Appendix L

MARSHALL DAY
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GREY LYNN TUNNEL
ASSESSMENT OF NOISE EFFECTS

Rp 002 20180726 | 13 February2019

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TABLE OF CONTENTS

EXECUTIVE SUMMARY

This report provides an assessment of noise effects from the construction and operation of the Grey Lynn Tunnel.

Daytime construction noise emissions and night-time operational noise are the primary issues of note.

The assessment discusses the guideline noise criteria from the Auckland Unitary Plan (Operative in Part) (AUP (OP)); outlines the acoustic effects assessment methodology; predicts noise levels and assesses the potential impacts from the construction and operation of the Project.

It is recommended that the guideline criteria contained in the AUP (OP) are adopted. The aim is to achieve compliance with these criteria where practicable. In accordance with Section 16 of the Resource Management Act 1991 (RMA) the best practicable option should be adopted to ensure that noise effects do not exceed a reasonable level and where there is predicted temporary exceedance, appropriate noise mitigation measures should be put in place.

The predictions contained in this assessment are conservative and cover the anticipated envelope of potential noise effects based on the current construction methodology for the project. However, the assessment is also broad enough to cover the anticipated effects envelope, should alternative construction techniques be used.

Construction noise has been predicted using equivalent noise source data from other similar projects and from information contained in NZS 6803: 1999 and BS 5228-1: 2009. Tables are provided that show potential worst-case noise levels from the construction activities proposed. The predictions are based on assumptions and estimates detailed in the indicative construction methodology. There may be some variation in the actual methodology or equipment used to carry out the work as the final decision would be made by the lead Contractor. However, the Construction Noise and Vibration Management Plan will contain the procedures necessary for identifying, mitigating and managing any potential noise issues through an adaptive management approach, as has historically occurred on various large infrastructure projects in Auckland.

Some activities are predicted to temporarily exceed the relevant noise limits and may therefore require activity-specific management and mitigation. These will be addressed via Activity Specific Noise and Vibration Management Plans.

General acoustic management and mitigation measures are recommended to be implemented throughout the course of the Project as a best practice provision, including maintenance of equipment to a high level and the avoidance of unnecessary noise such as the use of horns, tonal reverse alarms or clearing excavator buckets by hitting the ground.

Overall, the construction of the Grey Lynn Tunnel is predicted to result in noise levels that are generally within the applicable noise limits, with some exceptions. Whilst construction noise levels are higher than ongoing operational levels, it is commonly accepted that for any construction to occur, acoustic criteria must be less stringent, with the understanding that construction is a temporary activity with a finite duration. With appropriate mitigation and management measures in place the effects of construction noise will be minor.

Operation noise from the proposed plant room has been predicted using SoundPLAN noise modelling software. With the proposed conceptual acoustic mitigation measures in place, plant room noise is predicted to comply with the relevant night-time noise limit. It is concluded that the operational noise effects would be less than minor.

1.0 INTRODUCTION

Watercare Services Limited (Watercare) is the water and wastewater service provider for Auckland. Watercare is proposing to construct a wastewater interceptor from Western Springs to Tawariki Street, Grey Lynn (Grey Lynn Tunnel). The Grey Lynn Tunnel will connect to the Central Interceptor at Western Springs.

The potential acoustic amenity impacts on residential receivers from the construction of the Grey Lynn Tunnel is the principal issue of concern. It is noted that tunnelling is a continuous activity (i.e. operates 24/7) once it commences, therefore potential night-time effects from regenerated noise have also been considered. The operation of the Grey Lynn Tunnel is anticipated to generate noise of little appreciable significance, given the absence of mechanical ventilation and air filtration systems.

This report and assessment is submitted to accompany an application for resource consents and a notice of requirement by Watercare for the construction, operation and maintenance of the Grey Lynn Tunnel.

2.0 PROJECT OVERVIEW

The Grey Lynn Tunnel involves the elements shown in the drawings and outlined in more detail in the reports which form a part of the application. These elements are summarised in the following sections.

