IN THE MATTER OF	the Fast-track Approvals Act 2024 ("FTAA")
AND	
IN THE MATTER OF	an application for approvals by **NZ Transport Agency Waka Kotahi** to construct, operate and maintain the State Highway 1 Wellington Improvements project

---

STATE HIGHWAY 1 WELLINGTON IMPROVEMENTS PROJECT
TECHNICAL REPORT #14
GROUNDWATER ASSESSMENT

Table of Contents

1. INTRODUCTION

1. My name is Eric van Nieuwkerk. I am a Technical Principal – Hydrogeology at WSP and am involved in the Project as a Specialist in Groundwater.

2. The groundwater assessments I present in this report were completed in close collaboration with Andrew Raj (Principal Hydrogeologist at WSP) and Kevin Ledwith (Senior Hydrogeologist at Tonkin and Taylor). In addition, I have relied on previous investigations documented in the reports listed below in Section 3 at paragraph 34.

Qualifications and experience

3. I have a Master of Science degree in Hydrogeology from the Free University of Amsterdam, The Netherlands, which I obtained in 1998. I have 27 years' experience as a Hydrogeologist working for local government, and research institute and various consultancies in The Netherlands, New Zealand and Australia. I have been involved as technical specialist and project manager in various environmental risk assessments, water resource investigations and civil engineering projects in both urban and rural areas. My experience most relevant to this Project is:

   i. **Mackays to PekaPeka (M2PP) expressway project, Kāpiti Coast, New Zealand for NZTA/Waka Kotahi:** This project involved the design and construction of a 13 km-long expressway on the Kāpiti Coast, North of Wellington. The expressway is constructed through soft sedimentary deposits that are prone to land settlement and also support high-valued sensitive wetlands. I developed the design for several water supply wells, undertook aquifer testing and well performance assessments on these wells, and provided guidance to drilling contractors during installation of these wells. I also assessed the effects of groundwater abstraction from these wells as part of resource consent application. I designed and assessed the effects of several construction dewatering projects using 3D-groundwater models in Visual MODFLOW. I undertook groundwater level monitoring and wetland water level monitoring during the construction phase and provided compliance reporting to the Regional Council

   ii. **Sunshine Coast Airport Expansion Project Hydrogeology Assessment, Sunshine Coast, Queensland, Australia for Sunshine Coast Council:** My responsibility in this project was the analysis of hydrogeological information and development of a hydrogeology conceptual site model for the construction of a new runway. I developed a 3D groundwater and solute transport model (Modflow and MT3DMS) and undertook predictive modelling scenarios to assess changes in groundwater levels and groundwater flow paths to inform the design and construction of a new runway and associated infrastructure. I assessed pathways and control options to manage a large PFAS groundwater contamination plume. I assessed the potential risks of saline intrusion to nearby sensitive wetlands. I was also responsible for the project's management, client contact and reporting

   iii. **Te Kaha Stadium in Christchurch for Christchurch City Council (2021-2022):** I was involved in the dewatering design for temporary works as part of the development of the new stadium. I set up a groundwater monitoring network, undertook hydraulic testing, analysis of monitoring data and reporting. I reviewed the hydrogeology and groundwater quality information and developed the hydrogeology conceptual model. I also developed a 3D numerical groundwater model (Modflow) to assess dewatering volumes, rates and drawdowns during temporary works dewatering. I completed an assessment of groundwater effects and consenting. I was responsible for all analysis, work package management, client contact and liaison with other experts

   iv. **The Cambridge to Piarere (C2P) expressway, Waikato for NZTA/Waka Kotahi (2021-2025):** This project involves the development of a new expressway between Cambridge and Piarere, Waikato, which includes large cuts below the groundwater table requiring permanent drainage. I was the hydrogeology technical lead for the consenting phase of the project. I was responsible for analysis of hydrogeological information and development of a hydrogeology conceptual model for the construction of the new C2P expressway. I arranged field investigation programmes for drilling and installing monitoring wells, water quality sampling and testing, and flow gauging of streams and wetlands. I liaised with the client and legal team, as well as various subject matter experts to understand environmental risks. I developed several 2D and 3D-numerical groundwater models (SEEP/W and Modflow-Surfact) to assess risks of reduction of baseflow in streams and wetlands (i.e., stream depletion) and well interference effects in the wider area

v. **Hawkes Bay Expressway (HBEX), Napier, Otaki to North Levin (O2NL) expressway, Levin, and Northland Corridor Road of National Significance (NRONS) expressway, Northland, for NZTA/Waka Kotahi:** I am currently involved in three other ongoing expressway projects as hydrogeologist and hydrogeology technical reviewer, supporting other hydrogeologists involved in those projects with construction dewatering designs, groundwater effects assessments and analysis and interpretation of field investigation data

4. On 29 and 30 October 2025 I visited the Terrace Tunnel, Basin Reserve and Mt Victoria Tunnel and surrounding areas of the Project accompanied by other experts. During the site visit I familiarised myself with the conditions at the sites, including the topography, land use, road layout, buildings and structures and stormwater features. I also confirmed the location of pre-existing monitoring wells near the proposed alignment and suitability of their use for ongoing groundwater level monitoring. I measured depth of well, groundwater level and location coordinates of the encountered wells.

Code of conduct

5. I confirm that I have read the code of conduct for expert witnesses contained in the Environment Court Practice Note 2023. This assessment has been prepared in compliance with that code, as if it were evidence being given in Environment Court proceedings. In particular, unless I state otherwise, this assessment is within my area of expertise and I have not omitted to consider material facts known to me that might alter or detract from the opinions I express.

Project description

6. A full description of the proposal by NZ Transport Agency Waka Kotahi (NZTA) to construct, operate and maintain the State Highway 1 Wellington Improvements project (Project) is provided in Part B of NZTA's substantive application. The upgrade involves road widening, realignments, and the construction of new under and overpasses, and tunnels. At several locations, the proposed new road alignment may intercept groundwater. This report describes the potential effects on groundwater associated with the Project, and outlines requirements to monitor, remedy and mitigate any adverse effects on groundwater and associated effects such as land settlement.

7. Excavations potentially below the groundwater table for tunnelling and trenching are proposed at four sites along the Project alignment, which are at the Terrace Tunnel, the Basin Reserve, the Mt Victoria Tunnel, and Ruahine Street / Wellington Road. These are the four key sites considered in this hydrogeology assessment report.

8. **Terrace Tunnel:** a new tunnel will be constructed adjacent to the east of the existing Terrace Tunnel with the base at a level of ~15.5 m RL (reduced level; New Zealand Vertical Datum 2016) at the southern portal rising to ~28.7 m RL at the northern portal. The tunnel will be constructed by sequential excavation method via a road header. The southern 10 m of the tunnel will be constructed as a cut and cover tunnel. The current tunnel is a drained tunnel which means groundwater ingress is continuously drained and seeps into the existing tunnel’s stormwater system. The new tunnel is also proposed to be a drained tunnel. The tunnel will have a concrete lining with a waterproofing membrane in the arch, so that groundwater will only ingress through a base drainage at the invert level (i.e., the drainage system will be located at its lowest internal point). It is expected that the new tunnel will remain fully submersed with groundwater similar to the existing Terrace Tunnel, despite groundwater ingress being drained by the tunnel’s stormwater system at the base of the tunnel.

9. **Sussex Street Trench at Basin Reserve:** a partially covered trench will be constructed at Sussex Street directly west of the Basin Reserve. This structure will permanently drain groundwater. The new trench will be excavated as a ramp to a maximum depth ~9.0 m (~5.2 m RL) at the intersection with Buckle Street. The trench will be excavated into the shallow groundwater table and may partially

be excavated in an underlying artesian aquifer. The sides of the excavation will be permanently stabilised with approximately 14 m deep piled retaining structures, and the base will be permanently drained from 0.5 m below the new road level. The retaining walls are expected to comprise either:

i. Contiguous piles, which have small gaps between adjacent piles (and are therefore expected to allow some groundwater flow between piles); or

ii. Secant piles, which overlap and provide a less permeable, more watertight barrier

The trench will require temporary dewatering and depressurisation to dry out the construction trench and to reduce groundwater pressures during construction. A 0.5 m undercut will be incorporated at the base of the excavation to enable installation of the drainage system

10. **Mt Victoria Tunnel:** a new tunnel will be constructed adjacent to the existing Mt Victoria Tunnel on the north side with the base at a level of ~33.5 m reduced level (RL) at the eastern portal falling to ~21.5 m RL at the western portal. The tunnel will be constructed by drilling and blasting. The portals will be constructed by cut and cover methods. The current tunnel is a drained tunnel which means groundwater ingress is continuously drained and seeps into the tunnel's stormwater system. The new tunnel is also proposed to be a drained tunnel. The tunnel will have a concrete lining with a waterproofing membrane in the arch, so that groundwater will only ingress through a base drainage at the invert level. It is expected that the new tunnel will remain fully submersed with groundwater similar to the existing Mt Victoria tunnel, despite groundwater ingress being drained by the tunnel's stormwater system at the base of the tunnel

11. **Ruahine Street / Wellington Road (Hataitai):** several construction activities will occur for the Project along Ruahine St and Wellington Rd in Hataitai:

i. A new stormwater pipe is proposed to be installed beneath Ruahine Street and Wellington Road to the east of Mt Victoria. Open trenching construction methods will be considered where the stormwater pipe is to be installed at shallow depth generally above the groundwater table, which requires minimal dewatering during construction. Where the pipe will be installed at considerable depth below the groundwater table, it will be installed mainly through 'trenchless' methods (i.e., horizontal directional drilling) and no excavation and dewatering will be required during construction. However, up to four launch/receiving pits will be installed for the trenchless pipe installation. These pits will have a footprint of about 10 m by 10 m, and will be excavated to a variable depth with the deepest pit excavated up to 20 m deep from the current ground level. The deepest pit is located near 72 Wellington Rd

ii. Several cut slopes for road widening are proposed along Ruahine St and Wellington Rd in Hataitai. Groundwater drainage (e.g., retaining wall weep holes or toe drains) will be installed where these cut slopes intercept the groundwater table

Scope of assessment

12. My assessment addresses the following matters:

i. The methodology I have used to assess the effects of the Project

ii. The existing environment

iii. Effects of the Project during operation

iv. Effects of the Project during construction

v. Recommended measures to avoid, remedy or mitigate adverse effects

Assumptions and exclusions in this assessment

13. All assumptions and exclusions relevant to this hydrogeology assessment report are listed in Section 4 through to Section 7 below.

2. EXECUTIVE SUMMARY

Existing environment

14. The Project is located within Wellington's dense urban environment, where construction activities interact with complex geological and groundwater systems. Four key locations, being The Terrace, Basin Reserve, Mt Victoria, and Ruahine Street/Wellington Road, have been identified as areas where groundwater effects are most likely to occur, and the groundwater assessment is specifically focused on these sites.

15. At **The Terrace**, the Project involves construction of a new tunnel adjacent to the existing Terrace Tunnel, which was excavated in the 1970s through weathered and fractured greywacke bedrock beneath a steep urban hillside. Historical construction records show that groundwater was encountered at shallow depths prior to excavation, with significant drawdown occurring as test drives approached fractured and crushed zones. Groundwater inflows were concentrated along fault-related crush zones and clay seams, with some inflows described as heavy and increasing after rainfall, indicating hydraulic connectivity to near-surface recharge. Tunnel excavation historically caused measurable ground subsidence, including damage to buildings and services, particularly where fault zones and weak rock were present. More recent monitoring confirms that groundwater levels remain well above tunnel level in parts of the alignment, and seepage continues through tunnel joints, evidenced by visible seepage and calcite deposits. The Kumutoto Stream flows from the hillside west of the existing Terrace tunnel to the northern portal of the Terrace tunnel, where it discharges into the stormwater network (i.e., it becomes a 'piped awa').

16. The **Basin Reserve** occupies a low-lying, sediment-filled basin that has a unique geological and hydrogeological history. The subsurface consists of thick sequences of alluvial, lacustrine, swamp, and marine sediments overlying bedrock, producing a complex, layered stratigraphy with strong vertical anisotropy. Groundwater investigations show shallow groundwater levels and artesian conditions may occur, driven by recharge from the upper catchment and restricted vertical drainage due to fine-grained sediment layers confining underlying more permeable gravels. Numerous monitoring wells and historic investigations confirm upward hydraulic gradients across much of the site. Because of these conditions, construction near the Basin Reserve has the potential to cause land settlement due to dewatering and depressurisation during the construction phase or in the operational phase. There are no sensitive environments, such as streams or wetlands with significant ecological values, identified within the likely area of impact of the Project around the Basin Reserve.

17. At **Mt Victoria**, the proposed new tunnel will be constructed adjacent to the existing tunnel to the north, which was excavated in the 1930s through greywacke bedrock beneath a forested hill reserve. Investigations from a 1973 pilot tunnel show that the bedrock is variably weathered and fractured, with groundwater mostly associated with brecciated fault zones and crushed argillite beds. Groundwater levels measured in piezometers indicate low recharge to the deeper bedrock, reflecting the small contributing catchment and significant interception of rainfall by vegetation and shallow weathered rock. The conceptual model suggests that groundwater flow occurs mainly through isolated fracture zones, with significant compartmentalisation and rapid spatial variability in groundwater levels. As a result, while groundwater inflows are expected to be limited overall, localised inflows may still occur where the tunnel intersects fractured or crushed zones.

18. The **Ruahine Street / Wellington Road** area lies on the eastern slopes of Mt Victoria in Hataitai and involves installation of a new stormwater pipe, potentially requiring excavation to depths of up to 20 m. The geology in this area consists of weathered greywacke bedrock underlying residential streets. Groundwater levels recorded in monitoring wells indicate shallow groundwater in some locations,

particularly at lower elevations. Four wetlands have been identified in the Town belt along Ruahine Street, near the intersection with Goa Street.

19. No part of the Project lies within the Hutt Aquifer Protection zone.

20. Historic data on the aquifer hydraulic conductivities (i.e., a measure of permeability), groundwater levels fluctuations, and groundwater quality (including groundwater contamination state) is available from various reports at all four of the key Project sites. Further geotechnical and hydrogeological field investigations have been undertaken for the Project, which complements the available data. In addition, ongoing groundwater level monitoring is underway to confirm baseline conditions. The available data and information collected from previous projects and field investigations data from the current Project have all been used to confirm the assumptions used to assess the potential effects.

Summary of potential adverse effects without mitigation

21. The proposed construction and operation of the Project could have an impact on groundwater which may cause adverse environmental effects and affect nearby buildings, structures and groundwater users. The effects are associated with the drawdown of groundwater levels and a change in groundwater flow directions as a result of the installation of temporary or permanent stormwater conveyance and groundwater drainage systems for the Project. I consider that the potential adverse effects of the Project (without mitigation) could include the following:

   i. Land settlement and potential impacts on nearby structures as a result of groundwater level drawdown from dewatering at the Project, either for Project construction or throughout its operation

   ii. Stream and wetland depletion can occur as a result of temporarily or permanently lowering groundwater levels. Lowering of groundwater levels can also affect the ability to abstract from nearby water supply wells

   iii. Change in groundwater flow direction or velocities that could affect groundwater quality and affected groundwater could subsequently flow into receiving surface water or water supply wells, potentially affecting ecological values or the water's suitability to use for water supply

   For each of these effects specific assessment frameworks have been adopted to support the assessment of effects. The potential ecological effects of this depletion are considered in separate reports.

22. I have assessed the unmitigated potential **land settlement effects during the operational phase** of the Project as follows:

   i. The proposed new Terrace and Mt Victoria tunnels will both be permanently drained tunnels similar to the existing tunnels. No land settlement from groundwater drawdown is expected at either the Terrace or the Mt Victoria tunnels, and the effects are therefore assessed as 'negligible'. Nonetheless, mechanical settlement associated with the construction of the tunnels could cause land settlement and potential damage to buildings and structures near the Terrace or the Mt Victoria tunnels (as discussed further in the geotechnical / resilience assessment by Mr Brabhaharan)

   ii. Based on the information currently available, it would appear that soils prone to land settlement from groundwater drawdown are mostly present in the Basin Reserve area. The proposed Sussex St Trench at Basin Reserve will be constructed as a permanently drained structure which will cause groundwater drawdown, and this could result in land settlement. The potential damage category is assessed as 'slight' (i.e., 'low' impact) leading to minor cracks that could be repaired

   iii. Weathered greywacke at Ruahine Street and Wellington Road could also be susceptible to land settlement from groundwater drawdown, but to a lesser extent. Several cut slopes for road widening are proposed along Ruahine St and Wellington Rd in Hataitai. Where the cuts intercept groundwater, localised lowering of groundwater levels could occur. The associated settlement is expected to be limited to the immediate vicinity of the cut slopes, and therefore unlikely to extend to adjacent buildings. The impact on neighbouring structures from groundwater induced land settlement is assessed as 'negligible'

23. I have assessed the unmitigated potential **effects on nearby sensitive environments and water users during the operational** phase of the Project:

i. The Kumutoto Stream flows directly adjacent to the existing Terrace Tunnel and the proposed new southbound tunnel to the east. Based on the information from monitoring wells along The Terrace the groundwater appears to be deep, and it is unlikely that the Kumutoto Stream receives groundwater inflows in its upper reaches around the Terrace Tunnel. The upper reaches of the Kumutoto Stream will not be affected by the additional groundwater drainage from the proposed new Tunnel. The Kumutoto Stream is a 'piped awa' in its lower reaches and forms part of the city's stormwater system. The lower reaches of the Kumutoto Stream are therefore not considered a natural stream. The impact of the Project on the baseflow of the upper reaches of the Kumutoto Stream is assessed as 'negligible'

ii. There are no sensitive environments, such as streams or wetlands with significant ecological values identified within the likely area of impact of the Project around Basin Reserve

iii. The identified Ruahine St wetlands are at a notable distance away from the proposed new Mt Victoria Tunnel. It is unlikely that the proposed new Mt Victoria Tunnel will affect the Ruahine St wetlands

iv. However, a cut slope will be excavated directly downgradient of the identified Ruahine St Wetland 2 for the upgrade of the access road to the adjacent sport fields. Permanent drainage will be required to keep the cut slope stable and the access road dry. This will affect the baseflow of the wetland, and this potential effect (without mitigation) has been assessed as 'high'

v. Because all listed water supply wells near the Project's key sites are either likely to be abandoned, or at a distance of more than 500 m away, it is unlikely that any water supply well will be affected by the Project

24. I have assessed the unmitigated potential water quality **effects on nearby sensitive environments and water users during the operational phase** of the Project:

i. Contaminated groundwater from nearby contaminated sites could be drawn towards permanent dewatering systems. These systems will typically discharge to the stormwater system and may subsequently be discharged to the sea at the harbour outflows of the stormwater system. Selected Land Use Register (SLUR) sites, which are sites at which historic land uses could have caused soils and groundwater to become contaminated, have been identified near the four key sites. However, contaminated land and groundwater quality investigations for the Project show the risks of groundwater contaminants affecting surface water due to the Project in the operational phase are 'negligible'

ii. The recently constructed Waterview and City Rail Link tunnels in Auckland both experienced groundwater ingress with extremely high alkalinity, leading to calcite precipitation inside the tunnel and the stormwater and drainage water collection systems. If pH and alkalinity are raised in the tunnel drainage water due to this process, then the impact on water quality of the receiving environment (i.e., various piped awa and the sea to which they discharge) could be 'high'

25. I have assessed the unmitigated potential **land settlement effects from groundwater drawdown during the construction phase** of the Project as follows:

i. Because the presence of soils prone to land settlement from dewatering is very limited at Mt Victoria Tunnel and Terrace Tunnel, it is unlikely that any groundwater induced land settlement will occur at those sites during the construction phase

ii. At the Basin Reserve, groundwater drawdown from dewatering or depressurisation for constructing the Sussex Street trench could cause some land settlement which could affect nearby buildings and structures. The potential damage category could be 'slight' (i.e., 'low' impact) leading to minor cracks that could be repaired

iii. Very slight to slight damage (i.e., 'negligible' to 'low' impact) could occur to the nearest properties along Ruahine Street and Wellington Rd during the construction phase as a result of the proposed temporary launch/receiving pit dewatering

iv. Several other structures such as culverts, pipelines, stormwater retention basins and bridge piers will be installed at shallow depths below the groundwater table, and this requires temporary construction pit dewatering to create dry working conditions. Based on the anticipated works required to construct the Project, the impacts of these dewatering activities are expected to be small-scale, localised and would only occur temporarily

26. I have assessed the unmitigated potential **effects on nearby sensitive environments and water users during the construction phase** of the Project as follows:

   i. The impact from the Project on the baseflow of the Kumutoto Stream is the same as in the operational phase (paragraph 97) and is assessed as ‘negligible’

   ii. There are no sensitive environments, such as streams or wetlands with significant ecological values identified within the likely area of impact of the Basin Reserve

   iii. The temporary dewatering during installation of the launch/receiving pits for the new stormwater pipeline along Ruahine Street and Wellington Road, will likely temporarily reduce the baseflows of the identified Wetland 2 at Ruahine Street. The potential impact during the construction phase is assessed as ‘moderate’

   iv. No actively used groundwater supply wells are present near the four key sites of the Project. As such, no impacts from groundwater drawdown caused by the construction activities are expected

27. I have assessed the unmitigated potential **water quality effects on nearby sensitive environments and water users during the construction phase** of the Project as follows:

   i. As stated in paragraph 24, contaminated groundwater from nearby contaminated sites could be drawn towards permanent dewatering systems, which could also occur in the construction phase. Contaminated land and groundwater quality investigations for the Project show the risks of groundwater contaminants affecting surface water due to the Project in the construction phase are 'negligible'

Proposed measures to avoid, remedy or mitigate adverse effects, including conditions

28. The following mitigation measures are recommended to minimise adverse effects on groundwater from the Project during the construction and operational phase:

   i. A concrete lining with a waterproofing membrane in the arch of both the new Terrace Tunnel and Mt Victoria Tunnel to minimise groundwater ingress via the tunnel liner into the tunnel

   ii. Secant pile or contiguous pile walls will be installed at the Sussex Street trench to minimise groundwater inflows during the construction phase

29. The following monitoring is recommended to confirm and implement measures to minimise potential adverse effects during the construction and operational phase:

   i. The preparation and implementation of a **Groundwater and Settlement Monitoring and Contingency Plan (GSMCP)** is recommended to support the monitoring and management of dewatering during construction and for 1 year after the completion of construction. The GSMCP provides a guide to managing potential settlement and deflections of nearby buildings and structures caused by the Project on buildings and services adjacent to the works

   ii. Stormwater quality monitoring as part of the **Stormwater Management Plan (SMP)** is recommended to confirm if water quality of stormwater collected by the drainage systems in the tunnels is suitable for discharge to the environment. Water treatment of the tunnel drainage water as part of a SMP may be required if the water quality does not meet the guideline values

30. Mitigation and ongoing monitoring measures (to be included in relevant management plans, including a **GSMCP** and **SMP**) are proposed to avoid, remedy or mitigate actual or potential adverse effects of the Project. These include monitoring for effects, GSMCP measures such as damage repair, and SMP measures such as water treatment. If these measures are appropriately implemented no residual adverse effects on groundwater are expected of the Project.

31. Table 14 provides a more complete overview of effects and recommended mitigation.

3. ASSESSMENT METHODOLOGY

Introduction

32. The hydrogeology assessment undertaken for the Project, includes a literature study, interrogation of databases, field investigations, analysis and interpretation of data and information, technical discussions with other technical experts, and the development and use of numerical groundwater modelling, to understand the existing environment and assess effects. Further details of the applied methodology and the assessment framework are described in this section.

Literature review and desktop study

33. Data and information relevant to the Project have been obtained from publicly available online databases from several research institutes (Land Care Research, GNS Science, NIWA) and the Greater Wellington Regional Council (GWRC), New Zealand Geotechnical Database (NZGD) and Wellington Water Ltd (WWL). This data and information cover land uses, geology, surface water, underground 3-water services (i.e., stormwater, wastewater and drinking water infrastructure), existing water supply wells, and groundwater abstraction resource consents within the project area.

34. Various relevant technical reports of the project area and general textbooks on hydrogeology, geology and hydrology have been reviewed to support the hydrogeology investigations, as follows:

i. Anderson, M.P., Woessner, W.W., 1992, Applied Groundwater Modelling, Simulation of Flow and Advective Transport, Academic Press, ISBN 0-12-059485-4

ii. BECA, 2015. Terrace Tunnel Duplication – I&R Geotechnical Interpretive Report

iii. Begg, J.G., Johnson, M.R., 2000, Geology of the Wellington area. Institute of Geological & Nuclear Sciences (GNS) 1:250 000 geological map

iv. Beca, 2015, Terrace Tunnel Duplication I & R Geotechnical Factual Report, 18 March 2015, 3321339, NZ1-10053735-10 2.2

v. Booker, D. J., & Whitehead, A. (2020). Retrieved from eFlows Explorer: An interactive tool for exploring how water resource limits can be set: https://shiny.niwa.co.nz/eflowsexplorer/

vi. Burland J B, 1997. Assessment of risk of damage to buildings due to tunnelling and excavation, Earthquake Geotechnical Engineering, Ishihara (ed), Balkema, Rotterdam, pp. 1189-1201

vii. CIRIA, 1996, Prediction and Effects of Ground Movement caused by Tunneling in Soft Ground beneath Urban Areas, Project Report 30, March 1996

viii. CIRIA, 2016, Groundwater control: design and practice, second edition, C750, ISBN: 978-0-86017-755-5

ix. Freeze, R.A., Cherry, J.A., 1979, Groundwater, Prentice-Hall, Inc., ISBN 0-13-365312-9

x. Harrison, E., 2019, Ecosystem health in Wellington City urban streams. Greater Wellington Regional Council, Publication No. GW/ESCI-T-19/123, Wellington

xi. Kruseman, G.P., de Ridder, N.A., 2000, Analysis and Evaluation of Pumping Test Data, Second Edition (Completely Revised), ILRI, Publication 47, ISBN 90 70754 207

xii. Mair, R.J., Taylor, R.N., Burland, J.B., 1996, Prediction of Ground Movements and Assessment of Risk of Building Damage due to Bored Tunneling, ISBN 90 5410 856 8, 1996 Balkema, Rotterdam

xiii. Maxey, G. B., 1964, Hydrostratigraphic Units, Journal of Hydrology 2 (1964) 124-129; © North-Holland Publishing Co., Amsterdam

xiv. NIWA, 2012, Climate Map: Wellington, Median Annual Total Rainfall, based on data from the period 1981-2010

xv. N.D Perrin, 1973. Geological Log of Pilot Tunnel, Mount Victoria Tunnel Duplication. NZ Geological Survey Geology Sections

xvi. N.D Perrin, 1990. Wellington Urban Motorway extension – Engineering Geological Assessment. NZ Geological Survey Engineering Geology Report EGI 90/005

xvii. NZTA and AECOM, 2012, Wellington Tunnel Alliance Victoria to Cobham Drive Scheme Assessment, Geotechnical Investigations Factual Report, V1424 – RE – CV – 6 – 021, 30 April 2012

xviii. NZTA and OPUS, 2010, Basin Reserve Improvements, Preliminary Geotechnical Appraisal Report, January 2010

xix. NZTA and OPUS, 2012, Transportation Improvements Around the Basin Reserve, Contract 380PN, Scheme Assessment Geotechnical Report, March 2012

xx. NZTA and Memorial Park Alliance, 2015, Memorandum, UND-15-ENV-MM Underpass Groundwater Monitoring Summary, 21 August 2015

xxi. Opus (2016). Basin Reserve Cricket Museum Stand – Geotechnical Report

xxii. OPUS, 2013, Basin Bridge Project, Technical report 20: Assessments of Groundwater Effects, 5 -C1617.A3, June 2013

xxiii. Tonkin and Taylor, 2014 Basin Reserve Bridge – Groundwater Factual Report GFR

xxiv. Works Consultancy Services, 1991, Tunnel-Link, Wellington Urban Motorway Extension Terrace Tunnel to Mt Victoria Tunnel, Groundwater Study – Stage 2, C830B.09, May 1991

xxv. Works Central Laboratories, 1990, Wellington Urban Motorway Extension, Terrace Tunnel to Mount Victoria, Additional Site Investigations 1989, Part I – Factual Report, Central Laboratories Report, 90-22226.00, May 1990

35. In addition, I have considered the following technical assessments reports of specialists involved in the project:

i. Geotechnical, Natural Hazards and Resilience Report by Pathmanathan Brabhaharan

ii. Historic Heritage Assessment Report by Carolyn Hill

iii. Stormwater Assessment Report by Tom Bassett

iv. Contaminated Land Assessment Report by Kevin Tearney

v. Freshwater and Wetland Ecology Report by Joshua Markham

Field investigations

36. The following field investigations undertaken for the Project have been used in the hydrogeology assessment:

i. Visits of key sites (i.e., Mt Victoria Tunnel, the Basin Reserve and the Terrace Tunnel) with several hydrogeologists and geologists involved in the Project to familiarise with local conditions, confirm the presence of pre-existing monitoring wells that can be repurposed for the Project for ongoing groundwater level monitoring and identify additional locations for monitoring wells and hydraulic testing of key geological formations

ii. Several pre-existing monitoring wells were encountered and inspected during the site visits by hydrogeologists. A selection of these will be used for ongoing groundwater monitoring. The drilling of new monitoring wells and installation of water level loggers for continuous groundwater level monitoring has been completed in June 2026, with hydraulic testing undertaken during drilling works (i.e., packer testing) and in the completed monitoring wells (i.e., slug testing). Hydraulic testing is used to estimate the permeability (or hydraulic conductivity) of soils and rock formations. The hydraulic conductivity influences groundwater flow rates and fluxes, as well as groundwater level responses to changes, such as the activation of drainage and dewatering systems. The hydraulic conductivity is therefore an important parameter in the assessment of groundwater effects

iii. Figure 1 in Annexure A shows the locations of the selected pre-existing and the proposed new monitoring wells. Further technical details of the monitoring wells are included in Section 4 below, in Table 4 through to Table 10. Drilling and bore maintenance works was completed in June 2026, which included the install of new monitoring wells and redevelop (i.e., flush) of several pre-existing monitoring wells. All new and redeveloped existing monitoring wells are also equipped with automatic water level loggers for continuous monitoring, and hydraulic conductivity tests (i.e., falling and rising head testing) was undertaken on these monitoring wells. For six of the pre-existing monitoring wells and four newly installed wells the automatic water level loggers have already been installed December 2025 and February 2026, an multiple months of groundwater level records are available. These water level loggers continuously record groundwater levels. Groundwater quality sampling and testing will be undertaken in a selection of monitoring wells near The Terrace and Basin Reserve. The available information from the recently completed field investigations in relation to groundwater levels, hydraulic conductivity testing and groundwater quality testing have been used to inform the effects assessments. Ongoing monitoring will provide baseline monitoring data that will be considered in the context of detailed design

Approach to assessing effects

37. In addition to site visits, I have liaised with several of the Project's stormwater experts, geotechnical experts, road and tunnel design technical leads, and with NZTA, in various meetings and phone calls throughout the period June 2025 – June 2026, to discuss our understanding of the groundwater resources, assessment approaches and options for management of effects.

38. Based on the above information, discussions with various experts, and results from various field investigations, I have developed Hydrogeology Conceptual Models of the groundwater system in the vicinity of the four key sites of the Project in collaboration with the Project design team hydrogeologists. The Hydrogeology Conceptual Models form the basis for the development of numerical models, which have been used to inform the design and to quantify the effects of the Project on groundwater resources.

39. The data and information described above and presented in Section 4 underpin the Hydrogeology Conceptual Models, which is a pictorial representation of the groundwater flow system (Anderson and Woessner, 1992) combined with a summary of key aspects that define the groundwater system.

40. To support this assessment of effects on groundwater resources of the Project's proposed activities, numerical groundwater modelling investigations have been undertaken. Cross-sectional numerical groundwater models were developed in SEEP/W. SEEP/W is a finite element numerical modelling software for groundwater flow in porous media, developed by Seequent (2021). SEEP/W can model simple saturated steady-state problems or sophisticated saturated / unsaturated transient analyses. In addition, an AnAqSim analytical element model was developed by the Project design team to inform the design. This model was also used in the assessment of groundwater related environmental effects. A technical description of the numerical groundwater modelling and the modelling results are included in Annexure C.

41. The groundwater modelling was undertaken in accordance with the following document:

    i. Barnett B, Townley LR, Post V, Evans RE, Hunt RJ, Peeters L, Richardson S, Werner AD, Knapton A and Boronkay A., 2012, Australian groundwater modelling guidelines, Waterlines report, National Water Commission, Canberra, ISBN: 978-1-921853-91-3

42. The groundwater modelling is used to inform how groundwater levels and groundwater flows change due to the construction and operation of the Project. These changes can potentially have impacts on environmental values, nearby water users or nearby properties as described in detail in Sections 5 and 6 below. The assessment framework used for the groundwater effects assessments is described in paragraph 43.

Assessment Framework

43. The proposed construction and operation of the Project could have an impact on groundwater which may cause adverse environmental effects and affect nearby buildings, structures and groundwater users. The effects are associated with the drawdown of groundwater levels and a change in groundwater flow directions as a result of the installation of permanent stormwater conveyance and

drainage systems for the state highway. The potential adverse effects that can arise from these processes and their assessment framework used in this report are as follows:

i. Land settlement and potential impacts on nearby structures as a result of groundwater level drawdown from dewatering at the Project, either for Project construction or throughout its operation. Table 1 lists how the potential impacts from land settlement on nearby buildings and structures are classified in this assessment

ii. Stream and wetland depletion can occur as a result of temporarily or permanently lowering groundwater levels. The potential ecological effects of this depletion are considered in separate reports. Table 2 lists how the potential impacts from groundwater level drawdown on baseflows to nearby streams and wetlands are classified in this assessment

iii. Change in groundwater flow direction or velocities that could affect groundwater quality and affected groundwater could subsequently flow into receiving surface water or water supply wells, potentially affecting ecological values or the water's suitability to use for water supply. Ecological water quality guidelines for 95-percentile species protection (ANZG, 2018) can be considered to classify the potential water quality impacts of receiving surface water. The Drinking Water Standards for New Zealand (Water Services Authority/Taumata Arowai, 2022) can be considered to assess water quality effects on water supply wells. For the purpose of this assessment the classifications listed in Table 3 are used to assess water quality impacts

iv. The lowering of groundwater levels can potentially affect a well-owner's ability to abstract water from their water supply wells. Figure 28 (Annexure A) provides a schematic overview of an abstraction well. If a well is pumped, the well water level will be drawn down. Pumping from a nearby well or drainage/dewatering system may draw water levels down in the subject well. Natural groundwater fluctuations may also cause the subject well water level to decline. If the sum of the self-induced drawdown, the drawdown caused by nearby pumping, and seasonal groundwater level decline, causes the well water level to fall below the pump intake (Figure 28 in Annexure A), no water can be taken from the well. The potential impact on the ability to abstract water from an existing well near the Project is used to assess well interference impacts

Table 1: Building damage from land settlement assessment framework (after Burland, 1997; Mair et al, 1996; CIRIA, 1996)

Effects Assessment Terminology	Category of Damage	Description of Degree of Damage	Limiting Tensile Strain (%)	Max Slope of the Ground	Maximum Settlement of the Building (mm)	Description of Typical Damage
Negligible	0	Negligible	<0.05	–	–	Hairline cracks less than about 0.1 mm.
	1	Very slight	0.05–0.075	<1/500	<10	Fine cracks easily treated during normal decoration. Damage generally restricted to internal wall finishes. Close inspection may reveal some cracks in external brickwork or masonry. Typical crack widths up to 1 mm.
Low	2	Slight	0.075–0.15	1/500 – 1/200	10–50	Cracks easily filled. Re-decoration probably required. Recurrent cracks can be masked by suitable linings. Cracks may be visible externally and some repointing may be required to ensure weathertightness. Doors and windows may stick slightly. Typical crack widths up to 5 mm.
Moderate	3	Moderate	0.15–0.3	1/200 – 1/50	50–75	Cracks require opening up and patching by a mason. Repointing of external brickwork and possibly replacement of a small amount of brickwork. Doors and windows sticking. Service pipes may fracture. Weathertightness often impaired. Typical crack widths 5 – 15 mm or several greater than 3 mm.

Effects Assessment Terminology	Category of Damage	Description of Degree of Damage	Limiting Tensile Strain (%)	Max Slope of the Ground	Maximum Settlement of the Building (mm)	Description of Typical Damage
High	4	Severe	>0.3	1/200 – 1/50	>75	Extensive repair work involving breaking out and replacing sections of walls, especially over doors and windows. Frames distorted, floors sloping noticeably, walls leaning or bulging. Some loss of bearing in beams. Service pipes disrupted. Typical crack widths 15–25 mm depending on number of cracks.
Very high	5	Very severe	>0.3	>1/50	>75	Major repair involving partial or complete rebuilding. Beams lose bearing, walls lean badly and require shoring. Windows broken with distortion. Danger of instability. Typical crack widths greater than 25 mm depending on number of cracks.

Table 2: Wetland and stream baseflow reduction effects assessment framework

Magnitude	Baseflow Effect
Very high	Complete depletion of flow likely; more than 80% reduction in flow
High	Estimated loss in flow is between 30% and 80%
Moderate	Estimated loss in flow between about 15% and 30%
Low	Estimated loss in flow between about 5% and 15%
Negligible	Estimated loss in flow is less than 5%

Table 3: Water quality impact assessment framework

Magnitude	Baseflow Effect
High	Ecological water quality guideline values are likely to be exceeded in nearby receiving surface water, or drinking water standards of New Zealand are likely to be exceeded in a nearby water supply well.
Low	Water quality in nearby receiving surface water or a nearby water supply well may be affected, but guideline values or drinking water standards are unlikely to be exceeded.
Negligible	No impact: no water quality change is anticipated in nearby receiving surface water or a nearby water supply well

4. EXISTING ENVIRONMENT

Introduction

44. This section describes the existing environment for the project area relevant to groundwater.

Site description

45. The proposed works for the improvements of SH1 as part of the Project will all occur within the urban built environment of the city of Wellington. Wellington is densely populated and the built environment along the Project alignment is mostly dense, with a mix of older residential areas (mostly timber framed weather-board cottages), concrete commercial/industrial buildings and inner-city roads, streets and alleyways. The hills surrounding Wellington are mostly covered with a mix of native bush and exotic forests and are generally much less urbanised.

