REPORT

Tonkin+Taylor

Groundwater and Settlement Effects Assessment

Prepared for
Watercare Services Limited

Prepared by
Tonkin & Taylor Ltd

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Executive Summary

Whenuapai Redhills Wastewaters Servicing Scheme includes a new wastewater pipeline that will be built in stages to redirect wastewater from the north-western parts of Auckland to the Rosedale Wastewater Treatment Plant.

This report covers Phase 2 of works and includes construction of approximately 4 km of wastewater pipeline between a new Break Pressure Tank (BPT) at 32 Mamari Road, Whenuapai and the existing Hobsonville Pump Station at 2A Buckley Avenue, Hobsonville.

Watercare has engaged Tonkin & Taylor Ltd (T+T) to undertake an Assessment of Effects for Groundwater and Settlement for the Phase 2 of the Whenuapai Redhills Wastewaters Servicing Scheme.

The assessments undertaken by T+T address groundwater drawdown-induced settlement effects and mechanical settlement effects and have been prepared to inform the application for resource consent. The overall results of this assessment are set out below:

Surface ground settlement

- Ground settlement effects arising from construction and operating the proposed shafts on surrounding structures is assessed to generally be negligible to very slight (1). The risk of damage is focused around shaft 16 and shaft 17. Mitigation measures such as adopting robust construction methods to seal shafts from leakage will aid in minimising settlements.
- Ground settlement effects arising from dewatering at shaft 17 and to a lesser extent shaft 16 will result in settlement across SH18. Adopting construction methods that minimise shaft leakage to just wetting is assessed to result in settlements of 8 mm and less. It is recommended that WSL consult with Waka Kotahi on this matter.
- Surface ground settlement due to mechanical settlement arising for shaft deformation is assessed to be negligible.
- Surface ground settlement due to tunnelling arising from volume loss is expected to be less than 10 mm for the Massey connector through to shaft 17.
- Surface ground settlement at electrical substation located at 439 Hobsonville Road is expected to occur as a result of excavations for shaft 17. Adopting construction methods that minimise shaft leakage to just wetting is assessed to result in settlements of 11 mm and less at the sub-station, reducing the risk of damage to structures on this property. Consultation with the asset owner should be had to determine acceptable threshold of ground settlement for the structures at the property.
- Surface ground settlement at a bowling club located at 1 Memorial is limited to 1V:2000H or 0.05% grade which is likely to be within the tolerance of what can be practically graded and maintained for a pitch.
- Surface ground settlement at the Hobsonville Pump Station owned by WSL may be as large as 70 mm (60 mm mechanical settlement in additional to 10 mm settlement arising from dewatering) if further engineering controls are not implemented. However, through the use of lateral bracing and active slide rail shields, these settlements can be reduced to less than 15 mm. WSL will need to assess acceptable thresholds for ground settlement for their pump station structure.

Groundwater

- Effects upon groundwater levels are expected to be localised to within generally 100 m distance of the proposed excavation and temporary in nature. Upon excavation completion and sealing of the shafts, groundwater levels are expected to return to levels prior to construction.

- There is no impact expected upon neighbouring groundwater bores.
- Groundwater that is not temporarily intercepted by the dewatering associated with shaft construction take will be diverted around the proposed secant piled structures. However, the diverted groundwater is still expected to continue in the same general direction and flow rate.
- No significant groundwater depletion to nearby streams and wetland is expected.

Monitoring

- A groundwater and settlement monitoring programme during construction will be a requirement of Consent. The requirements of the monitoring programme, including Alert and Alarm levels, will be set out in a Groundwater and Surface Monitoring Contingency Plan (GSMCP). The plan includes groundwater drawdown, surface settlement, building settlement monitoring and visual observation monitoring.

Overall, the assessment of groundwater and settlement effects indicates that the ground settlement and dewatering effects arising from the phase 2 Whenuapai Redhills Wastewaters Servicing Scheme, can be managed to within acceptable limits where thresholds are known. Where thresholds are not identified, specific tolerances and controls will need to be negotiated with stakeholders utilising the settlements provided in this assessment.

1 Introduction

1.1 General

Watercare Services Limited (WSL) has engaged Tonkin and Taylor Limited (T+T) to undertake a groundwater dewatering and excavation induced settlement assessment, including proposed monitoring and mitigation measures, to support the lodgement of the Assessment of Effects (AoE) for the Whenuapai Redhills Wastewaters Servicing Scheme with Auckland Council. Results of this assessment are presented in this Groundwater and Settlement Effects Report (GWSER), which has been prepared in accordance with T+T's offer of service dated 29 April 2021[1].

Our initial assessment was presented to WSL as a draft[2] in November 2022. In December 2022, WSL advised T+T of minor changes in pipeline alignment and the addition of manholes MH1 and MH2. At WSL's request, a revision to the GWSER has been undertaken in accordance with T+T's Variation V01 dated 2 June 2023[3]

1.2 Overview

Whenuapai Redhills Wastewaters Servicing Scheme includes a new wastewater pipeline that will be built in stages to redirect wastewater from the north-western parts of Auckland to the Rosedale Wastewater Treatment Plant.

This report covers Phase 2 of works and includes construction of approximately 4 km of wastewater pipeline between a new Break Pressure Tank (BPT) at 32 Mamari Road, Whenuapai and the existing Hobsonville Pump Station at 2A Buckley Avenue, Hobsonville. This can generally be broken down into three components:

- **The Massey Connector gravity pipeline**: Approximately 1 km long and 2100 mm in outer diameter, running generally south-east from the Break Pressure Tank (BPT) and passing beneath SH18 to 27 Trig Road, Whenuapai.
- **The Northern Interceptor (Main) gravity tunnel:** Approximately 2.8 km long and 2100 mm in outer diameter, running north-east from 27 Trig Road towards the pump station, generally parallel to the southern edge of State Highway 18 (SH18).
- **The pump station connection**: Approximately 40 m in length and 600 mm in diameter, running between the Northern Interceptor tunnel and the existing Hobsonville Pump Station. This includes excavations for two manholes, MH1 and MH2.

Nine circular shafts are proposed along the extent of the project, inclusive of the excavations for MH1 and MH2.

[1] Tonkin + Taylor Ltd, Offer of Service, Northern Interceptor Phase 2, Assessment of Groundwater & Settlement Effects and Monitoring & Contingency Plan, Job No. 1015506, dated 29 April 2021

[2] Tonkin & Taylor Ltd, 11 November 2022. Groundwater and Settlement Effects Assessment. Whenuapai to Redhills Wastewater Servicing Package 2, Job Number 1015506 v1

[3] Tonkin + Taylor Ltd, Variation Order, Northern Interceptor Phase 2, Revision of the Geotechnical AoE, Contaminated Land DSI Updates, Draft SMP and Ground Investigation, Job No. 1015506, dated 2 June 2023.

Table 1.1: Shaft details

Shaft ID	Inner Shaft Diameter (m)	Approximate Shaft Depth (m)	Pipe Invert Level (m RL)	Depth of piles (m)	Secant Pile Configuration	Type	Launch/Reception of TBM
BPT	7.5	9.4	28.88	14.0	900 mm dia at 700 mm c/c	Permanent Access	Reception
Shaft 8 (LS8)	13.0	17.7	7.54/5.32 *	19.5	900 mm dia at 700 mm c/c	Permanent Access	Launch
Shaft 10 (RS10)	6.75	26.7	4.79	34.1	750 mm dia at 550 mm c/c	Temporary	Reception
Shaft 12 (LS12)	11.25	18.6	4.19	21.0	750 mm dia at 550 mm c/c	Temporary	Launch
Shaft 14 (RS14)	7.5	25.3	3.57	28.0	900 mm dia at 700 mm c/c	Permanent Access	Reception
Shaft 16 (LS16)	11.25	21.5	3.10	26.7	800 mm dia at 550 mm c/c	Temporary	Launch
Shaft 17 (RS17)	12.0	14.0	2.60	190	900 mm dia at 700 mm c/c	Permanent Access	Reception/Launch**
MH1	6.0	4.5	2.39	TBC	TBC	Permanent Access	Reception/Launch
MH2	2.5	5.1	1.86	TBC	TBC	Permanent Access	Reception

* Invert levels listed are for the pipeline connecting to the LS8 shaft from the BPT and RS10, respectively.
** RS17 is a reception shaft for the 2.1 m diameter tunnel boring machine and the launch shaft for the smaller TBM to install the 0.6 m diameter pipeline to MH1.

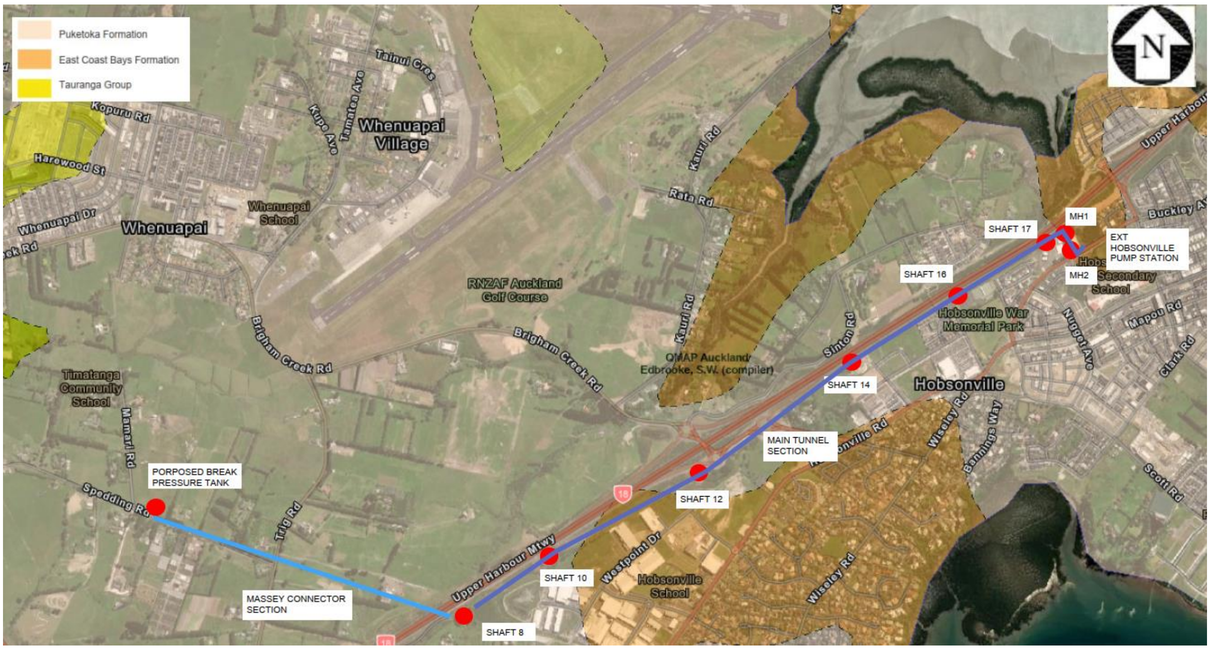

Figure 1.1: Approximate pipeline alignment and position of shafts

1.3 Scope of work

The following is our scope of works for this project:

- Obtain and review published geological and hydrogeological information available and any surrounding groundwater bore information provided by Auckland Council.
- Develop a geotechnical/ hydrogeological conceptual model for the excavation area based on site information from the GFR and excavation methodology information provided by WSL.
- Identify the zone of influence of construction, and the existing buildings and utilities (within that zone) that may be affected by excavation and dewatering induced ground movements.
- Evaluates within the zone of influence:
  - The potential magnitude and distribution of ground movement for buildings and utilities through excavation-induced mechanical settlement analysis.
  - The potential impacts on groundwater from diversion / dewatering activities (i.e. drawdown-induced ground settlement) through numerical groundwater modelling.
- Combine the developed settlements (drawdown and mechanical) to derive a surface settlement contour map, including:
  - Excavation-induced mechanical settlement.
  - Groundwater drawdown-induced settlement.
- Compare the derived total settlement values against the location of identified points of interest (existing buildings, structures, and utilities) close to the excavations that are potentially sensitive to ground settlement. Where assets are owned by Watercare, the effect will be assessed by the designers and therefore is not included as part of this report.
- Recommend appropriate measures to avoid, remedy or mitigate any potential or actual effects that may arise causing adverse effects on buildings and utilities, including providing a management framework through a Groundwater and Settlement Monitoring and Contingency Plan (GSMCP).

- Prepare this Assessment of Groundwater and Settlement Effects Report to support an Assessment of Effects on the Environment.

1.4 Referenced documents

This report is based on the following documents:

- Whenuapai-Redhills Wastewater Servicing Rising Main/Gravity Main (Massey), Geotechnical Ground Investigation Factual Report, dated January 2022, Version 4, job number 1015174, prepared by Tonkin + Taylor Ltd.
- Whenuapai-Redhills Wastewater Servicing Rising Main/Gravity Main (Main Alignment) – Shaft RS17, Geotechnical Ground Investigation Factual Report, dated February 2022, Version 4, job number 1015174, prepared by Tonkin + Taylor Ltd.
- Whenuapai-Redhills Wastewater Servicing Rising Main/Gravity Main (Main Alignment) – Shaft RS14, Geotechnical Ground Investigation Factual Report, dated February 2022, Version 3, job number 1015174, prepared by Tonkin + Taylor Ltd.
- Whenuapai-Redhills Wastewater Servicing Rising Main/Gravity Main (Main Alignment) – Shaft LS16, Geotechnical Ground Investigation Factual Report, dated February 2022, Version 4, job number 1015174, prepared by Tonkin + Taylor Ltd.
- Whenuapai-Redhills Wastewater Servicing Rising Main/Gravity Main (Main Alignment) – Shaft LS12, Geotechnical Ground Investigation Factual Report, dated February 2022, Version 3, job number 1015174, prepared by Tonkin + Taylor Ltd.
- Whenuapai-Redhills Wastewater Servicing Rising Main/Gravity Main (Main Alignment) – Shaft RS10, Geotechnical Ground Investigation Factual Report, dated March 2022, Version 3, job number 1015174, prepared by Tonkin + Taylor Ltd.
- Whenuapai Redhills Package 2 (WRP2) – Tunnel, Shafts, Structures, BPT (Break Pressure Tank) and Hobsonville Pump Station connection, Construction Methodology, Revision A1, dated 25 March 2022, prepared by Brian Perry Civil Ltd.
- Northern Interceptor Phase 2 Westgate to Hobsonville, Geotechnical Interpretive Report, Massey Connector Section, dated 26 June 2022, Revision 4, document reference 417039-MMD-01-XX-RP-J-0001, prepared by Mott MacDonald.
- Whenuapai to Redhills Wastewater Northern Interceptor Phase 2 and Gravity Connector Sewer, Preliminary Design Report, dated 11 July, 2022, Revision 2, document reference 417039-MMD-00-XX-RP-X-003, prepared by Mott MacDonald.
- Northern Interceptor Phase 2, Westgate to Hobsonville, Detailed Design – For Review drawings, dated 25-11-22, Issue 2, document reference 417039-MMD-00-XX-DR-Z-0001, prepared by Mott MacDonald.

2 Site setting

2.1 Site description

The site is located approximately 14 km north-west of the Auckland CBD between Whenuapai and Hobsonville. The proposed pipeline alignment is approximately 4 km long and extends from the junction between 32 Mamari Road to the Hobsonville Pump Station.

The alignment crosses Auckland Council owned land, privately owned land and SH18 owned by Waka Kotahi.

2.1.1 Northern interceptor (Main) Tunnel

The Main Tunnel runs north-east from the eastern extent of the Massey Connector at 27 Trig Road and continues towards the Hobsonville Pump Station. The alignment is bordered by SH18 to the north and future urban, residential, open space, business, and light industrial zones and stormwater treatment ponds to the south. The alignment is within Watercare' designation.

2.1.2 Massey connector

Land use along the proposed alignment comprises residential lots along Spedding Road and grassed paddocks between Spedding Road and the northern SH18 berm. The eastern extent of the Massey Connector alignment passes beneath SH18 and terminates within the property of 27 Trig Road, Whenuapai.

The ground elevation along the Northwest section is gently inclined, with a relatively steady increase in gradient from 37 mRL at Mamari Road to 50 mRL at the junction between Spedding Road and Trig Road. The ground elevation undulates significantly south-east of Trig Road and declines to approximately 20 mRL at the east extent of the alignment.

