere and at all times. These assumptions are those classically enforced in well hydraulics (e.g. Theis, Jacob-Lohman, etc) and are certainly limiting. However, when reasonably justified, they lead to elementary close-form solutions and allow subsequent operations which will allow accounting for cumulative effects.

The assumptions inherent in the method do not hold in the case of shallow tunnels where the groundwater level rapidly decreases above the tunnel, leading to unconfined conditions. This is the same as for classical well hydraulic equations, yet these methods continue to be used as first approximations for flow into a well in both confined and unconfined conditions.

Model conceptualisation

Due to the absence of deep hydrogeological investigations through the proposed micro tunnelled section between Manuka Rd and Shetland St (NOR 1) the following key parameters were assumed for a baseline case (typical hydrogeological conditions):

- Hydraulic Conductivity K = 1×10^{-3} m/day which corresponds to the mid- range hydraulic conductivity of a sandstone (Anderson, 2002). The tunnel would mainly go through the Nihotupu Formation which consists primarily of sandstone materials. The tunnel could also go through the Cornwallis Formation in some places but this formation also contains sandstone material
- Storage coefficient $S_s = 3.6 \times 10^{-5}$ m^{-1} which corresponds to the mid–range storage coefficient of a rock material (rock, fissured) (Anderson, 2002). Since the sandstones of the Nihotupu and Cornwallis formations are consolidated rock materials (e.g. sandstones) this value is appropriate

- Specified tunnel drawdown of 29m. This is the difference between the water level above the tunnel and the tunnel invert. Since there are no piezometric contours available, the 29m value was selected as a sensible estimate.

The tunnel annulus will remain open for the duration of drilling. As the tunnel is being drilled, groundwater will seep into the annulus between the rock formation and the 2.1m concrete pipe at a decreasing rate as the tunnel length increases. After completion, the annulus will be sealed with grout and seepage will stop. If the annulus is not sealed, remanent seepage can occur at a decreasing rate after the tunnel is completed. The following assumptions where made in regard to the construction methodology for the tunnel, which apply to hydrogeology:

- Tunnel Length = 900m
- Speed of tunnelling = 12m/day
- Completion time = 75days

Model results are presented in section 5.3.

Sensitivity analysis

The base case model uses mid- range values of hydraulic conductivities, storage coefficients, and tunnel drawdown. A sensitivity analysis which consisted in systematically varying these parameters is presented in Table 10-8. According to this analysis the model is not sensitive to changes in hydraulic conductivity for this range of base case values. However, it is very sensitive to changes in the Storage coefficient and specified tunnel drawdown. A high storage coefficient (e.g. S_s = 6.9 x10^{-5} m^{-1}) will result in a maximum discharge rate of almost 45m^3/day.

Table 10-8. Sensitivity analysis for tunnel discharge model

Scenario	Specified tunnel drawdown (m)	K (m/day)	S_s (m^{-1})	Qmax (m^3/day)
Base case	29	1x10^{-3}	3.6x10^{-5}	23.4
Low K	29	**1x10^{-5}**	3.6x10^{-5}	23.4
High K	29	**1x10^{-1}**	3.6x10^{-5}	23.4
Low S_s	29	1x10^{-3}	**3.3x10^{-6}**	2.2
High S_s	29	1x10^{-3}	**6.9 x10^{-5}**	44.9
DD =10	**10**	1x10^{-3}	3.6x10^{-5}	8.1
DD=20	**20**	1x10^{-3}	3.6x10^{-5}	16.2
DD=40	**40**	1x10^{-3}	3.6x10^{-5}	32.3
DD=50	**50**	1x10^{-3}	3.6x10^{-5}	40.4