46. The Project has four key locations at which the proposed works will likely interact with groundwater and potentially cause effects. This groundwater assessment is therefore specifically focused on those four key locations as described below:

    i. The **Terrace Tunnel** to the northwest of the project area was constructed in the 1970s and has three lanes (two northbound and one southbound). It was cut through the hillside beneath The Terrace. The proposed new tunnel will be constructed adjacent to the existing tunnel to the east

    ii. The **Basin Reserve** is a low-lying area located between the western slopes of Mt Victoria and the northeastern slopes of Mt Cook. It is currently in use as a cricket ground. In the 1800s the area formed the inland end of a large canal used for shipping that linked to the sea to the north. The area rose in the 1855 earthquake, causing the area to drain. The area was subsequently repurposed for use as a council reserve and sports field. The existing Arras Tunnel was constructed in 2014 which cuts through the lower northeastern slope of Mt Cook and links up to Buckle Street to the north of the Basin Reserve

    iii. The new **Mt Victoria Tunnel** on the eastern side of the Project will be constructed next to the existing tunnel, and will cut through Mt Victoria which is a large, forested council reserve with an elevation of up to 133 m RL[1]. The existing Mt Victoria Tunnel was constructed in the 1930s. It is 600 m long and has two lanes for motorised traffic in different directions, and an elevated foot and bike path on the north side. The proposed new tunnel will be constructed adjacent to the north of the existing tunnel

    iv. The **Ruahine Street / Wellington Road Stormwater** Pipe is a new stormwater pipe proposed to be installed at Ruahine Street to the east of Mt Victoria. The pipe will be installed through Ruahine Street and Wellington Road using a combination of techniques including open trenching and 'trenchless methods'. The trenchless methods could involve horizontal directional drilling (HDD) from launch and receive pits. This technique would be used where the pipeline is to be installed at larger depth of up to 20 m. Open trenching and the construction of launch and receiving pits may require temporary dewatering to lower the groundwater level during construction. In addition, the road upgrading along Ruahine Street and Wellington Road involves the partial lowering the road and the creation of several embankment cuts that require permanent groundwater drainage

47. No part of the Project lies within the Hutt Aquifer Protection zone. However, the easternmost parts of the Project are close to the Wellington Harbour Aquifer Zone, which is one of three parts of the Hutt Aquifer Protection Zone. Planning restrictions apply within the Hutt Aquifer Protection Zone, which aim to protect the quality and quantity of groundwater resources used for Wellington's drinking water supply

---

[1] Means metres above reduced level (m RL) with the New Zealand Vertical Datum (2016) as the reference level.

Surface water

48. The city of Wellington has an extensive network of streams, and many of them have been piped within the urban areas (Harrison, 2019). The piped sections of the streams (referred to as 'piped awa') are part of the city's stormwater network and are not considered to be natural streams. Harrison (2019) identifies the Pipitea, Waitangi, Kumutoto, and Waipapa Streams near the Project area (Figure 2 in Annexure A) which are all piped in the urban areas. I acknowledge that the indicated stream locations (Figure 2 in Annexure A) may not be accurate, due to the piped sections of the streams being buried and cannot be confirmed on the surface. The Stormwater Assessment Report from Tom Bassett includes further information on the possible locations of the urban streams and their piped sections based on historic information.

49. Estimated Mean Annual Low Flows (MALF) according to NIWA's NZ River Maps (Booker and Whitehead, 2020) ranges from 1 to 11.5 L/s in these streams as indicated in Figure 2 (Annexure A). Much of these flows are likely to be close to the stream's baseflow and could be from groundwater inflows.

Regional geology and hydrogeology

50. The bedrock that underlies much of Wellington comprises metamorphized Rakaia Terrane Triassic sandstone and mudstone (in some studies referred to as 'greywacke') which forms the topographic highs around the city, such as Mt Victoria, Mt Cook and Kelburn, the hillside west of the Terrace Tunnel. The Rakaia Terrane bedrock is heavily weathered at places, and there are several zones within the bedrock of heavy fracturing along fault lines. More recent Holocene alluvial valley-infill deposits and marine deposits are present in the lower-lying parts of the city. Large parts of the city have been cut and filled throughout the City's history, and an anthropogenic fill covers much of the lower-lying parts of the city.

51. The Wellington Region in which the Project area is located is an area of high seismicity in New Zealand. The region is traversed by several major active fault lines (Figure 3 in Annexure A). The Wellington Fault Line has a southwest-northeast orientation and traverses the Project area and the city to the west and northwest. Several smaller fault lines run through the urban area generally with a south-north orientation.

52. Across the Project area, five **hydrostratigraphic units**[2] (water bearing or confining geological formations) have been identified by the Project's hydrogeologists. These hydrostratigraphic units have been subdivided based on their parent geology, and their hydraulic conductivities, i.e., a measure of permeability. Hydraulic conductivities influence groundwater flow rates and fluxes, and are therefore important for assessment of potential groundwater effects. The following hydrostratigraphic units have been identified at the Project:

    i. **Fill:** this comprises various materials anthropogenic filling materials generally expected to be of low permeability (e.g. fine sands, silts and clay)

    ii. **Holocene swamp deposits:** swamp deposits typically include organic matter and peat interbedded with clay and silts. This is expected to be of low permeability although lateral hydraulic conductivity (i.e., a measure of permeability) can be notably higher than the vertical hydraulic conductivity

    iii. **Fine marine sediments:** this is formed predominantly by silts and sands with occasionally interbedded organic material generally expected to be of low permeability

    iv. **Alluvial sediments:** these typically comprise gravels and sands of moderate to high permeability. Where the matrix of the gravels is silts, the permeability can be notably lower

[2] A hydrostratigraphic unit can be defined as a part of a body of rock that forms a distinct hydrologic unit with respect to the flow of groundwater (Maxey, 1964)

v. **Fractured and weathered bedrock:** where the greywacke bedrock from the Rakaia Terrane formation is heavily fractured, the permeability can be relatively high, and the fracture zone can form an aquifer

vi. **Competent bedrock:** the material that forms the greywacke bedrock from the Rakaia Terrane formation has a very low permeability and can be considered impermeable. Where the bedrock is competent (also referred to as 'fresh') the bedrock has minimal fracturing hence very low permeability

53. Annual rainfall in Wellington City is about 1,250 mm (NIWA, 2012). Rainfall recharge of groundwater resources beneath Wellington is estimated to be 100 mm/year in the lower lying urban areas, and up to 200 mm/year in the forested hills around the city, according to GNS (2019)[3].

Geology and hydrogeology – Terrace Tunnel

54. The following is a summary of site investigation data that provide hydrogeological information for the Terrace tunnel area.

55. NZGS reports for construction of the Terrace tunnel (Perrin, 1973) noted the following observations:

i. Groundwater levels were within a few feet of the ground surface near the Terrace tunnel before test drive excavation was undertaken. Groundwater levels fell by 20 ft and 11 ft (i.e., approximately 6.1 m and 3.0 m respectively) locally, when the test drives approached

ii. Low-lying borehole water levels ranged from ground level to 8-10 ft (i.e., 2.4-3.0 m) below surface, while bores at greater elevation showed water levels varied from 10-20 ft (i.e., 3.0-6.1 m) below ground

iii. Near vertical bedding was identified during the test drive and tunnel excavation. The rock was very weathered to about 60 m at southern end, and not so deep at northern end. Mudstone (argillite) beds, sometimes quite thick when encountered were more brecciated (shattered) than the sandstone beds. Orientation of discontinuities were generally about 20 degrees clockwise from the tunnel alignment so extend for a good length along it. Several crush zones were considered associated with folding, therefore, only continue short distances as opposed to more regional fault associated crush zones

iv. The author noted dripping water from the ceiling. Inflows to the test drives occurred at crush zones and clay seams with groundwater ingress described as "very wet" to as much as comparable with "heavy rain". The flows decreased over time, indicating drawdown of the water table. Test drives noted dripping increased notably within a day of significant rainfall (BECA 2015, from discussion with N Perrin)

v. A crushed argillite zone was encountered in test drive A at set 182 (near the middle of the tunnel length). After the drive had continued a further 20 m this zone changed from dry to wet and collapsed, with approximately 300 gallons per hour (22.7 L/min) inflows of groundwater to the tunnel

vi. The excavation of the tunnel led to up to about 160 mm of subsidence of the ground above the tunnel, with a scarp in the ground surface at Dixon Street (New Zealand Geological Survey, 1978 and Newsome, 1978). The largest subsidence at Dixon Street was possibly associated with the investigation drive (C) that was formed 5 years prior to construction. Subsidence at the Terrace area was possibly associated with collapse in a fault zone, and at MacDonald Crescent was possibly associated with the weaker rock and lower overburden at the southern end of the tunnel. Considerable work was required to repair minor damage to houses and services damaged by the subsidence

56. Recent geotechnical drilling investigations for drill holes TCE01 to TCE05 near the Terrace Tunnel (locations shown in Figure 1 In Annexure A) show the Rakaia Terrane formation bedrock has various degrees of weathering and fracturing, with completely weathered horizons encountered in the top 5 to 10 m, and moderately weathered fractured rock further down. The core log of TCE04 drilled at Macdonald Crescent near the southern portal of the existing Terrace Tunnel is shown in Figure 4 in

[3] https://www.gns.cri.nz/data-and-resources/nationwide-groundwater-recharge-model/

Annexure A forms an example. Slightly to unweathered rock is generally encountered below 25 m depth. This is relevant as the groundwater levels recorded in all the drill holes were all measured below the completely weathered horizons. Completely weathered rock can be susceptible to consolidation from groundwater drawdown, albeit small. Moderately to unweathered rock materials are not susceptible to consolidation from groundwater drawdown. With groundwater levels currently below the completely weathered horizons, it is unlikely that any future groundwater drawdown would cause consolidation of the weathered rock.

57. Beca (2015) installed 3 piezometers in the area of the duplicate tunnel in 2014. Groundwater levels were measured in 2014 and appear to be about 15-20 m above the tunnel elevation near the middle of the tunnel (Table 4; levels recorded in 'metres below ground level', referred to as 'm bgl'). The location of these piezometers is shown in Figure 1 in Annexure A. These findings appear to be consistent with more recent measurements made by the Project's hydrogeologist on 29 October 2025 (also listed in Table 4).

Table 4: Groundwater data near the Terrace Tunnel

Monitoring well	Screening depth (m bgl)	Water level (m bgl) in 2014 (Beca, 2015)	Water level (m RL) in 2014 (Beca, 2015)	Water level (m bgl) on 29 Oct 2025	Water level (m RL) on 29 Oct 2025	Water level (m bgl) during testing between Jan and June 2026	Water level (m RL) during testing between Jan and June 2026	Estimated hydraulic conductivity (m/s)
NE01	16.5-20.0	12.3	39.7	11.5	40.5	12.5	39.5	5.9×10^{-8} to 8.2×10^{-8}
TD01	36.5-40.0	21.3	41.7	20.6	42.4	20.1	43.2	1.0×10^{-7} to 1.6×10^{-7}
TD02	28.5-32.0	14.9	35.1	11.0	39.0	11.7	38.3	(test failed)
TCE01	35.8-37.8	-	-	-	-	6.3	54.6	4.0×10^{-7} to 4.0×10^{-6}
TCE02	23.5-25.5					7.8	45.2	3.5×10^{-6} to 5.8×10^{-6}
TCE03	25.7-27.7					6.8	42.4	1.0×10^{-8} to 1.6×10^{-7}

58. Observations made on 29 October 2025 driving through the Terrace Tunnel suggest groundwater ingress occurs through most of the joints of the tunnel segments. Seepage was observed as well as calcite precipitate forming throughout the entire tunnel (photos included in Annexure B).

59. No hydraulic testing or pumping testing was undertaken for the construction of the existing Terrace Tunnel or any investigations thereafter. Hydraulic testing (i.e., falling and rising head testing) was recently successfully undertaken for the Project for several pre-existing monitoring wells and on the newly installed monitoring well TCE01, TCE02 and TCE03. A packer test (i.e., another form of hydraulic testing) was successfully undertaken in the drill hole for the new monitoring wells TCE01 (at a depth between 18.6 and 19.5 m bgl) and TCE03 (at a depth between 22.6 and 25.5 m bgl). The results are listed in Table 4.

60. Contour maps were derived from the available groundwater level data at the Terrace Tunnel for the Project as shown in Figure 5.

61. The **Hydrogeological Conceptual Model for the Terrace Tunnel** is presented in Figure 6 and Figure 7 and can be described as follows:

   i. Surface recharge in the study area is primarily attributed to two mechanisms: infiltration from unpaved surfaces and infiltration resulting from leaks and losses in stormwater drainage and piping infrastructure. The topsoil layer is notably thin and is underlain by completely weathered greywacke or sandstone at shallow depths (~ 5 m depth). This is underlain by fractured greywacke bedrock

   ii. The transition from highly or completely weathered rock to moderately or unweathered rock is relatively shallow, typically occurring at depths of approximately 3–10 m bgl. The current groundwater table is generally encountered below this depth

iii. The highly to completely weathered material is expected to exhibit higher bulk hydraulic conductivity compared to the less weathered underlying rock. Defects within this material may be self-healed, resulting in a soil-like bulk matrix. Groundwater flow within the moderately to unweathered rock is predominantly controlled by fracture flow through defects associated with tectonic activity. Crush zones are most frequently encountered within the softer argillite beds, and as a result, hydraulic conductivity may display anisotropy, with preferential flow occurring along the orientation of bedding. Large faults and crush zones are anticipated to act as conduits within the lower conductivity bulk rock mass

iv. Groundwater seepage to the existing tunnel during excavation was noted to increase following heavy rainfall, indicating at least some of the fracture sets are connected to near surface levels, though the delay between rainfall and seepage shows the flow path is restricted

Geology and hydrogeology – Basin Reserve

62. The following is a summary of site investigation data that provide hydrogeological information for the Basin Reserve area.

63. A map of "*The plan of the city of Britannia in Lambton Harbour*" from 1840 shows streams near the west and east sides of the Basin Reserve area. Perrin (1990) describes the subsequent change in plans after the 1855 earthquake. Perrin (1990) notes that the Basin Reserve is a topographic low area that was originally planned to be a basin for shipping, with a connection to Lambton Harbour via a canal along the line of Kent and Cambridge Terraces. Uplift associated with the 1855 earthquake (about 2 metres in this area) made this unfeasible. An historical stream channel is known to have passed through the Basin Reserve and along the Kent/Cambridge Terrace lineation, and this has been filled and the ground levelled for development and street formation. The approximate location of the streams is shown in Figure 1 in Annexure A on the Basin Reserve map.

64. Several monitoring wells (D1 to D104) were installed during groundwater investigations in the 1990s by Works Consultancy (1991). Groundwater levels were monitored during March to June 1990. They show an upward hydraulic gradient in most multi-level piezometers, with a downward gradient only in D5 and D6 (Opus, 2012).

65. Additional drilling and installation of piezometers was undertaken more recently (NZTA and AECOM, 2012), which are the BH series (details in Table 5). Groundwater levels were monitored in the new piezometers as well as the existing D series piezometers installed in 1990. The artesian pressure measured at the deep piezometer D2-D had reduced to ground surface, where previously it was 2 m above ground level. A basin profile is also given in this report, based on drilling information and a geophysics survey using geophones.

66. Opus (2016) reports that two piezometers were installed near the Basin Reserve cricket museum stand as part of this study. Details are included in Table 6.

67. Tonkin and Taylor 2014 installed a series of boreholes near the Arras tunnel and carried out a pumping test to contribute to the assessment of the potential effects of earthworks at the Arras tunnel. Opus (2013) installed two boreholes to the east of Basin Reserve at the intersection of Paterson and Dufferin Streets and used to carry out a depressurisation test to assess the effects of depressurising the artesian groundwater system during earthworks. Details of the piezometer depths and groundwater levels are included in Table 7.

68. NZTA and Memorial Park Alliance (2015) investigated the groundwater drawdown and land settlement effects associated with the construction of the Arras Tunnel along Buckle Street, adjacent to the proposed Sussex St Trench. The investigation showed there was no significant settlement of buildings. Therefore, NZTA and Memorial Park Alliance (2015) concluded that the sensitivity of surrounding building foundations to groundwater level change was less than initially expected.

Table 5: Groundwater data directly north of Basin Reserve (after NZTA and AECOM, 2012)

Borehole	Installed	Drilled (m)	Ground RL	PZ 1 (m)	PZ2 (m)	PZ3 (m)
BH601	25/11/12	44.7	7.861	4-6	13.5 (VW)[1)]	22 (VW)
BH603	7/12/11	40	5.49	8-10	18 (VW)	30.5 (VW)
BH604	7/12/11	36	7.41	4-6	15 (VW)	-
BH605	13/12/11	34	9.538	3.5-5.5	12-15	-
[1)] VW: vibrating wire piezometer; this is a pressure transducer grouted into the drill hole at the indicated depth						

Table 6: Groundwater data at Cricket Museum (after OPUS, 2016)

Monitoring well	Screened zone (m bgl)	Groundwater level (m bgl) on 21/04/2016	Groundwater level (m bgl) on 24/04/2016
BH1	5-10.5	3.70 and 3.94	3.65
BH2	8.5-12.5	0.5	0.0

Table 7 Groundwater data East of Basin Reserve (after OPUS, 2013; NZGD)

Borehole	Piezo	Depth (m)
TT5	BHTT5	18.5-21.5
TT6	BHTT6-VWP1	28.8
TT6	BHTT6-VWP2	19
TT6	BHTT6-VWP3	5
WS102	WS102	3

69. A series of D and D100 drilling and piezometer installations were made from the Basin Reserve to near the Terrace Tunnel south portal as reported by Works Central Laboratories (1990). In situ falling head tests were made in some drillholes during drilling and in installed piezometers. Some samples taken during drilling were also laboratory tested. (Note the data from this site investigation is also provided in Perrin, 1991) A summary of the test results is shown in Table 8. The interpretation of hydraulic conductivities from the tests was:

i. Gravels range from 8×10^{-8} to 6×10^{-5} , with an average of 8×10^{-6} m/s

ii. For clay and silt approximately 1×10^{-8} m/s was estimated

iii. Completely weathered bedrock range from 2×10^{-8} to 9×10^{-7} m/s, with an average of 4×10^{-7} m/s

Table 8: Groundwater data at Basin Reserve (after OPUS, 1991)

Borehole Number	From (m)	To (m)	Soil Type	Type of Test	Permeability (m/s)	Remarks
D1	11.5	11.7	Clayey SAND	Laboratory triaxial cell	3.90E-11	
D10	7.2	8.2	Gravelly SAND/GRAVEL	Insitu/Falling Head/Piezometer	2.00E-05	
D10	17.1	17.25	Sandy SILT	Laboratory triaxial cell	1.30E-08	
D101	7.57	9.5	0.45m SILT/0.15m Organic CLAY/0.9m Sand Silty GRAVEL/0.4m SAND	Insitu/Falling Head/Borehole	2.00E-07	
D101	8	9.45	0.1m Organic CLAY/0.9m Sandy Silty GRAVEL/0.4m SAND/0.1m SILT	Insitu/Packer Test/Borehole	1.00E-05	Unreliable
D102	3	4	0.3m CLAY/0.45m SAND	Insitu/Falling Head/Piezometer	2.00E-06	
D102	7.5	9	0.5m PEAT + CLAY/1.0m Sandy Silty GRAVEL	Insitu/Falling Head/Piezometer	7.00E-06	
D102	7.6	9	0.45m PEAT & CLAY/1.0m Sandy Silty GRAVEL	Insitu/Falling Head/Borehole	2.00E-07	

Borehole Number	From (m)	To (m)	Soil Type	Type of Test	Permeability (m/s)	Remarks
D102	15	17	0.3m CLAY/0.45m SAND/1.25m Silty Sandy GRAVEL	Insitu/Falling Head/Piezometer	2.00E-05	
D102	15.15	18.2	0.4m SAND/1.15m CLAY/1.45m Silty Sandy GRAVEL	Insitu/Packer Test/Borehole	1.00E-05	Unreliable
D103	8.1	10	Silty Sandy GRAVEL	Insitu/Falling Head/Borehole	2.00E-07	
D104	4.55	6.2	0.1m CLAY/1.5m Sandy CLAY (bedrock)	Insitu/Falling Head/Borehole	2.00E-07	
D105	3.2	4.26	Sand gravelly SILT	Insitu/Falling Head/Piezometer	7.00E-05	Unreliable
D105	11.5	13	1.1m CLAY/0.4m Sandy SILT & CLAY (bedrock)	Insitu/Falling Head/Piezometer	9.00E-07	
D106	6.02	8	1.0m Sandy gravelly SILT/1.0m CLAY/SILT	Insitu/Falling Head/Borehole	5.00E-08	
D108	7.6		Sandy gravelly SILT	Insitu/Falling Head/Borehole	3.00E-06	
D109	7.5	9	Sandy gravelly SILT	Insitu/Falling Head/Piezometer	5.00E-06	
D109	7.5	9	Sandy gravelly SILT	Insitu/Falling Head/Borehole	2.00E-05	
D11	8	10	Silty SAND & Sandy Silty GRAVEL	Insitu/Falling Head/Piezometer	1.00E-06	
D110	5.5	6.5	Sandy silty GRAVEL	Insitu/Falling Head/Borehole	4.00E-06	
D110	12.5	13.8	CLAY & SILT with gravel layers	Insitu/Falling Head/Borehole	1.00E-07	
D110	15.75	17	Sandy GRAVEL	Insitu/Falling Head/Borehole	8.00E-08	
D4	6.65	6.85	Silty SAND	Laboratory triaxial cell	1.80E-08	
D4	14.3	14.5	Organic SILT	Laboratory triaxial cell	8.90E-13	
D4	19	21	Silty SAND/GRAVEL/Organic Clay layers	Insitu/Falling Head/Piezometer	1.00E-07	
D4	22.25	22.35	Sandy GRAVEL	Laboratory triaxial cell	1.50E-09	
D5	4.5	6.5	Sandy GRAVEL	Insitu/Falling Head/Piezometer	2.00E-06	
D5	12	14	0.5m Sandy GRAVEL/SILT, SAND GRAVEL layers	Insitu/Falling Head/Piezometer	1.00E-05	
D6	4.5	6.5	Sandy silty GRAVEL	Insitu/Falling Head/Piezometer	7.00E-06	
D6	11.5	13.5	Sandy silty & clayey GRAVEL	Insitu/Falling Head/Piezometer	9.00E-06	
D7	5.7	6.7	Sandy GRAVEL, slightly clayey	Insitu/Falling Head/Piezometer	6.00E-05	
D7	17.6	17.8	Weathered bedrock	Laboratory triaxial cell	2.30E-08	
D8	8.9	8.9	Sandy GRAVEL	Laboratory triaxial cell	1.40E-08	
D8	11	13	Sandy silty GRAVEL	Insitu/Falling Head/Piezometer	2.00E-05	
D8	14.5	15.5	Silty Sandy GRAVEL	Insitu/Falling Head/Piezometer	8.00E-07	
D8	22	22.2	Sandy SILT	Laboratory triaxial cell	3.40E-10	

Borehole Number	From (m)	To (m)	Soil Type	Type of Test	Permeability (m/s)	Remarks
D9	8.5	9.5	Sandy GRAVEL	Insitu/Falling Head/Piezometer	5.00E-07	
D9	14.5	15.5	0.5m Sandy Clayey GRAVEL/0.5m gravelly CLAY	Insitu/Falling Head/Piezometer	4.00E-07	

70. Hydraulic conductivity testing was undertaken for the Project on the pre-existing monitoring well BH01 screened in the marine sediments at the Basin Reserve. In addition, four new monitoring wells were installed at the Basin Reserve for the Project, which are BR01 S, BR01 D, BR02 S and BR 02 D, as shown in Figure 1 in Annexure A. The results from the hydraulic testing and estimated hydraulic conductivities are listed in Table 9. An effective modulus (i.e., a measure of material compressibility and susceptibility to land settlement) of 15 MPa for Holocene swamp sediments and 30 MPa for Marine sediments was assumed, based on geotechnical design parameters values used for the Project.

Table 9: Results from hydraulic conductivity testing at Basin Reserve

Piezometer	Screening depth (m bgl)	Water level (m RL) during testing between Jan and March 2026	Estimated hydraulic conductivity (m/s)
BH01	5.0-10.5	3.27	1.6×10^{-7} to 2.5×10^{-7}
BR01-S	8.0-11.0	5.29	9.3×10^{-9} to 4.4×10^{-8}
BR01-D	15.5-18.5	-	(test failed)
BR02-S	7.0-10.0	2.94	9.3×10^{-9} to 2.0×10^{-7}
BR02-D	15.5-18.5	1.95	3.5×10^{-7} to 5.4×10^{-7}
WS102	1.0-3.0	-	(test failed)

71. Contour maps derived from the available groundwater level data at the Basin Reserve for the Project are shown in Figure 8 in Annexure A.

72. The **Hydrogeological Conceptual Model for the Basin Reserve** is presented in Figure 9 to Figure 11 (Annexure A) and can be described as follows:

i. The Basin Reserve is situated at a point where the valley to the south transitions from a relatively narrow corridor to a broader, sediment-filled basin. This geomorphological setting has resulted in significant sediment accumulation within the area. The sediment infill at the Basin Reserve consists of Pleistocene alluvium and lacustrine deposits, which are overlain by more recent Holocene alluvium. This depositional environment has produced complex stratigraphy characterized by lenses and layers of silts, clays, sands, and silty gravels. The resulting stratigraphy exhibits vertical anisotropy, with preferential groundwater flow occurring horizontally where coarser sediments are present and reduced vertical flow due to the presence of finer-grained layers

ii. Groundwater recharge at the Basin Reserve primarily originates from the upper catchment, with contributions from throughflow within the valley sediments and discharge from the surrounding rock into deep sediment layers. The combination of recharge from the upper catchment and restricted vertical flow within the valley sediments has led to the development of artesian conditions in the Basin Reserve area

iii. Groundwater is encountered at shallow depths, as local recharge is unable to migrate to greater depths due to the artesian conditions maintained by the upper catchment. Discharge from the Basin Reserve area is likely facilitated by shallow drainage features, where historic overland streams have been infilled or redirected into subsoil drains

iv. A west-to-east profile of the Basin Reserve shows that sediment thickness is greatest at the Basin Reserve itself, with progressively thinner sediment cover toward the valley sides, where the land rises and the underlying rock is closer to the surface

Geology and hydrogeology – Mt Victoria Tunnel

73. The following is a summary of site investigation data that provide hydrogeological information for the Mt Victoria tunnel area:

i. A pilot tunnel was excavated in 1973, north of the existing tunnel alignment (Perrin, 1973). Perrin (1973) used a series of tunnel excavation logs from the pilot tunnel to develop a local geological map of the tunnel walls encountered during excavation of the pilot tunnel. These included notes on defects in the bedrock and groundwater occurrences. The existing Mt Victoria tunnel was excavated entirely through greywacke bedrock of varied weathering and fracturing conditions. The greywacke comprises fine grained partly recrystalised quartzose sandstone with laminations to beds of argillite (siltstone)

ii. Bedding planes encountered in the pilot tunnel are near vertical to east dipping. Brecciated fault zones, crush zones and clay gouges were encountered during excavation at near vertical orientation. Most but not all of these crushed zones had associated dripping/seepage, while much of the tunnel excavation was noted as dry to damp. Crushed zones mostly occur within the argillite beds and form gouge material. Test pits were also excavated beyond the portals of the tunnel. The only test pit near the tunnel alignment was TP02, near BH01 to the west of the tunnel (Figure 1 in Annexure A). In the test pit, extremely weak to weak sandstone below 0.1 m of topsoil was encountered, indicating no colluvium is present on the lower slopes. The test pit was excavated to 4.5 m depth and was dry to the full depth

iii. Observations made during the 30 October 2025 site visit suggest groundwater ingress occurs through most of the joints of the tunnel segments. Seepage was observed as well as calcite precipitate forming throughout the entire tunnel (photos included in Annexure B)

iv. Three piezometers were installed along the duplicate tunnel alignment, with packer tests made within two of them. Water levels were noted during the drilling of monitoring well BH04 located at the hill crest, most likely the level measured in the morning before drilling commenced. The first reading was at 8 m bgl, but lowered progressively as drilling advanced. Piezometers were re-visited once on 29 March 2012 after installation to measure groundwater levels. The level at the west end of the tunnel was 13 m bgl (in BH01), under the highest part of the hill was 55 m bgl (in BH04) and at the east end of the tunnel was dry to 30 m bgl (in BH05). A summary of available groundwater level data is included in Table 10. The location of the monitoring wells is shown in Figure 1 (Annexure A). It is acknowledged that BH05 was drilled into a pilot tunnel adjacent to the existing Mt Victoria Tunnel and is unlikely to represent the ambient groundwater level

Table 10: Groundwater data in Mt Victoria tunnel area (after Aecom 2012)

Borehole	Depth (m)	Response zone (m bgl)	Drilling reading (m bgl)	29 Mar 2012 reading (m bgl)
BH01	15.0	12.0 to 15.0	4.98	5.49
BH04	100.0	95.0 to 100.0	8.0 / 53.0	55.2
BH05	30.0	25.0 to 30.0	NA	dry
TP02	4.5	-	Dry	

74. Packer tests undertaken by NZTA and AECOM (2012) targeted slightly to moderately weathered bed rock consisting of sandstone and mudstone (i.e., Rakaia Terrane greywacke formation). The results of the packer test suggest a permeability range from 8×10^{-9} m/s to 5.0×10^{-7} m/s. However, the packer test results may not be accurate as only two pressure steps were applied, and detail of the tests were not provided in the report. From the information given for the tests BH03 took almost no flow and BH04 may have had some washout in the 95-100 m bgl span.

75. Further field investigations for the Project were undertaken, which included hydraulic conductivity testing in existing monitoring wells BH03 and BH06 near Mt Victoria Tunnel. BH03 is located near the west tunnel portal and is screened in fractured bedrock, and BH06 near the east tunnel portal and is screened in alluvial gravels. The results from the testing and estimated hydraulic conductivities are listed in Table 11.

Table 11: Results from hydraulic conductivity testing at Mt Victoria Tunnel

Borehole	Screening depth (m bgl)	Screened geology	Water level (m RL) during testing between Jan and March 2026	Estimated hydraulic conductivity (m/s)
BH03	4.0 - 25.0	Fractured bedrock	12.58	1.1×10^{-6} to 1.7×10^{-6}
BH06	7.0 - 10.0	Alluvial gravels	0.81	3.5×10^{-5} to 8.3×10^{-5}

76. Contour maps were derived from the available groundwater level data at Mt Victoria Tunnel for the Project as shown in Annexure A, Figure 12.

77. The **Hydrogeological Conceptual Model for the Mt Victoria** is presented in Figure 13 and Figure 14 in Annexure A and can be described as follows:

    i. The surface environment in the Mt Victoria tunnel area is characterized by tall trees and a thin layer of organic soil, which has likely developed from the existing vegetation. Colluvium is absent in the logged geological profile; instead, thin topsoil or fill is directly underlain by very weathered sandstone, even on the lower slopes beneath the tunnel portal. The groundwater system within the highly weathered surface may be locally perched, and if so, it may be disconnected from the deeper groundwater table in the less weathered but fractured greywacke bedrock encountered at tunnel alignment depth. Surface infiltration predominantly travels through the highly weathered zone and is largely captured by the established tree root system before reaching the less weathered rock below

    ii. The rainfall catchment area upgradient of the tunnel alignment is very limited, resulting in low recharge to the deeper greywacke. Consequently, groundwater levels observed in piezometers screened at the tunnel depth are low

    iii. Groundwater flow through the greywacke rock primarily occurs within crushed zones formed by tectonic activity. These zones are often found within argillite beds or laminations. Gouge material within these zones likely restricts groundwater movement, while gritty, crushed material provides the main fracture flow path

    iv. Groundwater levels within the greywacke may vary significantly over short distances due to compartmentalisation, which is a common feature in fracture flow systems. Preferential groundwater flow is expected along the orientation of bedding and defects, which are mostly near vertical and oriented north-south

Geology and hydrogeology – Ruahine Street/Wellington Road

78. The geology of the eastern slopes of Mt Victoria and the residential area along Ruahine St, Wellington Rd and Moxham Ave is characterised by weathered and fractured greywacke. No hydraulic conductivity data is available from previous investigations. From hydraulic conductivity testing of BH101b on the eastern slopes of Mt Victoria near the tunnel portal (location shown in Figure 1 in Annexure A) a hydraulic conductivity of 2.3×10^{-7} to 2.9×10^{-7} m/s was estimated. Based on hydraulic conductivity values from BH101b and BH03 (Table 11) a hydraulic conductivity of 5×10^{-7} m/s is assumed. An effective modulus (i.e., a measure of material compressibility and susceptibility to land settlement) of 50 MPa was assumed, based on geotechnical design parameters values used for the Project. Groundwater levels were recorded during installation of several monitoring wells along Ruahine St as listed in Table 12 (locations shown in Figure 1 in Annexure A).

Table 12: Groundwater data along Ruahine St (after Aecom 2012)

Borehole	GL (m RL)	Depth (m)	GWL (m bgl)	GWL (m RL)
BH06	26.8	10	0.83	25.97
BH07	33.9	15	4.72	29.18
BH08	23.8	15	2.98	20.82
BH09	36.6	15	8.8	27.8
BH10	19.1	11	4.44	14.66

Groundwater quality

79. Groundwater quality sampling and testing was undertaken for the Project near the Terrace Tunnel (in monitoring wells NE01 and TD02) and at the Basin Reserve (in monitoring wells WS102, BR01 S and BR02 S). The results are listed in Table 13. Shallow groundwater could be contaminated due to activities typically associated with urban and industrial land uses. The following sites of potential interest have been identified near the Project in the Contaminated Site Assessment report for the Project, and are listed on the Selected Land Use Register (SLUR)[4]; locations are shown in Figure 1 in Annexure A:

   i. Former landfill / cemetery sites at Kelburn Park and Mount Street Cemetery are identified on the SLUR near the Terrace tunnel

   ii. The Gazley Mitsubishi site north of Basin Reserve is a SLUR site of potential concern and some soil and possibly groundwater contamination could be expected there. A Detailed Site Investigation (DSI) and management plans for the construction works informed by the DSI have been prepared by the Project's contaminated land specialist

   iii. The Church of Latter Day Saints in between Moxham Ave and Ruahine Street is listed to have fuel storage on site on the SLUR. The site is located between Ruahine Street and Moxham Ave. This site has or had a 1000 gal (4500 L) tank probably for heating oil for the Latter Day Saints Church. According to the Project's contaminated land specialist Mr Kevin Tearney, there is no physical evidence of a tank being present (e.g., no tank slab, fill points or dip points). The site currently has an LPG fueled boiler. If there was an underground storage tank at the site as indicated on drawings from 1973, it was most likely removed in the mid-1970s during building extension works

   iv. Kilbirnie Park is classed as a HAIL (Hazardous Activities and Industries List) site and could potentially have groundwater contamination associated with landfilling

80. Blasting methods are proposed for the construction of the Mount Victoria tunnel. Blasting can release nitrogen components, such as ammonium and nitrate, into the environment, including groundwater and stormwater.

81. Groundwater quality at the Terrace Tunnel does not appear to be affected by the former landfill / cemetery sites at Kelburn Park and Mount Street Cemetery or any other localised source of contamination associated with urban and industrial land uses:

   i. The test results (Table 13) at the Terrace suggest the groundwater is freshwater with electrical conductivity less than 500 μS/cm

   ii. Nutrient levels (i.e., nitrate and dissolved reactive phosphorous) are relatively low and within the range of levels typical for urban environments. Ammoniacal-nitrogen is below detection limits and chloride levels are relatively low. These parameters would be elevated if the former landfills contaminated the groundwater at the Terrace Tunnel

   iii. All total petroleum hydrocarbon concentrations are below detection limit. As such, there is no evidence of petroleum hydrocarbon contamination of the groundwater at the Terrace Tunnel

   iv. Some concentrations of metals at the Terrace Tunnel are higher than those observed at the Basin Reserve, most notably copper, nickel and zinc. Those parameters are typically elevated near major transport corridors due to heavy traffic, for example, copper is released from brake pads, and zinc can leach from car tires

82. Groundwater quality at the Basin Reserve does not appear to be affected by the Gazley Mitsubishi site north of the Basin Reserve:

   i. The Project's contaminated land specialist Mr Tearney confirmed no petroleum hydrocarbon contamination of groundwater was detected at the Gazley site during field investigations

---

[4] The Selected Land Use Register records details of properties that have, or may have, been used for activities and industries included in the Hazardous Activities and Industries List (HAIL).

ii. All total petroleum hydrocarbon concentrations are below detection limit in the three monitoring wells in between the Gazley site and the proposed Sussex St Trench (BR01 S, BR02 S and WS102; location shown in Figure 1 in Annexure A). As such, there is no evidence of petroleum hydrocarbon contamination of groundwater at the Basin Reserve

Table 13: Groundwater quality test results at the Terrace Tunnel and Basin Reserve

Parameter	Terrace Tunnel		Basin Reserve		
	TD02	**NE01**	**BR01 S**	**BR02 S**	**WS102**
General					
Dissolved Oxygen (g/m^3) (field)	6.7	8.9	0.88	0.81	1.24
pH (field) (pH units)	5.7	5.3	6.9	6.5	6.17
Temperature (sample in field) (C°)	14.3	13.8	20.5	19.5	16.2
Sum of Anions (meq/L)	3.7	3.3	2.8	3.0	5
Sum of Cations (meq/L)	3.7	3.3	2.6	3.0	4.5
pH (pH Units)	6.1	5.6	7.4	7.3	8.2
Total Alkalinity (g/m^3 as CaCO3)	33.0	11.8	79.0	75.0	200
Bicarbonate (g/m^3 at 25°C)	40	14.4	96	92	240
Total Hardness (g/m^3 as CaCO3)	27	37	38	43	155
Electrical Conductivity (EC) (μS/cm)	426	399	279	328	482
Dissolved Boron (g/m^3)	0.09	0.08	-	-	
Total Boron (g/m^3)	0.097	0.08	-	-	
Dissolved Calcium (g/m^3)	5.4	2.1	9.6	10.6	46
Dissolved Iron (g/m^3)	< 0.02	0.05	-	-	
Dissolved Magnesium (g/m^3)	3.3	7.8	3.5	3.9	10
Dissolved Manganese (g/m^3)	0.0133	0.0133	-	-	
Total Manganese (g/m^3)	0.05	0.21	-	-	
Dissolved Potassium (g/m^3)	1.77	2.2	1.81	1.59	7.8
Dissolved Sodium (g/m^3)	70	58	42	48	28
Chloride (g/m^3)	63	71	37	47	29
Total Nitrogen (g/m^3)	-	-	13.9	0.46	2.1
Total Ammoniacal-N (g/m^3)	< 0.010	< 0.010	-	-	-
Nitrite-N (g/m^3)	< 0.002	< 0.002	0.03	< 0.002	0.007
Nitrate-N (g/m^3)	0.8	1.4	<0.02	0.006	0.02
Nitrate-N + Nitrite-N (g/m^3)	0.8	1.4	0.04	0.006	0.027
Total Kjeldahl Nitrogen (TKN) (g/m^3)	-	-	13.8	0.45	2.1
Dissolved Reactive Phosphorus (g/m^3)	0.011	0.004	-	-	-
Total Phosphorus (g/m^3)	-	-	1.12	0.093	0.72
Sulphate (g/m^3)	61	47	7.4	7.7	10.6
Dissolved Organic Carbon (DOC) (g/m^3)	3.3	2.4	-	-	-
Metals, dissolved					
Dissolved Aluminium (g/m^3)	0.019	0.073	-	-	-
Dissolved Arsenic (g/m^3)	< 0.001	< 0.001	0.003	< 0.001	< 0.001
Dissolved Cadmium (g/m^3)	0.0001	< 0.00005	< 0.00005	< 0.00005	< 0.00005
Dissolved Chromium (g/m^3)	0.0021	0.0007	< 0.0005	< 0.0005	< 0.0005
Dissolved Copper (g/m^3)	0.52	0.08	0.001	< 0.0005	0.0035
Dissolved Lead (g/m^3)	< 0.0001	0.0001	0.00025	< 0.00010	< 0.0001
Dissolved Mercury (g/m^3)	< 0.0001	< 0.0001	-	-	-
Dissolved Nickel (g/m^3)	0.0189	0.0074	0.0005	0.0008	0.0007
Dissolved Zinc (g/m^3)	0.041	0.029	< 0.0010	0.0018	0.0099
Metals, totals					
Total Aluminium (g/m^3)	2.1	6.4	-	-	-
Total Arsenic (g/m^3)	< 0.0011	0.0014	0.0138	0.0042	0.0046
Total Cadmium (g/m^3)	0.000094	< 0.000053	0.00013	< 0.000053	< 0.000053

Parameter	Terrace Tunnel		Basin Reserve		
Total Chromium (g/m^3)	0.0142	0.0185	0.0139	0.0078	0.0022
Total Copper (g/m^3)	0.64	0.152	0.06	0.0119	0.0071
Total Lead (g/m^3)	0.0038	0.0147	0.033	0.0123	0.0033
Total Mercury (g/m^3)	< 0.00011	< 0.00011	-	-	-
Total Nickel (g/m^3)	0.024	0.022	0.0096	0.0067	0.00183
Total Zinc (g/m^3)	0.059	0.127	0.054	0.038	0.025
Total Petroleum Hydrocarbons					
C7 - C9 (g/m^3)	< 0.10	< 0.10	< 0.10	< 0.10	< 0.10
C10 - C14 (g/m^3)	< 0.2	< 0.2	< 0.2	< 0.2	< 0.2
C15 - C36 (g/m^3)	< 0.4	< 0.4	< 0.4	< 0.4	< 0.4
Total hydrocarbons (C7 - C36) (g/m^3)	< 0.7	< 0.7	< 0.7	< 0.7	< 0.7

Potentially sensitive buildings and structures near the Project

83. There are several buildings near each of the four key sites described in paragraph 46 that could potentially be affected by land settlement effects. Historical buildings are typically more sensitive to these effects than more modern buildings. The Historic Heritage Assessment report notes the presence of historic buildings in proximity of the four key sites described in paragraph 7 above. This includes buildings on The Terrace and adjoining streets, Basin Reserve perimeter (e.g., Cricket Stand and National Cricket Museum), Dufferin St, Paterson Rd, Austin St, Ruahine St and Wellington Rd.