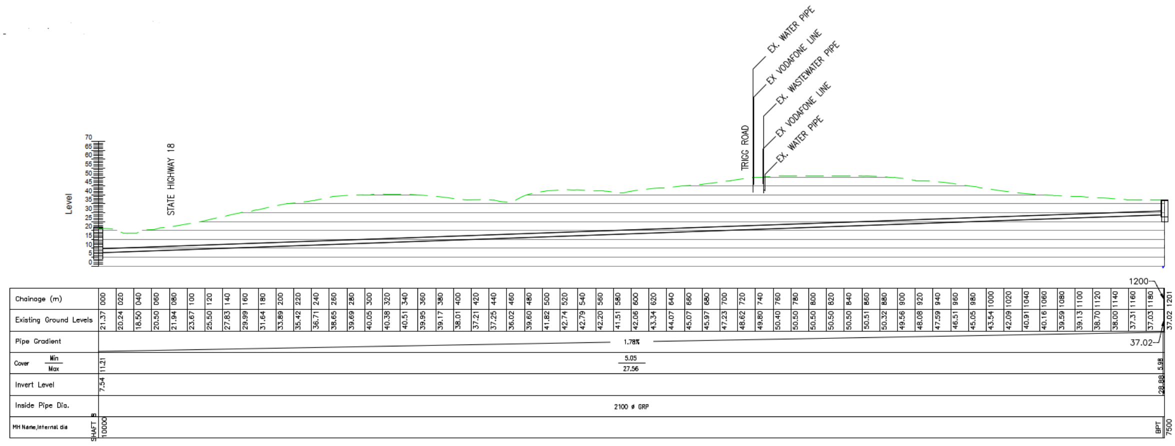

Figure 2.1: Elevation profile of the Massey Connector referenced from Mott M MacDonald For Review drawings, dated 25-11-22

3 Proposed construction methodology

The proposed construction methodology has been summarised from the memorandum prepared by Brian Perry Civil Ltd, dated 25 March 2022[4].

The proposed underground structures for the Northern Interceptor that are part of this assessment includes the BPT shaft at the western extent of the alignment, the six shafts to be constructed along the southern side of SH18, and the two smaller shafts near the Hobsonville Pump Station to allow for the installation of MH1 and MH2. An Earth Pressure Balance (EPB) tunnel boring machine (TBM) is to be used for the excavation from the BPT through to shaft 17 (by the Hobsonville Pump Station). Trenchless construction methods such as Trenchless Horizontal Direction Drilling (HDD) are proposed from shaft 17 through to MH1 and MH2; and open trench excavation for the remaining alignment to connecting into the existing Pump Station.

The details of the shafts are summarised in Table 1.1 and the locations are shown on Figure 1.1. The proposed concept construction methodology for all shaft structures, except MH1 and MH2, comprises secant piles arranged in a circular arrangement extending through the overburden soils (Tauranga Group Alluvium and ECBF soils) and embedding into the ECBF rock. Lateral stiffening support may be provided by the use of ring beams if required by the designer.

The construction methodology for MH1 and MH2 is specified to comprise an excavation support system that provides seepage control. The details of the construction methodology have not yet been confirmed. For the purposes of this assessment, we have adopted secant piles with a 3 m embedment below the base of the excavation for MH1 and sheet piles with a 1.5 m embedment below the base of the excavation for MH2. We note that the pile embedment depths adopted are for the purposes of assessing groundwater drawdown. Further assessments are required to assess pile depths for stability purposes.

All shaft excavation supports have been specified to impede groundwater flow into the shafts during construction, and therefore limit groundwater drawdown. Groundwater modelling undertaken to support the consent application incorporates a low permeability liner through the soils at the site.

It is expected that construction of the secant shafts will take approximately 24 months in total (from site preparation works through to sealing and reinstatement) although potentially longer (up to 36 months maximum). MH1 and MH2 are expected to take up to 60 days.

Following construction, a permanent water-tight concrete lining will be installed around the shaft walls and base, forming a complete seal from groundwater intrusion.

[4] Brian Perry Civil Ltd, Construction Methodology, Whenuapai Redhills Package 2 (WRP2) – Tunnel, Shafts, Structures, BPT (Break Pressure Tank) and Hobsonville Pump Station connection, Revision A1, dated 25 March 2022

4 Ground model

4.1 Geological setting

4.1.1 Published geology

The published geological map[5] of the area indicates that the site is underlain by mud, gravel, peat, lignite, tephra and pumice of the Puketoka Formation (Tauranga Group) overlying sandstones and mudstones of the East Coast Bays Formation (Waitemata Group). The Waitemata Group materials form the basement rock of the area. The location of the shaft sites in the context to the regional geology is presented on Figure 4.1 below.

- Puketoka Formation of the Tauranga Group alluvial soils now referred to as the Takanini Formation (soft to very stiff clays and silts, or loose to dense sands with the occasional organic or peat layer), which are expected to be encountered from surface or near surface under fill.
- East Coast Bays Formation (ECBF) of Warkworth Subgroup (Waitemata Group) comprising alternating sandstone and mudstone with variable volcanic content and interbedded volcaniclastic grits.

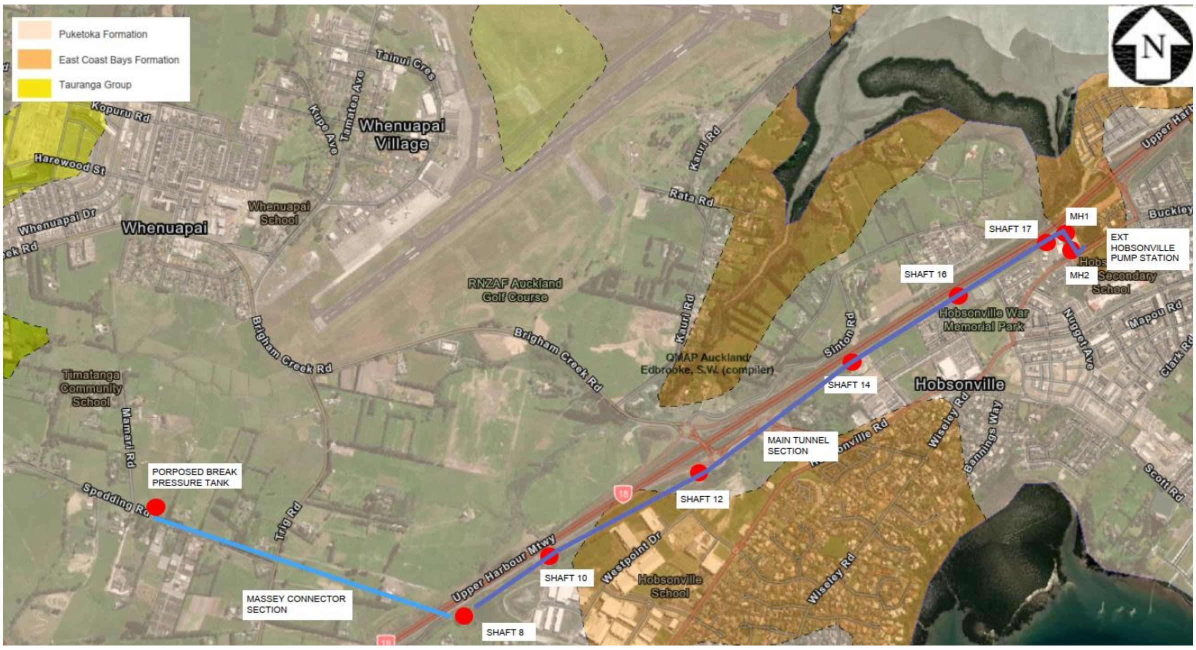

Figure 4.1: Published surface geology map for the site area

4.1.2 Site investigations

T+T carried out geotechnical investigations and presented them in various Geotechnical Factual Reports (GFR) respective of each shaft location. A single GFR[6] was presented in September 2021 to compile the various shaft GFRs. The GFRs provide a comprehensive summary of the investigations and includes copies of borehole logs, CPTs and laboratory test data. A summary of all relevant investigations undertaken for this study are presented below in Table 4.1 and presented in Appendix A.

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[5] Kermode, L.O. 1992: Geology of the Auckland urban area. Scale 1:50,000. Institute of Geological & Nuclear Sciences geological map 2. 1 sheet + 63 p. Institute of Geological & Nuclear Sciences Ltd., Lower Hutt, New Zealand.
[6] Tonkin & Taylor Ltd. September 2021. Whenuapai-Redhills Wastewater Servicing Gravity Main (Main Alignment) – Collated Geotechnical Ground Investigation Factual Report. T+T Ref: 1015174.v1

Appendix A of this report includes copies of the borehole logs, CPTs and groundwater monitoring data for ease of reference.

Table 4.1: Summary of the investigations used to assess each shaft location

Shaft ID	Machine Boreholes	sCPTs
BPT	BH201	sCPT201
Shaft 8 (LS8)	BH101 BH102 BH103	sCPT101
Shaft 10 (RS10)	BH108 BH109	sCPT102
Shaft 12 (LS12)	BH114 BH115	sCPT103
Shaft 14 (RS14)	BH121 BH122 BH123	sCPT104
Shaft 16 (LS16)	BH127 BH128	sCPT105
Shaft 17 (RS17)	BH132 BH133 BH134	sCPT106
MH1 & MH2	NZGD ID BH_69990 NZGD ID BH_69993 NZGD ID BH_69989 NZGD ID BH_69991	NZGD ID CPT_70054 NZGD ID CPT_70051

4.1.3 Geological interpretation

Our assessment of the subsurface geological conditions outlines how the factual data from the site investigations has been interpreted to develop a simplified geological model for each shaft, that we consider suitable for the purposes of this assessment. The models are based on the geomorphology, published geology and previously completed project specific investigations.

The subsurface of the shaft locations is interpreted to comprise of the following geological units:

1 Puketoka Formation;
2 Weathered East Coast Bays Formation; and
3 East Coast Bays Formation rock.

Material descriptions can be generalised as follows:

Puketoka Formation deposits which have been broken down into two sub-units based on organic content. The first unit typically comprised of soft to very stiff clays and silts, or loose to dense sands. These occurred as mixtures and interbedded layers. The second unit comprised of organic silt and peat with occasional woody material.

East Coast Bays Formation deposits have also been broken down into two units based on the weathering profile of the rock. Weathered ECBF is characterised as residual soil to moderately weathered rock, consisting of uncemented or weakly cemented sandstone and mudstone. ECBF rock is characterised as slightly weathered to unweathered ECBF rock, consisting of cemented sandstone and mudstone.

A ground model for each shaft location used in this assessment is presented in Appendix B. A summary of unit depths is presented in Table 4.2 below.

Table 4.2: Summary of the geological unit assumptions at each shaft location

Shaft	Unit	From (m RL)	To (m RL)
BPT	Puketoka	37.0	31.5
	Weathered ECBF	31.5	24.0
	ECBF rock	24.0	Depth
Shaft 8 (LS8)	Puketoka	21.5	17.0
	Weathered ECBF	17.0	16.6
	ECBF rock	16.6	Depth
Shaft 10 (RS10)	Puketoka	30.5	19.3
	Weathered ECBF	19.3	15.7
	ECBF rock	15.7	Depth
Shaft 12 (LS12)	Puketoka – Organic	20.7	12.15
	ECBF rock	12.15	Depth
Shaft 14 (RS14)	Puketoka	26.3	20.0
	Weathered ECBF	20.0	15.0
	ECBF	15.0	Depth
Shaft 16 (LS16)	Puketoka	23.4	2.4
	Weathered ECBF	2.4	-5.9
Shaft 17 (RS17)	Puketoka	14.0	11.0
	Puketoka – Organic	11.0	3.5
	Puketoka	3.5	-9.0
MH1	Puketoka	6.9	Depth
	Weathered ECBF	-10.9	Depth
MH2	Puketoka	7.0	-9.7
	Weathered ECBF	-9.7	Depth

4.2 Geotechnical Parameters

Based on results of geotechnical investigations and testing in this area, and past experience with similar ground conditions, the geotechnical analysis parameters presented in Table 4.3 below are considered appropriate for our assessment of settlement effects.

Table 4.3: Geotechnical parameters

Geological Unit	Unit Weight	Strength		Elastic parameters		Consolidation parameters
	ɣ	c'	ϕ'	E'	ν	M
	kN/m³	kPa	°	MPa	-	(MPa)
Puketoka Formation inorganic	17	2	28	25	0.45	20
Puketoka Formation organic	16	2	26	10	0.45	10
Weathered ECBF	18	0	36	45	0.3	50
ECBF rock	20	50	40	300	0.3	150

4.3 Hydrogeological setting

This report section outlines the hydrological information collected of the conceptual hydrogeology setting at each shaft location. This information is outlined in detail within the geotechnical factual reporting[6].

With particular reference to each shaft location, Table 4.4 outlines the piezometers which were used during our hydrological assessment. Groundwater monitoring was undertaken in each of these piezometers and the location is presented in Appendix F. Specific groundwater levels adopted for each shaft location is detailed in Section 5.

For excavations at MH1 and MH2, our assessment is reliant on groundwater data from Shaft 17 and observed nearby bodies of water.

Table 4.4: Summary of the piezometer installations particular to each shaft location

Shaft ID	Machine Boreholes	Collar RL (AHD1946)	Piezometer type	Tip Install depth or screen depth	Geological Unit
BPT	BH201	36.3	Vibrating Wire	8.5	ECBF
Shaft 8 (LS8)	BH103	21.66	Vibrating Wire	17.0	ECBF
Shaft 10 (RS10)	BH109	27.4	Vibrating Wire	22.0	ECBF
Shaft 12 (LS12)	BH114	17.68	Standpipe	10.5-13.5	ECBF
	BH115	21.6	Standpipe	5.0-8.0	ECBF
	BH115	21.6	Standpipe	16.0-19.0	ECBF
Shaft 14 (RS14)	BH123	28.8	Vibrating Wire	25.12	ECBF
	BH122	28.42	Standpipe	4.0-9.0	Puketoka Formation
	BH122	28.42	Standpipe	11.0-15.0	ECBF
Shaft 16 (LS16)	BH128	22.83	Vibrating Wire	20.00	ECBF
	BH127	24.45	Standpipe	24.45	Puketoka Formation
	BH127	24.45	Standpipe	24.45	ECBF
Shaft 17 (RS17)	BH132	15.1	Standpipe	4.0-7.0	Puketoka Formation
	BH132	15.1	Standpipe	13.0-16.0	Puketoka Formation
	BH134	15.6	Standpipe	11.5-14.5	Puketoka Formation

Further to groundwater monitoring, various investigation locations had permeability tests conducted. This consisted of lugeon testing or rising/falling head testing. The results and summary of the investigations used to inform hydraulic conductivity is given in Table 4.5, and the locations of these investigations is presented in Appendix F.

Table 4.5: Hydraulic conductivity results.

Shaft ID	Machine Borehole	Test Section	Geological Unit	Hydraulic Conductivity (m/s)	Test method
Shaft 8 (LS8)	BH101	12-18	ECBF	7.5E-07	Lugeon
		24-30	ECBF	2.4E-09	Lugeon
	BH103	13-20.25	ECBF	6.0E-08	Lugeon
		9-30	ECBF	1.0E-07	Lugeon
Shaft 10 (RS10)	BH108	11.61-6.61RL	ECBF	7.2E-09	Lugeon
	BH109	9.74-3.74	ECBF	1.7E-09	Lugeon
Shaft 12 (LS12)	BH114	10.5-13.5	ECBF	2.7E-07	Slug test
		12.0-16.0	ECBF	2.9E-09	Lugeon
	BH115	5.0-8.0	ECBF	2.2E-08	Slug test
		16.0-19.0	ECBF	1.1E-06	Slug test
Shaft 14 (RS14)	BH121	12.0-25.5	ECBF	9.1E-08	Lugeon
Shaft 16 (LS16)	BH127	6.6-9.6	Puketoka Formation	1.1E-07	Slug test
		21.0-24.0	ECBF	4.0E-08	Slug test
	BH128	21.6-24.6	ECBF	1.3E-07	Slug test
Shaft 17 (RS17)	BH132	4.0-7.0	Puketoka Formation	2.4E-06	Slug test
		13.0-16.0	Puketoka Formation	2.7E-08	Slug test

5 Dewatering assessment and resulting ground settlement

The section below presents our assessment of groundwater draw down at the proposed shaft locations. Given the pipeline shall be installed by HDD and TBM, both of these trenchless methods have the capability to operate with full face support, which will restrict ingress of groundwater into the tunnel through zones where higher inflows could occur. As a result, we have assessed that negligible groundwater drawdown will occur as a result of the tunnelling and therefore, groundwater drawdown has been ignored for tunnelling.

5.1 Hydrogeological conceptual model

For the Northern Interceptor (main) tunnel, geological modelling was undertaken using a combination of manual review of ground investigation logs and Leapfrog software[7]. Geological sections were generated for six shaft locations (shaft 8, 10, 12, 14, 16, and 17) are shown in Appendix B. These sections show that Puketoka Formation overlies weathered East Coast Bays Formation (ECBF) which overlies competent ECBF rock. These sections also show localised lenses of organic material identified within the Puketoka formation.

For the Massey Connector, the ground model presented in Geotechnical Interpretive Report[8] prepared by Mott MacDonald has been adopted.

For MH1 and MH2, ground models have been derived from nearby investigations as outlined in Table 4.2.