Figure 1 indicates the proposed tunnel alignment and shaft site locations. The tunnelling will be undertaken within a 40m corridor centred on the alignment shown in the figure.

Figure 1: Overview of Indicative Tunnel Alignment

2.1 Grey Lynn Tunnel

The Grey Lynn Tunnel involves construction, operation and maintenance of a 1.6km gravity tunnel from Western Springs to Tawariki Street, Grey Lynn with a 4.5m internal diameter, at an approximate depth of between 15 to 62m below ground surface, depending on local topography. The tunnel will be constructed northwards from Western Springs using a Tunnel Boring Machine (TBM). The Grey Lynn Tunnel will connect to the Central Interceptor at Western Springs via the Western Springs shaft site.

2.2 Tawariki Street Shaft Site

The Grey Lynn Tunnel also involves construction, operation and maintenance of two shafts and associated structures at Tawariki Street, Grey Lynn ("Tawariki Street Shaft Site").

The Tawariki Street Shaft Site will be located at 44, 46 and 48 Tawariki Street where the majority of the construction works will take place. Construction works will also take place within the road reserve at the eastern end of Tawariki Street and a small area of school land (St Paul's College) bordering the end of Tawariki Street (approximately 150m^2).

The Tawariki Street Shaft Site will involve the following components:

2.2.1 Main Shaft

- A 25m deep shaft, with an external diameter of approximately 10.8m, to drop flow from the existing sewers into the Grey Lynn Tunnel;
- Diversion of the Tawariki Local Sewer to a chamber to the north of the shaft. This chamber will be approximately 12m long, 5m wide and 5m deep below ground, and will connect to the shaft via a trenched sewer;
- Diversion of the Orakei Main Sewer to a chamber to the south of the shaft. This chamber will be approximately 10m long, 5m wide and 11m deep below ground, and will connect to the shaft via a pipe-jacked sewer;
- Construction of a stub pipe on the western edge of the shaft to enable future connections (that are not part of this proposal) from the CSO network;
- Construction of a grit trap within the property at 48 Tawariki St to replace the existing grit trap located within the Tawariki Street road reserve. The replacement grit trap will be approximately 16m long, 5m wide and 13m deep below ground;
- Permanent retaining of the bank at the end of Tawariki Street to enable the construction of the chamber for the Orakei Main Sewer. The area of the bank requiring retaining will be approximately 44m long, 3m wide and 2m high; and
- An above ground plant and ventilation building that is approximately 14m long, 6m wide and 4m high. An air vent in a form of a stack will be incorporated into the plant and ventilation building and discharge air vertically via a roof vent. The vent stack will be designed with a flange to allow future extension of up to 8m in total height and approximately 1m in diameter in the unexpected event of odour issues.

Refer to Appendix B for the proposed concept design of the Tawariki Street shaft site and connection sewers.

2.2.2 Tawariki Connection Sewer Shaft – Secondary Shaft

A secondary shaft will be constructed at the Tawariki Street Shaft Site to enable the connection of future sewers (that are not part of this proposal) from the Combined Sewers Overflows ("CSO") network. This will involve the following components:

- A 25m deep drop shaft with an external diameter of approximately 10.2m; and
- A sewer pipe constructed by pipe-jacking to connect the secondary shaft to the main shaft.

2.3 Construction Timeframe

The construction works for the main shaft, chambers and tunnel will occur at the same time as works for the Central Interceptor. Construction will be up to 2 ½ years total duration. The construction of the main shaft and chambers is estimated to take approximately 12 months initially, followed by a hiatus of several months waiting for the TBM to arrive at Tawariki Street Shaft Site. This will be followed by approximately 9 months of activity to remove the TBM and complete the internal structure of the main shaft.

The secondary shaft will be constructed in conjunction with the future sewers at a later date but (subject to need) within a 10-year period following construction of the main shaft and tunnel. The construction period for the secondary shaft and future sewer connections is estimated to be up to 2 years total duration.