Potentially sensitive environments or water users near the Project

84. Several water supply wells are listed on GWRC's wells database as shown on the map in Figure 17 (Annexure A). Most of the wells shown are more than 500 m distance from any of the four key sites of the Project described in paragraph 46. Only one well is located relatively close to the proposed Terrace Tunnel, which is BQ31/0022 at 140 m west of the tunnel portal. However, this well appears to be located on an abandoned site and is unlikely to be still in use.

85. Several existing piped sections of Kumutoto, Waitangi and Waipapa Streams are adjacent to proposed excavations for the Project (Figure 2 in Annexure A).

86. The ecology specialist Joshua Markham identified potentially significant wetlands on the east side of Mt Victoria in the Town Belt along Ruahine Street (Figure 18). Whilst more than 500 m south of the proposed new Mt Victoria tunnel, this wetland is close to the proposed new stormwater main and the proposed access road realignment into Hataitai Park.

5. ASSESSMENT OF OPERATIONAL EFFECTS

87. The proposed construction and operation of the Project could have an impact on groundwater which may cause adverse environmental effects, and affect nearby buildings, structures and groundwater users. The following potential adverse effects in the operational phase are assessed in this section:

    i. Land settlement as a result of Project dewatering, either for the Project construction or throughout its operation, and potential impacts on nearby structures

    ii. Stream and wetland depletion can occur as a result of temporarily lowering groundwater levels. The potential ecological effects of this depletion are considered in separate reports

    iii. Change in groundwater flow direction or velocities that could affect groundwater quality when contaminants are drawn into a drainage system, or into a well that is used for water supply. Contaminants could also be diverted into groundwater receiving streams or wetlands after a change in flow direction

    iv. The lowering of groundwater levels can potentially affect a well-owner's ability to abstract water from their water supply wells, as described in paragraph 43

88. The drawdown of groundwater levels and change in groundwater flow can be caused by various activities related to the operation of the Project, such as:

    i. The installation of permanent drainage systems in the Terrace and Mt Victoria tunnels, and in the Sussex St trench

    ii. Cutting and filling activities at all four key sites. Where these intercept the groundwater table, the cut slopes will require permanent groundwater drainage for stability and to keep the land at the toe of the cut slope dry

89. The Project's potential effects in the operational phase have been assessed using 2D cross-sectional numerical groundwater models in SEEP/W and in AnAqSim, an analytical element model. Both are described in Annexure C.

Land settlement effects caused by groundwater drawdown

90. Land settlement can occur from dewatering activities as a result of reduced pore water pressure and subsequent consolidation of the soil structure. Certain soil materials are particularly vulnerable to settlement, such as unconsolidated and saturated clay and peat. If land settlement occurs, nearby structures and services could be damaged, particularly if the land subsidence is uneven meaning that one part of a site settles more than another part. This process is referred to as differential settlement and damage is most likely to occur where differential settlements are the largest. Because groundwater levels are drawn down in the shape of a cone during abstraction any land settlement resulting from dewatering is likely to be differential. The maximum differential settlement will occur where the groundwater gradient is largest. Vertical land settlement from groundwater drawdown can be calculated based on the compressibility and the thickness of the compressible soil layer, using the one-dimensional compression equation listed in CIRIA (2016).

91. Land settlement could occur both in the construction and operational phase. It will be most severe relatively close to the point of groundwater abstraction, which will be the Project's permanent drainage system or any dewatering for pits or trenches during the construction phase. It can therefore be expected that if land settlement were to occur, it would be most significant within the proposed designation boundaries.

92. The proposed new Terrace and Mt Victoria tunnels will both be permanently drained tunnels similar to the existing tunnels. Groundwater drawdown has been assessed and is shown in Figure 23 and Figure 24 in Annexure A. No land settlement from groundwater drawdown is expected at either the Terrace or the Mt Victoria tunnels, because the groundwater table is below any weathered soil materials that could be prone to land settlement from groundwater drawdown. Fractured bedrock has very low compressibility and groundwater drawdown induced land settlement is unlikely to occur.

The effects are therefore assessed as 'negligible' in accordance with the assessment framework outlined in Table 1.

93. Nonetheless, mechanical settlement associated with the construction of the tunnel as described in the Geotechnical, Natural Hazards and Resilience Report by Pathmanathan Brabhaharan, could cause land settlement and potential damage to buildings and structures near the Terrace or the Mt Victoria tunnels, but this is not caused by groundwater drawdown.

94. Based on the information currently available, it would appear that soils prone to land settlement from groundwater drawdown are mostly present in the Basin Reserve area (i.e., marine sediments and Holocene swamp deposits, Figure 10 in Annexure A). Weathered greywacke at Ruahine Street and Wellington Road could also be susceptible to land settlement from groundwater drawdown, but to a lesser extent. Hardly any soils prone to land settlement from groundwater drawdown are present near the tunnels at Mt Victoria and the Terrace. In addition, the groundwater table is relatively deep at the Terrace and Mt Victoria tunnels and mainly present within the moderately to unweathered fractured bedrock formations which are not prone to land settlement induced from groundwater drawdown (described in paragraph 56).

95. The proposed Sussex St Trench at Basin Reserve will also be constructed as a permanently drained structure. This will cause groundwater drawdown which could result in land settlement effects. I have completed a groundwater drawdown assessment (Figure 25 in Annexure A) and have also considered the numerical groundwater modelling assessment undertaken for the design of the Project (as described in Annexure C). Based on these assessments, land settlement effects from groundwater drawdown can be in the order of up to 50 mm, assuming a compressibility of 15-30 MPa (paragraph 74), although the differential settlement slope will likely not exceed 1/500. Relying on the effects categories indicated in Table 1, the potential damage category could be 'slight' (i.e., 'low' impact) leading to minor cracks that could be repaired (further discussed in the mitigation section in paragraph 137). It is worth noting that NZTA and Memorial Park Alliance (2015) found no damage to buildings from land settlement after the construction of the nearby Arras Tunnel, which was constructed and dewatered to the same depth as the deepest part of the Sussex St Trench.

96. Several cut slopes for road widening are proposed along Ruahine St and Wellington Rd in Hataitai. Where the cuts intercept groundwater, localised lowering of groundwater levels could occur. The associated settlement is expected to be limited to the immediate vicinity of the cut slopes, and therefore unlikely to extend to adjacent buildings. The impact of any land settlement on neighbouring structures induced by groundwater drawdown is assessed as negligible.

Effects on sensitive environments and nearby water users

97. The Kumutoto Stream flows directly adjacent to the existing Terrace Tunnel and the proposed new southbound tunnel to the east (Figure 16 and Figure 15 in Annexure A and Photo 3 in Annexure B). Based on the information from monitoring wells along The Terrace the groundwater appears to be deep, and it is unlikely that the Kumutoto Stream receives groundwater inflows in its upper reaches around the Terrace Tunnel where the streambed is well-above the groundwater table (Figure 12 to Figure 14 in Annexure A). The Kumutoto Stream is piped downstream from the 'Woof Woof Ruff' dog park above the northern tunnel portal and becomes part of the city's stormwater network. From that point, the Kumutoto is no longer a natural stream. Because the upstream part of the Kumutoto Stream is hydraulically disconnected from groundwater, the impact of the Project on the baseflow of the Kumutoto Stream is assessed as 'negligible'. There are no other sensitive environments, such as streams or wetlands with significant ecological values, identified near The Terrace Tunnel. In addition, no actively used groundwater supply wells are present near the Terrace Tunnel.

98. There are no sensitive environments, such as streams or wetlands with significant ecological values, identified within the likely area of impact of the Project around Basin Reserve. In addition, no actively

used groundwater supply wells are present near the Basin Reserve. As such, no impacts from groundwater drawdown caused by the Project are expected.

99. The proposed Sussex St Trench will be constructed partially perpendicular to the groundwater gradient (see Figure 8 in Annexure A), which could in theory cause the groundwater table to mound on the upgradient side. However, the trench will be permanently drained alleviating any upgradient groundwater mounding. Therefore, no groundwater mounding and drainage issues caused by the construction of the Sussex St Trench are anticipated.

100. The identified Ruahine St wetland is at a notable distance away from the proposed new Mt Victoria Tunnel. In addition, it is noted that the current tunnel is permanently drained and closer to the Ruahine St wetland. It is unlikely that the proposed new Mt Victoria Tunnel will affect the Ruahine St wetland.

101. However, a cut slope will be excavated directly downgradient of the identified Ruahine St wetland for the upgrade of the access road to the adjacent sport fields, as shown in Figure 19 in Annexure A. Because the groundwater table is very shallow (which supports the wetland), permanent drainage will be required to keep the cut slope stable and the access road dry. This will likely affect the baseflow of the wetland as follows:

    i. SEEP/W cross-sectional modelling was undertaken along cross section C-C (Figure 1 in Annexure A; model description included in Annexure C, Figure 35). The results are shown in Figure 20 in Annexure A. The groundwater table will fully detach from the base of the wetland within a year, and groundwater will no longer flow to the wetland

102. The same SEEP/W model was used to assess the potential mitigation of sheet piling the cut slope. The results are shown in Figure 21 and show that sheet piling will help raise the upgradient groundwater levels beneath the wetland, but will not avoid the full detachment of the groundwater table and thus cause 100% reduction of baseflows.

103. Therefore, the effect of the Project on the Ruahine Street wetland (i.e., Wetland 2) has been assessed as ‘high’ (without mitigation). It is important to highlight that the wetland also receives inflows from up-catchment runoff, which are unaffected by the proposed road cut drainage. The ecological effects are described in a separate report.

104. There are no actively used groundwater supply wells present near Ruahine Street.

105. Because all listed water supply wells near the Project’s key sites are either likely to be abandoned, or at a distance of more than 500 m away, it is unlikely that any water supply well will be affected by the groundwater drawdown caused by the permanent drainage of the tunnels, trench and cut slopes.

Groundwater quality effects

106. Contaminated groundwater from nearby contaminated sites could be drawn towards permanent dewatering systems. These systems will typically discharge to the stormwater system and may subsequently be discharged to the sea at the harbour outflows of the stormwater system.

107. SLUR sites have been identified near the four key sites (refer to paragraph 79). The following sites could potentially pose a risk in the operational phase of the project:

    i. Former landfill / cemetery sites at Kelburn Park and Mount Street Cemetery are identified on the SLUR near the Terrace tunnel. Because the current tunnel is drained, any nearby groundwater contamination would be drawn into the existing dewatering system. The new tunnel will also be drained permanently. However, no evidence of a legacy groundwater contamination plume associated with the former landfill / cemetery sites at Kelburn Park and Mount Street Cemetery were found (refer to paragraph 81). No groundwater contamination in concentrations over and above levels that could typically be expected in urban areas were found at the Terrace (Table 13). The risk of groundwater contamination above background concentrations being drawn into the new Terrace tunnel permanent drainage system and subsequently impacting receiving surface water is considered ‘negligible’

ii. Underground storage tanks for petrol were present at the Gazley property close to the proposed Sussex St trench. The Sussex St trench will be constructed as a permanently drained structure. However, no petroleum hydrocarbons were detected in any of the tested monitoring wells, and as such there is no evidence of a legacy groundwater contamination plume being present. The risk of groundwater contamination above expected background concentrations being drawn into the new Sussex St Trench permanent drainage system and subsequently impacting receiving surface water is considered ‘negligible’

108. The recently constructed Waterview and City Rail Link tunnels in Auckland both experienced groundwater ingress with extremely high alkalinity, leading to calcite precipitation inside the tunnel and the stormwater and drainage water collection systems. Whilst this groundwater ingress water is diluted during wet weather, there are concerns about environmental impacts from the discharge of this highly alkaline drainage water during dry weather when no dilution occurs. If pH and alkalinity are raised in the tunnel drainage water due to this process, then the impact on water quality of the receiving environment (i.e., piped awas and the sea which they discharge) could be ‘high’.

6. ASSESSMENT OF CONSTRUCTION EFFECTS

109. The proposed construction and operation of the Project could have an impact on groundwater which may cause adverse environmental effects and affect nearby buildings, structures and groundwater users. The effects are associated with the drawdown of groundwater levels and a change in groundwater flow directions as a result of temporary dewatering or depressurisation during the construction of the Project.

110. The following potential adverse effects are assessed in this section:

   i. Change in groundwater flow direction or velocities that could affect groundwater quality when contaminants are drawn into a drainage system, or into a well that is used for water supply. Contaminants could also be diverted into groundwater receiving surface water after a change in flow direction

   ii. Land settlement as a result of Project dewatering, either for the Project construction or throughout its operation, and potential impacts on nearby structures

   iii. Stream and wetland depletion can occur as a result of temporarily lowering groundwater levels. The potential ecological effects of this depletion are considered in separate reports

   iv. The lowering of groundwater levels can potentially affect a well-owner's ability to abstract water from their water supply wells, as described in paragraph 43

111. The drawdown of groundwater levels and change in groundwater flow can be caused by various activities related to the operation of the Project, such as:

   i. All excavations for the tunnels, trench and cut slopes below the groundwater table at all four key sites will require temporary dewatering during construction, noting that for some of these sites permanent drainage systems will be installed after completion of the construction phase (refer to previous section)

   ii. For other sites involving excavations only, temporary lowering of the groundwater table or depressurisation during the construction phase may be required. This is mainly for the installation of structures such as culverts, pipelines or bridge piers. After the construction has been completed all dewatering activities will cease

112. The Project's potential effects during the construction phase have been assessed using 2D cross-sectional numerical groundwater models in SEEP/W and the AnAqSim analytical element model, as described in Annexure C.

Land settlement effects caused by groundwater drawdown

113. As discussed in paragraph 90 to 94, land settlement can occur from dewatering activities as a result of reduced pore water pressure (caused by groundwater drawdown) and subsequent consolidation of the soil structure.

114. The dewatering at the Mt Victoria Tunnel and Terrace Tunnel will both be permanent, but the dewatering will commence in the construction phase. However, their greatest impacts are likely to occur when the groundwater drawdown will reach its maximum extent, which will be after the construction is completed. Because the presence of soils prone to land settlement from dewatering is very limited at Mt Victoria Tunnel and Terrace Tunnel, it is unlikely that any groundwater induced land settlement will occur at those sites.

115. A trench with a bridge crossing will be constructed at Sussex Street at the Basin Reserve. This trench will require temporary dewatering and depressurisation to dry out the construction trench and to reduce groundwater pressures during construction. I have assessed the effects of temporary dewatering and depressurisation with 2D numerical groundwater modelling in SEEP/W (description included in Annexure C). The drawdown effects were assessed to be 2 m at a distance of 10 m away from the trench (Figure 25 in Annexure A), which could cause land settlement of up to 35 mm during the construction phase. Without mitigation, this could cause damage to the nearby Cricket Stand in

the Basin Reserve. In addition, an AnAqSim modelling assessment was completed to inform the design of the Project as described in Annexure C, which confirm the findings from the SEEP/W modelling. The potential damage category could be ‘slight’ (i.e., ‘low’ impact) leading to minor cracks that could be repaired (further discussed in the mitigation section in paragraph 137). As mentioned in paragraph 95, it is worth noting that NZTA and Memorial Park Alliance (2015) found no damage to buildings from land settlement after the construction of the nearby Arras Tunnel, which was constructed and dewatered to the same depth as the deepest part of the Sussex St Trench.

116. However, impermeable secant pile walls could be installed along the perimeter of the Sussex Street trench which will avoid lateral groundwater inflow. Only vertical and radial groundwater inflow through the bottom of the trench will occur, which could be reduced notably by extending the depth of secant pile wall to the top of the bedrock. Some inflow through the base of the excavation remains likely, so there remains some potential for associated ground settlement.

117. The proposed stormwater upgrade along Ruahine Street / Wellington Rd involves the construction of four launch/receiving pits for the trenchless stormwater main installation, as described in paragraph 11. Based on the geological and hydrogeological information listed in paragraph 78 and in Table 12, temporary dewatering is required for up to 30 days in all four pits during the construction phase. Sheet pile walls will be installed at launch/receiving pits, and this will limit the groundwater inflows into the pit during construction.

118. The largest maximum required groundwater drawdown is 14.2 m in the deepest pit near 72 Wellington Rd. Cross-sectional numerical modelling using SEEP/W (model described in Annexure C) was undertaken to assess the drawdowns at various distances from the pit (Figure 26 in Annexure A):

i. Groundwater levels drawdown at a distance of 15 m from the pit: 5.4 m

ii. Groundwater levels drawdown at a distance of 50 m from the pit: 1.5 m

119. The thickness of the weathered greywacke materials beneath the properties east of Ruahine St are assumed to be up to 20 m deep, and have a compressibility of 50 MPa (paragraph 78). Competent greywacke bedrock is assumed to lay beneath the weathered greywacke which has minimal compressibility and is not considered susceptible to land settlement. Land settlement from groundwater drawdown is assumed to only occur in the weathered greywacke materials and assessed to be as follows:

i. Up to 21 mm at a distance of 15 m from the pit

ii. Up to 6 mm at a distance of 50 m from the pit

120. Very slight to slight damage (i.e., ‘negligible’ to ‘low’ impact) could occur to the nearest properties along Ruahine Street and Wellington Rd during the construction phase as a result of the proposed temporary launch/receiving pit dewatering.

121. Several other structures such as culverts, pipelines, stormwater retention basins and bridge/overpass piers will be installed at shallow depths below the groundwater table, and this requires temporary construction pit dewatering to create dry working conditions. In some cases (sub)artesian aquifer water pressures may need to be reduced by lowering the groundwater levels beneath the construction pit for pit floor and pit wall stability reasons. This is referred to as depressurisation. Both dewatering and depressurisation will temporarily draw down groundwater levels around the construction pits, and this could cause land settlement effects.

122. The precise locations where construction dewatering may be required will be determined at the detailed design stage. In general terms, dewatering for the installation of culverts, pipelines, stormwater retention basins and bridge/overpass piers are expected to be completed in a relatively short timeframe of several days or weeks at each site, and only require minimal dewatering.

123. Based on the anticipated works required to construct the Project, the impacts of these dewatering activities for other structures are expected to be 'low' as they are small-scale, localised and would only occur temporarily.

Effects on sensitive environments and nearby water users

124. The Kumutoto Stream flows directly adjacent to the existing Terrace Tunnel and the proposed new southbound tunnel to the east (Figure 16 and Figure 15 in Annexure A and Photo 3 in Annexure B). The impact from the Project on the baseflow of the Kumutoto Stream is the same as in the operational phase (paragraph 97) and is assessed as 'negligible'.

125. There are no sensitive environments, such as streams or wetlands with significant ecological values identified within the likely area of impact of the Basin Reserve.

126. The temporary dewatering during installation of the launch/receiving pits for the stormwater renewal along Ruahine Street and Wellington Road, will likely temporarily reduce the baseflows by 25% of the identified wetland at Ruahine Street (Figure 18 in Annexure A). The potential impact is assessed as 'moderate' in accordance with the assessment framework in Table 2.

127. No actively used groundwater supply wells are present near the four key sites of the Project. As such, no impacts from groundwater drawdown caused by the construction activities are expected.

Groundwater quality effects

128. If contaminated groundwater is present near the construction dewatering or depressurisation activities, these could potentially be drawn into the dewatering systems and into the construction pits or trenches. If this dewatering water is discharged untreated to the stormwater system that flows out into the harbour, this could potentially impact marine water quality and ecological values near the harbour discharge points of the stormwater system.

129. Several SLUR sites near the four key sites could locally have contaminated groundwater (refer to paragraph 79) and could potentially pose a risk during the construction phase of the Project:

i. Former landfill / cemetery sites at Kelburn Park and Mount Street Cemetery. Because the current tunnel is drained, any nearby groundwater contamination would be drawn into the existing dewatering system. The new tunnel will be constructed with temporary dewatering systems, and this could change the pathway of any nearby groundwater contamination. However, no evidence of a legacy groundwater contamination plume associated with the former landfill / cemetery sites at Kelburn Park and Mount Street Cemetery were found. No groundwater contamination in concentrations over and above levels that could typically be expected in urban areas were found at The Terrace. The risk of groundwater contamination above background concentrations being drawn into the new Terrace tunnel permanent drainage system is considered 'negligible'

ii. Underground storage tanks for petrol were present at the Gazley property close to the proposed Sussex St trench. The Sussex St trench will be constructed with temporary dewatering and possible depressurisation systems, and this could change the pathway of any nearby groundwater contamination. However, no petroleum hydrocarbons were detected in any of the tested monitoring wells, and as such there is no evidence of a legacy groundwater contamination plume being present. The risk of groundwater contamination above expected background concentrations being drawn into the new Sussex St Trench permanent drainage system is considered 'negligible'

iii. A possible petrol storage tank at the Church of Latter Day Saints located between Ruahine Street and Moxham Ave. Whether this has caused groundwater to be contaminated is unconfirmed, but considered unlikely by the Project's contaminated land expert Kevin Tearney. Nonetheless, it is unlikely that any groundwater contamination would be drawn into the nearby launch/receiving pit for the stormwater upgrade along Ruahine Street during the construction phase. This is because the dewatering is relatively short (~30 days) and the construction methodology includes sheet piling which notably limits lateral groundwater flow and any potentially contaminated groundwater. The risks of groundwater contaminants being drawn into the Ruahine Street launch/receiving pits for the stormwater upgrading is considered 'negligible'

iv. Kilbirnie Park, which could potentially have groundwater contamination associated with landfilling. However, no dewatering or groundwater drainage is likely to be required for the Project near Kilbirnie Park, hence no groundwater effects are expected near Kilbirnie Park

130. Blasting methods are proposed for the construction of Mount Victoria tunnel. Blasting can release nitrogen components, such as ammonium and nitrate, into the environment, including groundwater and stormwater. However, the impacts on groundwater quality are expected to be limited and localized, noting that ammonium and nitrate are likely present at background values in the groundwater environment at both tunnels. It is unlikely that ecological water quality guideline values will be exceeded in groundwater-receiving surface water near the tunnels, and as such the risks are classed as 'low'. However, appropriate stormwater quality management measures are recommended (see paragraph 134).

131. The recently constructed Waterview and City Rail Link tunnels in Auckland both experienced groundwater ingress with extremely high alkalinity, leading to calcite precipitation inside the tunnel and the stormwater and drainage water collection systems. Whilst this groundwater ingress water is diluted during wet weather, there are concerns about environmental impacts from the discharge of this highly alkaline drainage water during dry weather when no dilution occurs. If pH and alkalinity are raised in the tunnel drainage water due to this process, then the impact on water quality of the receiving environment (i.e., piped awas and the sea which they discharge) could be 'high'. Appropriate stormwater quality management measures are recommended (see paragraph 138).

7. MEASURES TO AVOID, REMEDY OR MITIGATE ACTUAL OR POTENTIAL ADVERSE EFFECTS

132. This section describes the proposed measures and monitoring to avoid, remedy or mitigate actual or potential adverse effects of the Project.

133. The preparation and implementation of a **Groundwater and Settlement Monitoring and Contingency Plan (GSMCP)** is recommended to support the effects management and monitoring for dewatering during construction and for 1 year after the completion of construction. The GSMCP provides a guide to managing potential settlement generated by groundwater drawdown and deflections of nearby buildings and structures caused by temporary excavations on buildings and services adjacent to the works. The GSMCP describes the type of monitoring that will occur and trigger levels that will be used to prompt actions and responses in the form of additional measures that are to be taken. Further details are included in Annexure D.

134. Stormwater quality monitoring as part of the **Stormwater Management Plan (SMP)** is recommended to confirm if water quality of stormwater collected by the drainage systems in the tunnels is suitable for discharge to the environment. Water treatment of the tunnel drainage water as part of the SMP may be required if the water quality does not meet the guidelines. Further details about the SMP are included in the Stormwater Report.

Measures to avoid, remedy or mitigate effects during operation

135. The following mitigation measures are recommended to minimise adverse effects on groundwater from the Project during the operational phase:

i. A concrete lining with a waterproofing membrane in the arch of both the new Terrace Tunnel and Mt Victoria Tunnel to minimise groundwater ingress via the tunnel liner into the tunnel. Groundwater will be drained at the base of both tunnels and conveyed to the tunnel's stormwater system. This lining will reduce the risks of groundwater contamination or high alkaline groundwater being drawn into the tunnel drainage system. If a SMP as described in paragraph 138 is implemented also, the potential impacts on water quality are considered to be 'low'

Measures to avoid, remedy or mitigate effects during construction

136. The following mitigation measures are recommended to minimise adverse effects on groundwater from the Project during the construction phase:

i. Secant pile or contiguous walls should be installed at the Sussex Street trench to minimise groundwater inflows during the construction phase, thus minimising drawdown effects from temporary dewatering and depressurisation. If implemented, the potential damage from land settlement induced by groundwater drawdown (such as to buildings within the Basin Reserve) is 'very slight' (i.e., 'negligible' impact)

ii. Temporary augmentation of wetland flows by spray irrigation with clean water to the Ruahine Street wetland may be required to minimise effects during the installation of the proposed new stormwater pipe through Ruahine Street and Wellington Road. Spray irrigation would only be needed to keep the wetland wet if the nearby construction dewatering occurs during dry periods, because rainfall runoff will likely provide sufficient inflows most of the time. If implemented, the impact on the Ruahine Street wetland baseflows is 'low'. It is noted that the Ruahine Street wetland may eventually be affected by the permanent drainage for the proposed cut slope for the new access road to Hataitai Park

Recommended monitoring

137. During the construction and for at least throughout the first year in the operational phase of the Project it is recommended that the following monitoring measures are taken, which will be implemented as part of the GSMCP (outlined in paragraph 133 above, with further details presented in Annexure D):

i. Ongoing groundwater level monitoring at the Ruahine Street/Wellington Road stormwater renewal, Mt Victoria Tunnel, Basin Reserve and the Terrace Tunnel to confirm if groundwater level drawdowns remain within the expected range. Mitigation measures may be required to address adverse impacts, acknowledging that measures to reduce groundwater ingress into the tunnels could be cost-prohibitive. Damage to buildings and structures affected by land settlement from groundwater drawdown are to be repaired if groundwater level drawdowns are more than expected and have led to damage

ii. Ongoing tunnel drainage water quality monitoring at the Terrace Tunnel, Sussex St Trench and Mt Victoria Tunnel are recommended to confirm that groundwater ingress water that is discharged via the tunnel drainage system in the city stormwater system complies with the required water quality standards. Of particular concern is the possibility of high alkalinity resulting in high pH levels due to calcite dissolution from the tunnel liner, and the potential for existing groundwater contamination (including nitrogen from blasting residue at Mt Victoria Tunnel) to be drawn into the tunnel drainage system from contaminated sites nearby

iii. Water treatment of the tunnel drainage water as part of a SMP may be required if the water quality does not meet the norms. If an SMP as noted in paragraph 138 is implemented the potential impacts on water quality are considered to be low

138. During the operational phase of the Project, it is recommended that as part of ongoing stormwater quality monitoring for the SMP for both Mt Victoria and Terrace Tunnel sites, the following monitoring measures are taken:

i. Ongoing tunnel drainage water quality monitoring at the Terrace Tunnel, Sussex St Trench and Mt Victoria Tunnel are recommended to confirm that groundwater ingress water that is discharged via the tunnel drainage system in the city stormwater system complies with the required water quality standards. Of particular concern is the possibility of high alkalinity resulting in high pH levels due to calcite dissolution from the tunnel liner, and the potential for existing groundwater contamination to be drawn into the tunnel drainage system from contaminated sites nearby. Water treatment of the tunnel drainage water may be required as part of the SMP if the water quality does not meet the norms. If implemented, potential impacts on water quality are considered to be low

8. CONSULTATION

139. I understand that NZTA has engaged with various persons and entities in developing the Project.

140. A community engagement survey was held for the Project. The community survey was part of the wider engagement on the Project with the community and stakeholders. The survey was developed to get community feedback to help future project development.

141. The consequences of the Project for the natural environment was an issue raised in multiple comments. A key concern raised in the survey relates to the potential impacts of the Project on the Kumutoto Stream near the Terrace Tunnel. I have assessed the impacts on baseflow (i.e., groundwater inflow) to the Kumutoto Stream as 'negligible', as described in paragraph 97 and 124.

142. A previous version of this report was provided to Greater Wellington Regional Council (GWRC) and Wellington City Council (WCC) for an initial review. Review comments were made in relation to the following:

    i. GWRC indicated that further clarification on the assessments of baseflow impacts on the Kumutoto Stream and on the Ruahine Street wetland (Wetland 2) were required. I have assessed the impacts on baseflow (i.e., groundwater inflow) to the Kumutoto Stream as 'negligible', as described in paragraph 97 and 124. The assessment on the Ruahine Street wetlands are described in 99 through to 103, and 126

    ii. Further clarification was also requested by GWRC in relation to spray irrigation to mitigate baseflow reduction impacts on nearby wetlands. This is addressed in paragraph 136

    iii. GWRC sought clarification on which water quality standards and guideline values were used in the groundwater impacts assessment. In paragraph 43 (Assessment Framework) I refer to the ecological water quality guidelines for 95-percentile species protection (ANZG, 2018). It will be those parameters that will be tested for as part of a Stormwater Management Plan (SMP) as referred to in paragraph 134

    iv. WCC indicated that further clarification on the assessment of the groundwater induced land settlement at The Terrace was required. WCC raised concerns about the potential for land settlement to occur if weathered rock to a depth of 60 m was present near the southern portal of the Terrace Tunnel, as suggested in historic documents. However, more recent geotechnical and hydrogeological field investigations have confirmed that completely weathered rock (i.e., which may have some susceptibility of groundwater induced land settlement) at the Terrace Tunnel is no more than 10 m deep and above the groundwater table. This is addressed in paragraph 56

    v. GWRC and WCC indicated that the provision of a Groundwater and Settlement Monitoring and Contingency Plan (GSMCP) would help to better understand how groundwater induced land settlement risks are proposed to be managed during the construction of the Project. A GSMCP will be adopted for the Project; however, it has not been drafted at this stage as this is best done as a part of detailed design. Annexure D sets out the proposed requirements and framework for the GSMCP, including a clear description of the proposed approach to monitoring and follow up actions (i.e., Trigger Action Response Plan, TARP)

9. CONCLUSION AND RECOMMENDATIONS

143. The construction and the permanent operation of the proposed SH1 Improvement project involve various excavation and construction activities that could interact with groundwater and potentially cause adverse effects near the Mt Victoria Tunnel, the Basin Reserve and the Terrace Tunnel.

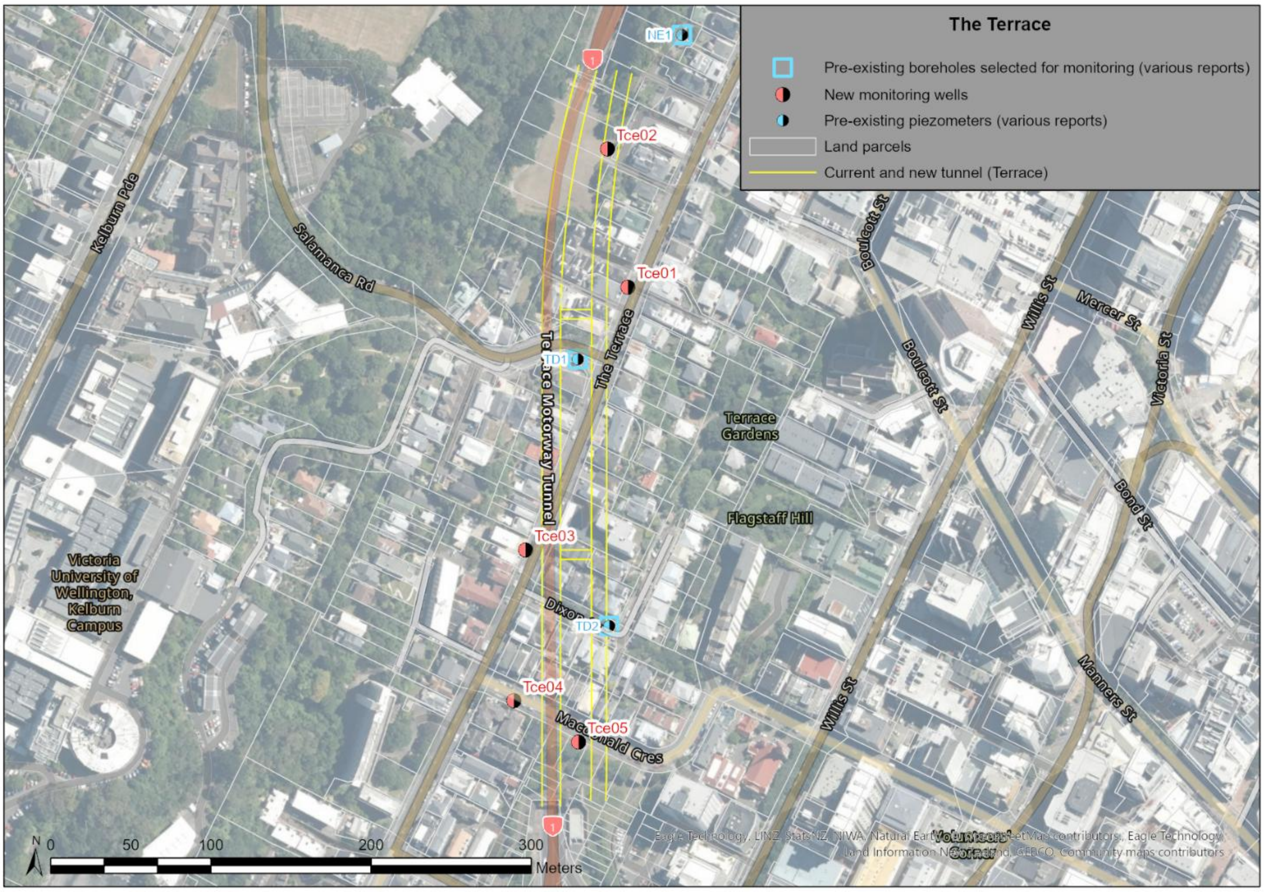
144. Mitigation and ongoing monitoring measures (to be included in relevant management plans, including a **GSMCP** and **SMP**) are proposed to avoid, remedy or mitigate actual or potential adverse effects of the Project. If these measures are appropriately implemented no residual adverse effects on groundwater are expected of the Project for most of the identified potential effects. However, the baseflow reduction effect on the existing Ruahine Street wetland is likely to be high and no effective mitigation measure has been identified. Table 14 provides an overview of effects and recommended mitigation.