Hydraulic conductivity estimates derived from in-situ falling/rising head tests and packer testing are shown in Table 2 in Appendix F summarising the range of values obtained from site specific testing. These data indicate hydraulic conductivity of the:

- **Puketoka Formation** encountered is in the order of **2.7E-8 m/s** (2.4E-3 m/day) to **2.4E-6 m/s** (2.1E-1 m/day); and
- **ECBF** encountered in the order of **1.7E-9 m/s** (1.5E-4 m/day) to **1.1E-6 m/s** (9.8E-2 m/day).

Groundwater levels / pressures for the hydrogeological conceptual were derived from site specific monitoring. Table 1 in Appendix F summarises the seasonal high and low records obtained from on-site monitoring instrumentation (combination of standpipe piezometers and vibrating wire piezometers).

Based on the groundwater level monitoring record, the aquifer type for the project area has been assessed as unconfined when groundwater levels are low. The available data suggests the aquifer type may be semi-confined (or confined) in some locations when groundwater levels / pressures are higher than during the summer, typically during winter months.

5.2 Numerical model method

5.2.1 Groundwater drawdown and inflows

For the Northern Interceptor (Main) tunnel shafts (shafts 8, 10, 12, 14, 16, 17 and MH1), Analytical Element Method (AEM) groundwater flow modelling software Analytical Aquifer Simulator (AnAqSim[9]) was used to estimate transient groundwater pumping rates and drawdown during the

[7] LeapFrog geological modelling software - Seequent
[8] Northern Interceptor Phase 2 Westgate to Hobsonville, Geotechnical Interpretive Report, Massey Connector Section, dated 26 June 2022, Revision 4, document reference 417039-MMD-01-XX-RP-J-0001, prepared by Mott MacDonald
[9] www.fittsgeosolutions.com

dewatering period for the shafts. AnAqSim is capable of modelling groundwater flow in three dimensions.

The model structure for each of the seven shaft locations has been assessed individually. In this context 'model structure' refers to:

- Number and elevation of model layers / domains
- Secant pile embedment depth (enforced in AnAqSim by elevation of domain elevation), refer Table 1.1
- Shaft diameters Refer Table 1.1 and on individual assumption drawings shown in section 5.3.
- Excavation depth (enforced in AnAqSim using 'head specified line boundaries (HSLB)' and 'spatially variable area sinks (SVAS)'
- Static water levels (initial heads):
  - Seasonal high groundwater levels to assess drawdown and pumping rates.
  - Seasonal low groundwater levels to assess drawdown and related settlement effects.
- Drawdown observation points positioned along four (4) orthogonal lines of section were incorporated into each AnAqSim model. Selection of observation points and lines of section location(s) were informed by structures with potential to be affected by drawdown-induced settlement.

Model structure information used to develop individual AnAqSim models are presented graphically alongside the model results presented in Section 5.3 (for ease of interpretation).

Hydraulic input parameters applied to each of the seven shaft models are consistent across all models (i.e. do not change between AnAqSim models). These values represent upper bound conditions (e.g. highest inferred hydraulic conductivity values) informed by our conceptual model presented in Section 5.1, and were selected for the purpose of assessing potential effects on the receiving environment. These include:

- **Puketoka Formation - organic and inorganic:**
  - Horizontal hydraulic conductivity (Kh): 1E-1 m/day
  - Specific yield (Sy): 0.1
- **Weathered ECBF:**
  - Kh: 1E-1 m/day
  - Sy: 0.05
- **Competent ECBF:**
  - Kh: 1E-1 m/day
  - Storativity (S): 5E-4 (no units)
- **Leaky barrier conductance:**
  - Represents the leakiness of the secant pile retention system
  - The "potential case" for effects assessment purposes was set to 1E-2 day^{-1} (conservative as this assumes a leaking wall), whereas the "expected case" was set to 1E-3 day^{-1}, refer Table 5.2: Model sensitivity anlaysis.

5.2.2 Drawdown-induced settlement

Drawdown-induced settlement was calculated using an incremental layer summation method using python programming. This approach calculated the decrease in pore water pressure and corresponding increase in effective stress at the centre of each incremental layer caused by the groundwater drawdown. Modulus of compressibility (mv) values adopted for analysis are shown in Table 5.1 for each geological unit identified from Leapfrog model.

Table 5.1: Adopted compressibility (mv) values

Leapfrog Model - Unit ID	Modulus of compressibility (mv) kPa
Puketoka (inorganic)	20,000
Puketoka (organic)	10,000
Weathered ECBF	50,000

Observation points were added to the AnAqSim model for assessing groundwater drawdown due to dewatering. The observation points positioned along four orthogonal lines of section represent areas of interest close to buildings and/or services which may be affected by drawdown-induced settlement.

Drawdown-induced settlement was calculated for each drawdown observation point using the following approach:

- Observation points (X,Y) obtained from lines of section.
- Geological contact elevation (Z) values (m RL) obtained from developed LeapFrog model.
- Static water level ($W_{initial}$) adopted from the hydrogeological conceptual model.
- Final groundwater level (W_{final}) obtained from the AnAqSim model results.
- 1D settlement assessment using an incremental layer-wise summation method calculated in a Python[10] script.
  - Divided the geological profile (H_{total}) into incremental units for calculation, in this case 0.1 m thick.
  - Assigned assumed constrained modulus to each unit.
  - Calculated the change in pore water pressure at the centre of each incremental layer caused by the groundwater drawdown (refer Equation 2).
  - Estimated the settlement of each incremental unit layer and sum the incremental settlement (refer Equation 1).
- The following assumptions were made for the settlement assessment:
  - Initial static water levels were considered hydrostatic.
  - ECBF rock was considered incompressible.

[10] www.python.org

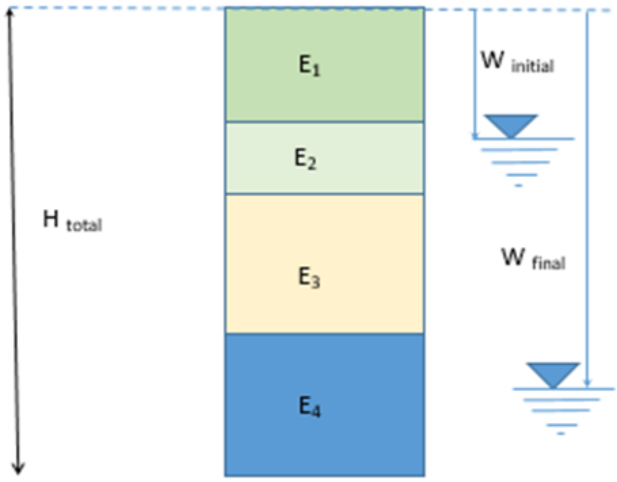


Figure 5.1: Example soil column and initial/final water level for calculating settlement using layer-wise summation method

Equation 1: Layer wise summation method:

$$S = \sum_{i=1}^{n} \left(\varphi \frac{\Delta P_i}{E_i} H_i \right)$$

$$S = total\ settlement\ caused\ by\ dewatering\ (m)$$

$$\Delta P = change\ in\ pore\ water\ pressure\ \text{(Equation 2)}$$

$$\Delta P_i = additional\ load\ of\ the\ calculated\ soil\ layer\ caused\ by\ dewatering\ (kPa)$$

$$\varphi = \text{empirical coefficient, defined as 1 in this calculation}$$

$$E_i = compression\ modulus\ of\ the\ calculated\ soil\ layer\ (kPa)$$

$$H_i = thickness\ of\ the\ calculated\ soil\ layer (m)$$

Equation 2: Change in pore water pressure:

$$\Delta P = \gamma_w (Water_{initial} - Water_{final})$$

$$\Delta P = change\ in\ pore\ water\ pressure\ (kPa)$$

$$\gamma_w = unit\ weight\ of\ water\ (kPa)$$

$$Water_{initial} = Piezomteric\ head\ before\ dewatering\ (m)$$

$$Water_{final} = Piezomteric\ head\ after\ dewatering\ (m)$$

5.2.3 Assumptions and limitations

Assumptions:

For modelling purposes we have assumed:

- Flat water table / no hydraulic gradient or direction.
- Rainfall recharge does not occur.
- Radius of AEM model set to 1 km from each shaft centre.
- Timestep 365 day selected to represent pseudo steady-state conditions[11].
- Hydrostatic conditions / groundwater levels in each geological unit are the same.
- For groundwater modelling, horizontal / uniform thickness geological units were assumed for consistency with simplifying assumption of AnAqSim software.
- For settlement calculations, variable elevation for geological contacts were based on the developed LeapFrog model (cross sections presented).
- Hydraulic conductivity was assumed to be isotropic: Kv = Kh.
- Competent ECBF rock was assumed to be incompressible.
- The settlement results documented reflect summer groundwater conditions. It is assumed that material above the recorded summer low has already consolidated and will not settle further with fluctuating groundwater due to dewatering in the shafts.

Limitations:

The dewatering assessment **did not** account for:

- Design:
  - Any variation in excavation level.
  - Any variation in retention embedment depth.
- The time of year that excavation dewatering/ pumping will commence and cease.
- Ground model:
  - An undulating geological profile (for the groundwater component/ AnAqSim model).
  - Any variation in modulus of compressibility values.
  - Any variation in hydraulic input parameters.
  - Any unidentified geological units.
- Infiltration (rainfall) recharge.

Implications:

The implications for the limitations are described below in relative terms **(i.e. greater or less than those modelled)** and include:

- Design:
  - If the excavation level adopted for design is shallower the inflows and drawdowns estimates are expected to be less, and vice-versa.
  - If the retention embedment depth adopted for design is shallower the inflows and drawdowns estimates are expected to be greater, and vice-versa.
- If dewatering/ pumping commences during the summer months peak flows are expected to be lower than winter flows.
- Ground model:
  - Drawdown and inflow estimates are generally a function of the boundary conditions, the thickness of geological units, and the hydraulic conductivity, Kv/Kh ratio, and

[11] We note that checks were completed for each model to ensure drawdown zone of influence did not reach the outer boundary conditions set at 1 km distance from shafts.

  - specific yield values for each unit. Mapping and estimation of these are subject to inherent uncertainty; however, our selection of the hydraulic input parameter values and adopted thicknesses of geological units are considered to be suitably conservative.
  - If consolidation settlement has already occurred in the dewatered units the observed settlement may be less than calculated. If more compressible soils exist the settlement estimates may be greater. However our selection of the modulus of compressibility values adopted are considered to be suitably conservative.
- Excluding infiltration recharge suggests that the modelled drawdown and settlement estimates are over-estimated.

5.2.4 Sensitivity analysis

There is inherent uncertainty associated with groundwater modelling. For deep shaft dewatering assessments, this uncertainty predominantly arises from uncertain performance of the retention system (i.e. its ability to impede and reduce groundwater inflow). Performance of the retention system is uncertain due to various contributing factors such as the type and the sequence of activities undertaken by the construction contractor, and the interaction with variable ground conditions. To assess the impact of this, a parameter sensitivity analysis on the retention conductance (leakiness) was completed.

A sensitivity analysis was undertaken to assess potential variability of the retention system/ leaky barrier boundary condition (refer Table 5.2).

Table 5.2: Model sensitivity analaysis

Model run ID	Leaky barrier conductance (day^{-1})	Interpretation
Run0	1E-02	**<u>Potential short-term scenario (leaky state):</u>** • Allows for temporary hydraulic defects in the retention system and associated leakage during the construction/ dewatering period. • In this event, we expect that the contractor would use practical measures to control these (e.g. by grouting or plugging gaps) to reduce groundwater leakage into the excavation. • Assumes remedial works occur over a short period to return to the expected long-term / average scenario as quickly as possible.
Run1	1E-03	**<u>Expected long-term / average scenario (water tight state):</u>** • Retention system performs as designed / intended. • Largely impermeable medium, but accounts for minor groundwater leakage through the retention system.

5.3 GeoStudio SEEP/W 2D method

For MH2 the effect of the groundwater drawdown and consolidation effects from the excavation work has been modelled through a 2-dimensional (2D) Seep/W models within GeoStudio software. A differing numerical model methodology was select for this excavation due to the surrounding site topography and limitations associated with AnAqSim to Model change in ground elevation.

Groundwater drawdown was assessed with seepage mitigation measures for the retained excavations (sheet pile walls) and an open base during construction allowing for up to 60 days of dewatering. The sheet pile walls have been modelled with sealed clutches. Modelling guidance on equivalent permeabilities and thicknesses of steel sheet piles was taken from ArcelorMittal[12].

Groundwater drawdown levels in this assessment have been based on observed groundwater levels from the groundwater monitoring programme and known bodies of water to the northeast (Wallace Inlet) and southwest (Limeburners Bay) of the site. Assessment of settlement effects are therefore based on the lowest observed groundwater level.

Details of the model setup are shown in the model outputs appended in Appendix D. The material parameters adopted are consistent with those for the AnAqSim models.

5.4 Empirical model method

For the BPT located on the Massey connector, groundwater drawdown estimates have been made using established simplified methods commensurate with the assumed nature of the ground model and general limited infrastructure surrounding the BPT shaft.

The lateral extent of groundwater drawdown away from an open excavation has been estimated using the following equation published in CIRIA (2016):

$R_o = C.(H-h_w)k^{0.5}$

where:

R_o is the radius of influence;

C is an empirical calibration factor (usually taken as 3000);

$(H - h_w)$ is the depth of water drawdown at the excavation; and

k is the permeability (hydraulic conductivity) of the soil.

The CIRIA method assumed a single uniform permeability within the drawdown zone. Groundwater monitoring within BH201 located at the proposed BPT sites indicates seasonal groundwater lows to be approximately at the interface of the weathered ECBF and overlying Puketoka Formation soils (approximately 5.5 m below existing ground level). As such, groundwater may drawdown up to 3.9 m with the majority of the groundwater drawdown occurring within the weathered ECBF. As discussed in 5.1, hydraulic conductivity estimates derived from in-situ falling/rising head tests and packer for weathered **ECBF** range in the order of **1.7E-9 m/s** (1.5E-4 m/day) to **1.1E-6 m/s** (9.8E-2 m/day). No specific hydraulic conductivity value is provided for weathered ECBF at BH201. As such, a hydraulic conductivity of 1.7E-9 m/s is conservatively adopted to estimate the radius of influence for drawdown.

5.5 Numerical model results

Model results are reported at 5 and 365 day time steps for inflows, and at 365 days (selected to represent pseudo steady state ie. long-term condition) for drawdown and settlement.

An overview of drawdown is provided in the section below. For ease of interpretation, figures presenting the locations of lines of section and assumption sketches used for AnAqSim modelling are presented alongside model results in Appendix C.

5.5.1 Shaft 17

The shaft 17 model results are shown in Appendix C.

[12] ArcelorMittal (2014): Impervious steel sheet pile walls – Design and practical approach

Adopting the leaky shaft excavation condition, maximum groundwater drawdown next to the shaft during winter conditions has been estimated to be 5 m, reducing to 0.5 m at 100 m distance. During summer conditions, the drawdown has been estimated to be 3 m next to the excavation reducing to 0.75 m at 100 m distance.

Modelled drawdown-induced settlement immediately outside the excavation has been estimated to be 50 mm adopting summer groundwater conditions. Surface ground settlement reduces to less than 10 mm at a distance of 70 m from the excavation.

Pumping rates have been presented adopting the winter high groundwater condition. Under this condition, pumping rates are estimated to be 80 m^3/day during the early stages of dewatering (ie. first few days). The pumping rate estimate reduces to 30 m^3/day at pseudo steady state (modelled timestep of 365 days).

Groundwater seepage is expected to cease into the shaft once the base and walls have been sealed.