2.4 Nearest Potentially Sensitive Receivers

There are a number of receivers that may potentially be adversely affected by noise from the Tawariki Street shaft site. The following table identifies these receivers, their zoning, use and distance to site.

Table 1: Receiver Locations

Address/location	Zoning / Usage	Distance to Works (setback distance, m)[1]
Marist Catholic School	Special Purpose / Education	40
29 Tawariki Street	Residential / Dwelling	40
33 Tawariki Street	Residential / Dwelling	27
35 Tawariki Street	Residential / Dwelling	25
36 Residential / Dwelling	Residential / Dwelling	44
37 Tawariki Street	Residential / Dwelling	25
38 Tawariki Street	Residential / Dwelling	30
39 Tawariki Street	Residential / Dwelling	22
40 Tawariki Street	Residential / Dwelling	21
41 Tawariki Street	Residential / Dwelling	20
42 Tawariki Street	Residential / Dwelling	Adj. West boundary (10m to dwelling))

Notes to table:

(1) Distance is from building façade to closest shaft site boundary

3.0 EXISTING ACOUSTIC BASELINE

To gain an understanding of the existing environmental noise baseline for dwellings in proximity to the proposed above-ground plant room, an attended noise measurement was carried out on 28 November 2018 between 10:00pm and 10:30pm. The weather at the time of the survey was clear skies with a light breeze present, and therefore within the allowable meteorological window prescribed in NZS6801:2008. The measurement was undertaken in accordance with the relevant standards[1]. The position, marked MP1 in the figure overleaf, is considered a representative location to measure the existing environment of receivers located around the proposed plant room.

Figure 2: Ambient Measurement Position

Source: https://unitaryplanmaps.aucklandcouncil.govt.nz/upviewer/

The measured noise levels are shown in Table 2.

Table 2: Measured Ambient Noise Levels

Measurement Position	Measurement		Measured Level (dB) (1)				Noise Source (2)
	Start Finish Times	Duration min:sec	L_{Amax}	L_{A10}	L_{Aeq}	L_{A90}	
MP1	22:12 pm 22:27pm	15:22	50	41	38	35	Wind in trees, crickets, distant aircraft, distant traffic, household noise, dog barking

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[1] AUP Standard E25.6.1 (1)

Notes to table:

(1) An explanation of technical terms is provided in Appendix A

4.0 ACOUSTIC PERFORMANCE STANDARDS AND LEGISLATION

4.1 Resource Management Act 1991 (RMA)

Under the provisions of the RMA there is a duty to adopt the best practicable option to ensure that noise (including vibration[2]) from any development does not exceed a reasonable level. Specifically, Sections 16 and 17 reference noise effects as follows.

Section 16 states that "every occupier of land (including any premises and any coastal marine area), and every person carrying out an activity in, on, or under a water body or the coastal marine area, shall adopt the best practicable option to ensure that the emission of noise from that land or water does not exceed a reasonable level".

Section 17 states that "every person has a duty to avoid, remedy, or mitigate any adverse effect on the environment arising from an activity carried on by or on behalf of the person, whether or not the activity is in accordance with –

(a) Any of sections 10, 10A, 10B and 20A; or

(b) A national environmental standard, a rule, a resource consent, or a designation"

This report uses the guiding principles of Section 16 and 17 of the RMA as noted above in assessing effects and recommending mitigation measures. It considers the potential construction and operational noise effects of the Grey Lynn Tunnel. The potential vibration effects associated with the construction of the Grey Lynn Tunnel are separately assessed in the Vibration Assessment.

4.2 Auckland Unitary Plan Operative in Part (AUP (OP))

The Tawariki Street Shaft site is on land with an underlying zone classification of *Residential – Mixed Housing Urban Zone* in the AUP (OP). The closest potentially sensitive sites are also within this zone. Saint. Pauls School and associated playing fields are zoned *Special Purpose – School Zone.*

Figure 3 shows the relevant AUP zones for the indicative designation boundary and surrounding area.