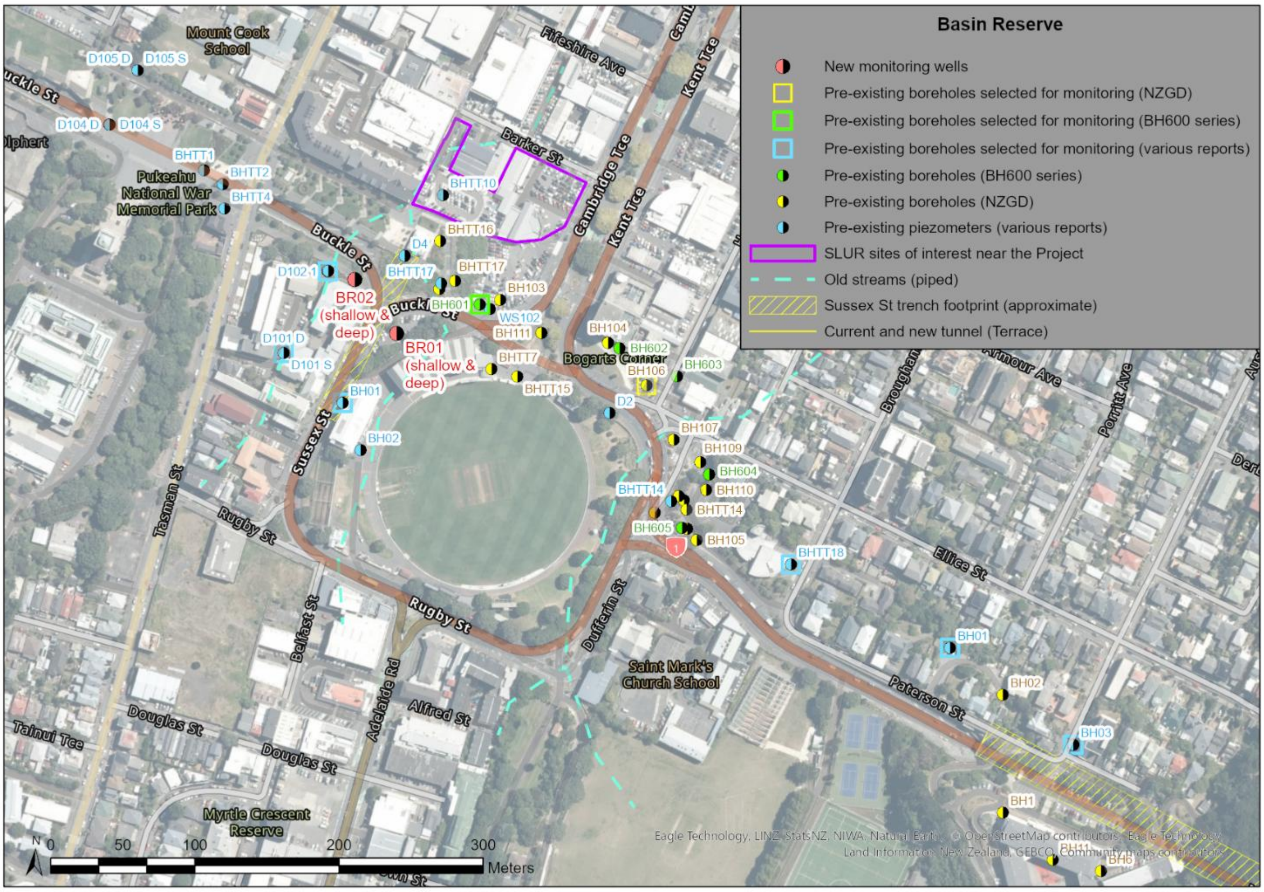
Table 14: Groundwater effects and mitigation assessment summary table

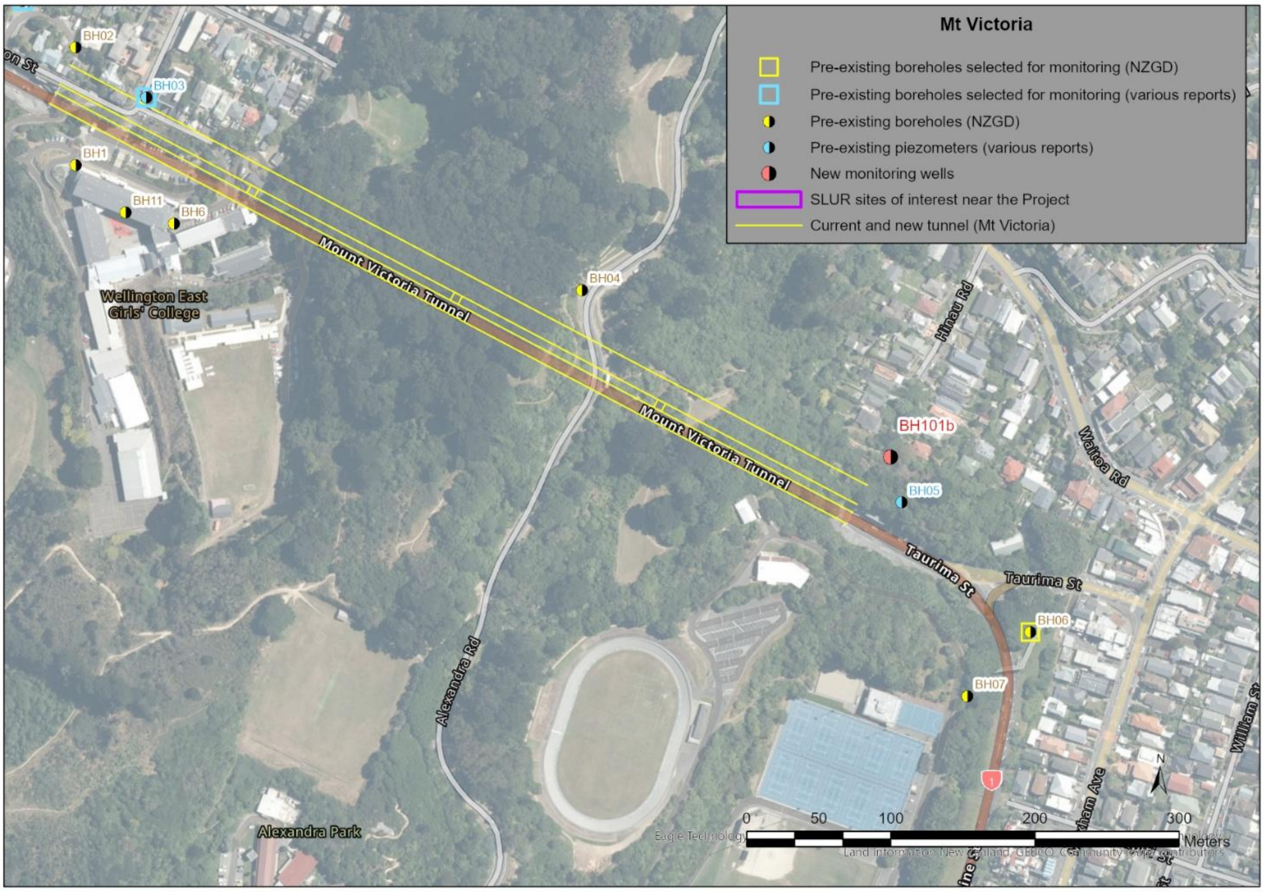
Location	Type of effect	Magnitude of unmitigated effect	Recommended mitigation and monitoring	Residual effect
Terrace Tunnel	Damage to nearby buildings and structures from groundwater induced land settlement from temporary dewatering (operational and construction)	Negligible (however, mechanical settlement effects could arise)	GSMCP monitoring (because of mechanical settlement effects) and mitigation measures such as damage repair	Negligible
	Baseflow reduction of upstream section of Kumutoto Stream (operational and construction)	Negligible	None	Negligible
	Groundwater or drainage water receiving surface water may be affected by groundwater contaminations being drawn into the groundwater drainage system which discharges stormwater (operational and construction)	Negligible	None	None
	Groundwater or drainage water receiving surface water may be affected by high alkaline drainage water which discharge to stormwater (operational and construction)	High	Lining of ceiling arch to reduce groundwater inflows and SMP monitoring and mitigation measures such as water treatment	Low
Basin Reserve	Damage to nearby buildings and structures from groundwater induced land settlement (construction)	Low	Secant pile walls to reduce groundwater inflows and GSMCP monitoring (because of mechanical settlement effects) and mitigation measures such as damage repair	Negligible
	Groundwater or drainage water receiving surface water may be affected by groundwater contaminations being drawn into the groundwater drainage system which discharges stormwater (operational and construction)	Negligible	None	Negligible
	Groundwater or drainage water receiving surface water may be affected by high alkaline drainage water which discharge to stormwater (operational and construction)	High	Secant pile walls to reduce groundwater inflows and SMP monitoring and mitigation measures such as water treatment	Low
	Stream and wetland baseflow reduction due to groundwater drawdown (operational and construction)	Negligible (there are none identified that could be affected)	None	Negligible

Location	Type of effect	Magnitude of unmitigated effect	Recommended mitigation and monitoring	Residual effect
Mt Victoria	Damage to nearby buildings and structures from groundwater induced land settlement from temporary dewatering (operational and construction)	Negligible (however, mechanical settlement effects could arise)	GSMCP monitoring (because of mechanical settlement effects) and mitigation measures such as damage repair	Negligible
	Stream and wetland baseflow reduction due to groundwater drawdown (operational and construction)	Negligible	None	Negligible
	Groundwater or drainage water receiving surface water may be affected by groundwater contaminations being drawn into the groundwater drainage system which discharges stormwater (operational and construction)	Negligible	None	Negligible
	Groundwater or drainage water receiving surface water may be affected by high alkaline drainage water which discharge to stormwater (operational and construction)	High	Lining of ceiling arch to reduce groundwater inflows and SMP monitoring and mitigation measures such as water treatment	Low
Ruahine Street / Wellington Road	Damage to nearby buildings and structures from groundwater induced land settlement from temporary dewatering (construction only)	Low	GSMCP monitoring and mitigation measures such as damage repair	Negligible
	Damage to nearby buildings and structures from groundwater induced land settlement due to cut slopes (operation and construction)	Low	GSMCP monitoring and mitigation measures such as damage repair	Negligible
	Groundwater or drainage water receiving surface water may be affected by groundwater contaminations being drawn into the groundwater drainage system which discharges stormwater (operational and construction)	Negligible	None	None
	Stream and wetland baseflow reduction due to groundwater drawdown at Ruahine wetland (construction only)	Moderate	Temporary spray irrigation during construction possible	Low
	Stream and wetland baseflow reduction due to groundwater drawdown at Ruahine wetland due to cut slopes (operational and construction)	High	Refer to ecology assessment	Refer to ecology assessment

ANNEXURE A: Figures and Maps







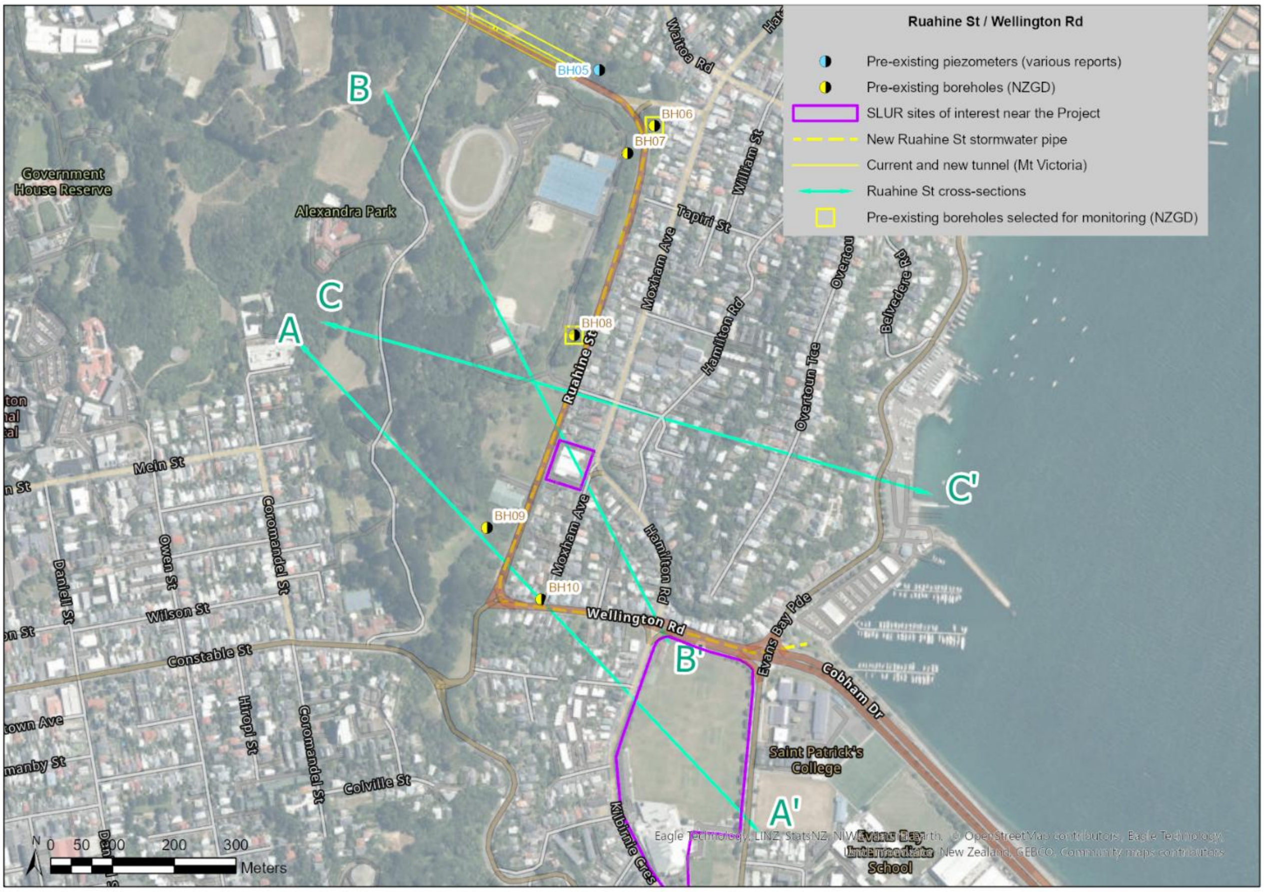


Figure 1 - Location of pre-existing and proposed new monitoring wells used for ongoing groundwater monitoring

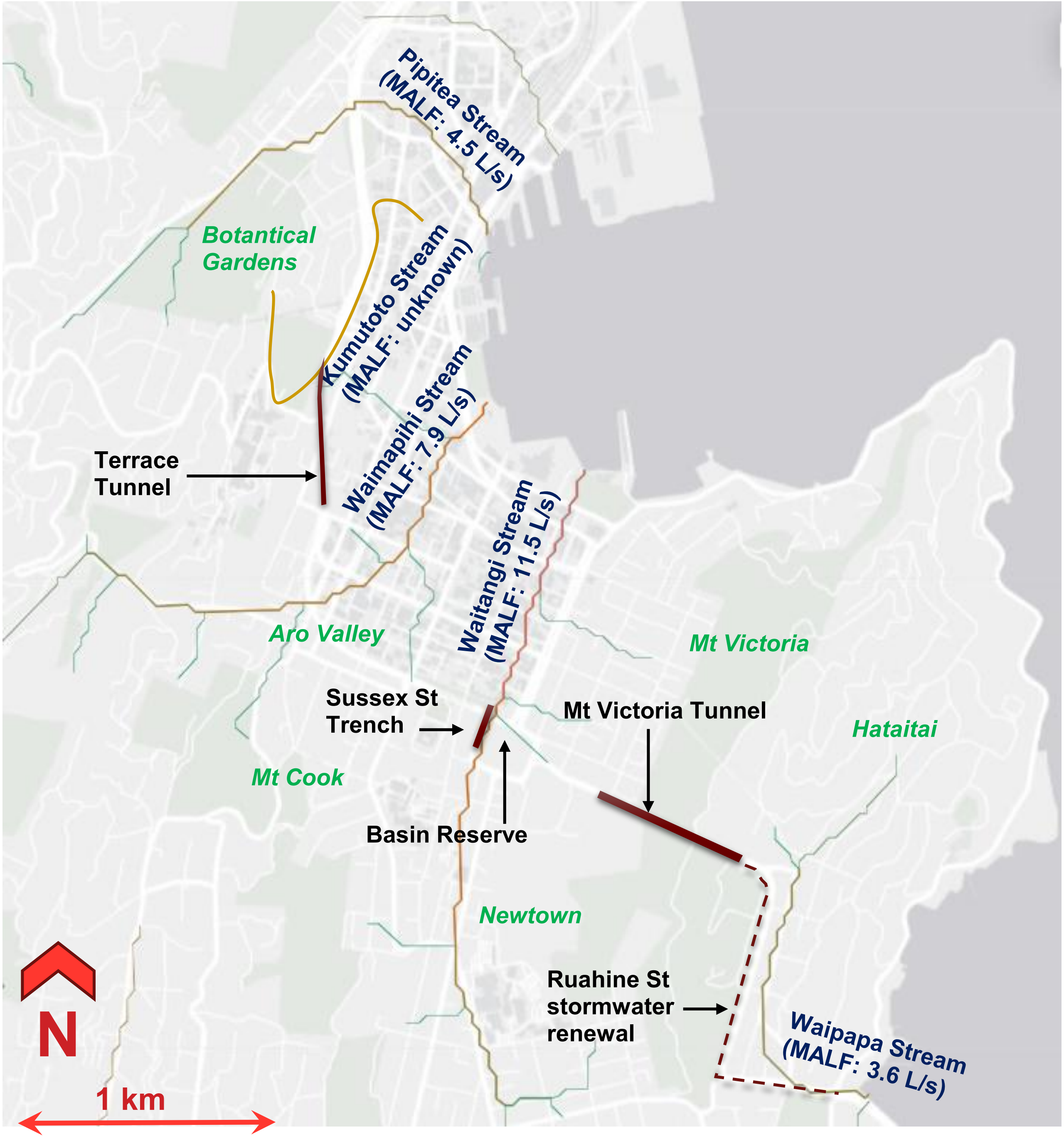


Figure 2 - Streams near the Project area (locations indicative)

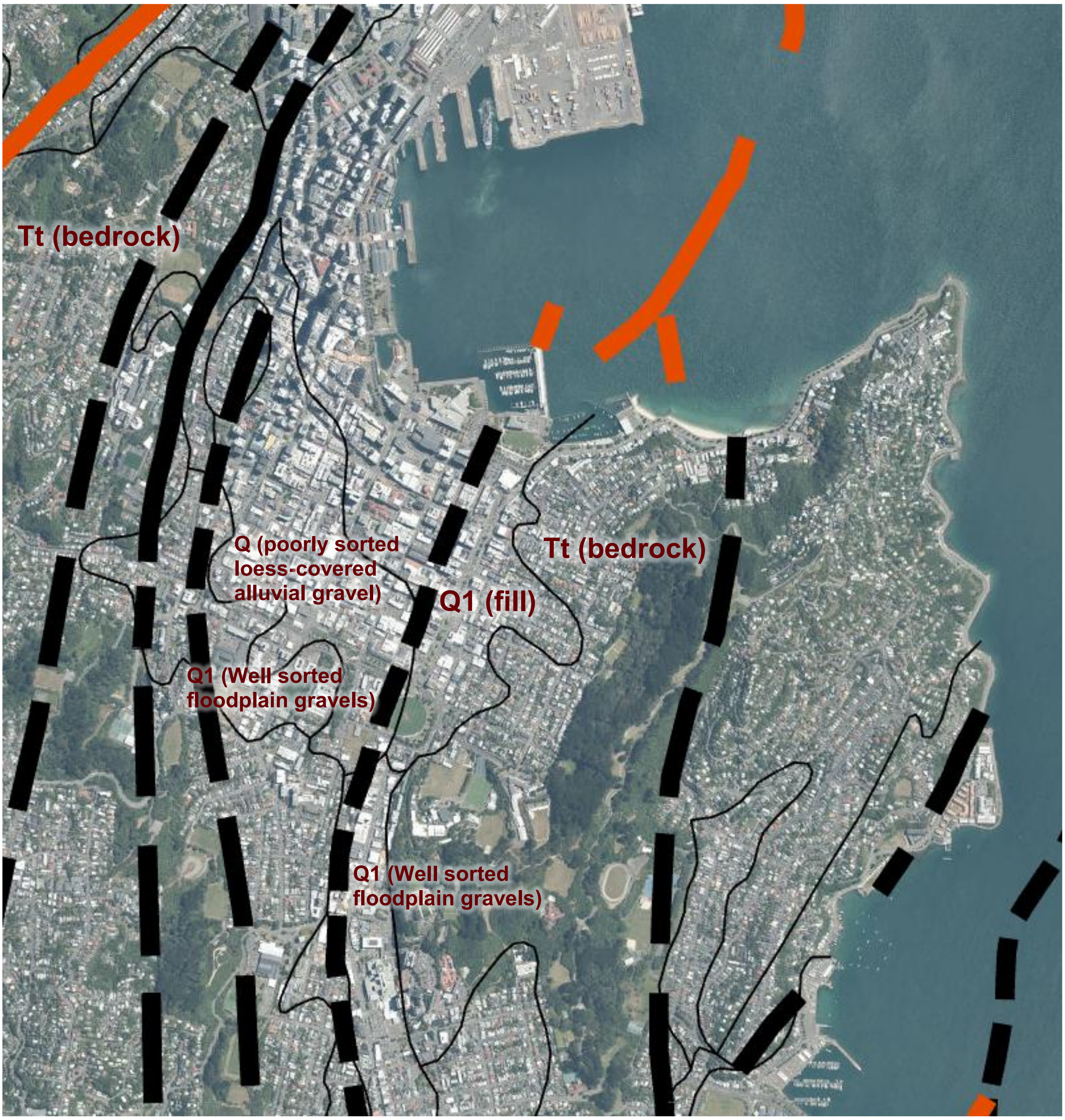


Figure 3 - Geological map and fault lines

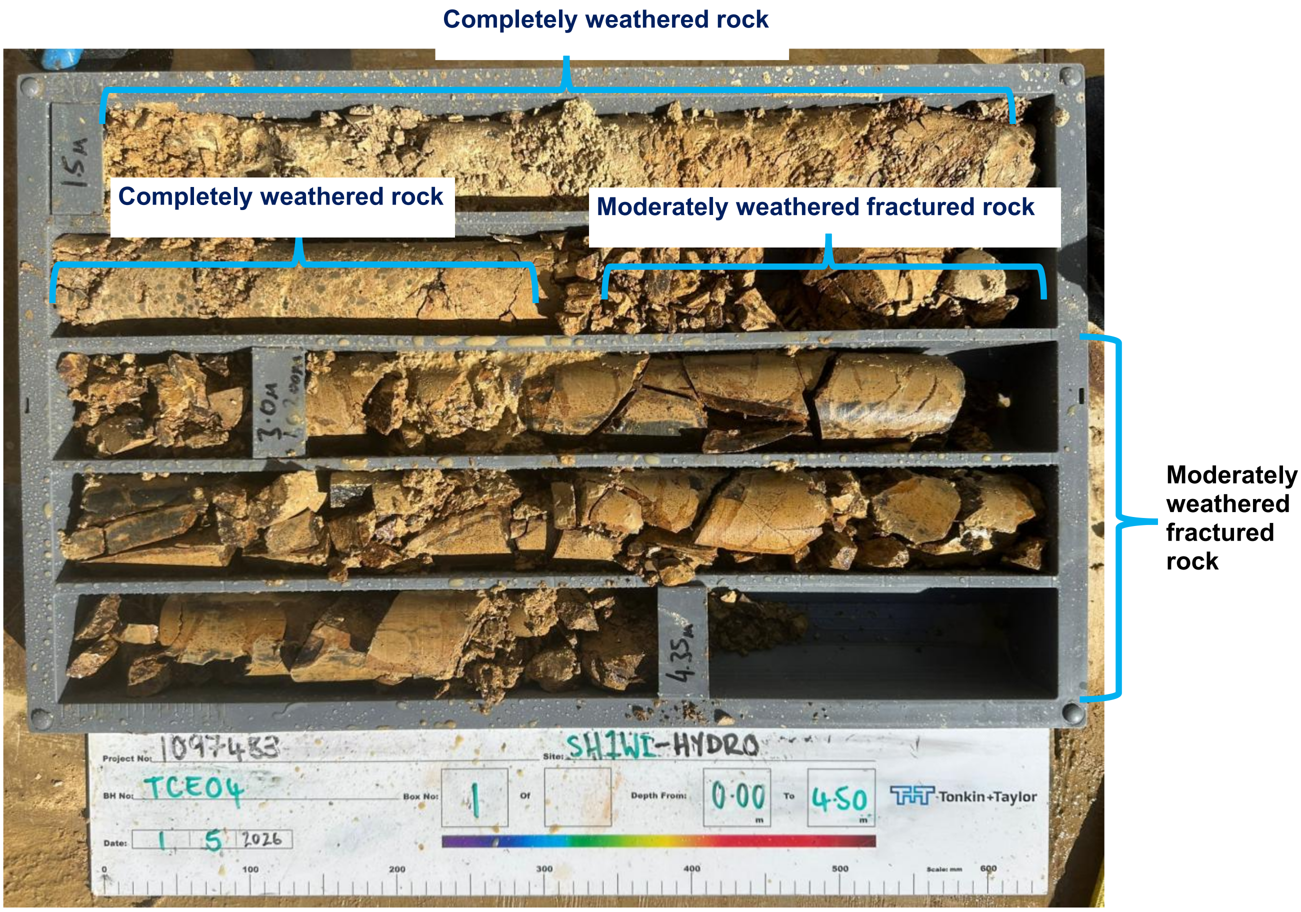


Figure 4 - Core photo of recently drilled geotechnical test hole TCE04 at the Terrace Tunnel

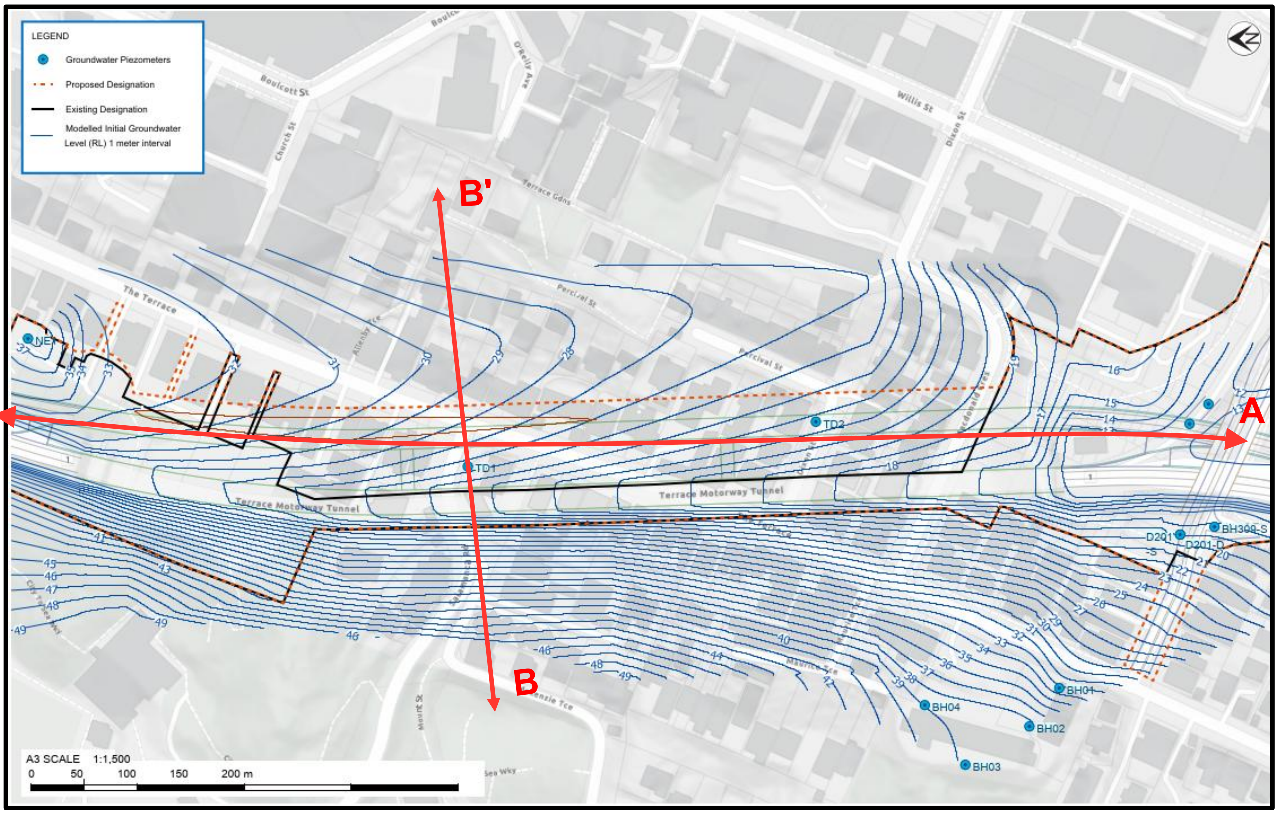


Figure 5 - indicative groundwater level contour map –Terrace Tunnel

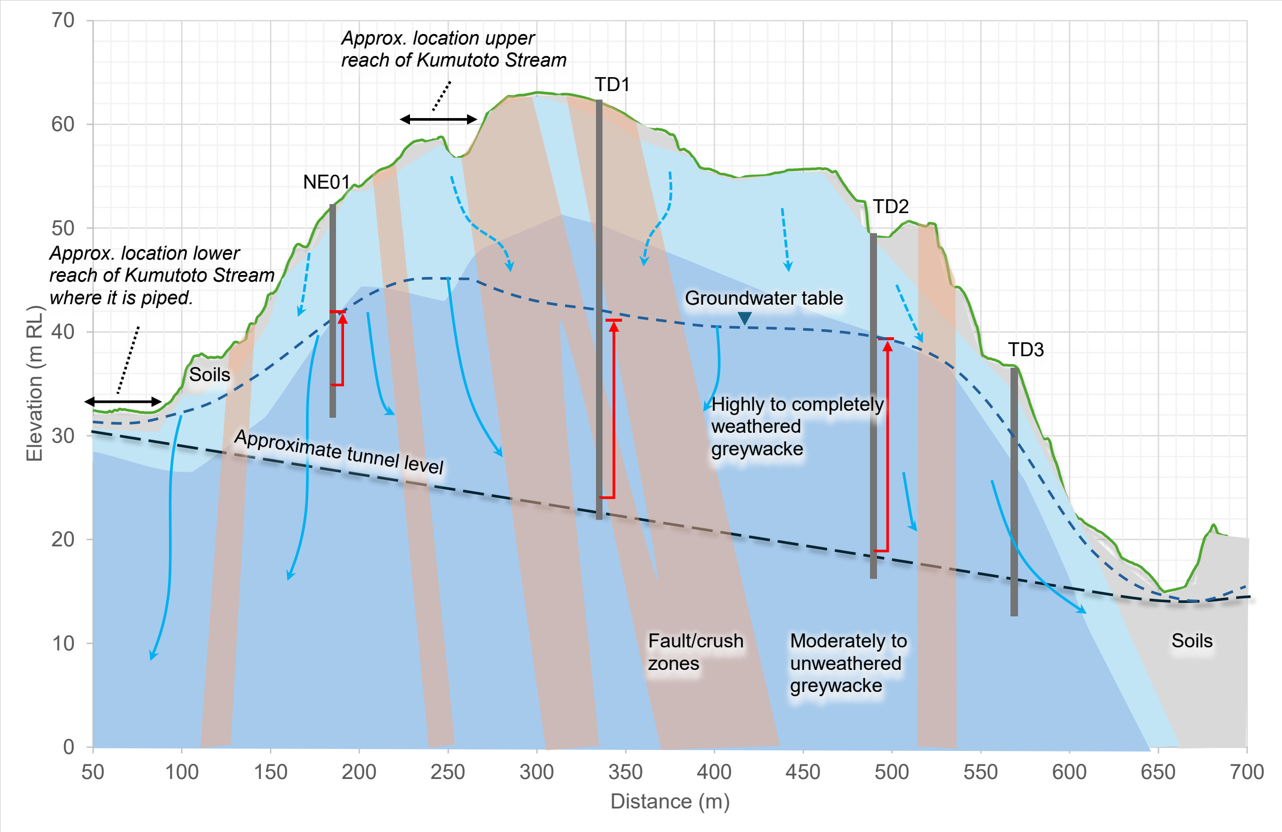


Figure 6 - Hydrogeological Conceptual Model cross section A-A' for Terrace Tunnel

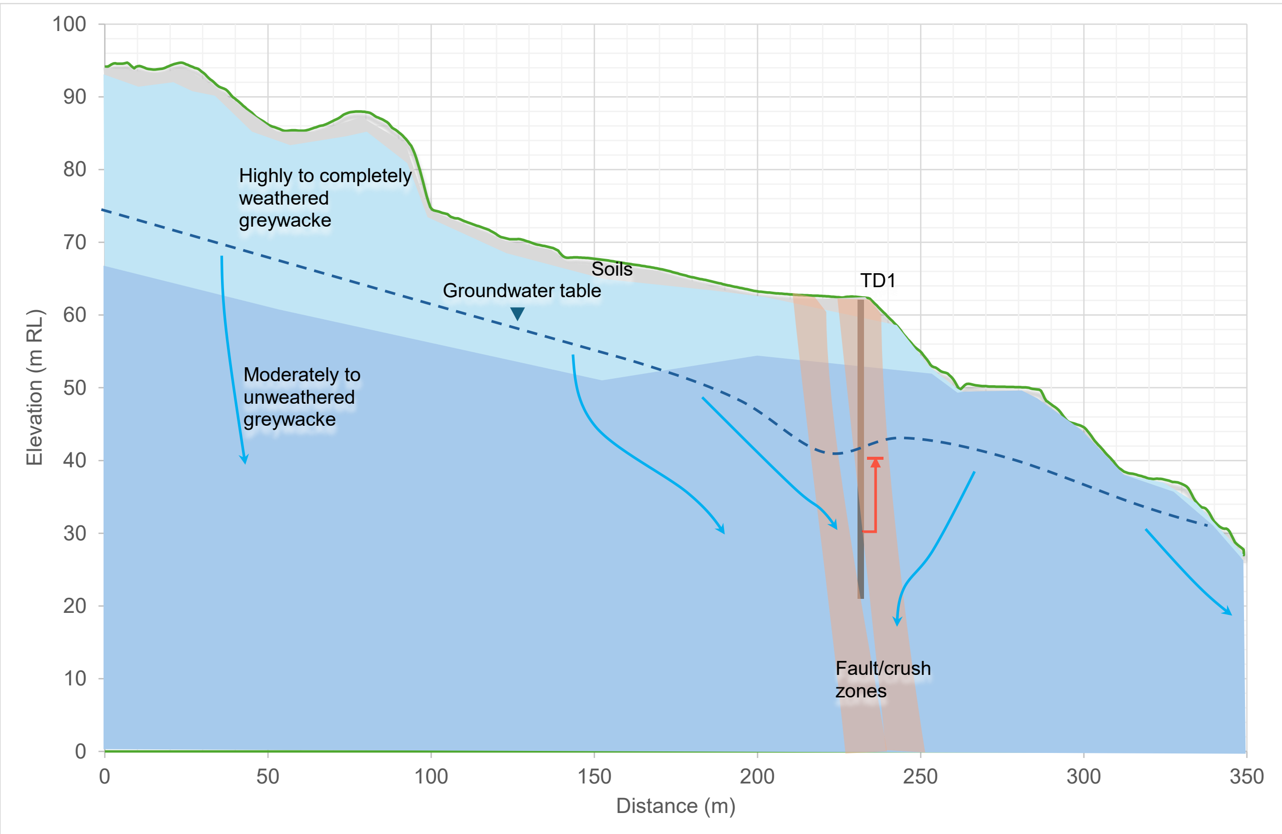


Figure 7 - Hydrogeological Conceptual Model cross section B-B' for Terrace Tunnel