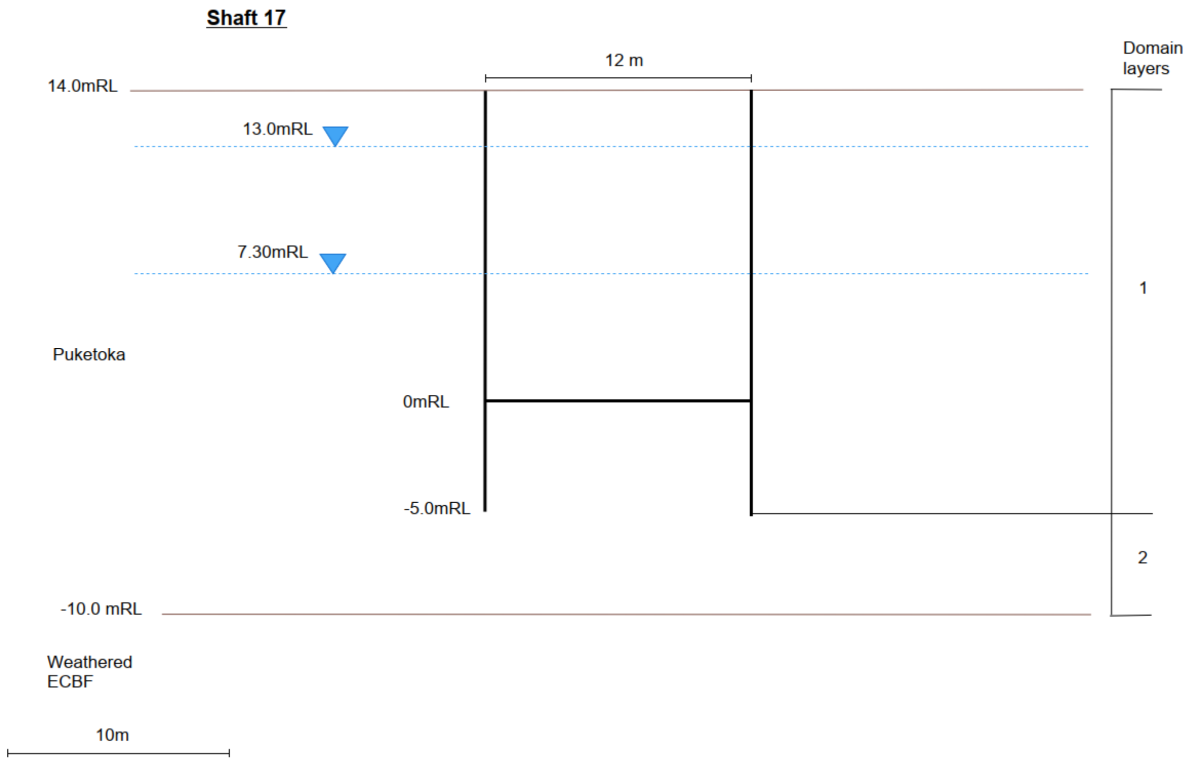


Figure 5.2: Shaft 17 assumption sketch for AnAqSim modelling

5.5.2 Shaft 16

The shaft 16 model results are shown in Appendix C.

Adopting the leaky shaft excavation condition, maximum groundwater drawdown next to the shaft during winter conditions has been estimated to be 5 m, reducing to 0.5 m at 100 m distance. During summer conditions, the drawdown has been estimated to be 2.8 m next to the excavation reducing to 0.4 m at 100 m distance.

Modelled drawdown-induced settlement immediately outside the excavation has been estimated to be 22 mm adopting summer groundwater conditions. Surface ground settlement reduces to less than 10 mm at a distance of 25 m from the excavation.

Pumping rates have been presented adopting the winter high groundwater condition. Under this condition, pumping rates are estimated to be 70 m^3/day during the early stages of dewatering. The pumping rate estimate reduces to 30 m^3/day at pseudo steady state (modelled timestep of 365 days).

Groundwater seepage is expected to cease into the shaft once the base and walls have been sealed.

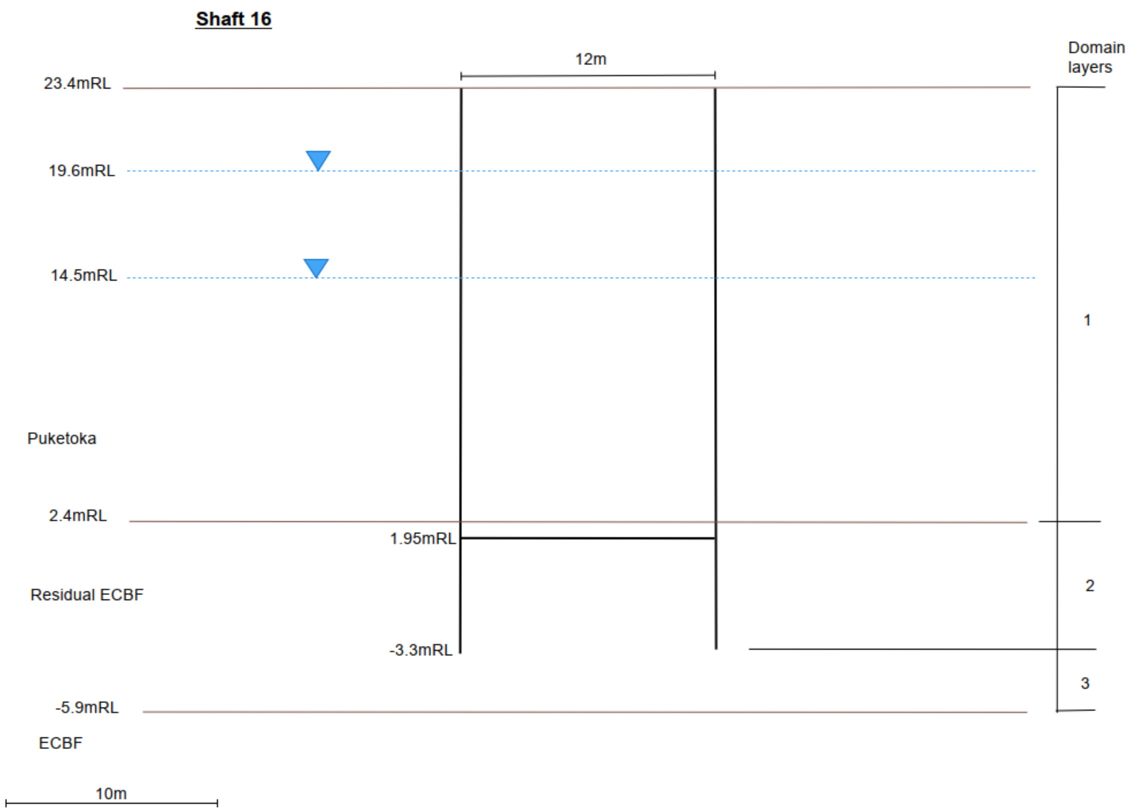


Figure 5.3: Shaft 16 assumption sketch for AnAqSim modelling

5.5.3 Shaft 14

The shaft 14 model results are shown in Appendix C.

Adopting the leaky shaft excavation condition, maximum groundwater drawdown next to the shaft during winter conditions has been estimated to be 1.75 m, reducing to 0.25 m at 100 m distance. During summer conditions, the drawdown has been estimated to be 1.5 m next to the excavation reducing to 0.3 m at 100 m distance.

Modelled drawdown-induced settlement immediately outside the excavation has been estimated to be 3.5 mm adopting summer groundwater conditions. Surface ground settlement reduces to less than 1.5 mm at a distance of 25 m from the excavation.

Pumping rates have been presented adopting the winter high groundwater condition. Under this condition, pumping rates are estimated to be 30 m^3/day during the early stages of dewatering. The pumping rate estimate reduces to 12 m^3/day at pseudo steady state (modelled timestep of 365 days).

Groundwater seepage is expected to cease into the shaft once the base and walls have been sealed.

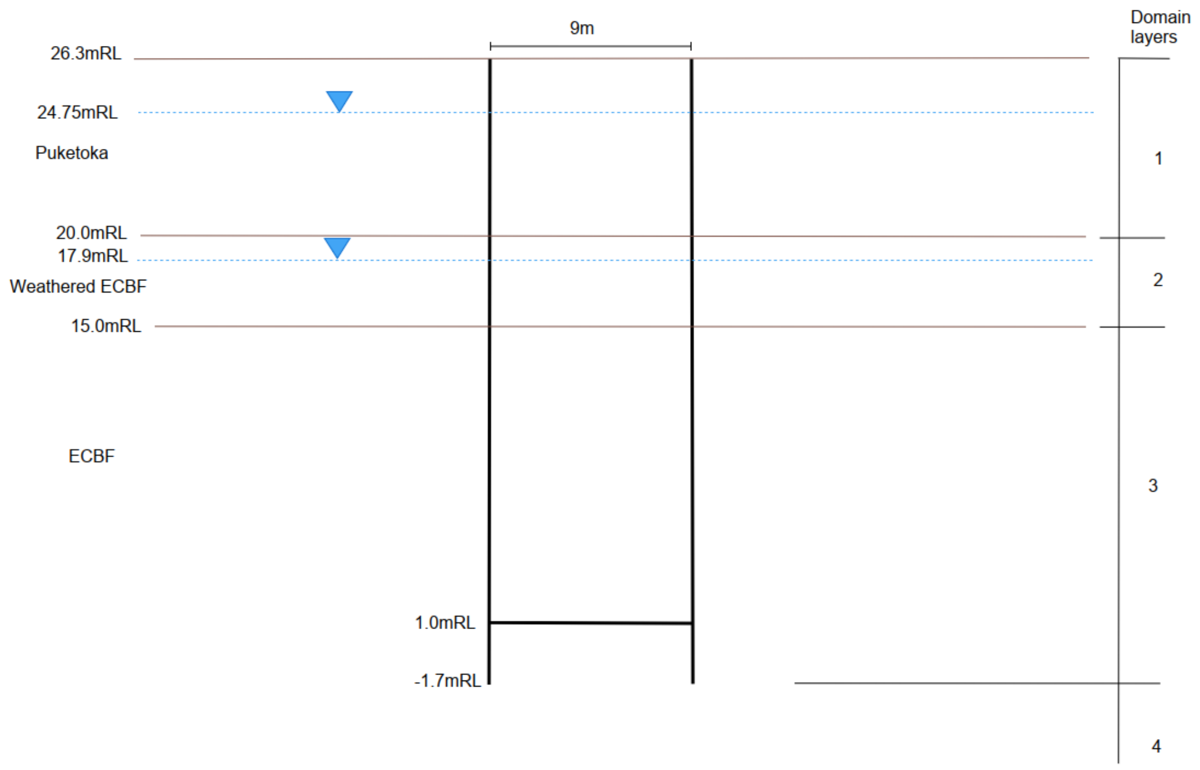


Figure 5.4: Shaft 14 assumption sketch for AnAqSim modelling

5.5.4 Shaft 12

The shaft 12 model results are shown in Appendix C.

Adopting the leaky shaft excavation condition, maximum groundwater drawdown next to the shaft during winter conditions has been estimated to be 2.2 m, reducing to 0.25 m at 100 m distance. During summer conditions, the drawdown has been estimated to be 1.6 m next to the excavation reducing to 0.2 m at 100 m distance.

Modelled drawdown-induced settlement immediately outside the excavation has been estimated to be 5 mm adopting summer groundwater conditions. Surface ground settlement reduces to less than 1 mm at a distance of 100 m from the excavation.

Pumping rates have been presented adopting the winter high groundwater condition. Under this condition, pumping rates are estimated to be 35 m^3/day during the early stages of dewatering. The pumping rate estimate reduces to 14 m^3/day at pseudo steady state (modelled timestep of 365 days).

Groundwater seepage is expected to cease into the shaft once the base and walls have been sealed.

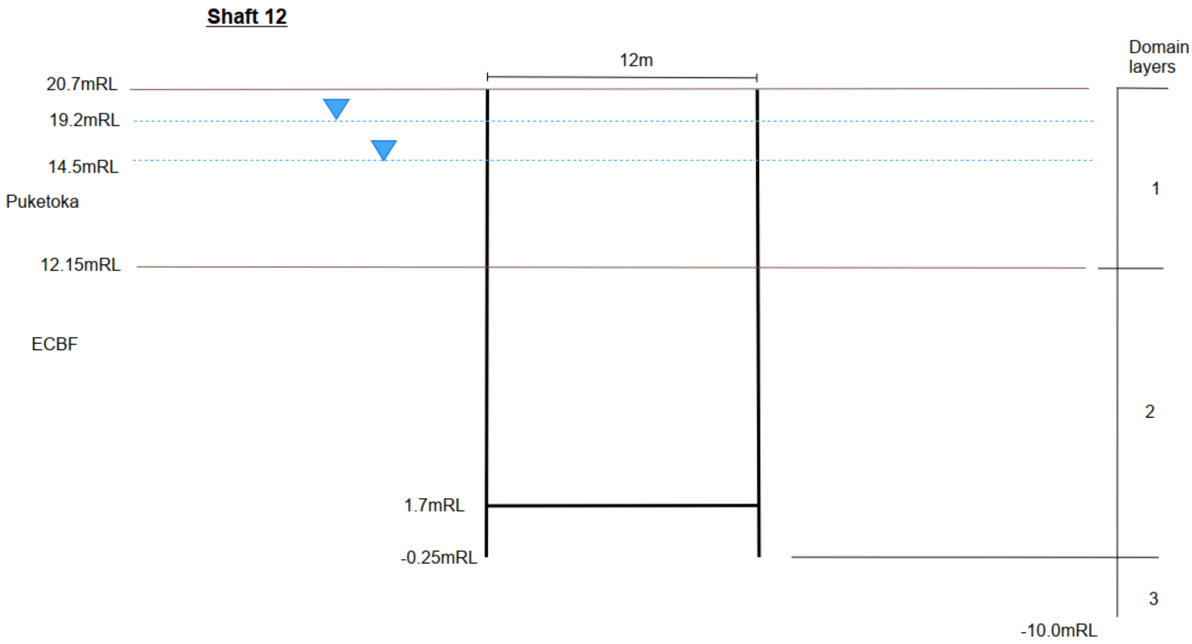


Figure 5.5: Shaft 12 assumption sketch for AnAqSim modelling. Note excavation level rounded to 1.7 m RL

5.5.5 Shaft 10

The shaft 10 model results are shown in Appendix C.

Adopting the leaky shaft excavation condition, maximum groundwater drawdown next to the shaft during winter conditions has been estimated to be 5.5 m, reducing to 0.5 m at 100 m distance. During summer conditions, the drawdown has been estimated to be 5.1 m next to the excavation reducing to 0.5 m at 100 m distance.

Modelled drawdown-induced settlement immediately outside the excavation has been estimated to be 14 mm adopting summer groundwater conditions. Surface ground settlement reduces to less than 4 mm at a distance of 100 m from the excavation.

Pumping rates have been presented adopting the winter high groundwater condition. Under this condition, pumping rates are estimated to be 60 m^3/day during the early stages of dewatering. The pumping rate estimate reduces to 35 m^3/day at pseudo steady state (modelled timestep of 365 days).

Groundwater seepage is expected to cease into the shaft once the base and walls have been sealed.

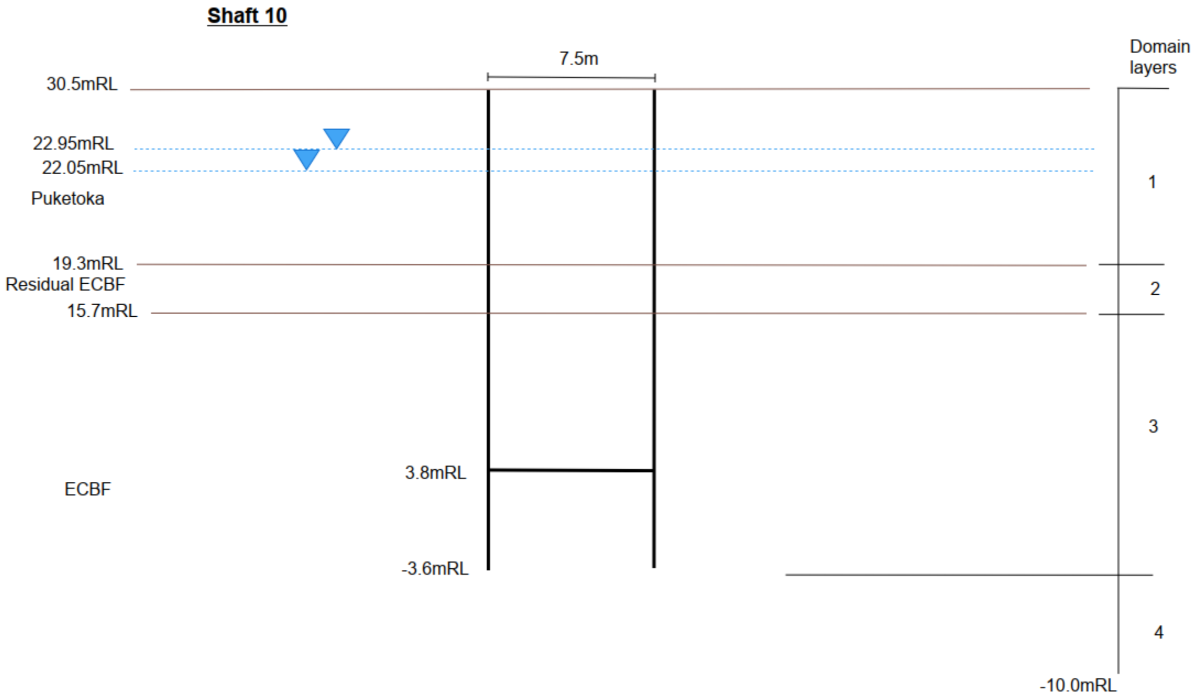


Figure 5.6: Shaft 10 assumption sketch for AnAqSim modelling

5.5.6 Shaft 8

The shaft 8 model results are shown in Appendix C.

Adopting the leaky shaft excavation condition, maximum groundwater drawdown next to the shaft during winter conditions has been estimated to be 8 m, reducing to 0.75 m at 100 m distance. During summer conditions, the drawdown has been estimated to be 6.2 m next to the excavation reducing to 0.5 m at 100 m distance.