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[2] RMA 1991 Part 1 Section 2 Interpretation: Noise includes vibration

Figure 3: AUP Zones

Shaft Site Designation (approximate extent)

Source: https://unitaryplanmaps.aucklandcouncil.govt.nz/upviewer/

The following details the AUP (OP) noise performance standards relevant to the identified receiving zones.

4.2.1 Construction Noise Limits

Standard E25.6.1 (3) of the AUP (OP) states that noise from any construction work activity must be measured and assessed in accordance with the requirements of New Zealand Standard NZS 6803: 1999 *"Acoustics - Construction Noise"*.

Standard E25.6.27 (1) sets out the noise limits for construction (refer to Appendix C). As the anticipated length of construction exceeds 20 weeks, Standard E25.6.7 (4) would apply, resulting in the construction noise limits set out in Table E25.6.27.1 decreasing by 5 decibels.

In summary, the reduced noise limits for noise affecting sensitive activities is 70dB L_{Aeq} / 85dB L_{Amax} between 7.30am and 6.00pm.

A Construction Noise and Vibration Management Plan (CNVMP) would be required, in accordance with Standard E25.6.29 (5).

4.2.2 Operation Noise Limits

Noise received by dwellings in residential zones

Standard E25.6.2 (1) of the AUP (OP) states that noise from any activity within the residential zone, when measured on another site in the same zone, must not exceed the limits in Table E25.6.2.1, reproduced as follows:

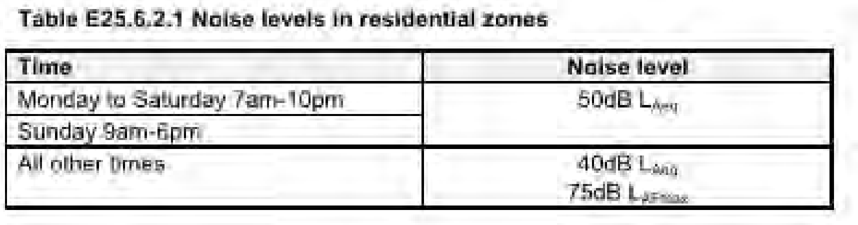

Table E25.6.2.1 Noise levels in residential zones

Time	Noise level
Monday to Saturday 7am-10pm	50dB L_{Aeq}
Sunday 9am-6pm	
All other times	40dB L_{Aeq} 75dB L_{AFmax}

4.3 Night-time Regenerated Noise Amenity

Night-time tunnelling beneath dwellings has the potential to cause sleep disturbance due to regenerated noise. This is where noise is generated in a room through the vibration of its walls, ceiling, floor and sometimes fittings.

MDA considers a suitable regenerated noise criterion to be 35 dB $L_{Aeq(15min)}$, which is deemed to be a satisfactory noise level for bedrooms in suburban areas or near minor roads[3]. A similar criterion has been adopted on another major infrastructure project[4] in Auckland for tunneling noise in hotel bedrooms between the hours of 10.00pm and 7.00am.

5.0 CONSTRUCTION NOISE ASSESSMENT

As typically occurs on large infrastructure projects such as the Grey Lynn Tunnel, a detailed construction programme would be developed prior to the commencement of construction activities. It is anticipated that this will be prepared by the lead contractor and incorporated into the project's Construction Management Plan. As such, the following preliminary assessment of construction noise has been based on an indicative construction methodology (refer to Appendix D).

5.1 Noise Prediction Methodology

Construction noise has been predicted in general accordance with the method detailed in Annex D[5] of NZS6803:1999. The method considers the sound power level, periods of operation, distance from source to receiver and screening of each source, as well as façade reflection and the degree of soft ground attenuation.

5.2 Predicted Noise Levels During Project Construction

The following table sets out the plant and activities anticipated to be used in the construction of the Tawariki Street Shaft and connection sewer. The table includes the per unit sound power level, a 10dB reduction from acoustic screening (refer to Appendices E and F for further details) and the minimum distance required to comply with the AUP (OP) reduced noise limit of 70dB L_{Aeq}.