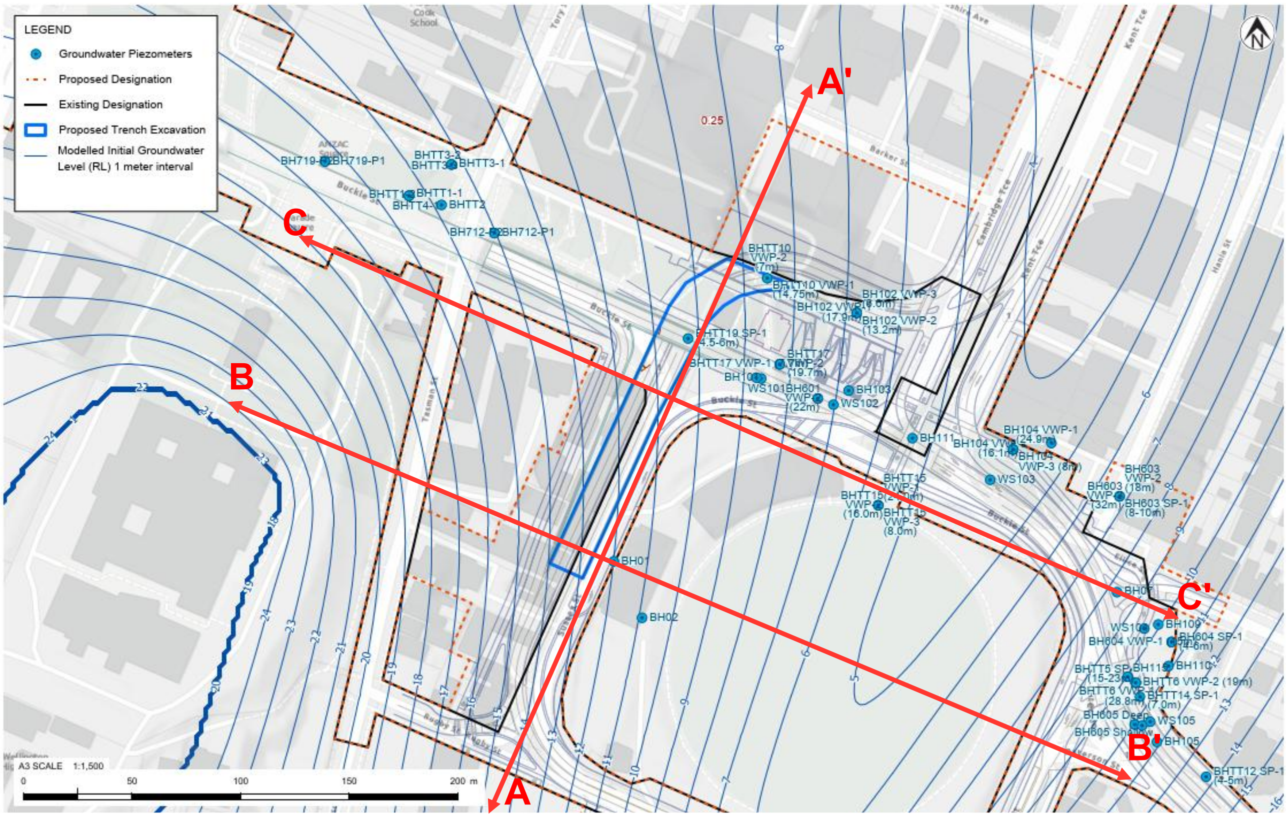


Figure 8 - indicative groundwater level contour map – Basin Reserve

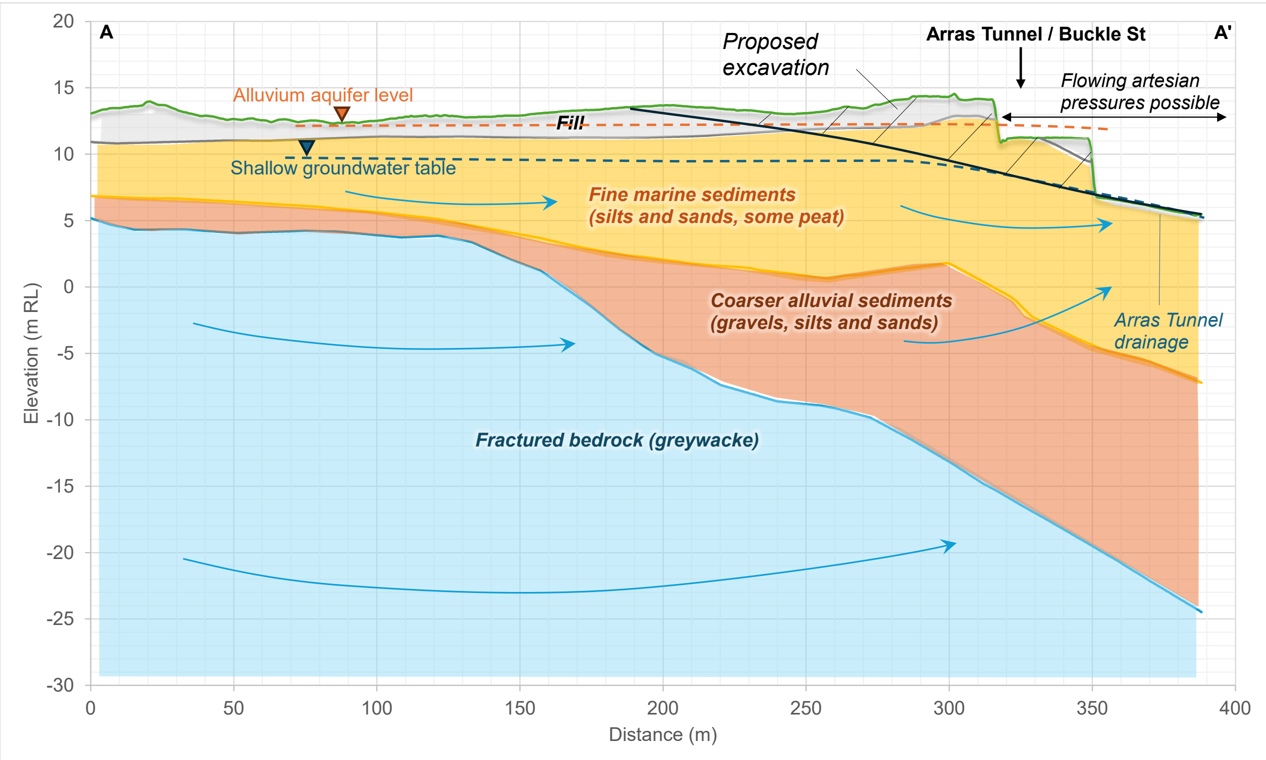


Figure 9 - Hydrogeological Conceptual Model cross section A-A’ for Basin Reserve

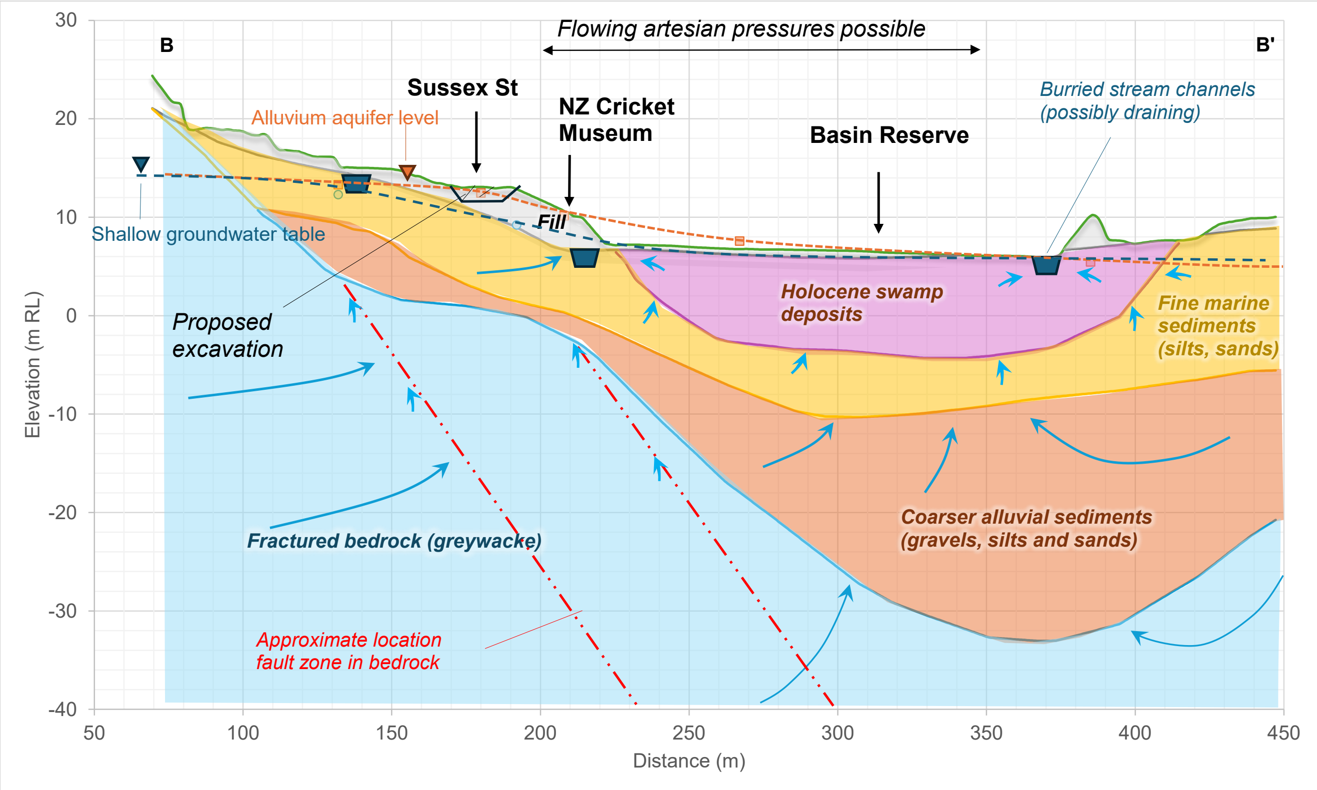


Figure 10 - Hydrogeological Conceptual Model cross section B-B' for Basin Reserve

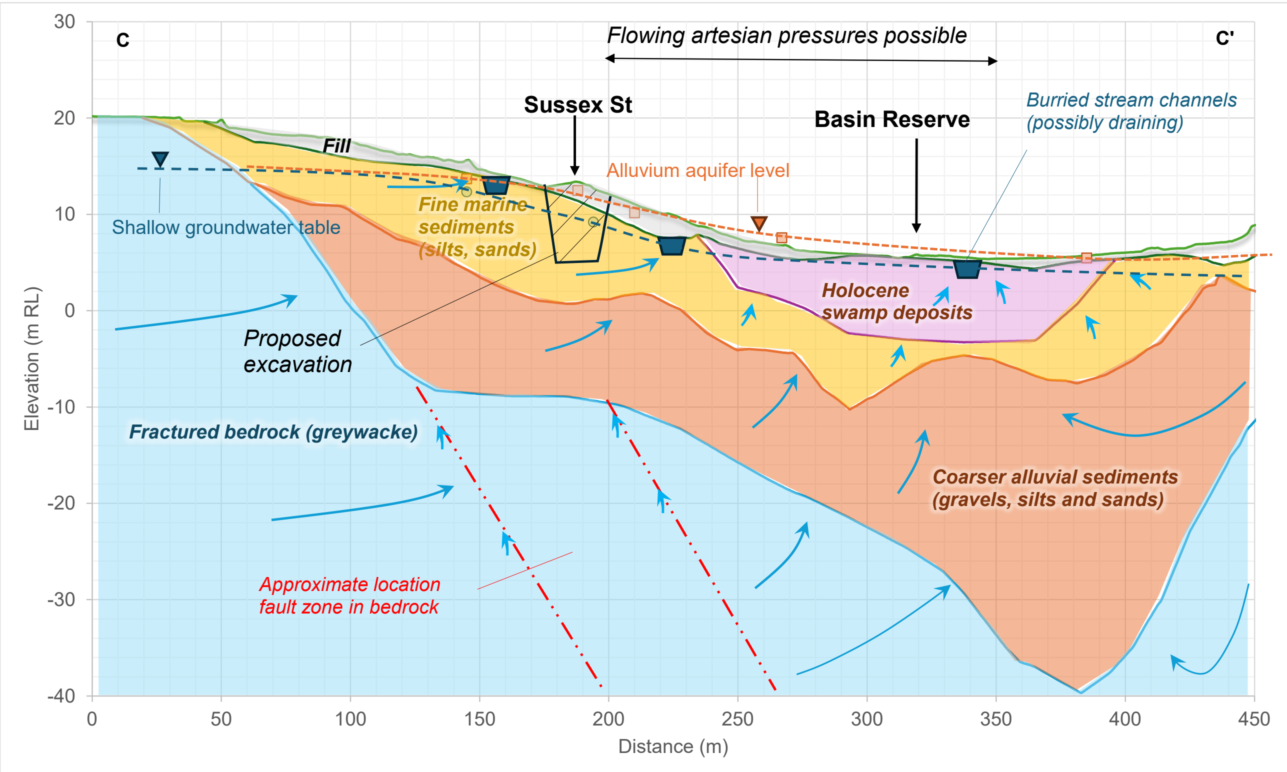


Figure 11 - Hydrogeological Conceptual Model cross section C-C' for Basin Reserve

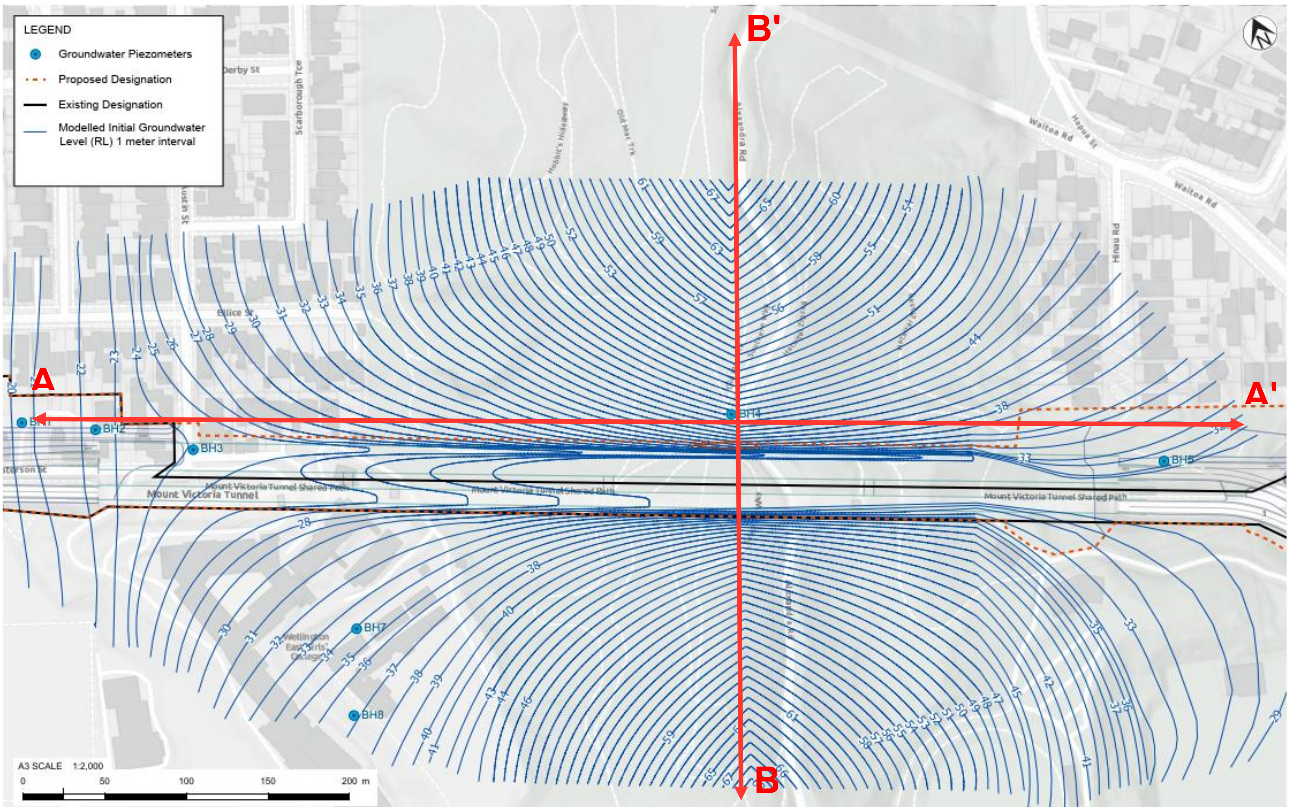


Figure 12 - indicative groundwater level contour map – Mt Victoria

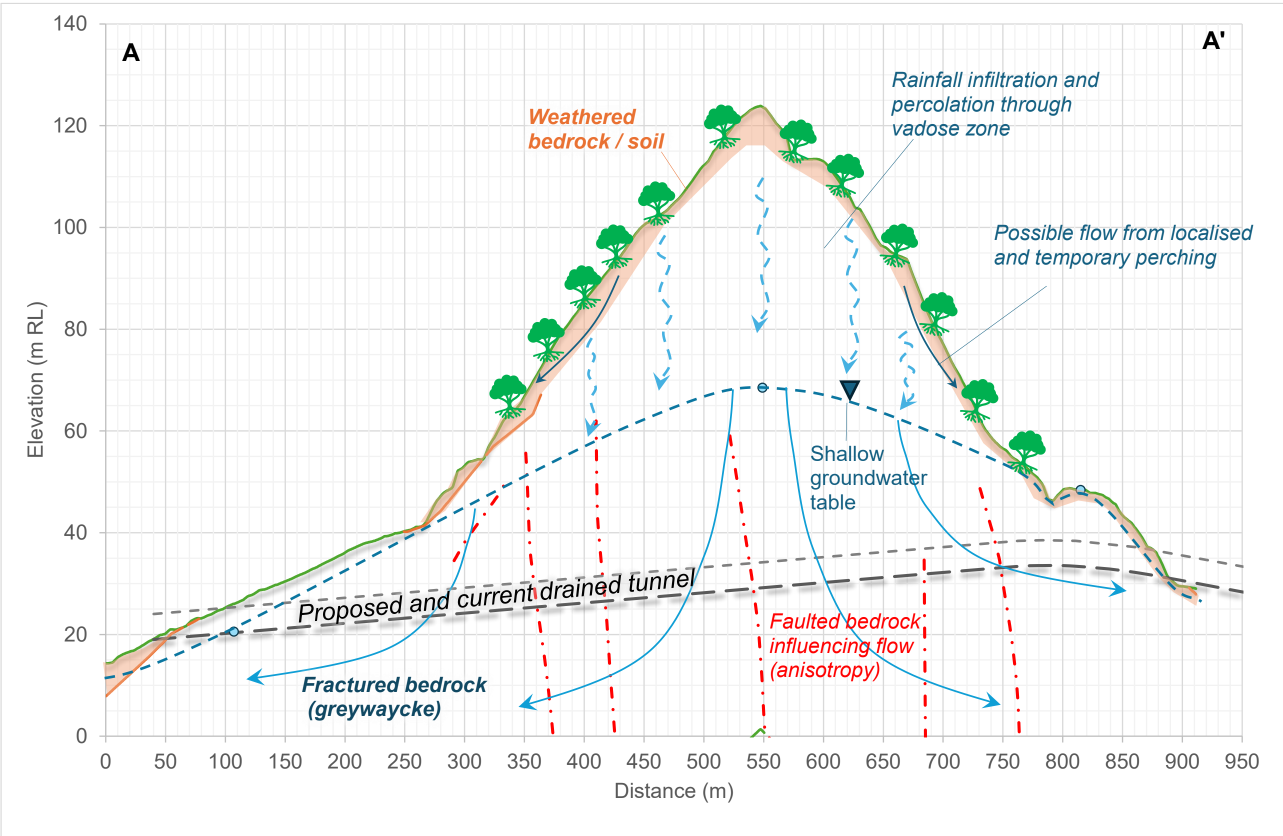


Figure 13 - Hydrogeological Conceptual Model cross section A-A' for Mt Victoria

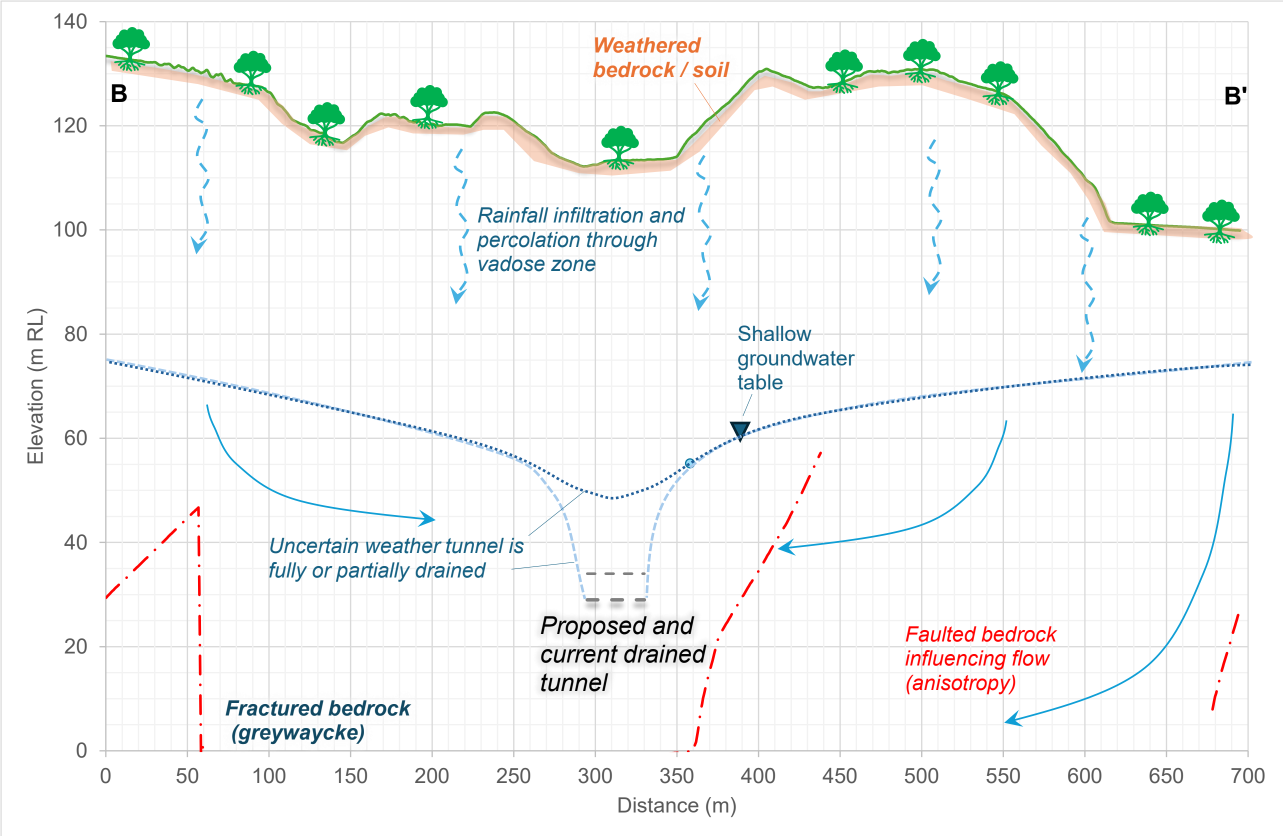


Figure 14 - Hydrogeological Conceptual Model cross section B-B' for Mt Victoria

Figure 15 - Kumutoto Stream and stormwater network near northern Terrace Tunnel portal

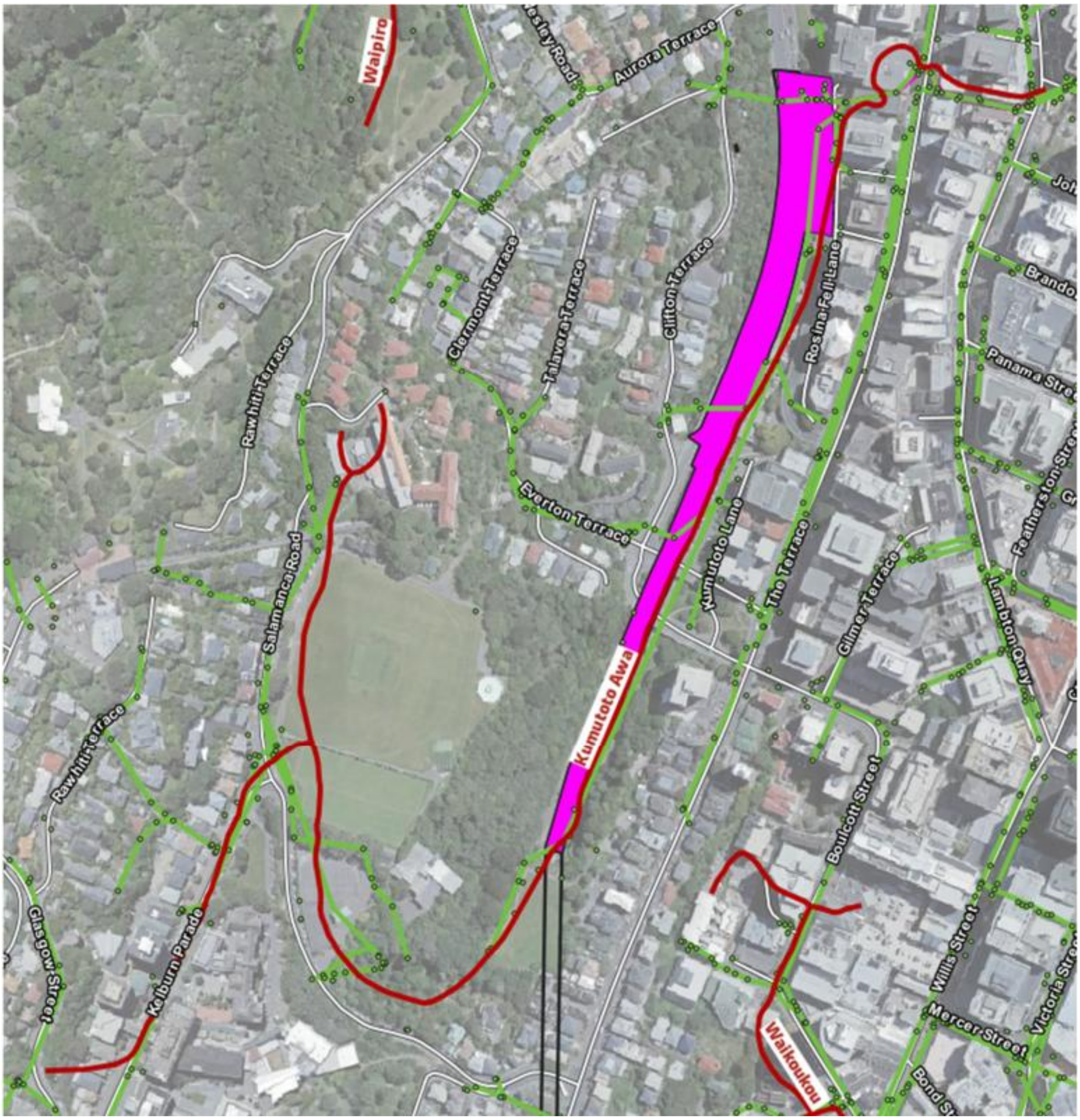


Figure 16 - Indicative location Kumutoto Stream near northern Terrace Tunnel portal

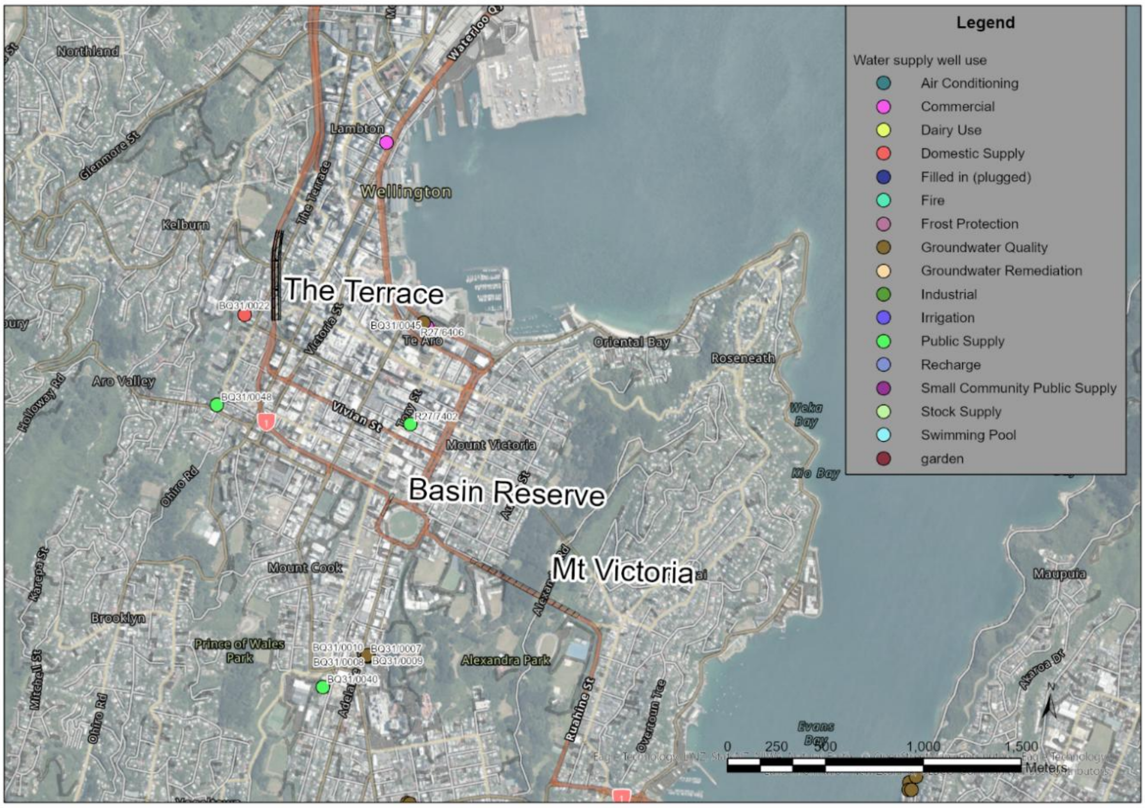


Figure 17 - Nearby water supply wells

Figure 18 - Identified wetland along Ruahine Street

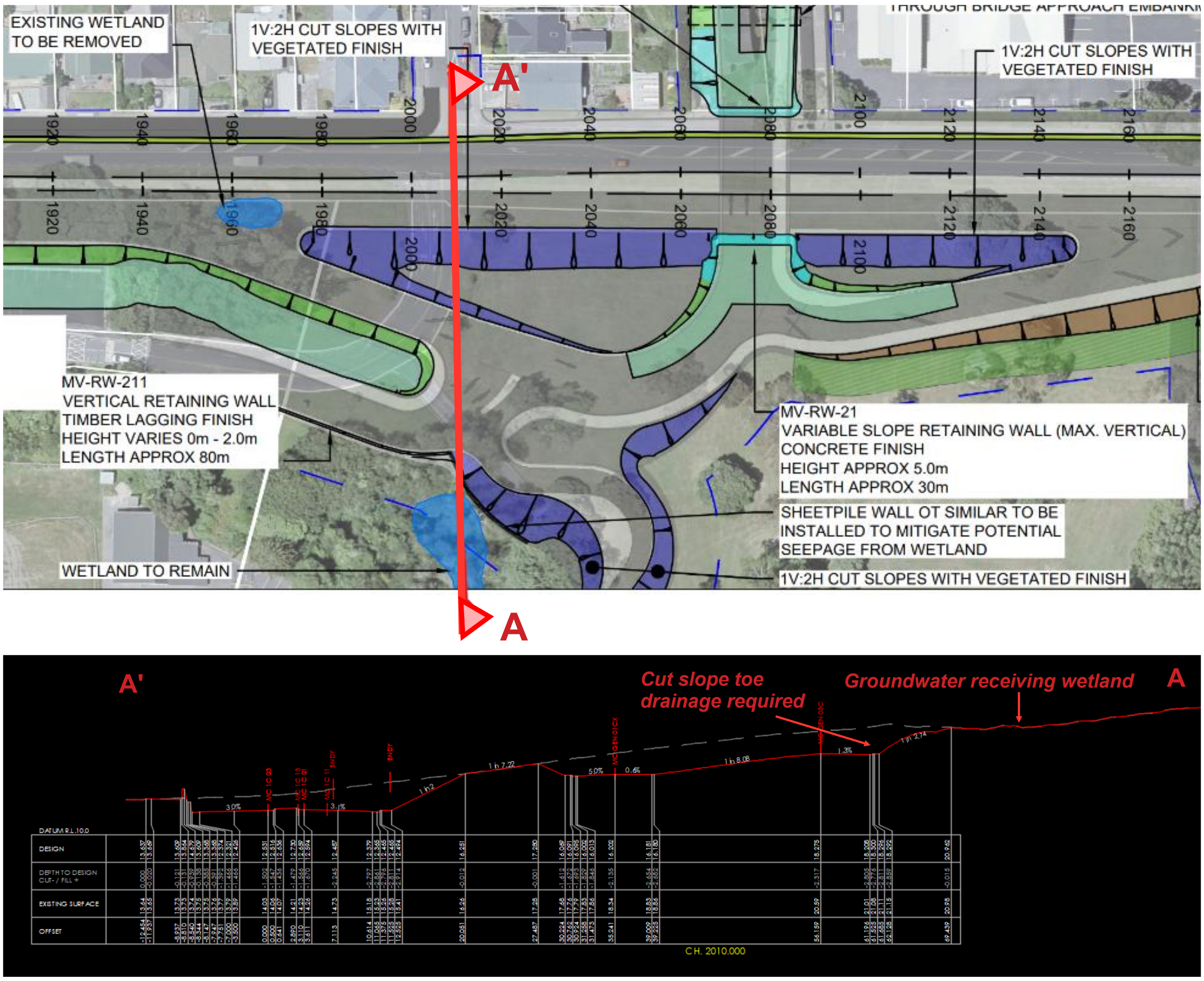


Figure 19 - Cut slope and drainage requirement near identified wetland along Ruahine Street

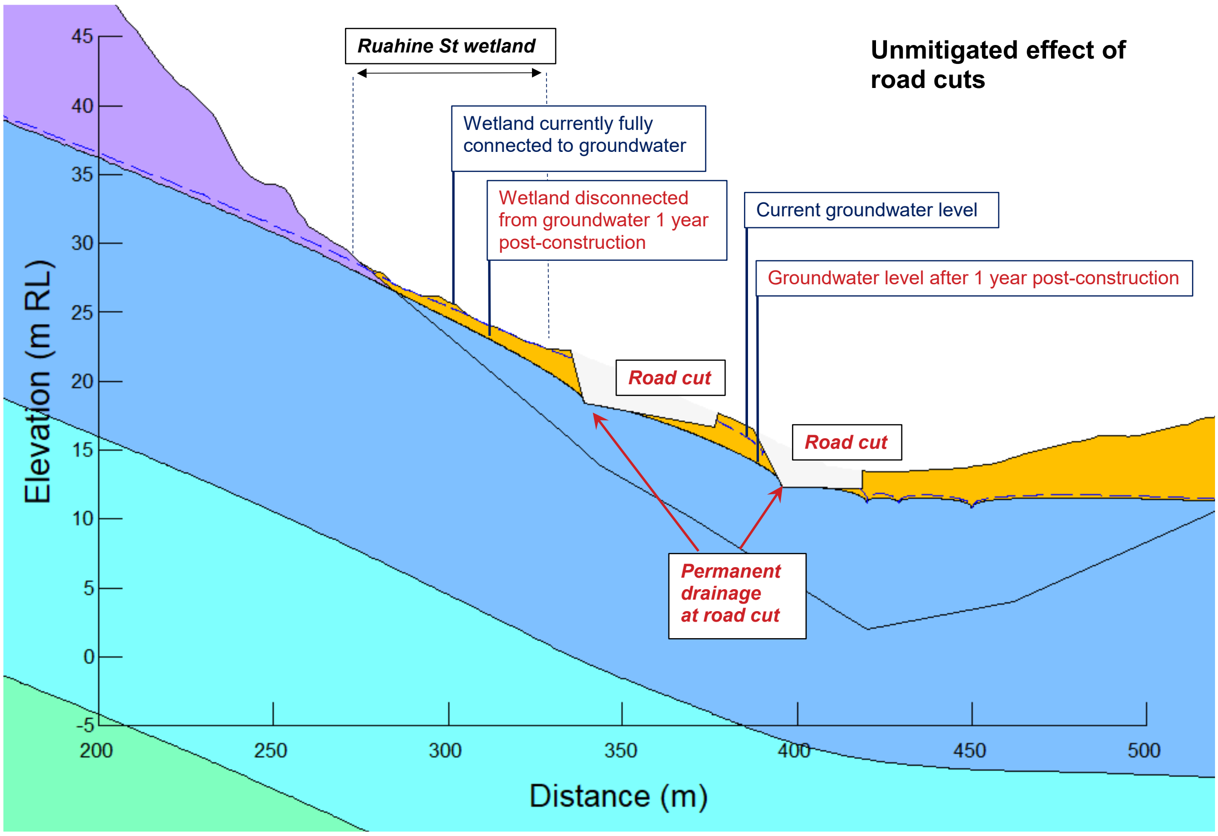


Figure 20 - SEEP/W cross-sectional modelling (from west to east along C-C', Figure 1) of cut slope drainage effects on Ruahine Street wetland – unmitigated effect

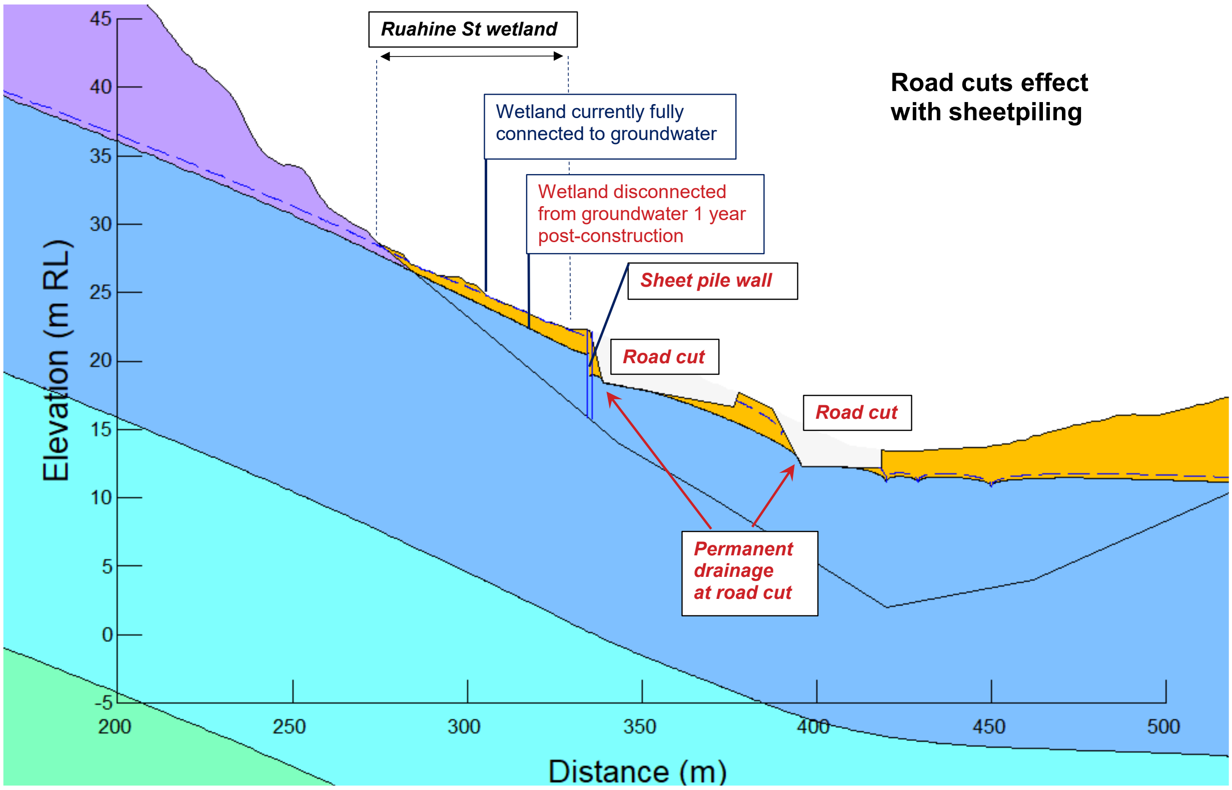


Figure 21 - SEEP/W cross-sectional modelling (from west to east along C-C', Figure 1) of cut slope drainage effects on Ruahine Street wetland – effect with sheet piling

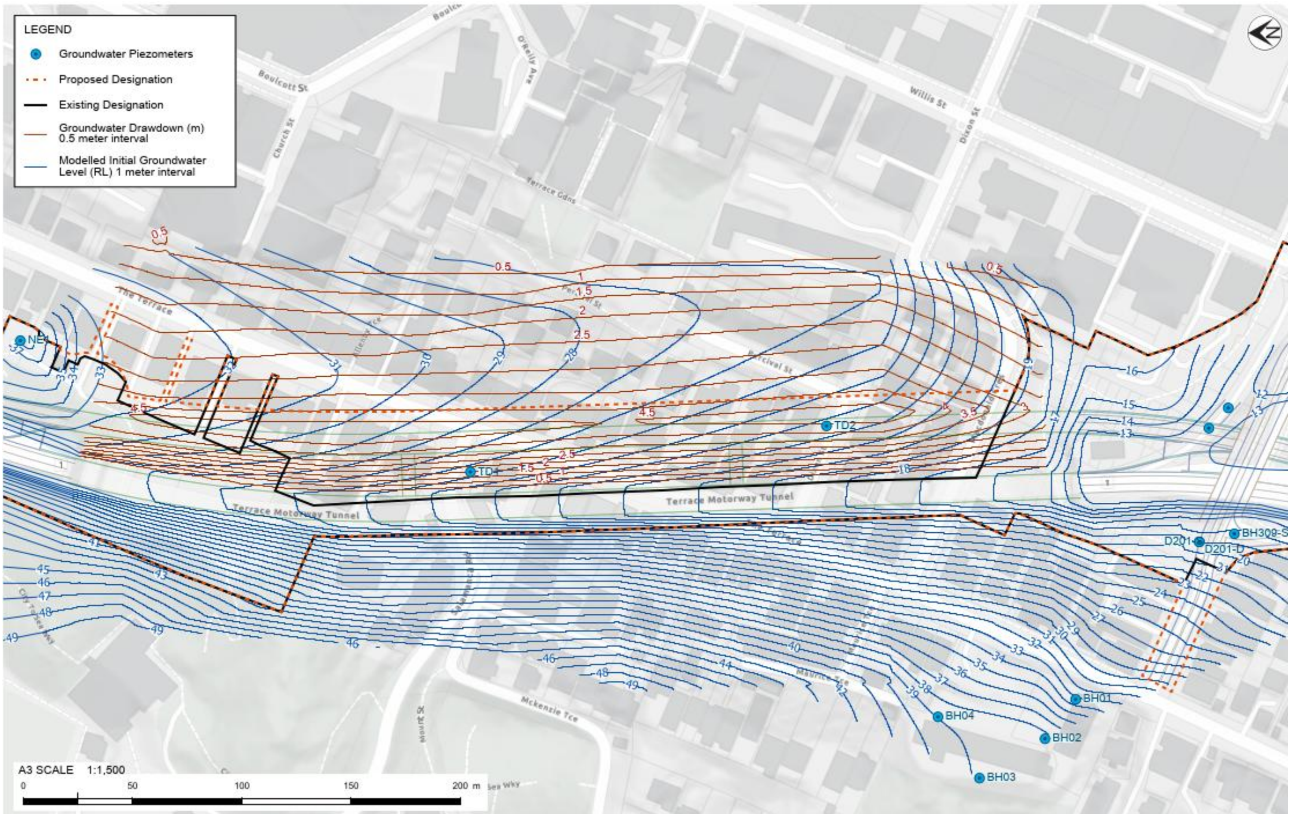


Figure 22 - Modelled groundwater drawdown at The Terrace tunnel caused by the Project