Modelled drawdown-induced settlement immediately outside the excavation has been estimated to be 17 mm adopting summer groundwater conditions. Surface ground settlement reduces to less than 10 mm at a distance of 25 m from the excavation.

Pumping rates have been presented adopting the winter high groundwater condition. Under this condition, pumping rates are estimated to be 110 m^3/day during the early stages of dewatering. The pumping rate estimate reduces to 40 m^3/day at pseudo steady state (modelled timestep of 365 days).

Groundwater seepage is expected to cease into the shaft once the base and walls have been sealed.

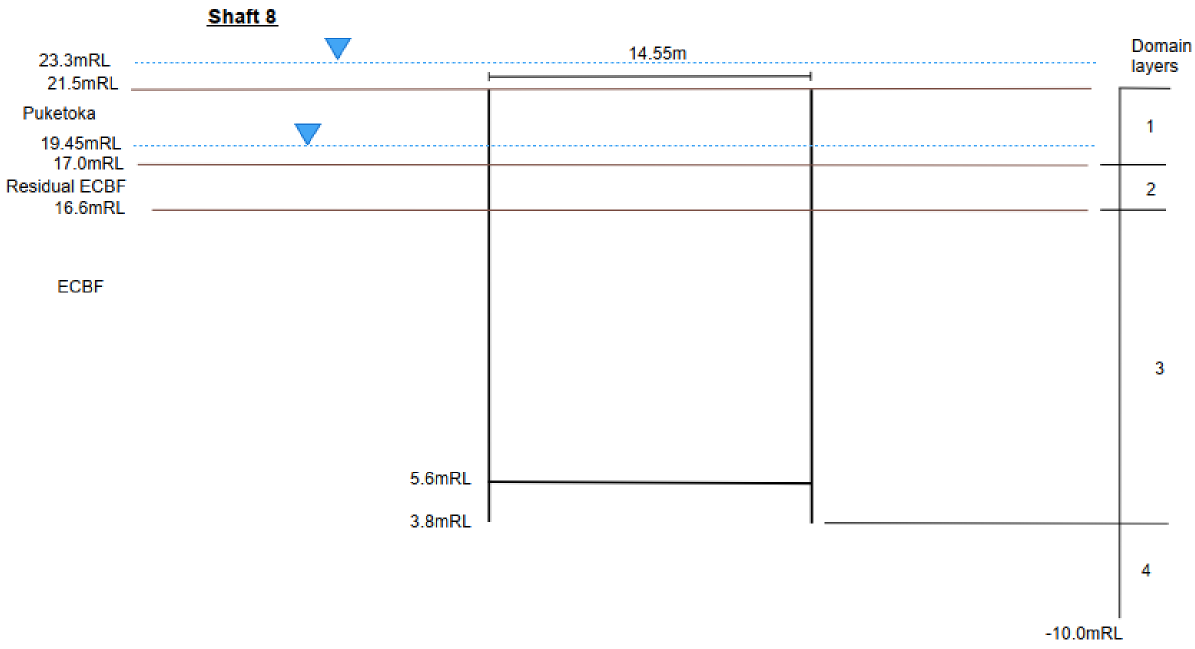


Figure 5.7: Shaft 8 assumption sketch for AnAqSim modelling

5.5.7 MH1 – secant piles

The MH1 model results are shown in Appendix C.

Adopting the leaky shaft excavation condition, maximum groundwater drawdown next to the manhole during winter conditions has been estimated to be 1.7 m, reducing to 0.04 m at 100 m distance. During summer conditions, the drawdown has been estimated to be 1.4 m next to the excavation reducing to 0.03 m at 100 m distance.

Modelled drawdown-induced settlement immediately outside the excavation has been estimated to be 22 mm adopting summer groundwater conditions. Surface ground settlement reduces to less than 10 mm at a distance of 10 m from the excavation.

Pumping rates have been presented adopting the winter high groundwater condition. Under this condition, pumping rates are estimated to be 25 m^3/day during the early stages of dewatering. The pumping rate estimate reduces to 6.5 m^3/day at pseudo steady state (modelled timestep of 180 days).

Groundwater seepage is expected to cease into the shaft once the base and walls have been sealed.

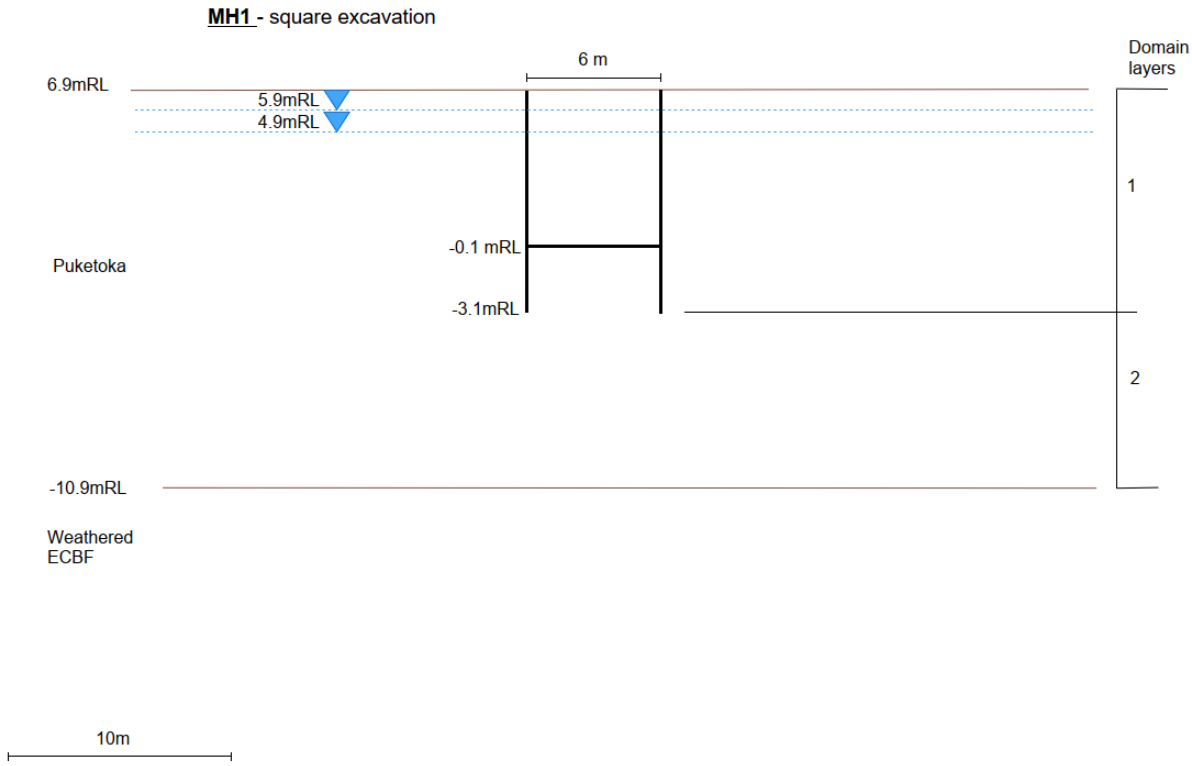


Figure 5.8: MH1 assumption sketch for AnAqSim modelling

5.5.8 MH2 sheet pile

The MH2 model results are shown in Appendix D.

Adopting sheet piles parameters consistent with recommendations in ArcelorMittal modelling guidance, maximum groundwater drawdown next to the shaft during summer conditions has been estimated to be 0.6 m next to the excavation reducing to less than 5 mm at 100 m distance.

Modelled drawdown-induced settlement immediately outside the excavation has been estimated to be 10 mm adopting summer groundwater conditions. Surface ground settlement reduces to less than 5 mm at a distance of 20 m from the excavation.

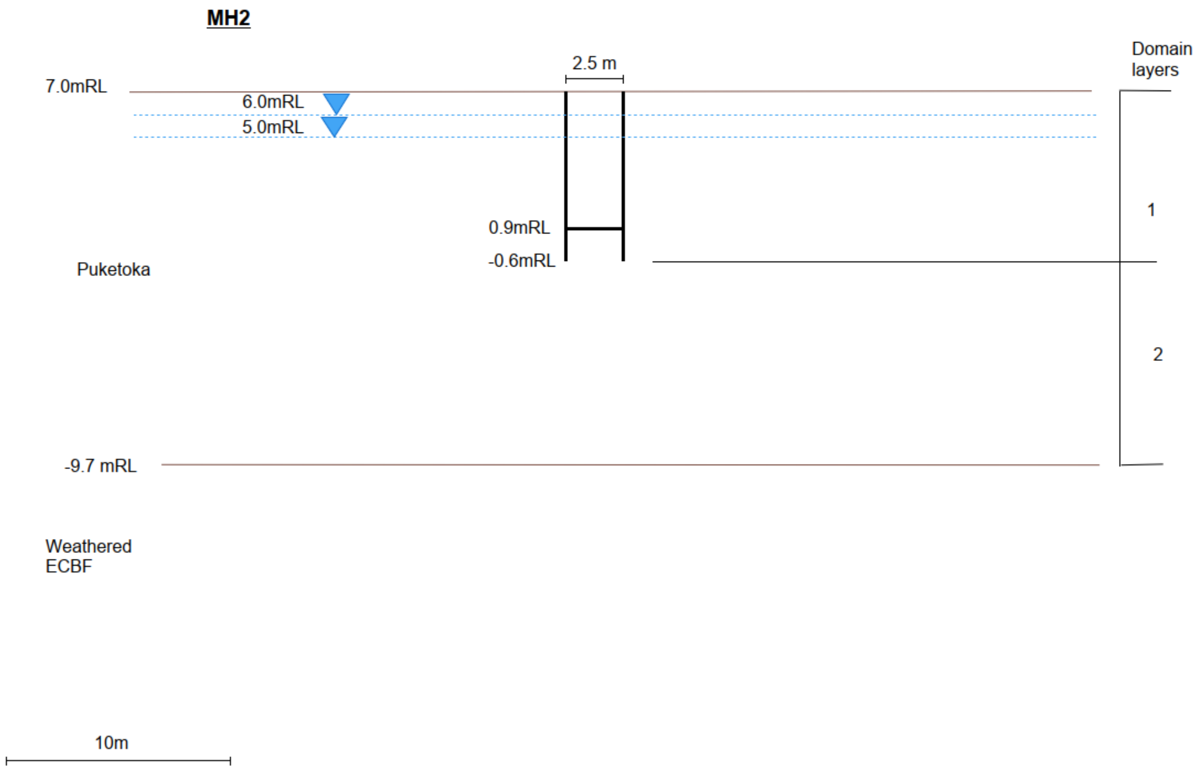


Figure 5.9: MH2 assumption sketch for SEEP/W 2D modelling

5.6 Empirical model results

Based on the CIRIA methodology, the maximum lateral extent of groundwater drawdown away from the BPT shaft is 12.5 m. Adopting the same 1D settlement calculation as presented in section 5.2 and a maximum groundwater drawdown of 3.9 m results in less than 6 mm of settlement.

6 Mechanical ground settlement assessment

The section below presents our assessment of surface ground settlement arising from:

1 Tunnelling arising from the over excavation of face commonly referred to as “volume loss”; and

2 Shaft excavation retention deformation resulting in soil relaxation and displacement into the excavation as the retention walls displace inwards.

6.1 Settlement due to tunnelling

6.1.1 Method

The method of New and O’ Reilly (1982)[13] has been used to assess the maximum magnitude and lateral extent of mechanically induced ground settlement due to construction of the proposed wastewater tunnel.

Tunnelling publications comprising experience from thousands of projects suggest that volume loss can vary from 0% to 4% depending partly on the ground conditions, but primarily from the specific capabilities of the TBM being used and contractor performance. From experience the following parameters can be adopted when assessing settlement associated with tunnelling as a result of volume loss. For our assessment, we have adopted a 0.7 m dia TBM for the 600 mm OD pipe and a 2.5 m dia TBM for the 2.1 m OD pipe.

Table 6.1: Parameters adopted for assessing settlement due to tunnelling volume loss

Parameter	0.7 TBM Dia. (m)	2.5 TBM Dia. (m)
Volume loss in Puketoka Formation (%)	2.5	2.5
Volume loss in residual ECBF (%)	N/A	1.0
Volume loss in HW to UW ECBF (%)	N/A	0.5
Trough width factor, K	0.5	0.5

The Massey connector tunnel alignment and the Northern Interceptor (Main) tunnel from shaft 8 to shaft 16 is expected to be excavated entirely within either ECBF. From shaft 16 through to shaft 17, excavations are likely to proceed within Puketoka Formation. From shaft 17 to MH2, the pipe reduces to a 600 mm outer and is likely to remain within Puketoka Formation.

6.1.2 Results

The surface ground settlements for different tunnel depths and volume losses is presented in Table 6.2. The results indicate that where the tunnel is excavated in ECBF, surface ground settlements are less than 5 mm with differential settlement less than 1V:2000H. Where the 2.5 m diameter tunnel is progressed through Puketoka formation, surface ground settlement increase but still remain below 10 mm with differential settlement less than 1V:850H for a tunnel centreline depth of 10 m. For the smaller 0.7 m diameter tunnel, surface ground settlements are expected to be less than 5 mm with differential settlement less than 1V:2000H. We note that the majority of the tunnel is at depths of 15 m to 20 m with shallow tunnel depths located locally around the BPT and shaft 8.

[13] Tunnelling induced ground movements; predicting their magnitude and effects, Barry M New, Myles P O’ Reilly, Ground Engineering Division, Transport and Road Research Laboratory, Crowthorne, 1991

Table 6.2: Summary of surface ground settlement resulting from tunnelling volume loss

Tunnel diameter (m)	Volume loss (%)	Depth to tunnel centreline (m)	Surface ground settlement (mm)	Maximum differential settlement
2.5	0.5 (HW - UW ECBF)	10	<5	less than 1V:2000H
	1.0 (residual ECBF)	10	<5	less than 1V:2000H
	2.5 (Puketoka Formation)	10	9.7	less than 1V:850H
		12	8.1	less than 1V:1200H
		14	6.9	less than 1V:1600H
		16	6.1	less than 1V:2000H
		18	5.4	less than 1V:2000H
		20	<5	less than 1V:2000H
0.7		4	<5	less than 1V:1800H
		10	<5	less than 1V:2000H
		14	<5	less than 1V:2000H

Given the proposed depth of the tunnel and ground conditions, we expected surface ground settlement due to tunnelling arising from volume loss to be less than 5 mm for the Massey connector through to shaft 16. From shaft 16 to shaft 17, surface settlements from tunnelling are expected to be less than 10 mm.

6.2 Trench excavations and MH2 sheet pile excavation

Based on discussions with the client, it is understood that approximately 10 m of open trenched excavations are proposed from MH2 to the existing Hobsonville Pump Station inlet chamber.

6.2.1 Method

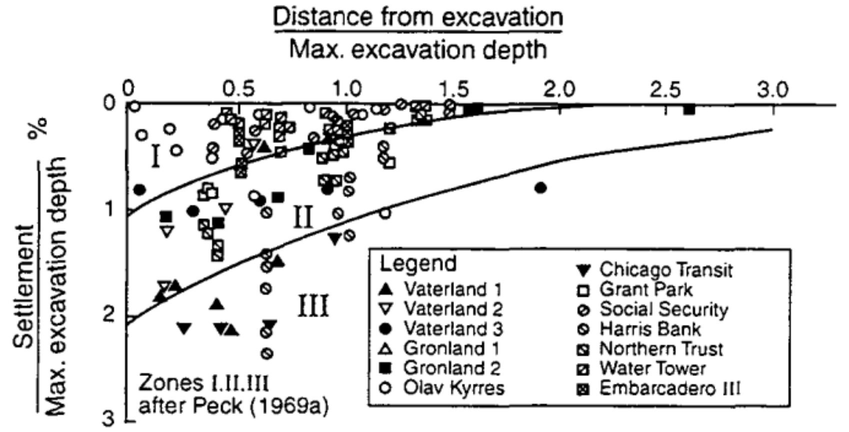
A single representative model has conservatively been adopted for the open trenched excavations and sheet pile excavations at MH2. To assess the zone of influence from mechanical settlement caused by the construction of the trenches, the following has been assumed:

- The maximum depth to invert for trenched sections is assumed to be 5.22 m bgl based on drawings provided. This equates to a maximum excavation depth of 5.72 m bgl, assuming 0.5 m of overdig for pipe bedding.
- The sub-surface geology along the trench generally comprises organic-rich Puketoka Formation soils. This unit has generally been logged as firm to stiff, fibrous and amorphous clayey peat.
- The excavation will be retained using trench shields. It has been assumed the contractor will undertake temporary works design to confirm the trench shields to be used have sufficient capacity (i.e. prop buckling etc.) to support the excavation.
- Corner and edge effects from trench excavations have been conservatively ignored.