The predictions are based on the assumption that work would be carried out during normal construction hours of 7.30am to 6.00pm (lower noise limits apply outside these hours).

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[3] Australian/New Zealand Standard AS/NZS 2107:2016 *"Acoustics - Recommended design sound levels and reverberation times for building interiors"*

[4] City Rail Link NoR – 35 dB $L_{Aeq\ (15\ min)}$ between 10pm and 7am

[5] Annex D refers to BS5228-1: 1997 (now superceded by BS 5228-1:2009)

Table 3: Predicted Construction Noise Levels (WITH SCREENING MITIGATION)

Activity	Equipment	Sound Power	Mitigation	Façade Noise Level (dB L_{Aeq})			Limit Setback (m)
		(dB L_{WA})	(dB)[1]	10	20	40	70dB L_{Aeq}
Tawariki Street Shaft and Chamber							
Excavation and Support	30T excavator (sheet piling)	116	0[2]	*91*	*85*	*78*	83
	30T excavator (digging)	103	-10	68	62	55	8
	3-axle truck	105	-10	70	64	57	10
	Hydraulic power pack	102	-10	67	61	54	7
	Generator	103	-10	68	62	55	8
	Shaft ventilation	102	-10	67	61	54	7
	Grout pump	107	-10	*72*	66	59	13
	Dewatering pump	97	-10	62	56	49	4
	Water treatment	95	-10	60	54	47	3
	Concrete truck + pump	107	-10	*72*	66	59	13
	Plate compactor	106	-10	*71*	65	58	11
Construction	30T excavator	103	-10	68	62	55	8
	20T mobile crane	99	-10	64	58	51	5
	50T crane	98	-10	63	57	50	4
	3-axle truck	105	-10	70	64	57	10
	Hiab truck	97	-10	62	56	49	4
	Dewatering pump	97	-10	62	56	49	4

Notes to table:

(1) Screening of -10dB provided by site acoustic barrier
(2) Due to the elevated nature of this activity the acoustic barrier would be ineffective

As set out in the table above noise from some construction activities, most notably intermittent sheet piling works, is predicted to exceed 70dB L_{Aeq}. This is not uncommon for large infrastructure projects undertaken in proximity to sensitive receivers. The predicted exceedances trigger the requirement for noise mitigation and effects management via a CNVMP.

As discussed, even with the proposed 3m high site hoarding in place, sheet piling noise would still exceed the 70dB L_{Aeq} limit at some receivers due to the elevated height of this noise source above the hoarding and would therefore need to be managed via the CNVMP to mitigate the otherwise appreciable potential noise effects from it. It is considered that with the management and mitigation measures in place effects from construction noise can be acceptably managed.

Refer to Appendix G for noise contour predictions of piling works associated with site construction. The contours indicate the 'envelope of effects'; receivers located within the 70dB L_{Aeq} contour are listed in the following table.

Table 4: Identified Receivers Predicted to Exceed Noise Limit During Sheet Piling

Receiver	Predicted Noise Level (70dB L_{Aeq} Noise Limit)
Marist Catholic School	72
29 Tawariki Street	72
33 Tawariki Street	73
35 Tawariki Street	76
36 Tawariki Street	73
37 Tawariki Street	79
38 Tawariki Street	76
39 Tawariki Street	82
40 Tawariki Street	77
41 Tawariki Street	84
42 Tawariki Street	83

Noisy construction should generally be programmed to occur between 7:30 am and 6:00 pm (normal construction hours), with no significant construction occurring outside these hours, Monday to Saturday. No construction should occur on Sundays nor outside normal construction hours unless supported by an Activity Specific Noise and Vibration Management Plan (ASCNVMP).

The CNVMP will be important in ensuring that any construction noise and resulting effects are practicably controlled.