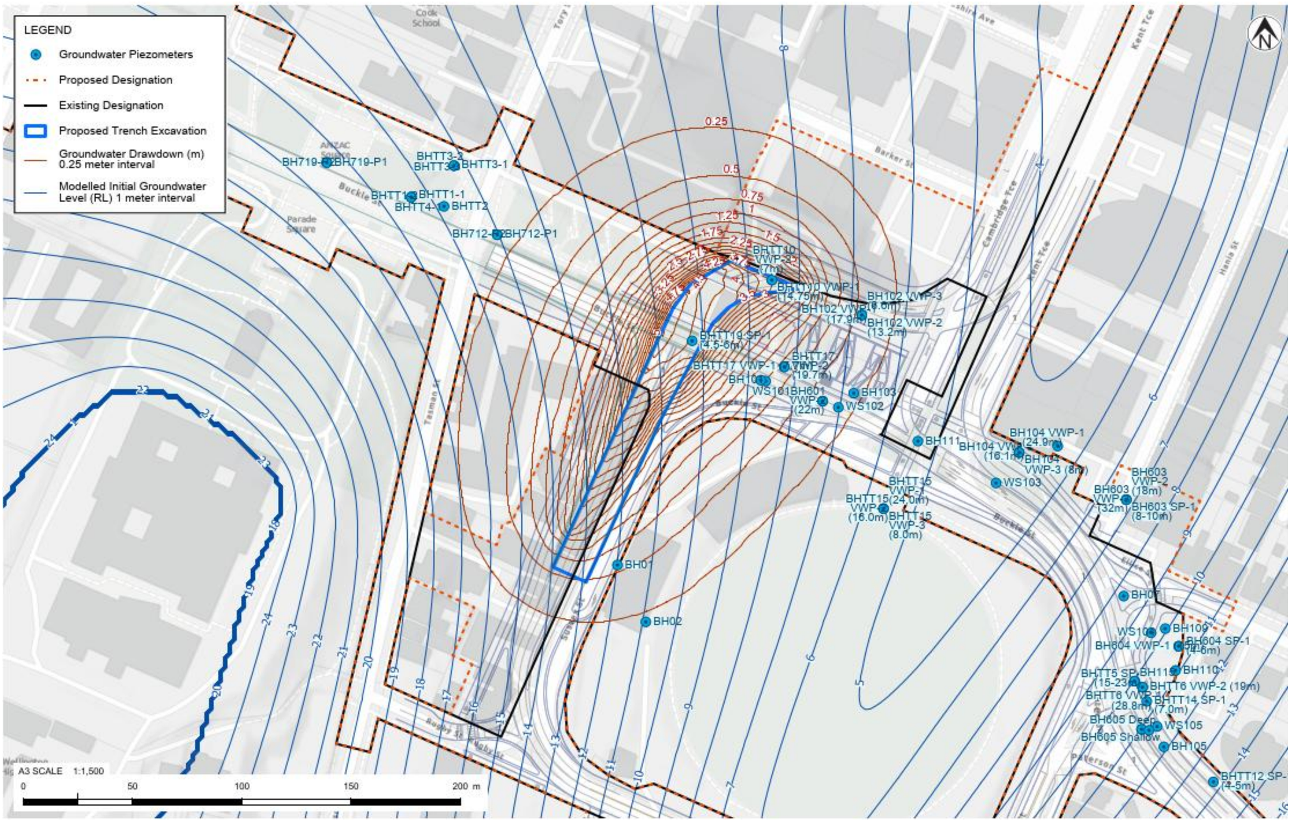


Figure 23 - Modelled groundwater drawdown at Basin Reserve caused by the Project

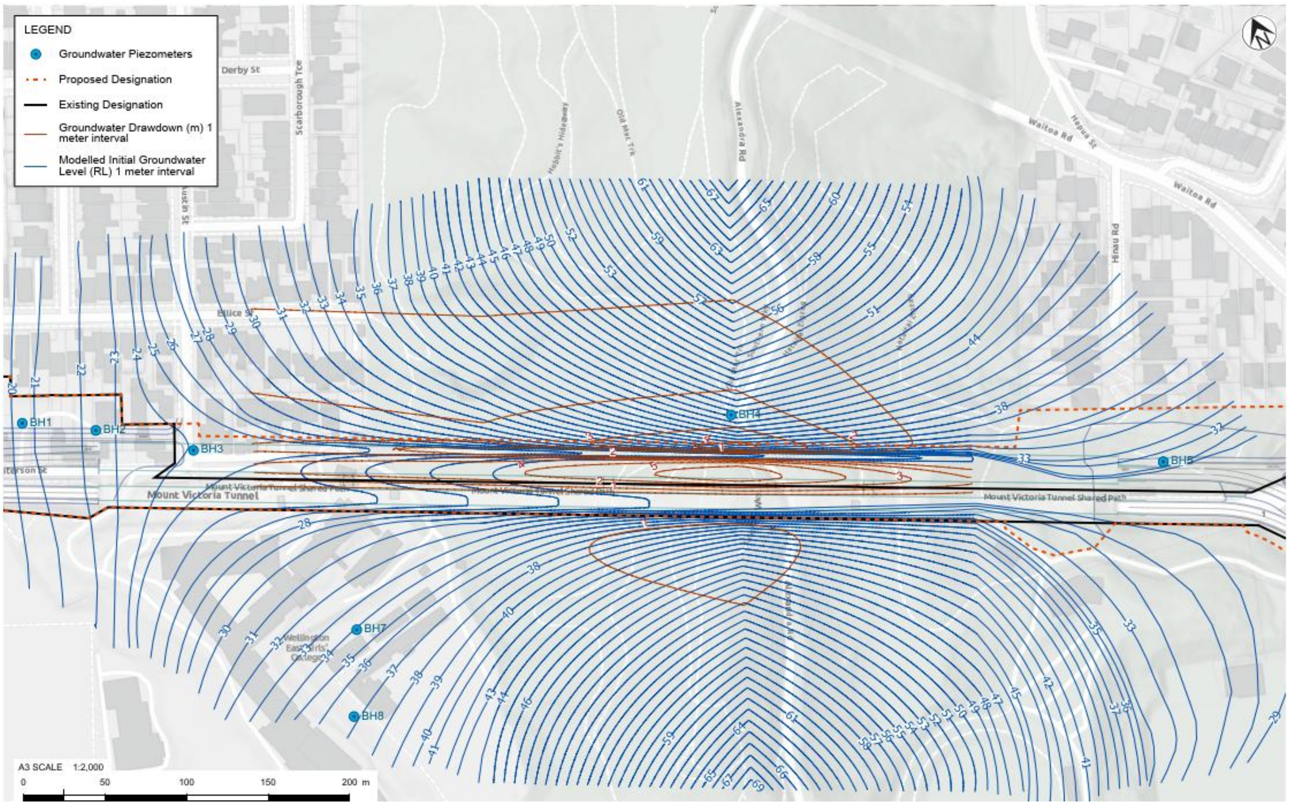


Figure 24 - Modelled groundwater drawdown at Mt Victoria tunnel caused by the Project

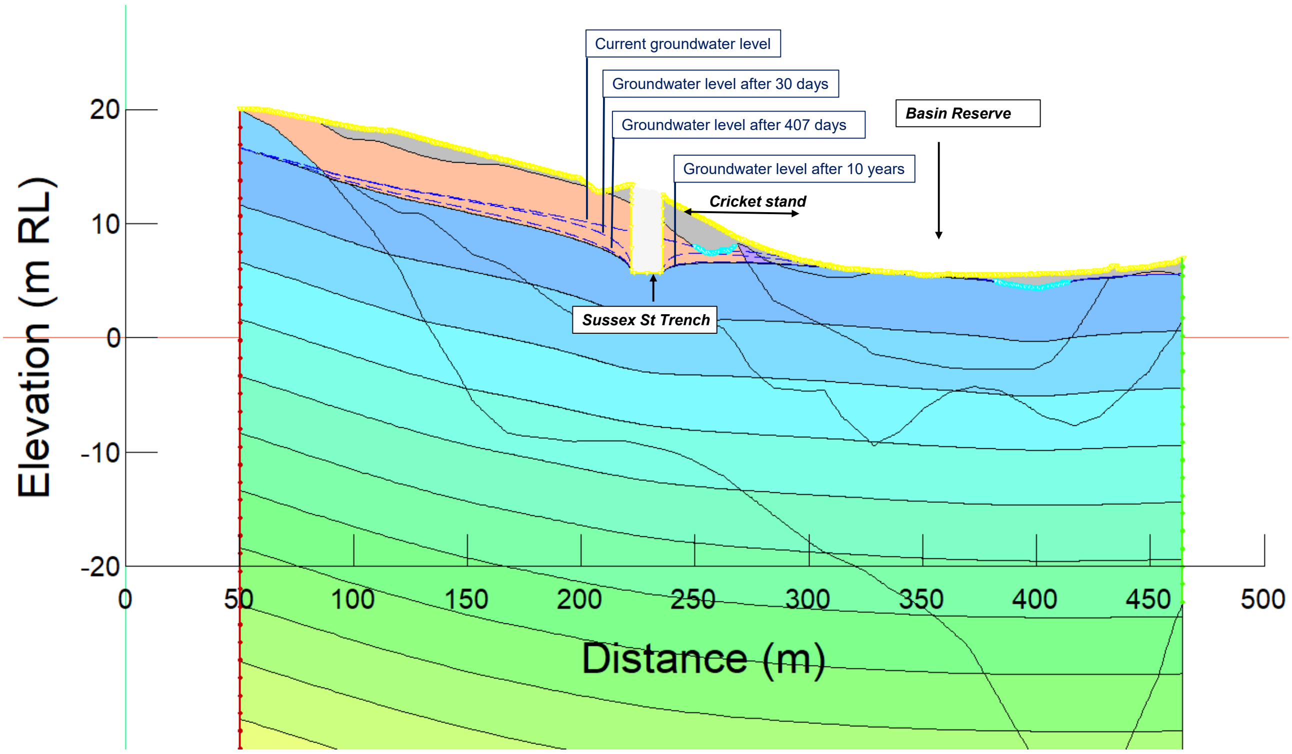


Figure 25 - SEEP/W 2D numerical groundwater modelling results at Basin Reserve

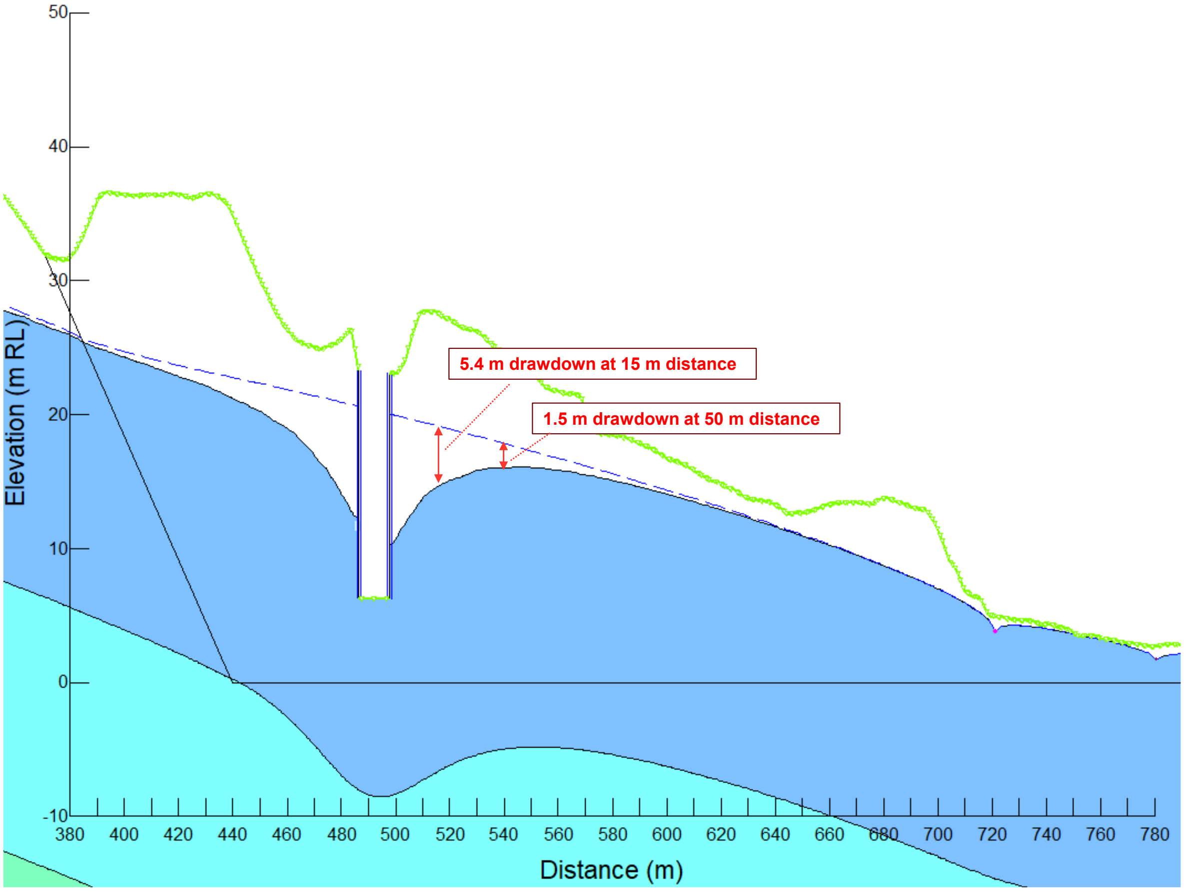


Figure 26 - SEEP/W 2D numerical groundwater modelling results at Ruahine Street / Wellington Rd

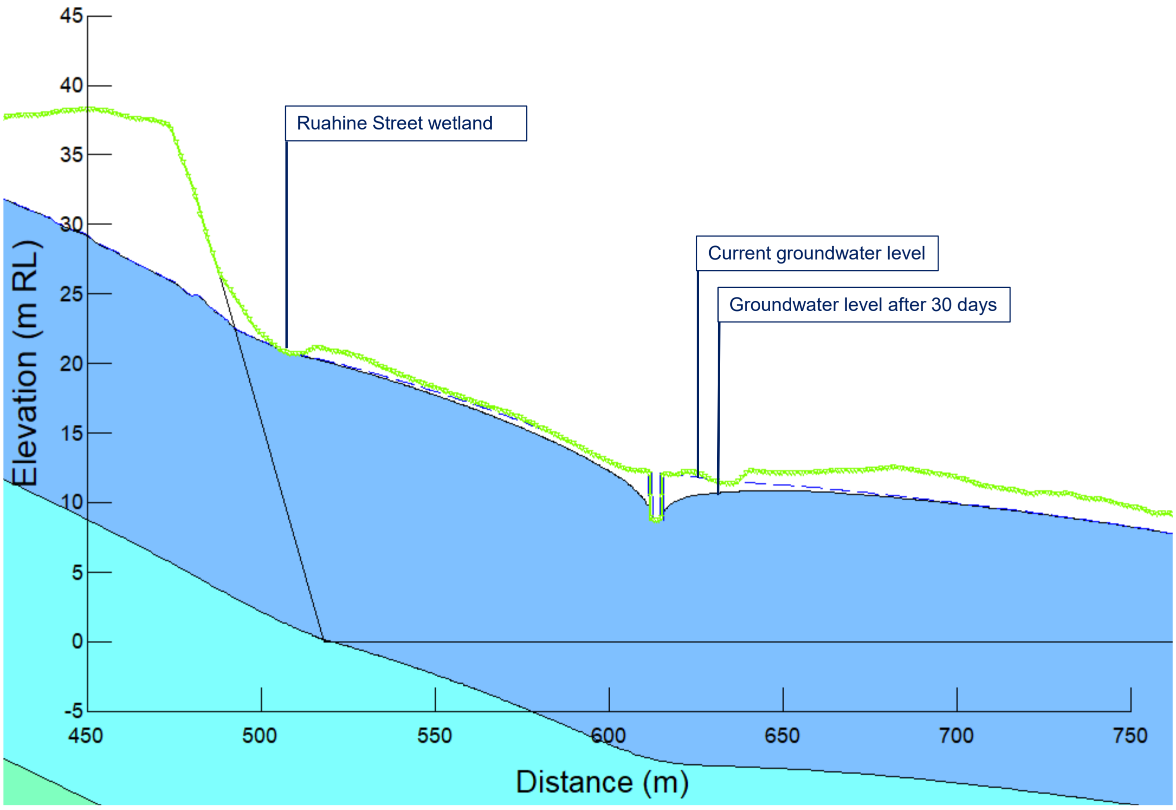


Figure 27 - SEEP/W 2D numerical groundwater modelling results showing impacts near Ruahine Street wetland

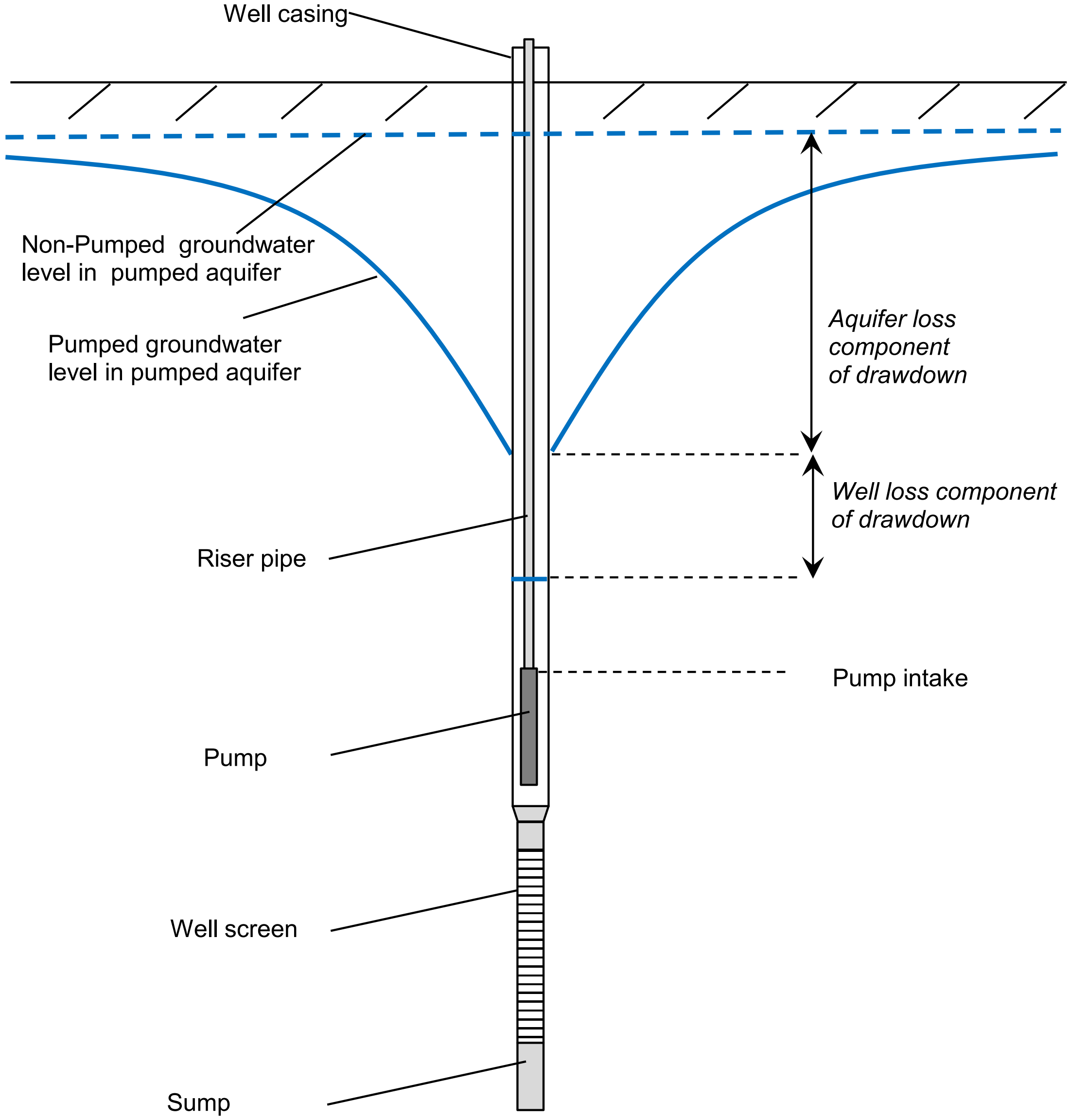


Figure 28 - Schematic overview of groundwater abstraction wells (subject well)

ANNEXURE B: Photographic observations during October 2025 site visits

Figure 29 - Groundwater ingress (seepage) and precipitate formatting at Mt Victoria Tunnel

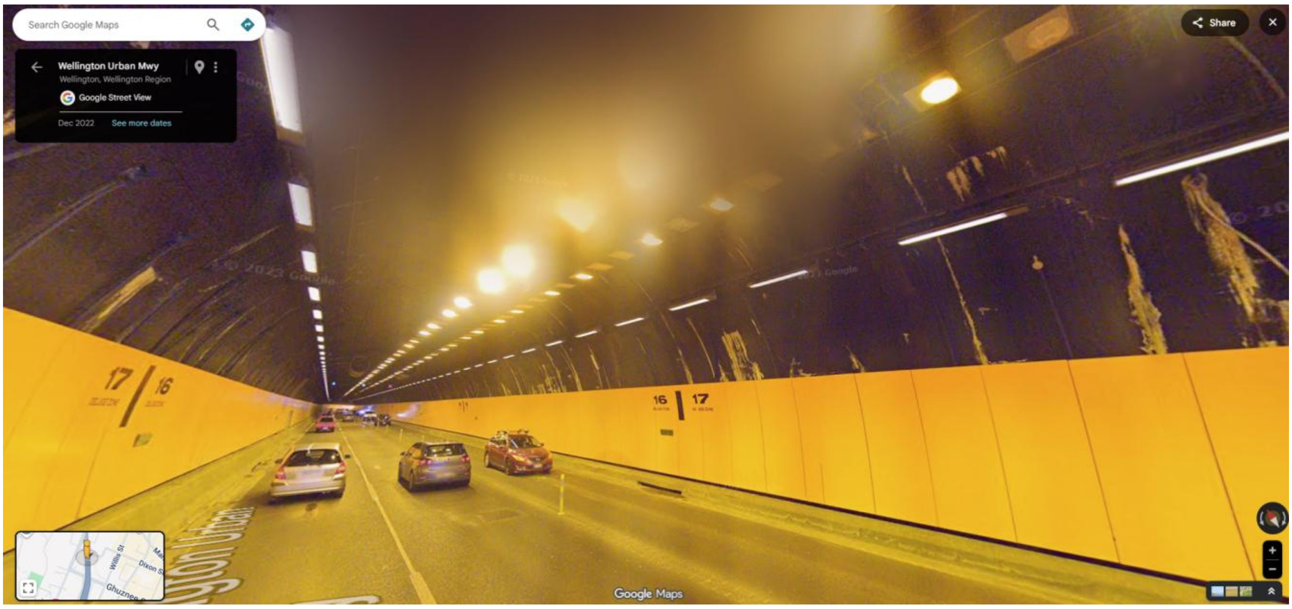


Figure 30 - Groundwater ingress (seepage) and precipitate forming at Terrace Tunnel

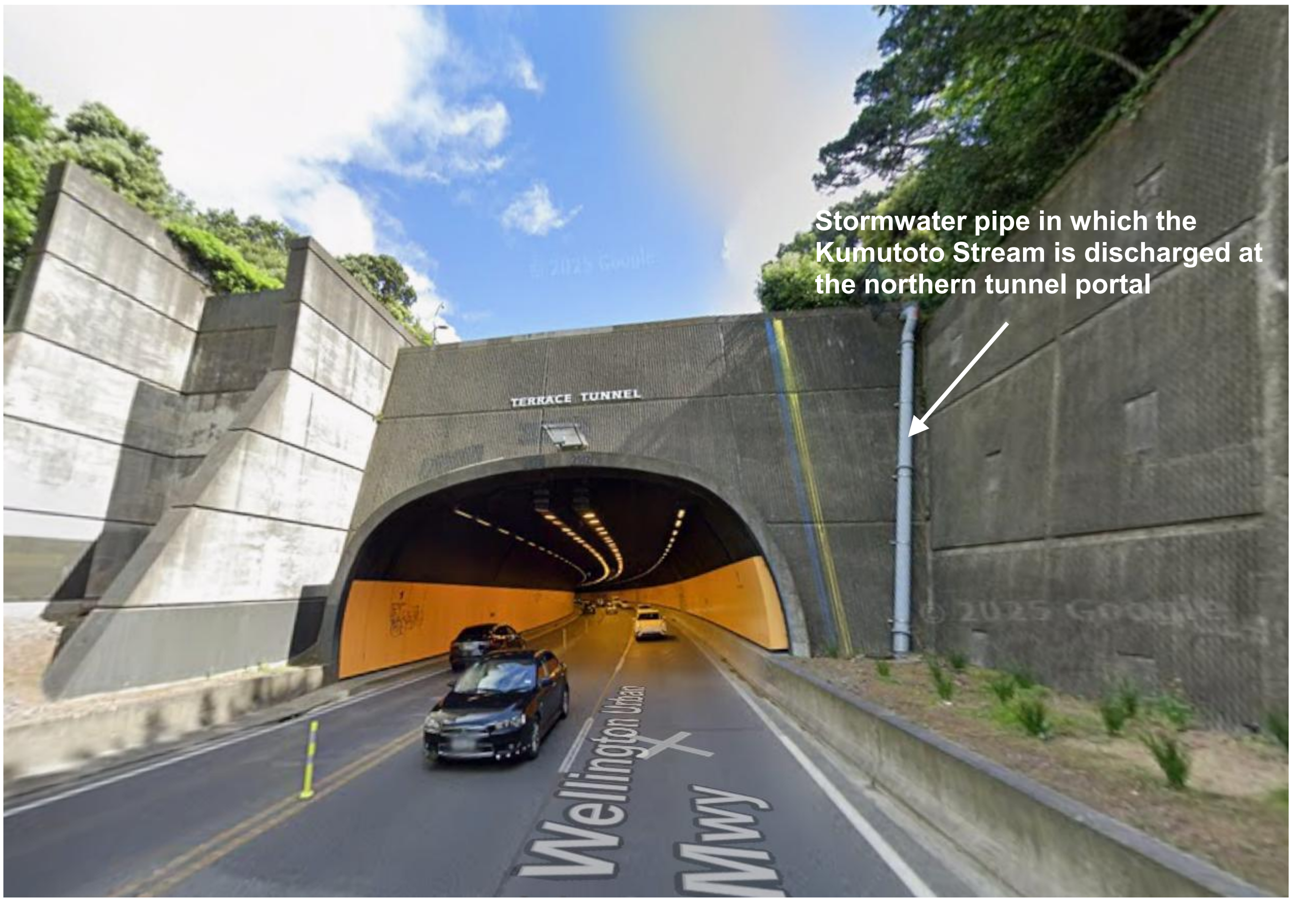


Figure 31 - Kumutoto Stream piped at northern portal of Terrace Tunnel

ANNEXURE C: Numerical groundwater modelling and modelling results

C1. Numerical groundwater modelling for Groundwater Assessment report

Three 2D-cross-sectional models have been developed for the Project to support the assessment of groundwater impacts caused by the construction and operation of the Project as follows:

- **Basin Reserve along cross-section C-C'** as indicated in Figure 8.
- **Ruahine Street along cross-section A-A'** as indicated in Figure 1.
- **Ruahine Street along cross-section B-B'** as indicated in Figure 1.
- **Ruahine Street along cross-section C-C'** as indicated in Figure 1.

The hydraulic conductivity values and boundary conditions are shown in the diagrams below (Figure 32 through to Figure 35). Each model is run as a transient (i.e., time-dependent) model which starts with a simulation of the current state (represented as average steady-state conditions) followed by 10 incrementally increase timesteps. The total modelling period is 10 years for the Basin Reserve (C-C') model and 31 days for the Ruahine Street (A-A') and Ruahine Street (B-B') models. The modelling period for the Ruahine Street (C-C') model is 10 years.

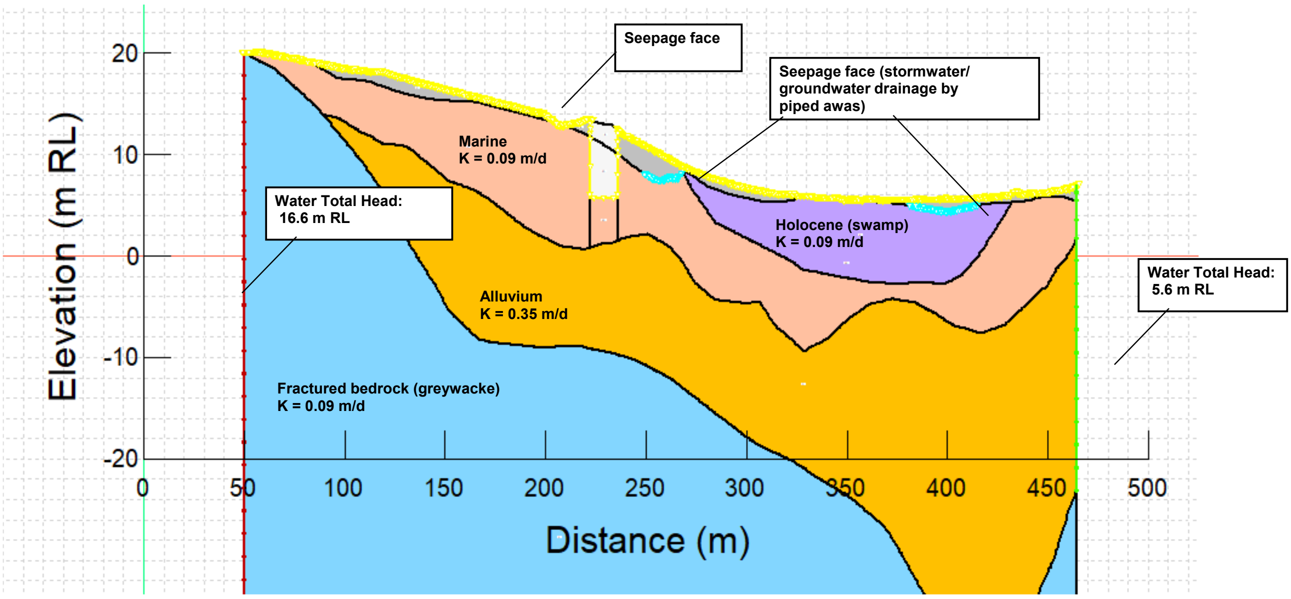


Figure 32 - SEEP/W 2D numerical groundwater modelling setup for Basin Reserve (C-C') model

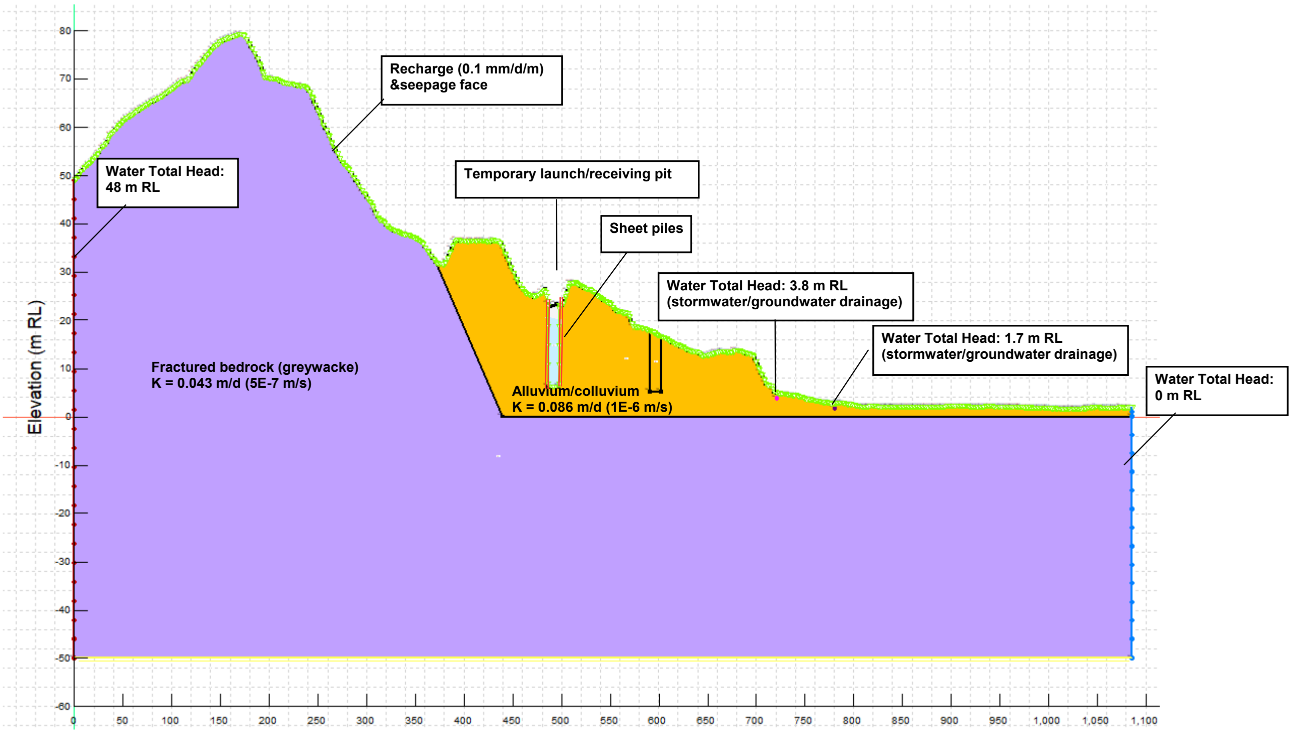


Figure 33 - SEEP/W 2D numerical groundwater modelling setup for Ruahine St (A-A') model

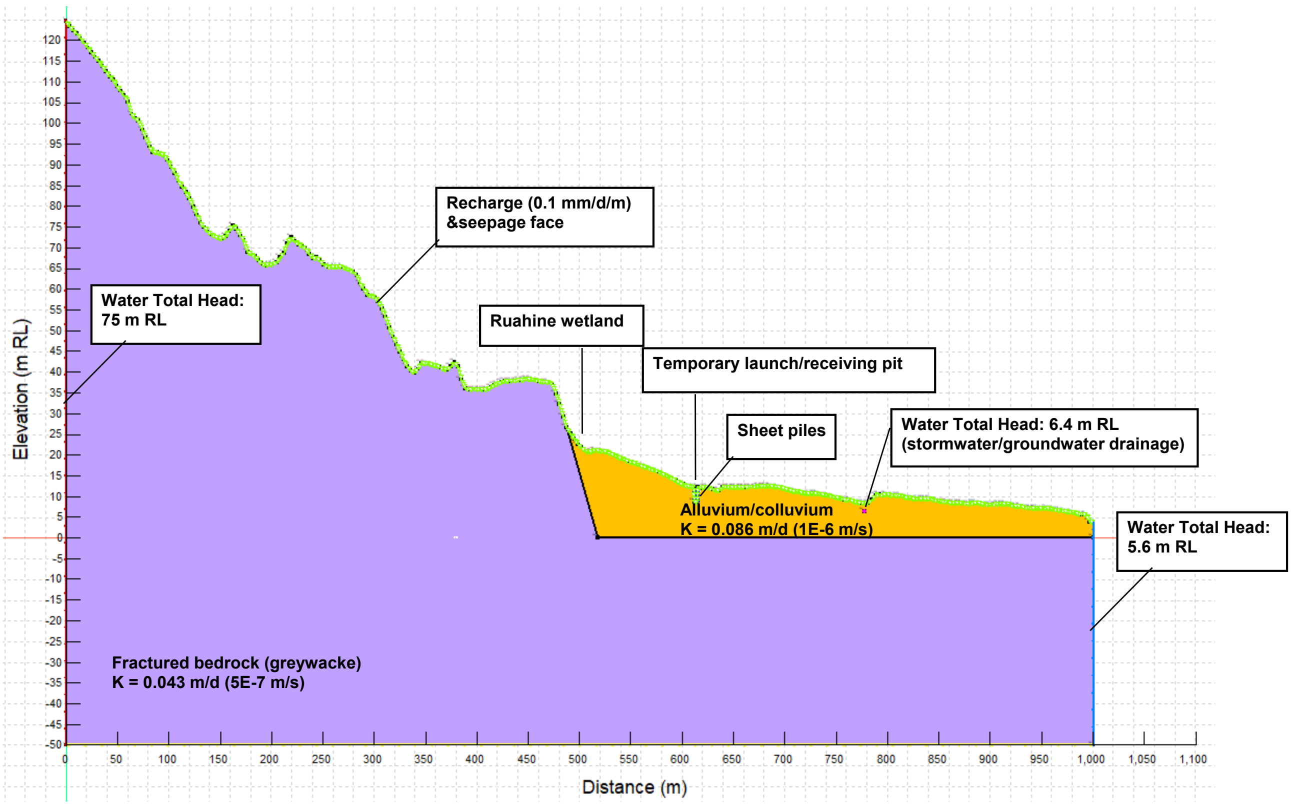


Figure 34 - SEEP/W 2D numerical groundwater modelling setup for Ruahine St (B-B') model

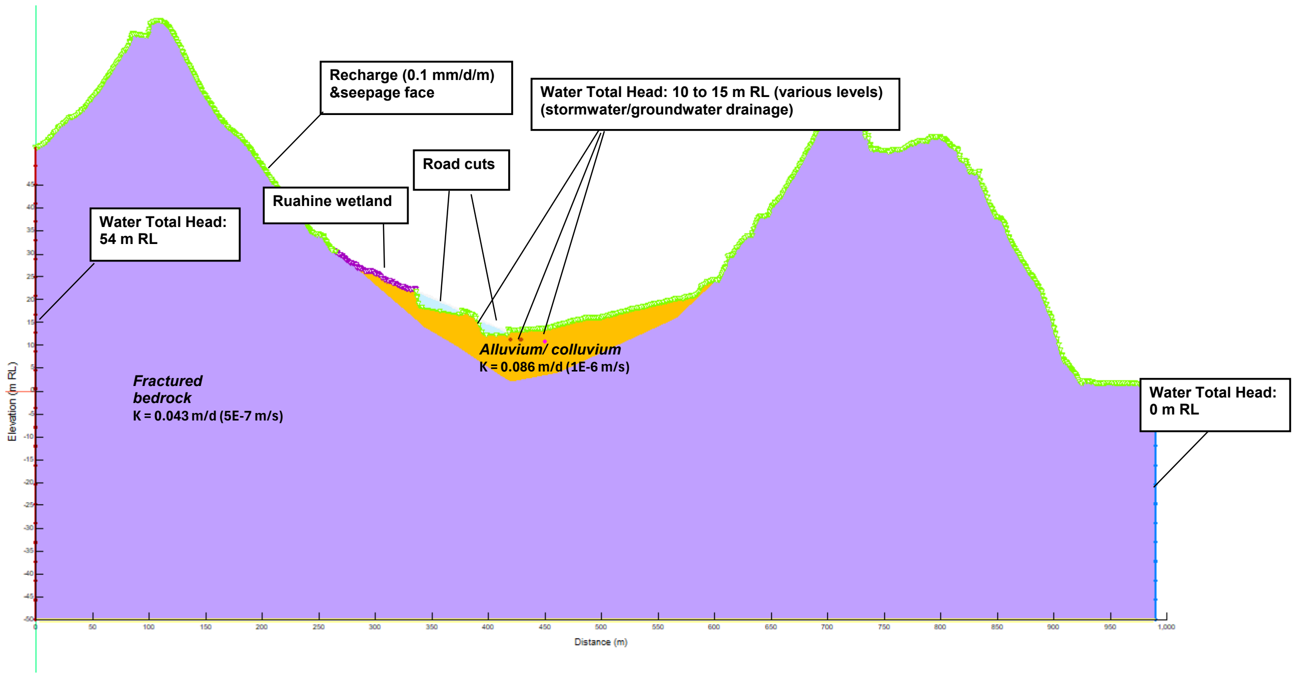


Figure 35 - SEEP/W 2D numerical groundwater modelling setup for Ruahine St (C-C') model

C2. Numerical groundwater modelling for design of the Project

Several 2D-cross-sectional models and an AnAqSim analytical elements model have been developed for the Project to support the design of the project as follows:

- Eight cross-sectional models were developed for The Terrace Tunnel design
- An AnAqSim analytical elements model was developed for the Sussex St Trench design at Basin Reserve
- Four cross-sectional models were developed for the Mt Victoria Tunnel design

These models have also been used to support the assessment of groundwater effects as described in the main body of this report. The technical details of these models are described below.