Mechanical settlements due to the open trench excavation has been assessed using the empirical method outlined in Clough and O'Rourke (1990)[14]. This journal presents an empirical design chart (Figure 10, duplicated as Figure 6.1 below) which provides an estimate of surface settlements for retaining walls in soft to firm clays.

[14] Clough, G.W., 1990. Construction induced movements of in situ walls. Design and performance of earth retaining structures, pp.439-470.

The controlling input parameters are the depth of excavation and the type of zone. For the purpose of this assessment, Zone I to II has been used, which corresponds to a retaining wall constructed in firm clay with average workmanship.



(c) Summary of measured settlements adjacent to excavations in soft to medium clay

Figure 6.1: Extract of design chart (Figure 10) from Clough & O'Rourke (1990)

6.2.2 Results

The results of the mechanical induced settlement for the analysed trench excavation is shown in Figure 6.2. This suggests settlements of approximately 60 mm for excavations of depth up to 6 m. This reduces to no movement estimated at approximately 14 m from the edge of the excavation. This depth is representative of both the trench works expected from MH2 to the Hobsonville Pump Station and the sheet pile excavations at MH2 if lateral bracing is provided.

Estimated settlements can be reduced through lateral bracing. This can be achieved by propping sheet pile excavations or using jacking slide rail shoring systems which provide active lateral excavation support. Through these systems, mechanical induced settlements can be reduced below 5 mm.

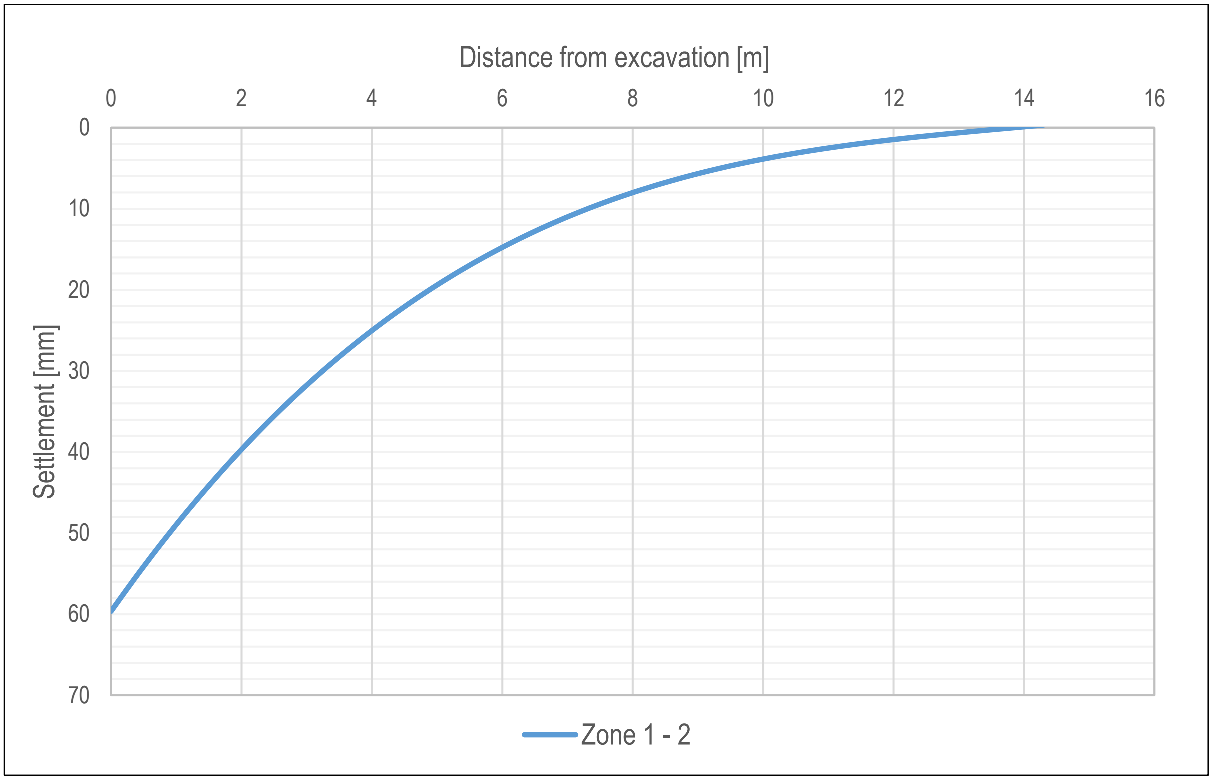


Figure 6.2: Graph of estimated mechanical settlements from open trench excavations assuming no lateral restraint

6.3 Shaft excavations

The section assesses the surface ground settlement resulting from the excavation of the secant piled shafts. As the chosen construction methodology for all shafts is circular secant pile retention system, these structures are inherently rigid by design as the structural system goes into compression to resist the lateral earth pressures, rather than deforming as seen with conventional cantilevered or tied-back retention systems. This results in very small deformations.

As such, we have only assessed the surface ground settlement associated with Shaft 16 to demonstrate that ground settlement arising from the mechanical deformations of the shafts are negligible. The ground conditions, size and depth of shaft 16 should result in among the largest effect of all the secant shafts and as such the results should be conservative when applied to remaining secant shafts. Given the negligible surface ground settlement that arise from the deformation of the shaft retention, the effects of mechanical ground settlement from shafts is not assessed in further detail beyond this section of the report.

6.3.1 Method

Two dimensional Fast Lagrangian Analysis of Continua (FLAC Version 7.0, Itasca Consulting Group) was used to model soil-structure interaction and estimate a settlement profile for shaft 16. Figure 6.3 shows the model generated in FLAC.

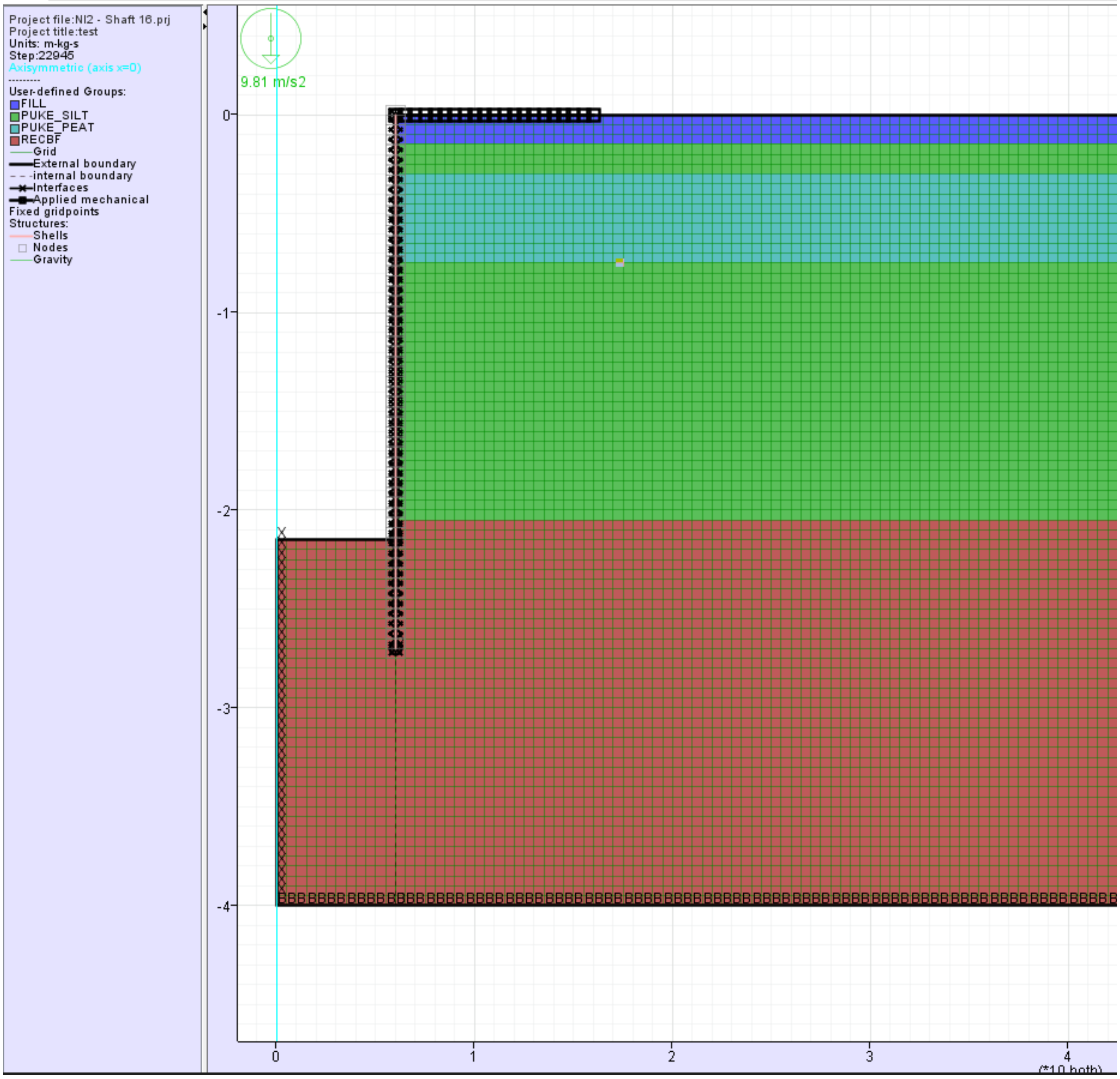


Figure 6.3: Cross section of FLAC model

The ground conditions are conservatively based on 2 nearby boreholes (WSL-21-BH127 and WSL-21-BH128) and 2 nearby CPTs (WSL-21-SCPT105 and WSL-21-SCPT105a). The ground model has been simplified by using horizontal layers as summarised in Table 6.3 below.

Table 6.3: Geological model used for modelling

Depth (m)	Geological model
0 to 1.5	Fill
1.5 to 3.0	Puketoka Formation
3.0 to 7.5	Puketoka Formation organic silts and peat
7.5 to 20.5	Puketoka Formation
20.5 to base	CW to HW ECBF

Table 6.4: Dimensions of shaft

Shaft depth (m)	Inner Shaft Diameter (m)	Support
21.5	11.25 m	800 mm secant piles at 550mm c/c spacing

The modelled groundwater level has been conservatively assumed to be at the ground surface outside of the shaft throughout the construction. The shaft will be excavated "dry", and therefore the groundwater level inside the shaft is assumed to be at the base of the excavation. This is considered appropriately conservative for the assessment of mechanical settlement as the secant piles will be subject to higher retention pressures compared to if groundwater drawdown was modelled.

A 35kPa surcharge has been modelled over a 7 m wide area directly behind the shaft opening to account for construction machinery. This is based on a 120-tonne crane on a 7 m x 5 m wide pad.

6.3.2 Structural properties of ring beam installation

The properties in Table 6.5 have been adopted in our model.

Table 6.5: Structural properties used for modelling

Structural member	Modelled structure element	Average Young's Modulus, E (GPa)	Poisson's ratio	Thickness (m)
Secant pile	Axisymetric shell element	13*	0.2	0.55**

*Average of pile young's modulus based on contribution from hard piles only (30MPa compressive strength concrete). The contribution of soft piles are conservatively ignored.

**Based on the overlap thickness of a 850 mm diameter piles at 550 mm centre-to-centre spacing.

6.3.3 Construction sequence

The following sequence has been assumed for the analysis:

1 Construct secant pile in a hard and soft sequence.

2 Excavate down to final depth (~21.5 m below ground surface modelled).

6.3.4 Assumptions and analysis limitations

The modelling results are based on the following assumptions:

1 An axisymmetric model has been used, which does not allow for the explicit modelling of unbalanced loading (variation in ground conditions, groundwater conditions or ground surcharges). However, we expect the effect on settlement is negligible.

2 The secant piles are interconnected and behave as a compression ring. Any adverse effect from loss of contact between secant piles has not been analysed.

3 Mechanical settlement associated with the construction of the tunnel connecting the shaft is considered to have a negligible contribution based on its size and depth. A detailed assessment has not been undertaken.

6.3.5 Results

Figure 6.4 below presents the settlement predicted on the ground surface due to the shaft excavation.

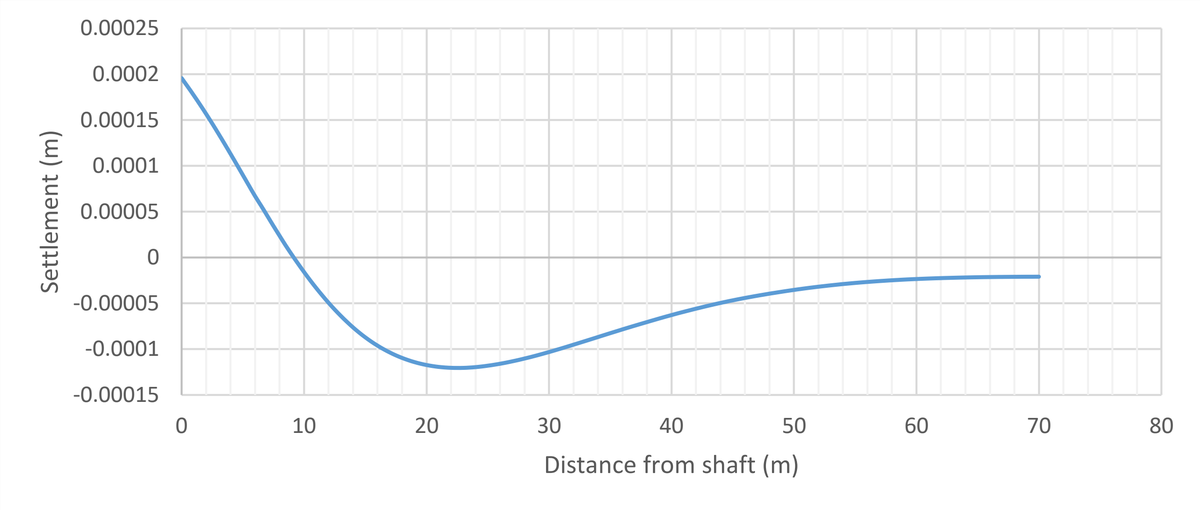


Figure 6.4: Ground surface settlement prediction

The results indicate that mechanical induced settlement from the construction of Shaft 16 is less than 1 mm.

6.4 Geotechnical effects

6.4.1 Overview and objective

Ground settlement associated with the project construction methodologies are expected to be derived from two sources, these being:

- Mechanical settlement – settlement due to the physical movement of the ground, as a result of lateral movement at the boundary of excavations. This type of settlement typically occurs within a short period of time and can be controlled by good construction practices and engineering solutions.
- Consolidation settlement – settlement due to an increase in effective stress associated with the lowering of groundwater levels. This settlement is dependent on the rate and extent of groundwater level lowering and the susceptibility of the soils to consolidation. This type of settlement typically occurs more slowly than mechanical settlement and can be controlled by specific pipeline and structure design, and by good construction methodologies, practices, and reduced open excavation durations.

The total settlement is the combination of these two sources. Where as the zone of influence is limited by the greater distance of the two sources.

The construction of the project has the potential to induce vertical and lateral ground movements that can affect the condition of structures within the zone of influence. For purpose of this assessment, we have considered the zone of influence as the area where total ground settlements associated with the projects may be equal to or greater than 10 mm or the theoretical threshold where more than negligible to very slight damage may occur. Settlement guidelines are presented below for the potentially affected structures.

6.4.2 Settlement tolerance

6.4.2.1 Buildings

The proposed works are expected to be constructed near existing structures, principally private dwellings. In general, a building structure's tolerance to total and differential settlement depends upon the materials used in construction as well as type of foundation system adopted (shallow versus piled), the quality of the structure, and the existing condition of the structure.

The limiting values of total settlement and angular distortion along with damage classification as presented in CIRIA PR30 1996 have been used as guidance for assessment of potential effects on residential buildings. In addition, the New Zealand Building Code states that designers should limit the probable maximum differential settlement of a building to 1V:240H (25 mm over a 6 m horizontal distance) under serviceability limit state loading, or total settlement to 50 mm unless the structure is specifically designed to resist damage under a greater settlement. While the NZ Building Code applies to these structures, the residential house dwellings are likely to have been designed using acceptable solutions outlined in NZS 3604 which is appropriate for ground with movements less than 25 mm. The ground deformation limits outlined in NZS 3604 are approximately equivalent to Risk Category 2 in CIRIA PR30. There are no residential buildings within the zone where surface ground settlements are assessed to be 10 mm or more.

It is important to note that the values presented in in the New Zealand Building Code are total amounts of movement over the life of a building. The buildings in the vicinity of the project are existing, with unknown histories; and therefore, may have already been subjected to some movement. To account for historical movement, buildings that are subjected to less than 10 mm vertical settlement and differential settlement slopes no greater than 1:1000, have been assessed to have a negligible risk to damage and would not need to be further assessed.