Terrace Tunnel

Method

Terrace Tunnel numerical modelling was undertaken using GeoStudio's SEEP/W finite element software (v2025.1).

Geometry

The geometry of the cross section was based on Leapfrog model outputs prepared by the Project team. Eight cross-sectional models were developed as shown in Figure 36 below. For cross sections 1 to 6 the proposed tunnel was centred at coordinates (0,0). Cross section 7 and 8 represent the road section south of the Terrace Tunnel.

Mesh

For each of the cross sections the model mesh consisted of the following:

- A combination of quadrilateral and triangular elements to accommodate the tunnel geometry.
- Global mesh size: 3 m × 3 m.
- Refined mesh: 1 m × 1 m assigned to the tunnel regions and adjacent areas.

Geology

Geological regions in all cross sections were based on the LeapFrog generated cross sections.

Hydraulic Conditions

The hydraulic conditions of the SEEP/W models were as follows:

- Model assumes fully saturated conditions.
- Permeability: 5×10^{-8} m/s.
- Saturated volumetric water content: 0.3.
- Anisotropy ratio: 1 (isotropic).

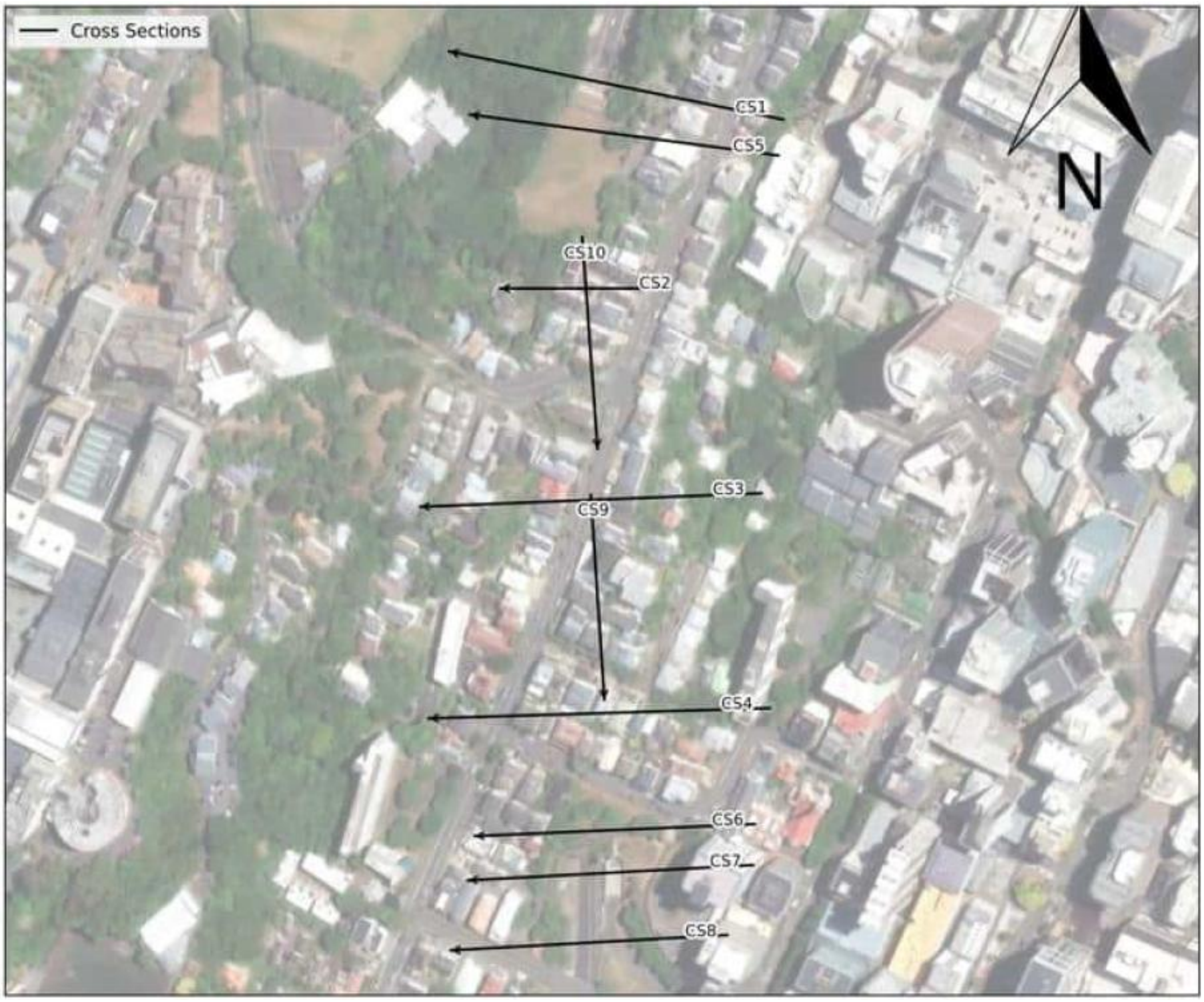


Figure 36 - Terrace Tunnel analysis cross section locations

Boundary Conditions

The following boundary conditions were applied:

- Recharge was applied to the model surface at a rate of 100 mm/year. This was based on the Nationwide Groundwater Recharge Model
- Fixed head boundaries positioned at the western and eastern borders of each model to represent the initial water table (Table 15)
- Tunnel drainage was modelled using fixed head boundaries at the tunnel perimeter set to the tunnel invert levels

Table 15: Boundary conditions Terrace Tunnel models

Cross section (CS)	Western boundary condition (m RL)	Eastern Boundary condition (m RL)
CS1	50	25
CS2	50	30
CS3	50	25

Cross section (CS)	Western boundary condition (m RL)	Eastern Boundary condition (m RL)
CS4	40	25
CS5	50	30
CS6	30	15.5
CS7	27.5	15.5
CS8	26	15.5

Assumptions

The following assumptions were applied to the model:

- The model is two-dimensional.
- The domain was assumed to be:
  - Homogeneous (uniform material properties throughout).
  - Isotropic (equal permeability in all directions).
- The coloured regions shown in the cross-section figures below represent distinct geological units. At this stage, the model is defined as a single domain, with no differentiation in hydraulic parameters between these units. This assumption will be revisited once site-specific data are collected, allowing for the application of variable hydraulic properties to the digitised geological regions if testing indicates such variability.
- Fully saturated conditions applied across the entire model domain.
- The groundwater level is fully drawn down to the base of each tunnel by the tunnel drainage systems.

Results

Groundwater levels of the pre-construction and post-construction models for all eight cross sections are shown as the blue dashed line in Figure 37 to Figure 50.

The results of the SEEP/W models show that groundwater is drained at the existing tunnel in the model. This water table is assumed to be the pre-construction historical season low groundwater level of the groundwater system at the Terrace Tunnel. The drawdown cone is enhanced in the post-construction model.

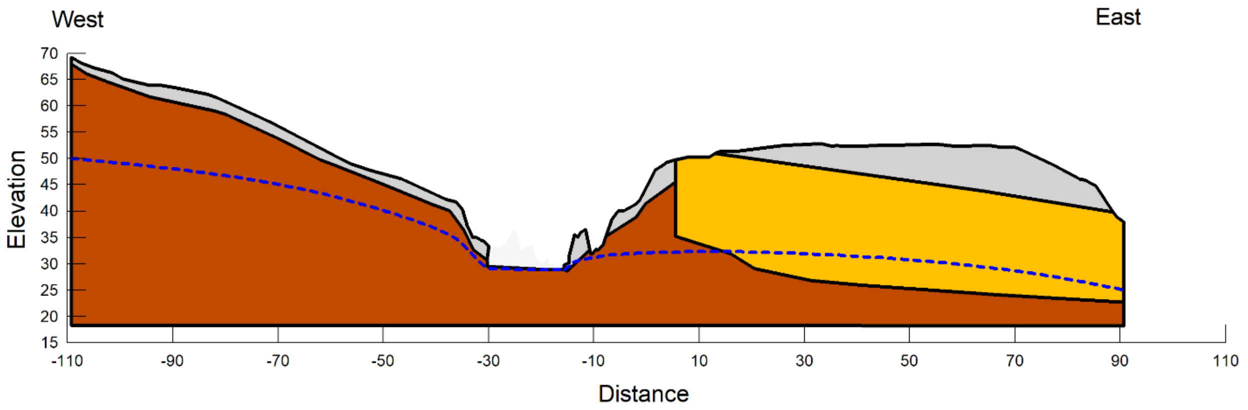


Figure 37 - Cross section 1 Pre-construction historic low model water table

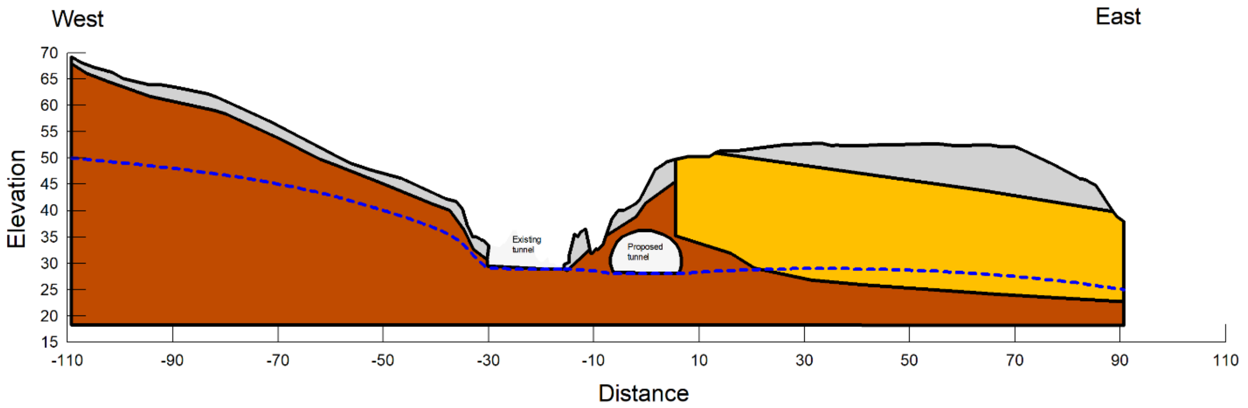


Figure 38 - Cross section 1 Post-construction model water table

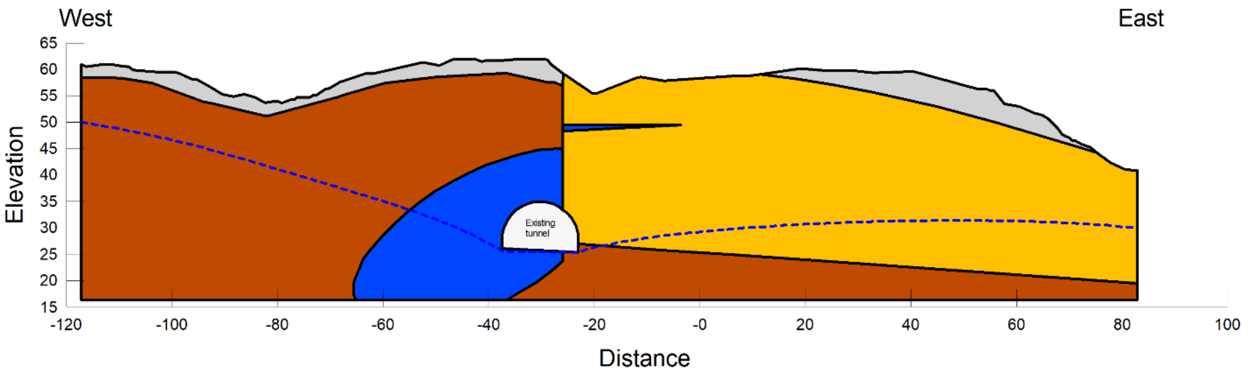


Figure 39 - Cross section 2 Pre-construction historic low model water table

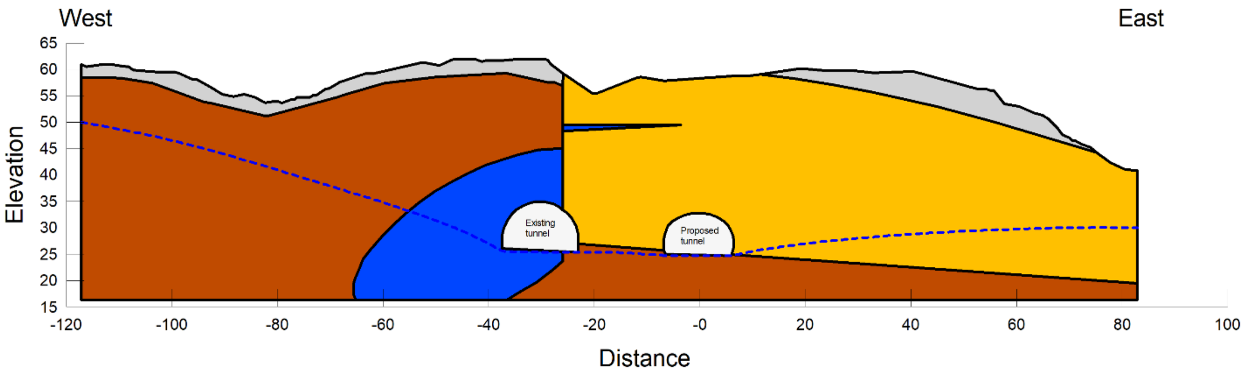


Figure 40 - Cross section 2 Post-construction model water table

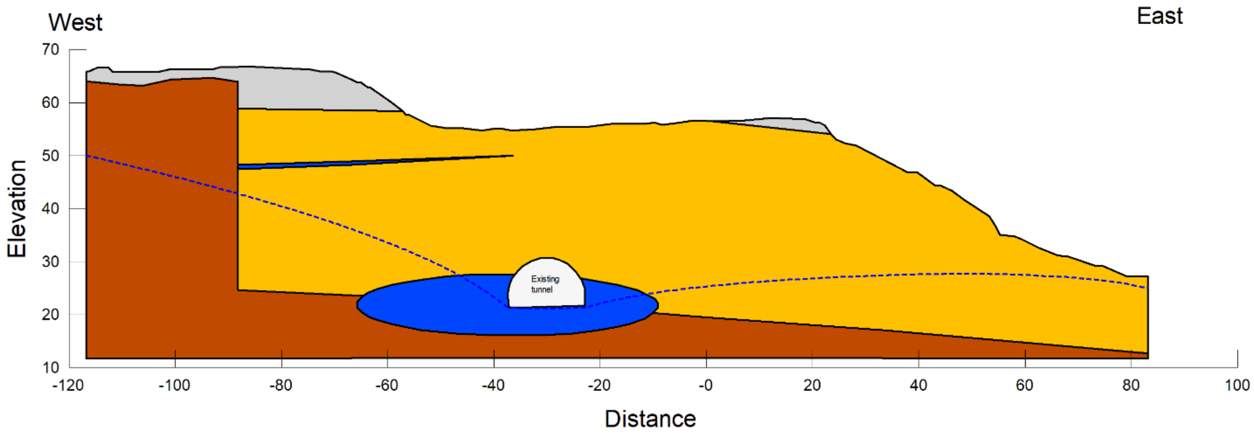


Figure 41 - Cross section 3 Pre-construction historic low model water table

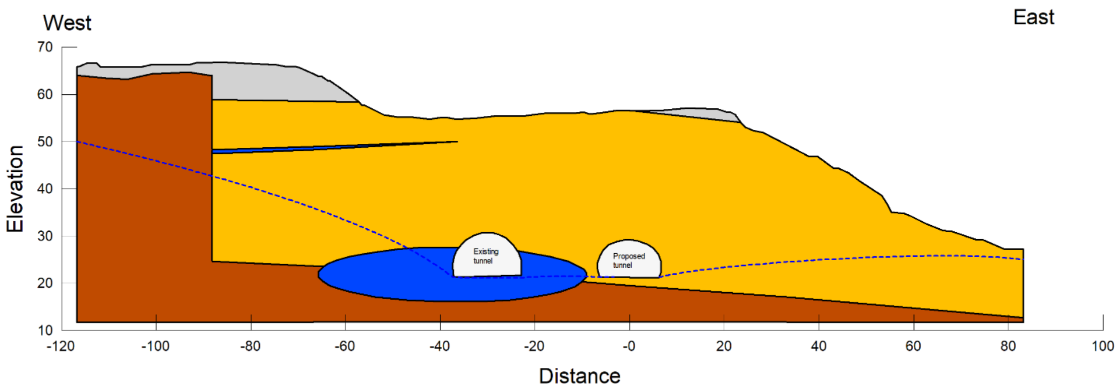


Figure 42 - Cross section 3 Post-construction model water table

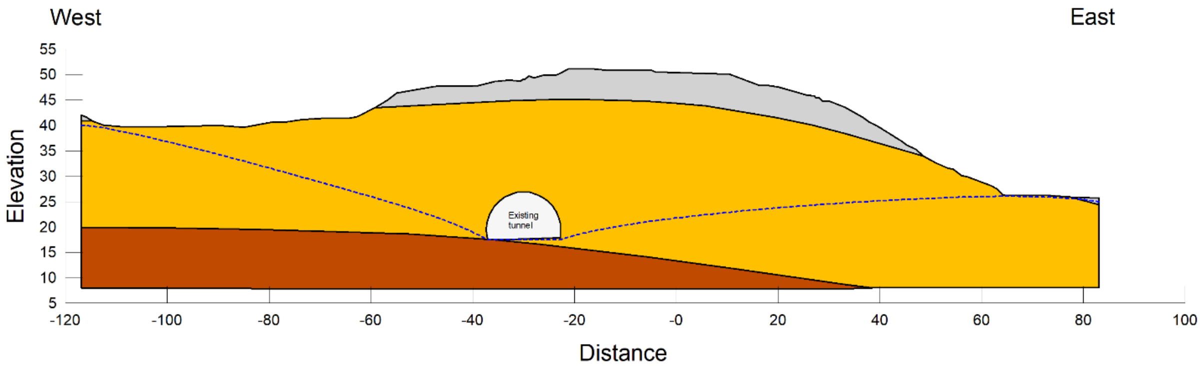


Figure 43 - Cross section 4 Pre-construction historic low model water table

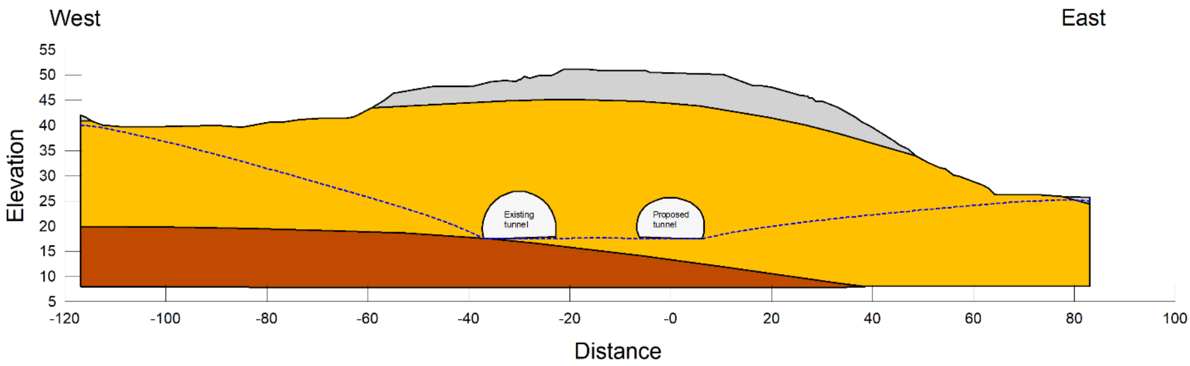


Figure 44 - Cross section 4 Post-construction model water table

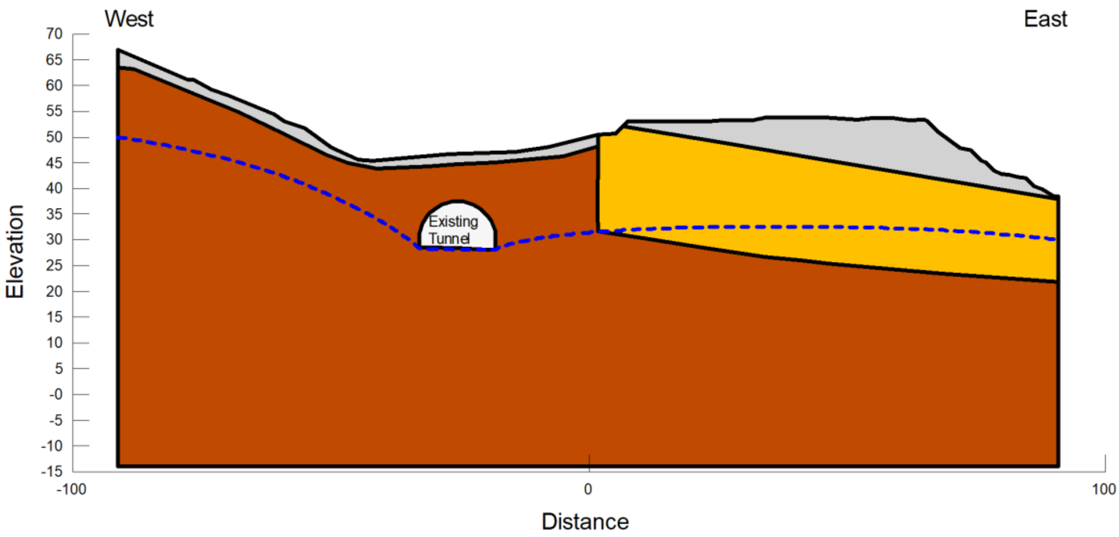


Figure 45 - Cross section 5 Pre-construction historic low model water table

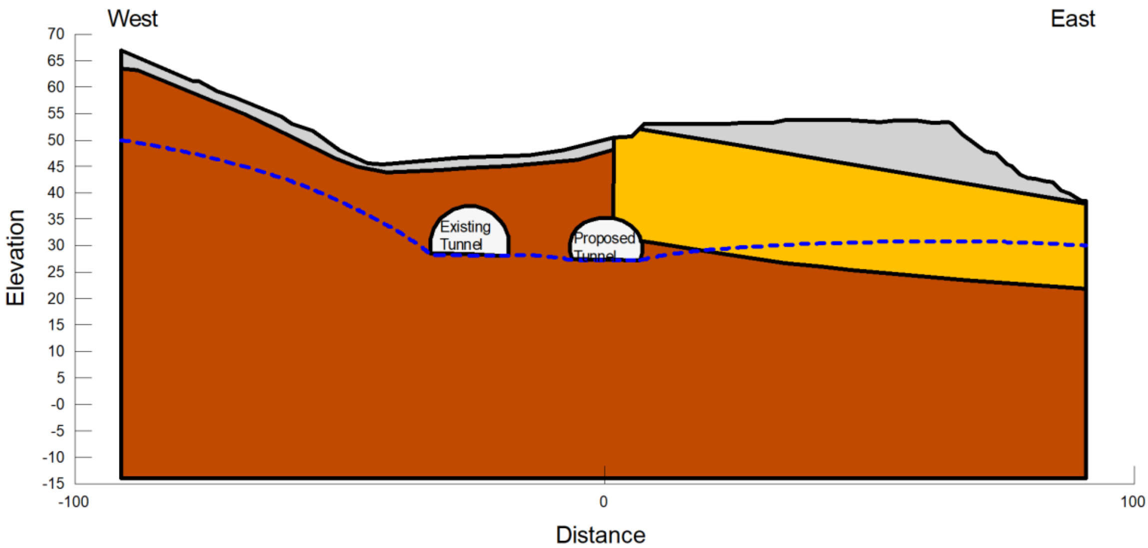


Figure 46 - Cross section 5 Post-construction model water table

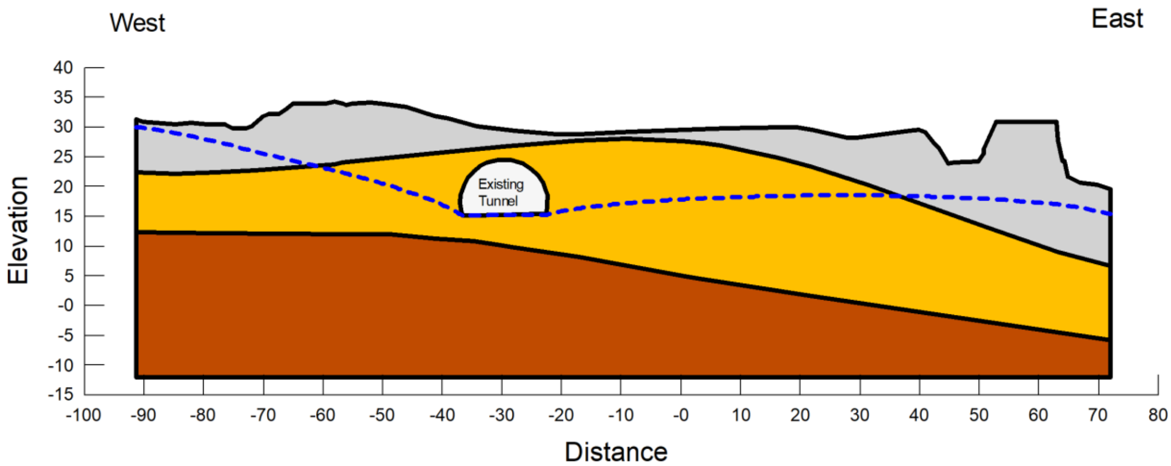


Figure 47 - Cross section 6 Pre-construction historic low model water table

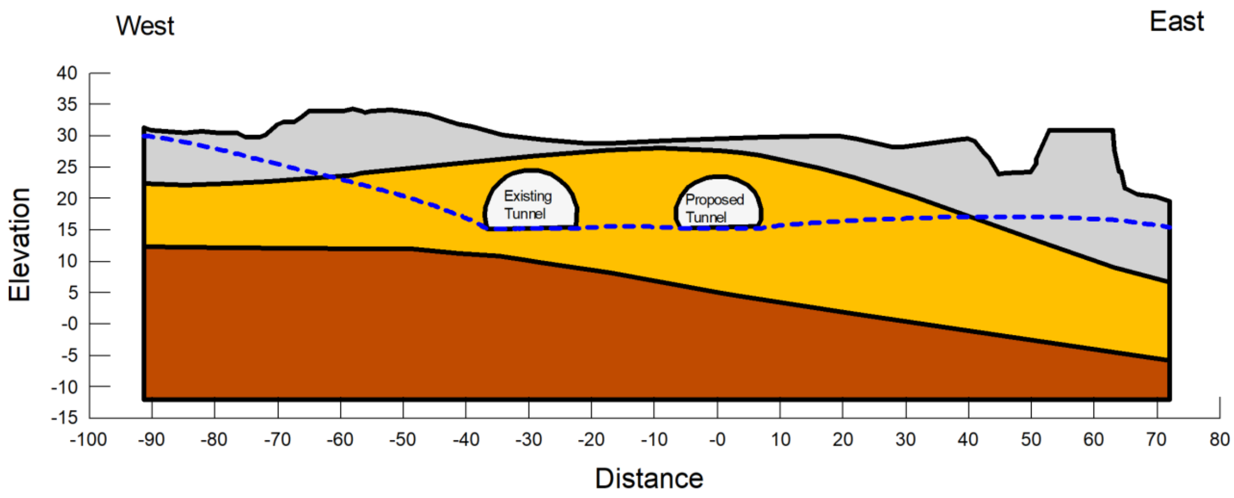


Figure 48 - Cross section 6 Post-construction model water table

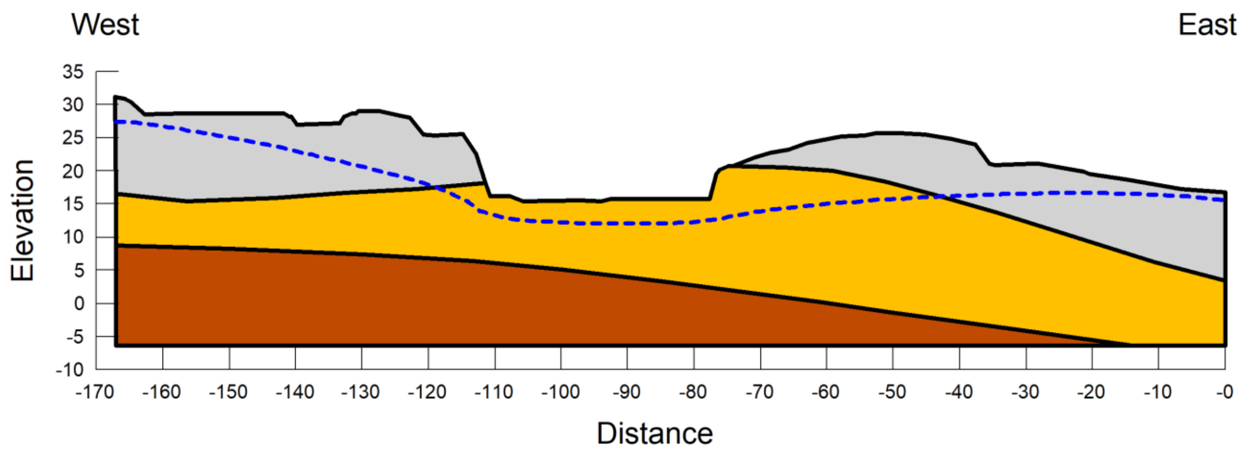


Figure 49 - Cross section 7 Pre-construction historic low model water table

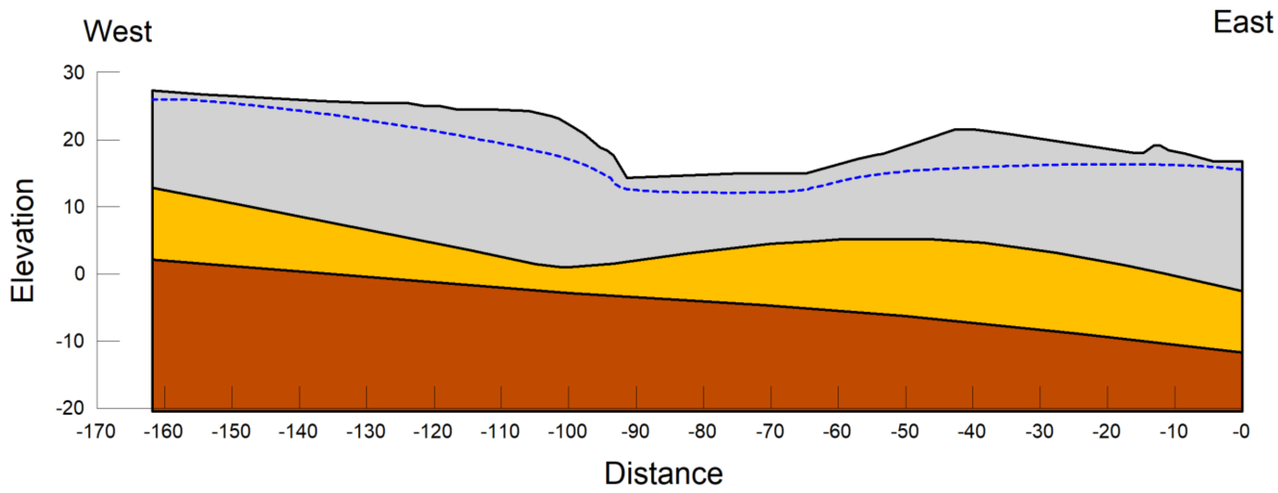


Figure 50 - Cross section 8 Pre-construction model water table

Sussex Street Trench

Method

Sussex Street trench numerical modelling was undertaken using the Analytical Aquifer Simulator (AnAqSim) based on the Analytical Element Method (AEM). The AEM approach implemented in AnAqSim allows simulation of steady-state and transient three-dimensional groundwater.

Model Discretisation

The model was discretised using Spatially Variable Area Sinks (SVAS), with coarser spacing adopted near the model edges with more refinement added closer to the modelled excavation.

Initial Conditions Model

Model Layers

The model comprised three uniform thickness hydrostratigraphic layers representing an upper unconfined and two lower confined aquifer systems at the site, as summarised in Table 16.

Table 16: Model Layer structure AnAqSim model

Layer	Elevation top (m RL)	Elevation bottom (m RL)	Aquifer type
1	30	0	Unconfined
2	0	-10	Confined
3	-10	-40	Confined

Boundary Conditions

The following boundary conditions were applied to the initial conditions model corresponding to the current conceptual understanding.

- **Southern and Eastern boundary 1:** Head Specified Line Boundaries (HSLB) corresponding to the current conceptual understanding of groundwater flow.
- **Northern boundary:** Constant head of 0 m RL corresponding to mean sea level.
- **Western and eastern boundary 2:** No-flow.
- **Drain boundaries:** Line sinks positioned within known valleys.

Model Calibration

Model calibration was undertaken using PEST, which applies statistical optimisation algorithms to determine a combination of parameters that provide a “best fit” between the modelled values from a numerical model and observed values. PEST is an industry-standard algorithm used widely in groundwater numerical modelling which employs an objective function to quantify model fit between observed and modelled heads, using linear regression methods to iteratively minimise the residuals.

Model parameter bounds were selected corresponding to the current conceptual understanding of the site. The adopted PEST parameter bounds are presented in Table 17. The parameters selected for the PEST analysis were horizontal hydraulic conductivity, hydraulic conductivity ratio, rainfall recharge, and conductance of the drain line boundaries.

As the current modelling purpose is to assess long-term steady-state conditions, parameters such as storativity and specific yield were not included in the calibration or assessment.

Table 17: PEST Calibration Parameter Bounds

Hydraulic parameter	Model layer	PEST Name	Initial value	Lower bound	Upper bound
Horizontal hydraulic conductivity (m/day)	1	kh_1	0.01	0.001	0.1
	2	kh_2	0.1	0.01	5
	3	kh_3	0.01	0.001	0.01
Rainfall recharge (m/day)	1	rch_1	2.7E-04	8.2E-05	4.1E-04
Kv/Kh hydraulic conductivity ratio	1	KvRat_1	0.1	0.01	0.5
	2	KvRat_2	0.1	0.01	0.5
	3	KvRat_3	0.1	0.01	0.5
Drain conductance	2	drain_1_start	5,000	1,000	50,000
	2	drain_1_end	5,000	1,000	50,000
	2	drain_2_start	5,000	1,000	50,000
	2	drain_2_end	5,000	1,000	50,000

Calibration Results

Modelled versus observed groundwater levels are plotted in Figure 51. Whilst most modelled groundwater levels are close to the observed value, some clearly deviate. The main reason is that most of the available groundwater records are spot measurements recorded in during different times and may not represent the groundwater conditions assumed in the model. The hydraulic parameters derived from the calibration achieved are summarised in Table 18.