For non-residential structures, these require specific detailed review and consultation with the asset owner. A review of aerial photographs and Google Street View identifies 4 properties at potential risk of experiencing adverse effects due to their proximity to the proposed works. These are:

- 439 Hobsonville Road and comprises an electrical sub-station with battery storage. Based aerial imagery and Google Street View, the structure appears to be a multiple level concrete structure likely comprising pre-cast tilt panels. As this property is likely to store equipment sensitive to differential settlement, specific settlement tolerances need to be assessed in consultation with the asset owner.
- 2 Buckley Avenue comprises the Watercare temporary pump station. This property is owned by Watercare (the Consent applicant).
- 1 Memorial Park Lane comprises the Hobsonville Bowling Club. The bowling pitch is likely to be sensitive to differential settlement, and specific settlement tolerances need to be assessed in consultation with the asset owner.
- Hobsonville Pump Station owned by WSL. The pump station is immediately next to the proposed MH2 and trenching excavations. As WSL are both the asset owner and the one lodging for consent, they will need to assess specific settlement tolerances to their assets and work with the contractor to assess a suitable controls to limit deflections to within limits they assess to be reasonable.

For the purposes of this assessment, the settlement criteria and risk classification terminology in Table 6.6 (which is generally based on the Burland (1995), and Mair et al (1996) classification) has been adopted for our assessment:

Table 6.6: Settlement criteria for properties and buildings along the proposed project alignment

Risk Category	Maximum settlement of building (mm)	Maximum differential settlement	Description of risk	General Category
0	-	-	Negligible: superficial damage unlikely	Aesthetic Damage
1	<10	< 1 in 500	Very Slight: Fine cracks easily treated during normal redecoration. Perhaps isolated slight fracture in building. Cracks in exterior visible upon close inspection. Typical crack widths up to 1 mm.	
2	10 to 50	1 in 500 to 1 in 200	Slight: Cracks easily filled. Redecoration probably required. Several slight fractures inside building. Exterior cracks visible, some repainting may be required for weather-tightness. Doors and windows may stick slightly. Typical crack widths up to 5 mm.	
3	50-75	1 in 200 to 1 in 50	Moderate: Cracks may require cutting out and patching. Recurrent cracks can be masked by suitable linings. Brick pointing and possible replacement of a small amount of exterior brickwork may be required. Doors and windows sticking. Utility services may be interrupted. Weather tightness often impaired. Typical crack widths are 5 to 15 mm or several greater than 3 mm	Serviceability Damage
4	> 75	1 in 200 to 1 in 50	Severe: Extensive repair involving removal and replacement of walls especially over door and windows required. Window and door frames distorted. Floor slopes noticeably. Walls lean or bulge noticeably. Some loss of bearing in beams. Utility services disrupted. Typical crack widths are 15 to 25 mm but also depend on the number of cracks.	
5	> 75	> 1 in 50	Major repair required involving partial or complete reconstruction. Beams lose bearing walls lean badly and required shoring. Windows broken by distortion. Danger of instability. Typical crack widths are greater than 25 mm but depend on the number of cracks	Structural Damage

6.4.2.2 Underground services

Published literature[15] indicates that the maximum differential settlement for cast iron pipes and brittle services with a diameter of 200 mm or greater is in the order of 1:250. Most of the major services near the proposed works appear to be more flexible materials such as concrete pipe, polyethylene pipes, electrical cables etc. Services running perpendicular to the excavation works are considered to be at the highest risk of damage. In general, where a service is parallel to the excavation works, it may experience horizontal displacement associated with ground loss at the

[15] O'Rourke, T D, and C H Trautmann. 1982. Buried pipeline response to tunnel ground movements. In Europipe 82 Conf., Basel, Switzerland, paper 1.

excavation face, as well as similar total settlement but with a gentler settlement slope, i.e. differential settlement.

Settlement tolerance of a service will be dependent on the condition of the current asset and its tolerance to deformation. An allowable differential ground settlement of 1:500 has been conservatively adopted for the services along the proposed excavation works to assess the potential for adverse effects. For any services passing within a zone of potential settlement, the service will need to be checked during detailed design for its tolerance to the predicted settlement magnitude and shape, with specific mitigation measures developed in the instance where tolerances may be approached or exceeded. Each service may differ in its acceptable movement tolerance (to be defined by the respective asset owner), so each shall be assessed individually during the detailed design stage.

A review of publicly available information retrieved from BeforeUdig and Auckland Council Geomaps indicates a series of public services within the zone of influence at shafts 12, 16 and 17. These are shown in Appendix E and summarised in Table 6.7.

Table 6.7: Summary of underground services within the 10 mm settlement contour line of shafts

Shaft ID	Services
Shaft 12	• Wastewater pipe local – Watercare Services Limited, 315 mm PVC-U • Two wastewater manholes – Watercare Services Limited, Unknown diameter and material • Wastewater pipe local – Watercare Services Limited, 150 mm PVC-U
Shaft 16	• Water pipe local – Watercare Services Limited, 180 mm polyethylene • Wastewater pipe local – Watercare Services Limited, 225 mm, PVC-U • Wastewater manhole – Watercare Services Limited, unknown diameter and material • Stormwater Pipe – Auckland Council - Stormwater, 900 mm concrete • Stormwater Manhole – Auckland Council – Stormwater, 2050 mm concrete • Stormwater Manhole – Auckland Council – Stormwater, 1800 mm concrete • Stormwater Pipe – Auckland Council – Stormwater, 750 mm concrete • Stormwater Pipe – Auckland Council - Stormwater, 1200 mm concrete • Stormwater Pipe – Auckland Council – Parks, 225 mm concrete • Stormwater Catchpit – Auckland Council – Parks • Unknown Ex GIS Servies
Shaft 17	• Wastewater pipe local (rising) – Watercare Services Limited, 140 mm polyethylene • Wastewater pipe local (rising) – Watercare Services Limited, 125 mm PVC-U • Wastewater pipe local – Watercare Services Limited, 400 mm Glass reinforced pipe (expected to be relocated)

Shaft ID	Services
	• Six Wastewater manhole – Watercare Services Limited, unknown diameter and material (One expected to be relocated) • Wastewater pipe local – Watercare services Limited, 180 mm polyethylene • Water pipe local – Watercare Services Limited, 150 mm polyethylene • Stormwater Pipe – Auckland Council – Stormwater, 525 mm concrete • Ex GIS Stormwater pipes – Auckland Council • Ex Stormwater manhole – Auckland Council • Ex GIS Vodafone – Vodafone (expected to be relocated) • Unknown Ex GIS Services • Ex Wastewater pipe • United Power Supply HV-EX line
MH1	• Wastewater pipe Local – Watercare Services Limited, 180 mm polyethylene • Wastewater pipe Local – Watercare Services Limited, 400 mm GRP
MH2	• Wastewater pipe Transmission Network (rising) – Watercare Services Limited, 400 mm polyethylene • Wastewater pipe Transmission Network – Watercare Services Limited, 500 mm polyethylene • Wastewater pipe Transmission Network – Watercare Services Limited, 150 mm polyethylene • Wastewater pipe Local – Watercare Services Limited, 180 mm polyethylene • Wastewater pipe Local – Watercare Services Limited, 400 mm GRP

6.4.2.3 Roadways and highways

Guidelines in relation to acceptable settlements in and around Waka Kotahi assets is not made widely available and is likely to differ from asset to asset.

In general, the most likely impact from surface settlement may only be limited to changes to the surface water flow regime, resulting from the minor changes in drainage grades. These should be checked as the design is progressed with Waka Kotahi to ensure that the overall surface water flow regime is not negatively affected.

6.5 Groundwater effects

6.5.1 Overview

These potential groundwater effects have been assessed:

- Effect of diversion – groundwater flow is diverted into and/or around structures which are installed in the ground such as temporary sheet piling and final permanent works.
- Effect on neighbouring bores – upstream groundwater levels may be increased by the damming effect of installed structures and reduced by the dewatering required to keep the excavation dry, which affects the ability of bores to provide a water supply.

6.5.2 Groundwater diversion

The effect of diversion is to alter the path of existing groundwater flow. This assessment method considers whether the groundwater flow is permanently diverted by the installed structures and causes increases or decreases in groundwater levels such that these affect lawful groundwater users.

6.5.3 Neighbouring bores

Neighbouring bores may be affected by groundwater level changes, occurring during dewatering activities. These effects are significant if the groundwater supply can no longer be obtained from these neighbouring bores.

6.5.4 Stream flow and wetland groundwater depletion

Dewatering activities may affect groundwater flowing into nearby streams and wetlands resulting in depletion. These effects can have an impact ecologically. Our assessment has looked at the following streams and wetland areas which have been identified by the project Ecologist to exist near the proposed works. In our assessment, we have assumed that there is a hydrological connection between shafts and respective wetlands at the BPC, Shaft 8 and Shaft 12.

<u>32 Mamari Road near the Break Pressure Tank– Palustrine Marsh Wetland</u>

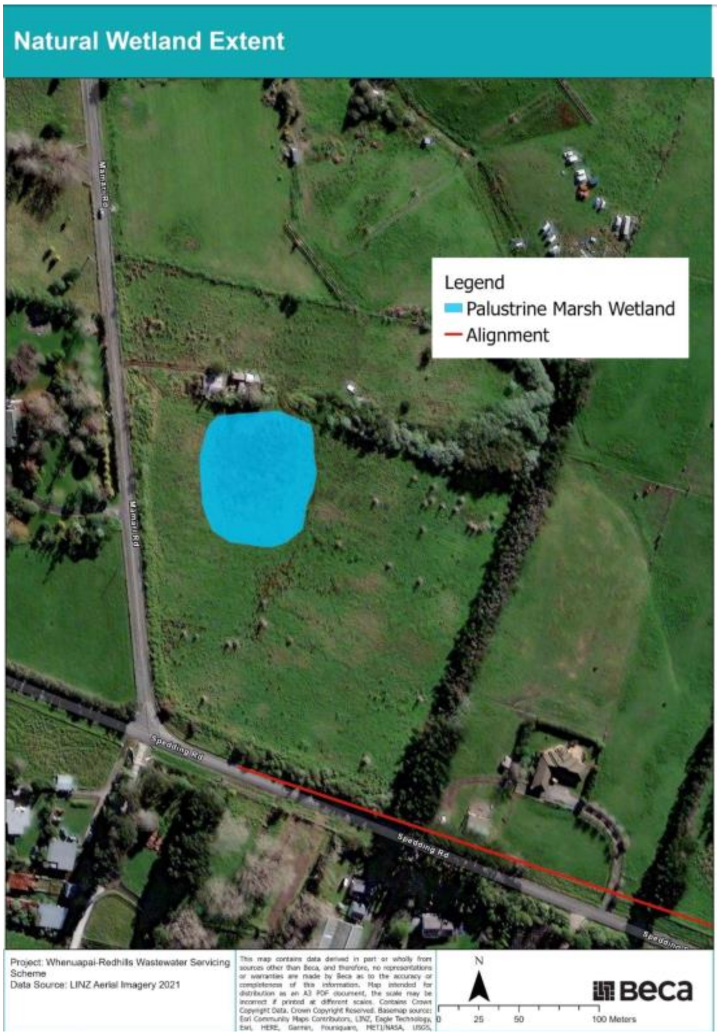


Figure 6.5: Wetland extent as identified by project Ecologist at 32 Mamari Road. Figure supplied by Beca 13 December 2022

<u>27 Trig Road near shaft 8- Riverine Wetland and Waiarohia Stream</u>

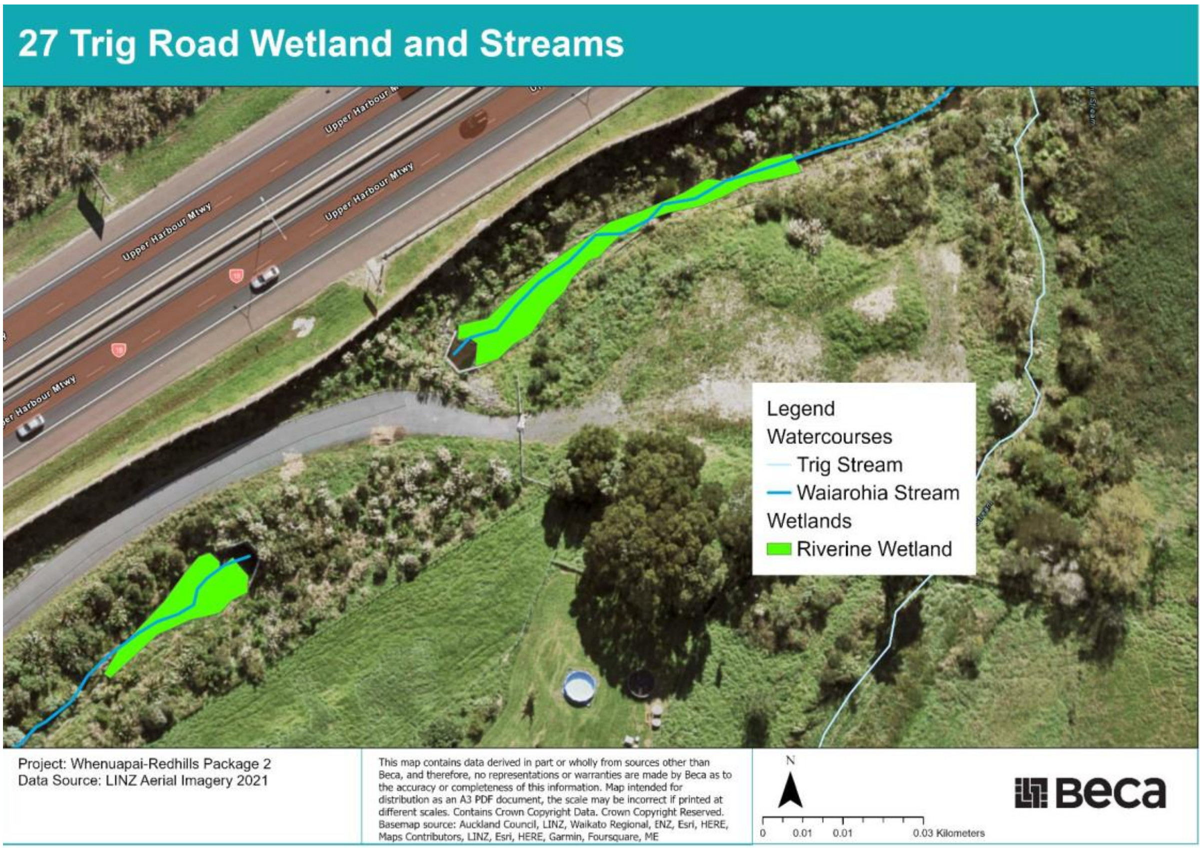


Figure 6.6: Stream and wetland extent as identified by project Ecologist at 27 Trig Road. Figure supplied by Beca 13 December 2022

SH18 Onramp from Bringham Creek Road near Shaft 12 – Wairohia Stream and Riverine Wetland

Figure 6.7: Stream and wetland extent as identified by project Ecologist at the SH18 on ramp from Brigham Creek Road. Figure supplied by Beca 13 December 2022

7 Effects assessment

This section presents the general range of effects that might arise from the project's construction activities, provides guidance on acceptable settlement amounts, and summarises the specific effects that are estimated for the areas considered to be of particular interest or critical to the project.

7.1 Geotechnical settlement effects

This section presents the general range of geotechnical effects that we have estimated that could occur from the project's construction activities and summarises the specific effects that are estimated for the areas considered to be of particular interest or critical to the project.

The estimated total ground settlements (where assessed) are summarised in the following sections and presented as a contour plan in Appendix E.

7.1.1 Assessment of effects on buildings

Total and differential settlement have been estimated for the at risk structures in Table 7.1. In the table, we also present our assessment for potential damage induced by ground settlement associated with the proposed excavation works of these structures.

Table 7.1: Summary of total settlements from excavation works at assessed structures

Location	Approximate Horizontal distance from edge of nearest excavation to structure[1]: (m)	Estimated Horizontal distance from edge of excavation to 10 mm or less settlement contour (m)	Estimated total maximum surface ground settlement (mm)	Estimated approximate maximum differential surface ground settlement across building	Damage category and description to existing structures[2]
439 Hobsonville Road (electrical substation)	19	70 – high shaft conductance 26 – low shaft conductance	25 – high shaft conductance 11 – low shaft conductance	Less than 1V:2500H - high shaft conductance Less than 1V:2000H - low shaft conductance	To be determined through asset owner engagement
1 Memorial Park Lane (bowling club)	~3.5m from centreline of pipe to edge of bowling pitch	N/A structure is sensitive to settlements less than 5 mm	~5 mm	Less than 1V:2000H	0 - 1 – "negligible to very slight"

1. Horizontal distances are approximate and calculated using aerial imagery. Horizontal distances should be confirmed during detailed design.

2. Damage category and description as described in Table 6.6.

439 Hobsonville Road – Electrical Sub-station

Surface ground settlement at 439 Hobsonville Road is associated with the groundwater drawdown effects associated with construction of shaft 17. As presented in sections above, surface ground settlement can be reduced below the values assessed in this report by robust construction that minimises leakage of the secant pile wall. Our assessment indicates that if leakage is kept to a minimum (i.e.. just wetting of the walls), surface settlements could reduce to 11 mm and less at the sub-station.

As specific settlement tolerance criteria is to be agreed with the asset owner of 439 Hobsonville Road, the specific type of damage that may occur cannot be qualified.

1 Memorial Park Lane – Bowling Club

Any ground deformations relating to the Bowling Club are likely to be induced by the tunnelling works. The potential differential settlement across the bowling pitch is assessed to be limited to 1V:2000H or 0.05% grade which is likely within the tolerance of what can be practically graded and maintained for a bowling pitch.

We recommend that pre and post building condition surveys are undertaken for all the structures listed in Section 7.1.1.

7.1.2 Assessment of effects on underground services

Maximum total and differential settlements are assessed to be 35 mm and 1V:750H respectively. At this level of total and differential settlement, we assess that the risk of damage to underground services is very slight. Table 7.2 summarises the settlement, differential settlement and qualitative risk of damage to the nearest underground service to the respective shaft.

Table 7.2: Summary of underground utilities qualitative risk assessment of the nearest service to the excavation

Shaft	**Horizontal distance to edge of closest services (m):**	**Estimated Maximum assessed surface settlement across services (mm)**	**Estimated Maximum assessed differential settlement across service (mm)**	**Qualitative Risk Assessment of Damage**
Shaft 12 • Wastewater pipe local – Watercare Services Limited, 315 mm PVC-U • Two wastewater manholes – Watercare Services Limited, Unknown diameter and material • Wastewater pipe local – Watercare Services Limited, 150 mm PVC-U	8 (315 mm PVC-U Wastewater Pipe)	Less than 5	Less than 1V:2000H	low
Shaft 16 • Water pipe local – Watercare Services Limited, 180 mm polyethylene • Wastewater pipe local – Watercare Services Limited, 225mm, PVC-U • Wastewater manhole – Watercare Services Limited,	8 (900 mm Concrete Stormwater)	Less than 15	Less than 1V:1500H	low

Shaft	Horizontal distance to edge of closest services (m):	Estimated Maximum assessed surface settlement across services (mm)	Estimated Maximum assessed differential settlement across service (mm)	Qualitative Risk Assessment of Damage
unknown diameter and material • Stormwater Pipe – Auckland Council - Stormwater, 900 mm concrete • Stormwater Manhole – Auckland Council – Stormwater, 2050 mm concrete • Stormwater Manhole – Auckland Council – Stormwater, 1800 mm concrete • Stormwater Pipe – Auckland Council – Stormwater, 750 mm concrete • Stormwater Pipe – Auckland Council - Stormwater, 1200 mm concrete • Stormwater Pipe – Auckland Council – Parks, 225 mm concrete • Stormwater Catchpit – Auckland Council – Parks • Unknown Ex GIS Services				
Shaft 17 • Wastewater pipe local (rising) – Watercare Services Limited, 140 mm polyethylene • Wastewater pipe local (rising) – Watercare Services Limited, 125 mm PVC-U • Wastewater pipe local – Watercare Services Limited, 400 mm Glass reinforced pipe (likely to be relocated) • Six Wastewater manhole – Watercare Services Limited, unknown diameter and material	10 (400 GRP Wastewater Pipe)	Less than 35	Less than 1V:750H	low

Shaft	Horizontal distance to edge of closest services (m):	Estimated Maximum assessed surface settlement across services (mm)	Estimated Maximum assessed differential settlement across service (mm)	Qualitative Risk Assessment of Damage
• Wastewater pipe local – Watercare services Limited, 180 mm polyethylene • Water pipe local – Watercare Services Limited, 150 mm polyethylene • Stormwater Pipe – Auckland Council – Stormwater, 525 mm concrete • Ex GIS Stormwater pipes – Auckland Council • Ex Stormwater manhole – Auckland Council • Ex GIS Vodafone – Vodafone • Unknown Ex GIS bdys • Ex Wastewater pipe • United Power Supply HV-EX line				
MH1 • Wastewater pipe Local – Watercare Services Limited, 180 mm polyethylene • Wastewater pipe Local – Watercare Services Limited, 400 mm GRP	Less than 5 m (400 GRP Wastewater Pipe)	Less than 20	Less than 1V:750H	low
MH2 • Wastewater pipe Transmission Network (rising) – Watercare Services Limited, 400 mm polyethylene • Wastewater pipe Transmission Network – Watercare Services Limited, 500 mm polyethylene • Wastewater pipe Transmission Network – Watercare Services Limited, 150 mm polyethylene • Wastewater pipe Local – Watercare Services Limited, 180 mm polyethylene	Less than 1.5 m	Less than 15 adopting lateral bracing support	Less than 1V:750H adopting lateral bracing support	low

Shaft	Horizontal distance to edge of closest services (m):	Estimated Maximum assessed surface settlement across services (mm)	Estimated Maximum assessed differential settlement across service (mm)	Qualitative Risk Assessment of Damage
• Wastewater pipe Local – Watercare Services Limited, 400 mm GRP				

7.1.3 Assessment of effects on roadways and highways

Surface ground settlement up to 18 mm may arise from dewatering at shaft 17 if the shaft is leaky. However, adopting robust construction methods to minimise leakage to just wetting, surface ground settlement can be reduced to approximately 8 mm and less reducing any settlement impact on SH18. It is recommended that WSL consult with Waka Kotahi to understand the level of tolerable settlement for SH18. Figure 7.1 presents the predicted surface ground settlement arising for dewatering at shaft 17.

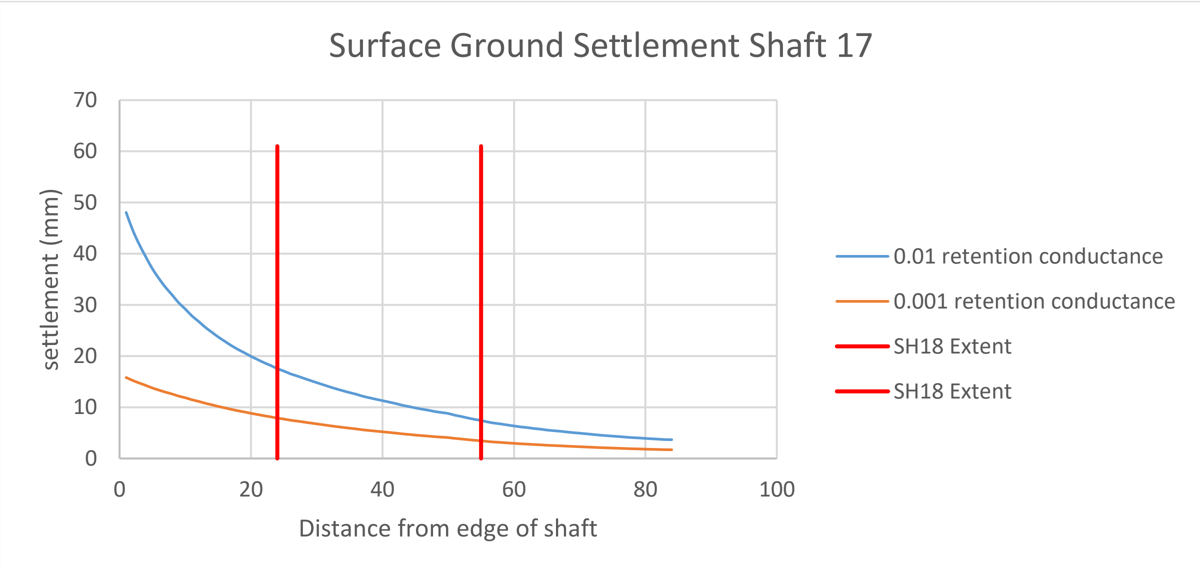


Figure 7.1: Surface ground settlement across SH18 at shaft 17

7.2 Groundwater effects

7.2.1 Groundwater diversion

The proposed shafts will be constructed within the groundwater table. Groundwater that is not temporarily intercepted by the dewatering associated with shaft construction take will be diverted around the proposed secant piled structures. However, the diverted groundwater is still expected to continue in the same general direction and flow rate. On this basis, we do not expect adverse effects relating to groundwater diversion.

7.2.2 Neighbouring bores

The closest recorded bore to the proposed excavation is bore ID WAT80324733-A and WAT60274220 located approximately 250 m to the south of BPT shaft. The bore is located well outside the zone of influence and as such is assessed to be not be impacted by the proposed works.

7.2.3 Stream flow and wetland groundwater depletion

32 Mamari Road near the Break Pressure Tank– Palustrine Marsh Wetland

Dewatering at the Break Pressure Tank is assessed to not significantly impact upon groundwater hydrological conditions in the palustrine marsh wetland at 32 Mamari Road. The marsh is located within a topographical depression which allows surface water flows to collect in the area of the marsh. Ground conditions comprising soils with varying proportions of clay result in limited vertical drainage which enables the ground to be near or at saturation especially during wetter periods of the year. The proposed excavations for the Break Pressure Tank will not alter the surface water flow to the marsh.

The proposed excavations for the Break Pressure Tank adopt a secant pile construction methodology, which provides seepage control. The shaft can be further patched to further reduce leakage during construction to effectively make the excavation watertight if groundwater depletion to the marsh were to be identified during construction.

On this basis, we consider that there is unlikely to be a change in hydrological function and negligible, if any effect to surface groundwater level (top 2-3 meters) is expected to occur at adjacent marsh as a result of dewatering activities.

27 Trig Road near shaft 8- Riverine Wetland and Waiarohia Stream

Dewatering at shaft 8 is assessed to not significantly impact upon groundwater flows into the Waiarohia Stream and Riverine Wetland. These water features are fed by water originating from further up the catchment. The surrounding ground is this area comprises soils with varying proportions of clay resulting in limited, if any, effect to surface groundwater flows. The proposed excavations for shaft 8 do not propose to divert water from either the Waiarohia Stream or associated riverine wetland.

On this basis, we consider that there is unlikely to be a change in hydrological function and negligible, if any effect to surface groundwater level (top 2-3 meters) is expected to occur at adjacent wetland as a result of dewatering activities.

SH18 Onramp from Bringham Creek Road near Shaft 12 – Wairohia Stream and Riverine Wetland

Dewatering at shaft 12 is assessed to not significantly impact upon groundwater flows into the Wairohia Stream and Riverine Wetland. These water features are fed by water originating from further up the catchment with shaft 12 located downstream. The surrounding ground is this area comprises soils with varying proportions of clay resulting in limited, if any effect, to surface groundwater flows. The proposed excavations for shaft 12 do not propose to divert water from either the Wairohia Stream or associated riverine wetland.

On this basis, we consider that there is unlikely to be a change in hydrological function and negligible, if any effect to surface groundwater level (top 2-3 meters) is expected to occur at adjacent wetland as a result of dewatering activities.

7.3 Long-term effects

Ongoing long-term groundwater drawdown is considered to be unlikely as the completed pipeline, and shaft structures should be a closed or watertight system in future. Groundwater levels are expected to return back to similar levels as prior to construction once excavations are sealed or backfilled. Settlements associated with construction are likely to occur in a relatively short period of time and subside once the excavations are complete and backfilled. Nevertheless, settlements that occur during construction are considered to be non-recoverable and therefore permanent, even when groundwater levels recover.

Backfilling of excavations should not result in long term changes in groundwater flow provided that the backfilling of excavations is carried out in accordance with appropriate geotechnical engineering recommendations, Watercare's Design Principles 07, and the Auckland Council Stormwater Code of Practice.

7.4 Monitoring programme

A groundwater and settlement monitoring programme during construction will be a requirement of Consent. The requirements of the monitoring programme, including Alert and Alarm levels, will be set out in a Groundwater and Surface Monitoring Contingency Plan (GSMCP). The plan includes groundwater drawdown, surface settlement, building settlement monitoring and visual observation monitoring.

Given the generally low levels of expected ground deformation associated with the proposed works, baseline monitoring of ground surface settlement and building settlement points is recommended to be undertaken prior to commencement of construction to establish seasonal variability. We would recommend this is undertaken for at least 12 months prior to commencement of construction unless it is not practical given a construction start date within the next 12 months. In the event that baseline monitoring cannot be undertaken for a minimum of 12 months; monitoring should commence as soon as practically possible.

In addition, the groundwater monitoring bores presented in Table 7.3 should be monitoring for a minimum period of 6 months prior to commencement of construction to establish baseline levels.

Table 7.3: Proposed groundwater monitoring locations

BH ID	Surrounding Land Use	Shaft
WSL-21-BH104	Wetland, SH18	LS8
WSL-21-BH109	SH18 berm	RS10
WSL-21-BH115s	On Ramp, stormwater	LS12
WSL-21-BH115d	On Ramp, stormwater	LS12
WSL-21-BH117	SH18 embankment	
WSL-21-BH118	Paddock	
WSL-21-BH122s	Watercare shaft for NH2WM	RS14
WSL-21-BH122d	Watercare shaft for NH2WM	RS14
WSL-21-BH127s	Residential Properties, SH18	LS16
WSL-21-BH127d	Residential Properties, SH18	LS16
WSL-21-BH128	Residential Properties, SH18	LS16
WSL-21-BH129	Residential Properties	

BH ID	Surrounding Land Use	Shaft
WSL-21-BH130	Residential Properties	
WSL-21-BH132s	Substation	RS17
WSL-21-BH132d	Substation	RS17
WSL-21-BH134	Bush & scrub land	RS17
WSL-21-BH201	Road, Watercare Utilities, Agricultural	
WSL-21-BH206	Paddock	

8 Conclusions

This Groundwater and Settlement Effects Report has been prepared by T+T to support an Assessment of Effects on the Environment (AEE).

The estimated total ground settlements (where assessed) are presented as settlement contours on the project site plan presented in Appendix E.

Based on our technical assessment, we conclude:

Surface ground settlement

- Ground settlement effects arising from construction and operating the proposed shafts on surrounding structures is assessed to generally be negligible to very slight (1). The risk of damage is focused around shaft 16 and shaft 17. Mitigation measures such as adopting robust construction methods to seal shafts from leakage will aid in minimising settlements.
- Ground settlement effects arising from dewatering at shaft 17 and to a lesser extent shaft 16 will result in some settlement across SH18. Adopting construction methods that minimise shaft leakage to just wetting is assessed to result in settlements of 8 mm and less. It is recommended that WSL consult with Waka Kotahi on this matter.
- Surface ground settlement due to mechanical settlement arising for shaft deformation is consider negligible.
- Surface ground settlement due to tunnelling arising from volume loss is expected to be less than 10 mm for the Massey connector through to shaft 17.
- Surface ground settlement at electrical substation located at 439 Hobsonville Road is expected to occur as a result of excavations for Shaft 17. Adopting construction methods that minimise shaft leakage to just wetting is assessed to result in settlements of 11 mm and less at the sub-station, reducing the risk of damage to structures on this property. Consultation with the asset owner should be had to determine acceptable threshold of ground settlement for the structures at the property.
- Surface ground settlement at Bowling club located at 1 Memorial is assessed to be limited to 1V:2000H or 0.05% grade, which is likely to be within the tolerance of what can be practically graded and maintained for a bowling pitch.
- Surface ground settlement at the Hobsonville Pump Station owned by WSL may be as large as 70 mm (60 mm mechanical settlement in additional to 10 mm settlement arising from dewatering) if further engineering controls are not implemented. However, through the use of lateral bracing and active slide rail shields, these settlements can be reduced to less than 15 mm. WSL will need to assess acceptable thresholds for ground settlement for their pump station structure.

Groundwater

- Effects upon groundwater levels are expected to localised to within generally 100 m distance of the proposed excavation and temporary in nature. Upon excavation completion and sealing of the shafts, groundwater levels are expected to return to levels prior to construction.
- There is no impact expected upon neighbouring groundwater bores.
- Groundwater that is not temporarily intercepted by the dewatering associated with shaft construction take will be diverted around the proposed secant piled structures. However, the diverted groundwater is still expected to continue in the same general direction and flow rate.
- No significant groundwater depletion to nearby streams and wetland is expected.

Monitoring

- A groundwater and settlement monitoring programme during construction will be a requirement of Consent. The requirements of the monitoring programme, including Alert and Alarm levels, will be set out in a Groundwater and Surface Monitoring Contingency Plan (GSMCP). The plan includes groundwater drawdown, surface settlement, building settlement monitoring and visual observation monitoring.
- The groundwater and settlement monitoring programme during construction will include monitoring of groundwater drawdown, surface settlement and building settlement monitoring. In addition, groundwater monitoring bores presented in Table 7.3 (referenced in this assessment) should be monitoring for a minimum period of 6 months prior to commencement of construction to establish baseline levels.

9 Applicability

This report has been prepared for the exclusive use of our client Watercare Services Limited, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

Recommendations and opinions in this report are based on data from discrete investigation locations. The nature and continuity of subsoil away from these locations are inferred but it must be appreciated that actual conditions could vary from the assumed model.

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