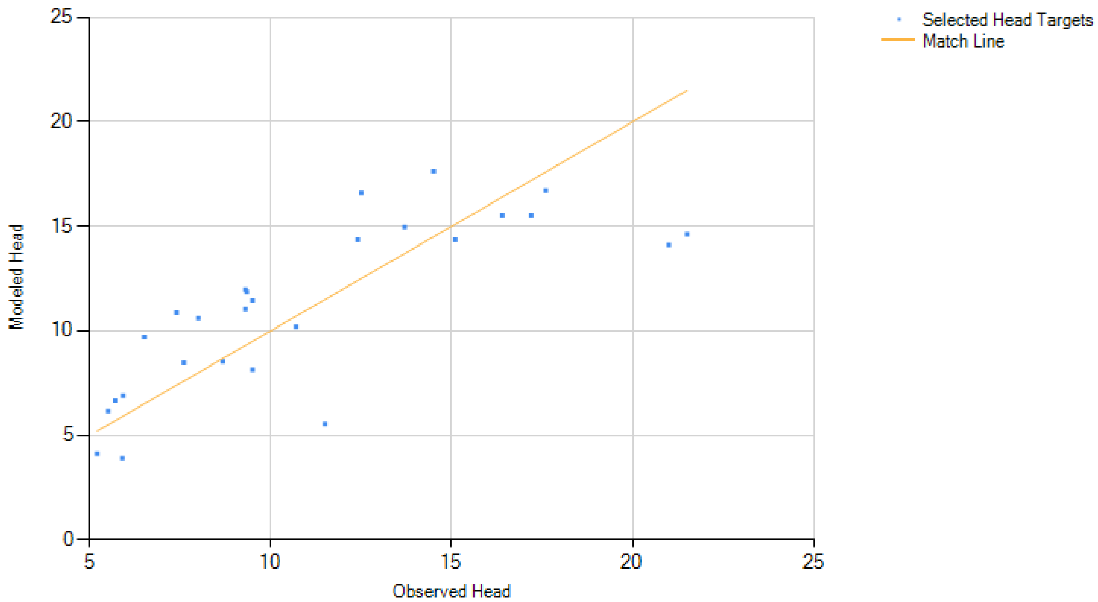


Figure 51 - Observed heads versus modelled heads

Composite sensitivities are also summarised in Table 18 to provide a measure of how changes in a model's parameters collectively influence its outputs. They are dimensionless values that quantify parameter influence, allowing direct comparison between parameters with different units and scales.

Table 18: Calibrated Parameters and Sensitivities

Hydraulic parameter	Model layer	PEST Name	Calibrated model parameters	Composite sensitivities	Sensitivity interpretation
Horizontal hydraulic conductivity (m/day)	1	kh_1	1.4×10^{-2}	0.25	**kh_2** has the highest sensitivity (1.37), so model outputs were most affected by this parameter.
	2	kh_2	9.6×10^{-1}	**1.37**	
	3	kh_3	8.4×10^{-3}	0.13	
Rainfall recharge (m/day)	1	rch_1	3.7×10^{-3}	**0.40**	Moderate sensitivity (0.40);
Kv/Kh hydraulic conductivity ratio	1	KvRat_1	0.09	0.09	Suggests vertical flow less influential in the model outputs.
	2	KvRat_2	0.10	0.01	
	3	KvRat_3	0.05	0.01	
Drain conductance	2	drain_1_start	1000	0.22	**drain2_e** exhibited high sensitivity (1.19)
	2	drain_1_end	1,000	0.35	
	2	drain_2_start	12,979	0.40	
	2	drain_2_end	25,251	**1.19**	

Final Conditions Model

The final conditions model is the same as the initial conditions model with the following modifications:

- A series of head-specified line boundaries (HSLBs) were used to represent the sloping drained trench base, including an additional 0.5 m undercut for drainage installation.
- A Leaky Barrier boundary condition was applied to Model Layer 1 (base at 0 m RL) to represent the proposed retaining structure.
    - This boundary simulates a partial barrier to groundwater flow which restricts but does not completely prevent seepage through the retaining wall system.
    - In AnAqSim, the leaky barrier is implemented as a zero-width line feature using line-doublet functions. The barrier line was positioned 0.1 m offset from the proposed excavation edge.
    - The degree of leakage through the barrier is controlled by the conductance parameter (C), defined as: C = K*/b, where b is the width of the barrier measured normal to the flow direction, and K* is the horizontal hydraulic conductivity of the barrier material.
    - The adopted b* value used for the model was set to 0.8 m to represent the thickness of the piles, where K* was set to 8×10^{-3} m/day (9.3×10^{-8} m/s), resulting in conductance of 1×10^{-2} day^{-1}.

Mount Victoria Tunnel

Method

Mount Victoria tunnel numerical modelling was undertaken using GeoStudio's SEEP/W finite element software (v2025.1). Four cross-sectional models were developed as indicated in Figure 52

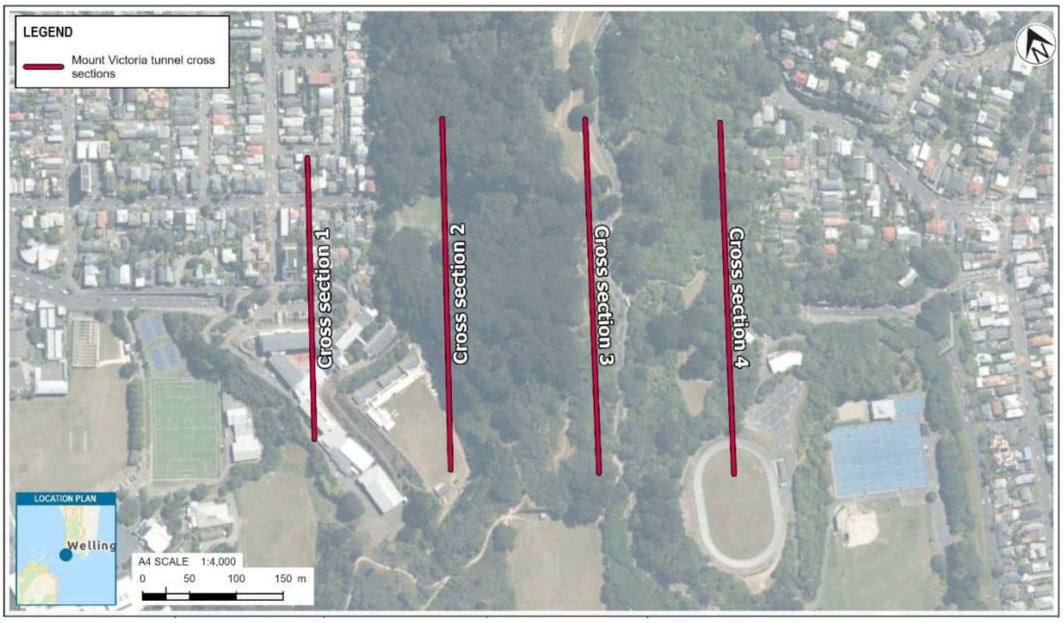


Figure 52 - Cross sections of 2D-cross-sectional models for Mt Victoria

Geometry

The geometry of the cross section was based on Leapfrog model outputs prepared by the Project team. For each cross section the proposed tunnel was centred at coordinates (0, 0) within the SEEP/W models.

Mesh

For each of the cross sections the model mesh consists of the following:

- A combination of quadrilateral and triangular elements to accommodate the tunnel geometry
- Global mesh size: 3 m × 3 m.
- Refined mesh: 1 m × 1 m assigned to the tunnel regions and adjacent areas.

Geology

Geological regions in all cross sections are based on the LeapFrog generated cross sections.

Hydraulic Parameters

The hydraulic conditions of the SEEP/W models are as follows:

- Model assumes fully saturated conditions.
- Permeability: 5×10^{-8} m/s.
- Saturated volumetric water content: 0.3.
- Anisotropy ratio: 1 (isotropic).

Boundary Conditions

The following boundary conditions were applied to the cross sections:

- Recharge was applied to the model surface at a rate of 100 mm/year. This was based on the Nationwide Groundwater Recharge Model
- Fixed head boundaries were positioned at the northern and southern borders of each model to represent the initial water table

Table 19 provides an overview of the applied head values per cross section. Tunnel drainage was simulated using fixed head boundaries at the tunnel perimeter set to the tunnel invert levels.

Table 19: Boundary conditions applied to outer model boundaries

Cross section	Northern fixed head (m RL)	Southern fixed head (m RL)
CS1	24	36
CS2	45	45
CS3	70	70
CS4	50	50

Assumptions

The following assumptions were applied to the model:

- The model was two-dimensional
- The domain was assumed to be:
  - Homogeneous (uniform material properties throughout)
  - Isotropic (equal permeability in all directions)
- The coloured regions shown in the cross-section figures below represent distinct geological units. At this stage, the model is defined as a single domain, with no differentiation in hydraulic parameters between these units. This assumption will be revisited once site-specific data are collected, allowing for the application of variable hydraulic properties to the digitised geological regions if testing indicates such variability
- Fully saturated conditions applied across the entire model domain
- The groundwater level is fully drawn down to the base of each tunnel by the tunnel drainage systems

Results

Groundwater levels of the pre-construction and post-construction models for all four cross sections are shown as blue dashed lines in Figure 53 to Figure 60.

The pre-construction SEEP/W models show groundwater already drained and lowered by the existing tunnel and pilot tunnel. In the post-construction model, increased drainage is simulated into the proposed tunnel, resulting in further lowering of groundwater levels.

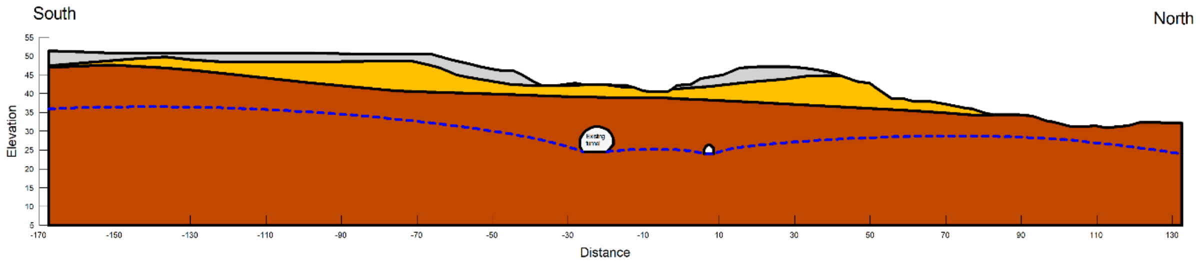


Figure 53 - Cross section 1 Pre-construction model water table

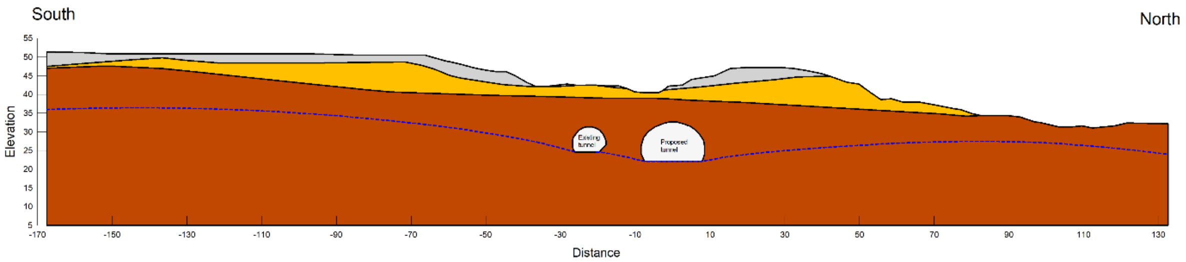


Figure 54 - Cross section 1 Post-construction model water table

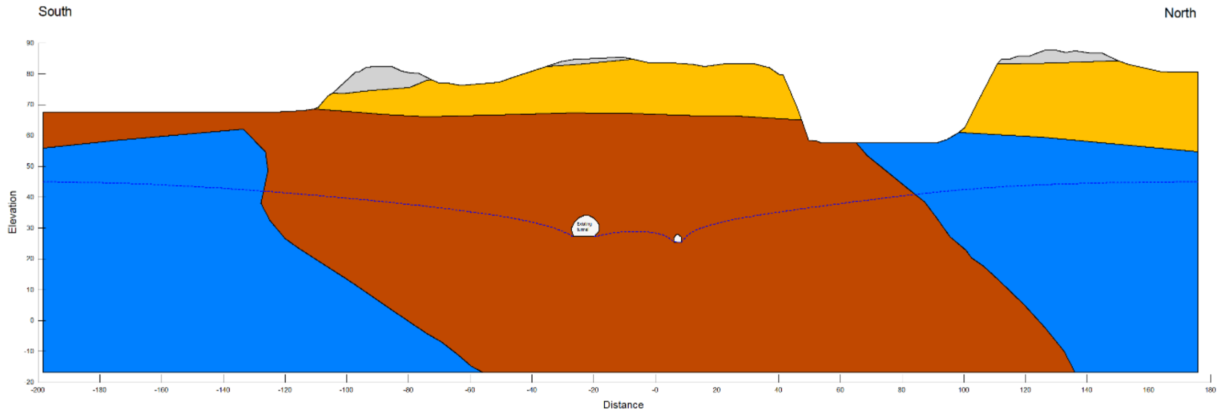


Figure 55 - Cross section 2 Pre-construction historic low model water table

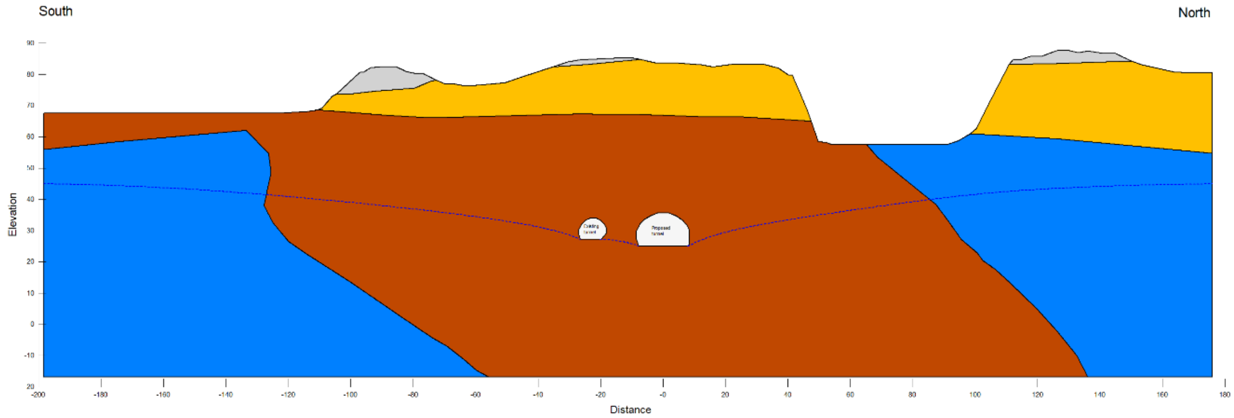


Figure 56 - Cross section 2 Post-construction model water table

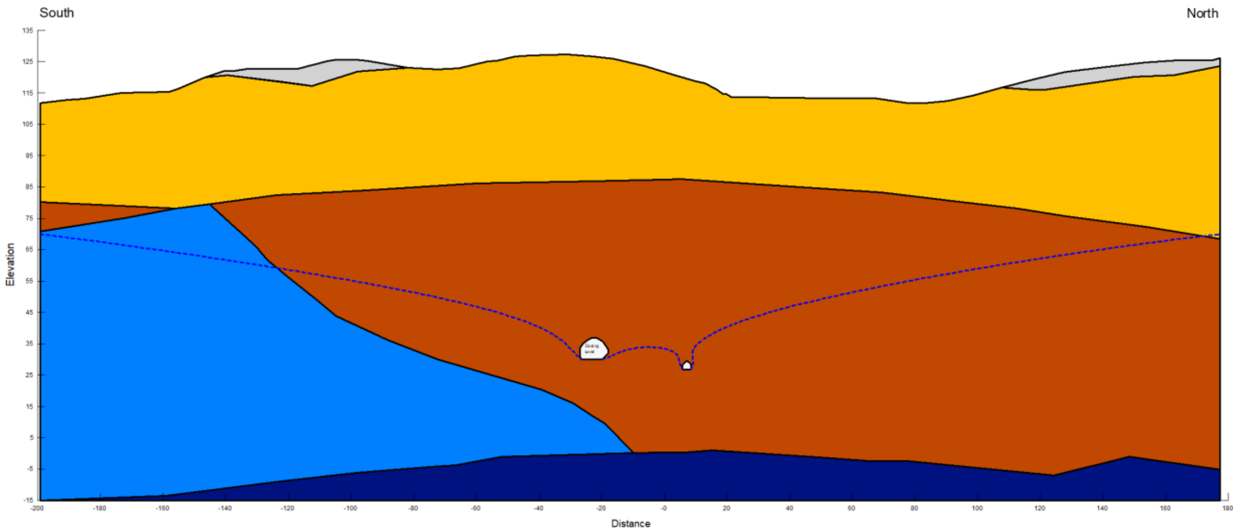


Figure 57 - Cross section 3 Pre-construction model water table

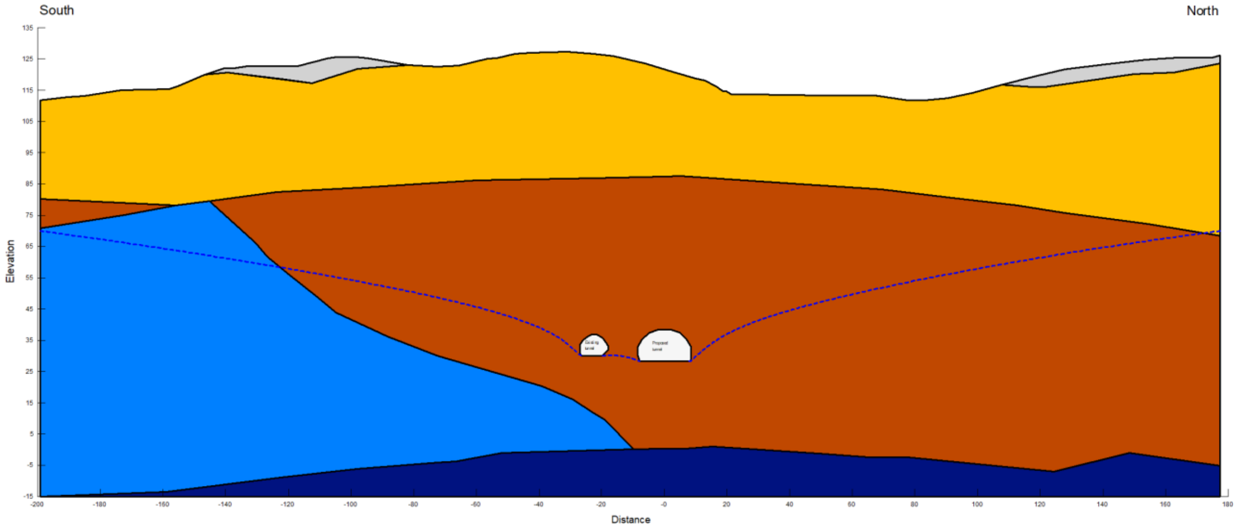


Figure 58 - Cross section 3 Post-construction model water table

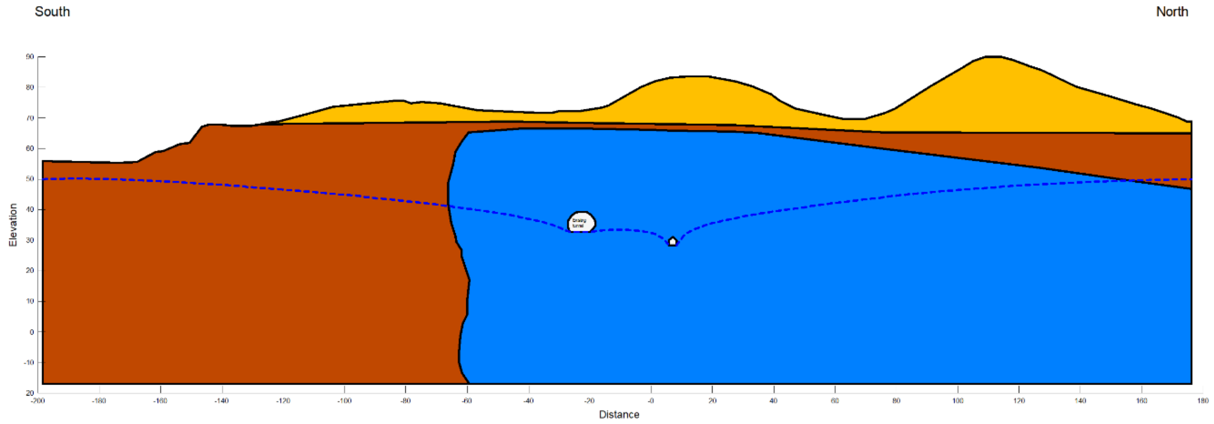


Figure 59 - Cross section 4 Pre-construction model water table

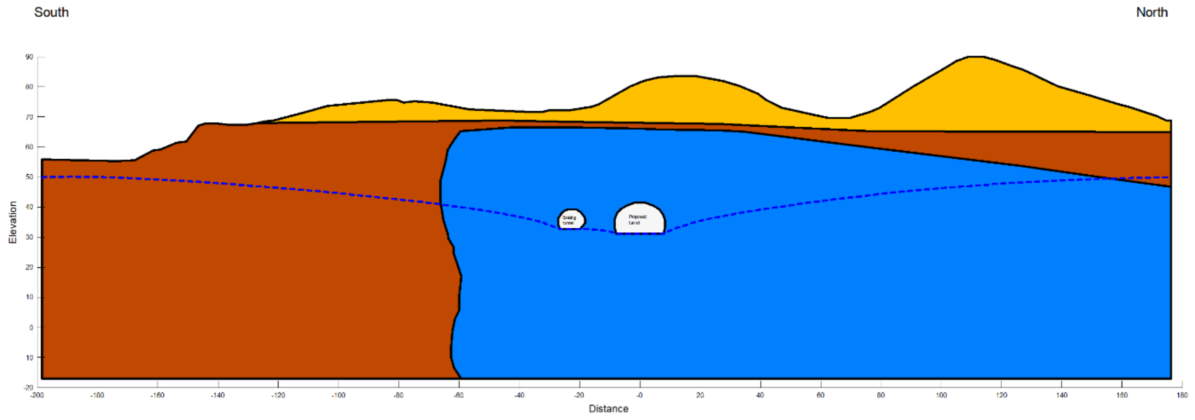


Figure 60 - Cross section 4 Post-construction model water table

ANNEXURE D: Groundwater and Settlement Monitoring and Contingency Plan (GSMCP) requirements

Introduction

The preparation and implementation of a Groundwater and Settlement Monitoring and Contingency Plan (GSMCP) is recommended to support the effects management and monitoring of land settlement effects caused by the **temporary and permanent groundwater level drawdown and mechanical settlement** by the Project. Monitoring will be required during the construction phase and for 1 year after the completion of construction. The GSMCP provides a guide to managing potential settlement and deflections of nearby buildings and structures caused by temporary excavations on buildings and services adjacent to the works.

The GSMCP will be drafted once the detailed design for the Project is available, so that the monitoring, trigger levels and management options reflect the detailed design and can be effectively implemented at the start of the Project. The details provided below serve to indicate what the content of a GSMCP will be.

The GSMCP will be submitted to Greater Wellington Regional Council for review and certification at least 1 month prior to the start of construction.

This GSMCP will set out the following:

- The at-risk buildings and other infrastructure as a result of the dewatering and excavation activities
- Recommended maximum levels for groundwater drawdown and trigger levels for settlement (i.e., alarm and alert levels as described below) of buildings and other infrastructure associated with the proposed earthworks and dewatering for the Project
- Contingency measures to be implemented should the alert or alarm levels be exceeded
- Measures to mitigate adverse effects (including cumulative effects) as a result of the dewatering and excavation activities involved in the early works and dewatering

Overview of monitoring requirements

This section provides an overview of the types of monitoring and reporting required for the excavation and construction works in accordance with the conditions of resource consents that permits the temporary construction and permanent operation dewatering at the proposed new Terrace Tunnel, Sussex St Trench, Mt Victoria and cut slopes and stormwater installation at Ruahine Street / Wellington Road in Hataitai.

Temporary works, ground settlement, building and structure deflections, and groundwater drawdown monitoring during the construction works will be undertaken to monitor whether the response of the surrounding buildings and structures remain within design tolerances and estimated range of settlement effects. This process allows for the geotechnical effects to be monitored and are to be used as an indication if mitigation measures are required.

In general, monitoring is subdivided into three separate stages reflecting the separate periods of construction works. These are summarised in Table 20.

Table 20: GSMCP monitoring and management stages

Stage	Monitoring and management
Pre-construction monitoring	This monitoring phase will provide baseline data against which effects resulting from the construction works can be assessed. The outcomes will form part of the input for the construction phase assessments. Pre-construction monitoring is to include: Condition surveys and visual inspections of selected nearby buildings (as described below), including photographs of nearby buildings, to define existing condition of the buildings. CCTV surveys of stormwater and wastewater assets Installation and monitoring of building and ground monitoring survey points to establish a settlement baseline. Groundwater level monitoring to establish baseline levels.
During construction monitoring	Monitoring during the construction phase is carried out to compare settlement against the set Alert and Alarm levels and enable the implementation of countermeasures in advance of potential adverse impacts occurring. Mitigation options set out in the GSMCP, may also be required to be implemented.
Post-construction monitoring	Post-construction measurements will be completed, if required, for 1 year after completion of dewatering or until the position markers are found to have stabilised and approval is given in writing by Greater Wellington Regional Council.

The key management roles for each organisation in relation to groundwater and settlement management during the construction of the project will be identified in the GSMCP. Monitoring details and records shall be submitted to Greater Wellington Regional Council by those indicated as responsible.

Proposed construction methodology

The GSMCP will set out a summary of the proposed earthworks and both temporary (during the construction phase) and permanent (operational) dewatering for the Project. Details relevant to the potential risks of groundwater drawdown and land settlement are listed, in line with those listed in paragraph 6 through to 11 in the main body of this report. Parts of the proposed construction methodology developed by the lead contractor, where they are relevant to the groundwater drawdown and land settlement risks, will be appended to the GSMCP.

Groundwater monitoring

The ongoing monitoring of groundwater levels will be proposed which should start before any excavations to establish the initial baseline conditions, throughout the earthworks and dewatering and beyond the end of the construction process.

Groundwater levels will be monitored using automatic pressure transducers set at collecting a reading at 15-minute intervals. This is considered the minimum frequency at which the data is retrieved and processed. All levels will be recorded to a minimum accuracy of ± 10 mm.

Groundwater drawdown as a result of the project works will be modelled by the Project's hydrogeologists and modelled drawdowns will be used to set the groundwater trigger levels. For groundwater level monitoring two alert levels, as opposed to a single alert and alarm level, are set because ground settlement itself will be measured (described below), and any mitigation or contingency will be based on those settlement responses. The groundwater alert trigger levels will provide notification that groundwater responses to the construction and dewatering works are nearing those estimated, and that such groundwater responses may be close to having implications for ground settlement.

To verify groundwater drawdown does not exceed estimated drawdown the first alert level is set as the lowest groundwater level considering seasonal variation, plus a dewatering drawdown. The second alert level will be set as an additional 0.5 m of drawdown. A schematic diagram of monitoring sites and groundwater alert levels is shown in Figure 61. If an Alert level is reached, the actions outlined below shall be carried out.

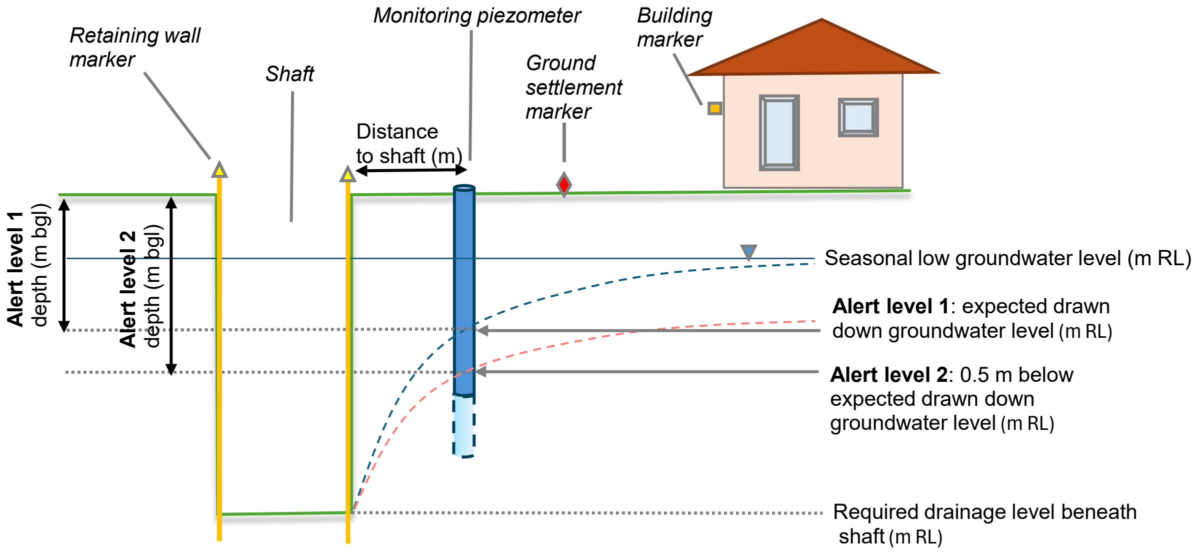


Figure 61 - Schematic diagram showing monitoring sites and groundwater alert levels (not to scale)

Settlement and deflection monitoring and building condition surveying

The settlement monitoring provides advance warning if the ground is settling more than estimated. This provides time for mitigation or rectification works to be identified and implemented.

The settlement monitoring will use visual inspections, ground settlement markers, building markers, and retaining wall markers for monitoring the area around the shaft / excavations. Condition surveys of nearby buildings, structures and services will also be undertaken. A possible methodology is detailed below, but generally comprises the following:

- Pre-construction monitoring to determine existing conditions and baseline measurements
- Monitoring during construction around active works areas at a higher frequency to determine actual ground movements and compare with estimates as the works progress
- Post-construction monitoring continued until potential ground movements have stabilised and no further potential for damage remains

A plan showing the proposed location of monitoring markers will be included in the GSMCP. This plan will be updated by the Contractor with as-built locations following installation of the monitoring points and their pre-construction survey.

Condition surveys and visual inspections

Pre- and post-construction condition surveys will be undertaken on all assets (buildings and services) determined to be susceptible to settlement. During construction additional visual inspections shall be undertaken with additional condition surveys performed as required by this plan.

The inspections and subsequent evaluations will be undertaken and reported on by a SQEP and shall include:

- For all inspections, approval from the asset owner shall be acquired
- Type and arrangement of foundations
- Condition of the existing asset including any pre-existing damage and the type of damage (e.g., aesthetic, serviceability impact)
- Photographic evidence of the above

Visual inspections of the external building fronts facing the excavation on either side of the excavation must be undertaken for the purpose of detecting any existing external damage or new external damage or deterioration of building fronts. The sites to be surveyed are the buildings identified at the Terrace Tunnel, Basin Reserve and Mt Victoria as being potentially at risk, within the area in which land settlements may occur as shown in Figure 22, Figure 23 and Figure 24 in Annexure A of this report. The exact buildings to be surveyed will be confirmed once the detailed design and construction methodology have been confirmed.

Settlement and deflection monitoring

The purpose of the ground surface and building monitoring is to confirm that actual settlements are within the estimated settlement range, as a precautionary measure. The expected maximum total and differential settlements of the Project are estimated to be noticeable but within the ‘slight’ damage category (i.e., ‘low’ impact) as outlined in paragraph 43 and Table 1 in the main body of this report. Trigger levels will be set which will be based on these estimated settlements.

Ground settlement and building markers will be installed in all directions around the tunnel and trench excavations. The locations will be indicated on the maps that will be included in the GSMCP. The locations will be selected as follows:

- All markers should be placed to allow for ease of access for pre and post construction monitoring
- Ground settlement markers between the shaft / excavation edge and the nearest buildings
- Building markers on the buildings near the excavation, with at least two markers on the building fronts closest to the excavation to allow for differential settlement to be determined
- Retaining wall deflection markers on each side of the Sussex St Trench excavation
- Differential settlement is to be determined by pairs of ground settlement markers and building markers

The purpose of the retaining structure monitoring is to confirm that actual deflections remain within the estimated range. Trigger levels are based on the estimated deflections, which will be confirmed by the Contractor at the pre-start meeting with Greater Wellington Regional Council’s compliance team.

Monitoring of Utilities and Infrastructure

Excavations will be undertaken in proximity to various existing underground services including water, wastewater, stormwater and electricity. These services are constructed of different materials to various standards, at different depths and locations and at different times. As such they are expected to have varying tolerances to deformation. Therefore, the settlement trigger levels will be based on a conservative assessment of damage to typical services. The asset owners shall be consulted to confirm deformation tolerances of the given assets, and the associated proposed monitoring is suitable.

Prior to construction, a pre-condition survey shall be undertaken on services. The survey may comprise a CCTV condition assessment of services accessed for relocation for up to 20 m from the excavation. This will be shown on a map to be included in the GSMCP, and will be carried out by a SQEP. It shall include the following but are not limited to:

- Existing levels of aesthetic damage
- Existing levels of serviceability damage
- Existing levels of structural damage
- Existing top of pipe/invert RL (GPS)
- Photographic/video evidence of (1), (2), and (3) above

If the total or differential settlement Alarm limits are reached during construction close to the excavation (less than 20 m), a post-construction survey will be done within six months of completion of the dewatering activity covering the items detailed above.

A copy of the pre- and post-construction survey report shall be forwarded to the Greater Wellington Regional Council Greater Wellington Regional Council within 15 working days of completing the reports along with a certificate from the SQEP who has certified that the survey has been completed in a professional manner and is an accurate assessment of the condition of the structure concerned.

The proposed ground settlement markers will be used for monitoring for the underground services as well. If the trigger levels are exceeded, the actions outlined below shall be carried out.

Settlement trigger levels

Two trigger levels are set for all settlement monitoring points:

- Alert: Measured settlements are still within normal levels but are approaching those predicted (≥ 70%) in the settlement assessment. Alert levels have been set to 70% of estimated settlement levels at the markers, or, for estimated settlement less than very slight damage levels (10 mm), alert levels have been set to 70% of damage levels (i.e., 7 mm)
- Alarm: Measured settlements have reached those predicted. Alarm levels have been set to 100% of estimated settlement levels at the markers, or, for estimated settlement less than damage levels (10 mm), alert levels have been set to very slight damage levels (i.e. 10 mm)

Details of the trigger levels for all monitoring points will be included as tables in the GSMCP.

Trigger levels are based on the estimated settlements calculated in the design and do not necessarily imply potential for damage to occur if they are exceeded. These trigger levels are based on an initial estimate of effects, and the Contractor may adopt more rigorous levels if needed for services condition surveys or because of further developments or changes in the design. At least 10 working days prior to adopting any change in trigger levels the Contractor will submit to Greater Wellington Regional Council the change in trigger levels for review.

Proposed Response, Mitigation and Contingency Plan

Groundwater levels

Dewatering of the excavations is expected and planned for as a part of the construction works. Groundwater monitoring will be used to confirm the groundwater levels are responding as estimated in the modelling.

Two alert levels for groundwater level monitoring have been set, as described above, with each alert level requiring specific management actions as follows:

- **Alert level 1 exceedance:**
  - The Contractor will notify and advise Greater Wellington Regional Council of the alert level exceedance within 24 hours
- **Alert level 2 exceedance:**
  - The Contractor will notify and advise Greater Wellington Regional Council of the alert level exceedance within 24 hours
  - The Contractor will increase the groundwater level, ground settlement and building deflection monitoring frequency to daily at the location of exceedances. This monitoring frequency will continue if a ground settlement or building deflection trigger level has been exceeded as well. If no further settlement or building deflection occurs, the frequency reverts back to weekly
  - Monitoring results are to be submitted to Greater Wellington Regional Council

If ground settlement and/or building deflection trigger levels are exceeded as well, further actions are required as outlined in below.

Ground settlement and building deflection

The settlement and deflection monitoring will be used to confirm if ground settlements and building deflections are within the estimated range. Responses and mitigation measures as outlined below are required in the unlikely event that alert or alarm trigger levels are exceeded:

- **Alert level exceedance:**

  "Alert" exceedance level means the ground settlement is still less than expected but approaching the estimated settlement. In the event of an alert exceedance the following steps shall be taken:

  - The Contractor will notify and advise Greater Wellington Regional Council of the alert level exceedance within 24 hours

- The Contractor will re-measure all monitoring stations within 20 m of the affected monitoring location to confirm the extent of apparent settlement and/or deflection. Re-measurements are undertaken for all these monitoring stations every two days until the written report has been submitted to Greater Wellington Regional Council

- Prepare to institute mitigation measures in consultation with the consultants. These may include measures to reduce dewatering, increase the stiffness of the support measures etc

- A written report, prepared by a SQEP, is to be submitted to the Council within five working days of the Alert Level exceedance

- **Alarm level exceedance:**

  "Alarm" exceedance means the ground settlement has reached or exceeds the estimated settlement, and any further settlement and/or deflection may cause damage to nearby buildings or structures. Note that asset damage is still not expected at this level. In the event of an alarm exceedance the Contractor will take the following steps:

  - Initiate a "stop work" and implementation of mitigation measures as outlined following the Alert level exceedance, or as otherwise required to minimise risks of damage to nearby buildings and structures

  - Notify Greater Wellington Regional Council within 24 hours of the Alarm Level exceedance and provide details of measurements taken

  - The works will be assessed by a SQEP to identify the reasons for the ground settlements and reconsider the design assumptions

  - Undertake a condition survey by a SQEP on any building or structure located adjacent to any monitoring station where the Alarm Level has been exceeded

  - The SQEP will recommend and oversee the implementation of mitigation measures such as additional ground support (e.g., additional struts or anchors) and / or groundwater recharge wells to reduce further settlement and/or deflection and prevent asset damage. The SQEP may also propose additional monitoring instrumentation be installed at the affected area

  - Within five working days of recommencement of works, a report will be prepared by the SQEP and submitted to Greater Wellington Regional Council of the alarm exceedance being identified. The report will include an analysis of all relevant monitoring data and comparison with the initial design, details of the mitigation measures implemented and the estimated risk of further ground settlement. The report will also include the results of the condition surveys and of asset damage, as well as any remedial works and/or agreements with affected parties

Response to damage

- **Building damage:**

Building monitoring is proposed to establish early warning systems against significant damage. Thus, if a building is found to have been damaged as a result of the construction works, either at a post-construction building survey or at one requested during construction, Greater Wellington Regional Council will be notified immediately as per the above. Should building repairs be required, these will be undertaken at the cost of the Contractor as soon as practicable. The timing and extent of repairs may vary depending on the building owner's requirements.

- **Utility and infrastructure damage:**

If trigger levels, in particular differential settlement trigger levels are exceeded at monitoring points related to utilities or infrastructure, the Contractor will immediately notify Greater Wellington Regional Council and the utility provider. A condition survey will be undertaken to determine the level and extent of any damage. Should the survey find that damage has occurred as a result of the construction works, the Contractor will notify Greater Wellington Regional Council and propose a methodology to repair the damage and prevent further damage.