5.3 Regenerated Noise During Night-time Tunnelling

Tunnelling beneath dwellings during the night-time will occur. To determine the potential for regenerated noise effects on residential receivers manifested as sleep disturbance, MDA has referenced previous project experience regarding regenerated noise versus slant distance[6] from tunnelling plant to receiver.

To comply with a regenerated noise criterion of 35 dB $L_{Aeq\ (15\text{-min})}$ (refer to Section 4.3 for criterion discussion) a minimum vibration slant distance of approximately 18m from buildings with bedrooms located on the ground floor and 15m from buildings with bedrooms on the first floor (building junctions provide vibration attenuation). Any building along the proposed alignment at a closer distance is at risk of exceeding the regenerated noise criterion.

Table 5 identifies the properties which will have the shallowest depth to pipe crown and therefore the shortest slant distance, based on the pipe crown being at the top of the proposed vertical alignment window[7]. All other properties are calculated to have a depth of 20m or greater and would therefore comfortably comply with the criterion.

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[6] The vector distance between the tunnelling source and the receiving building's foundation or floor level

[7] Watercare is seeking resource consent for a 40m wide horizontal corridor and 4m vertical corridor for the tunnel alignment

Table 5: Slant Distance Summary

Receiver Address	Zone/Use	Building Type	Min Depth to Pipe Crown (Slant Distance)[1]	Complies with Criterion? / Comment
Dwelling:				
30 Sackville Street	Residential – Single House Zone / Dwelling	Single-storey	18.0	Complies
2/30 Sackville Street	Residential – Single House Zone / Dwelling	Single-storey	18.0	Complies
32 Sackville Street	Residential – Single House Zone / Dwelling	Double-storey	16.9	Exceeds criterion. Consultation required
34 Sackville Street	Residential – Single House Zone / Dwelling	Double-storey	15.5	Exceeds criterion. Consultation required
37 Tawariki Street	Residential – Mixed Housing Urban Zone / Dwelling	Single-storey	18.4	Complies
39 Tawariki Street	Residential – Mixed Housing Urban Zone / Dwelling	Single-storey	18.0	Complies
No Dwelling:				
36 Sackville Street	Open Space / Hakanoa Reserve	None	15.3	No dwellings
38 Sackville Street	Residential – Single House Zone / Daycare	Single-storey	15.9	Not a dwelling

Notes to table:

(1) 2m has been subtracted off the baseline depths provided by McMillen Jacobs Associates to calculate the worst-case pipe crown distance within the vertical alignment window Watercare is seeking

Table 5 shows that two properties with dwellings located at 32 and 34 Sackville Street have a calculated worst-case slant distance of less than the 18m threshold and may therefore potentially experience regenerated noise above 35dB L_{Aeq} for no more than 1-2 days (based on an estimated tunnelling rate of 10-20m per day). The remaining properties in the table with a dwelling have a calculated worst-case slant distance of 18.0m or greater and are predicted to experience a regenerated noise level of or slightly below 35dB L_{Aeq}. The two properties without a dwelling, namely 36 and 38 Sackville Street, do not have dwellings therefore, there is no potential for adverse effects.

Based on the above, regenerated noise resulting from tunnelling vibration during the night-time will generally not result in any appreciable sleep disturbance effects. However, there may be instances where tunnelling noise is audible. Advance communication and consultation with the identified stakeholders is recommended to address any concerns. Pre and post construction building condition surveys may also need to be offered to alleviate resident's concern about potential building damage upon hearing the tunnelling noise.

5.4 Construction Traffic Movements on Road Network

Although not explicitly required by AUP (OP) provisions, given the size of the project, MDA has considered the potential noise impact of increased truck movements on the surrounding road network during construction.

The Commute Transportation Assessment[8] states that the highest number of truck movements will be during Stage 1 – the main shaft and chambers excavation, with an estimate of 64 peak movements per day (average of five movements per hour over a 12-hour working day) over a period of 12 months. Comparatively, Stage 2will generate significantly less truck movements. Stage 3 and secondary shaft construction is estimated to generate a similar or lower level of movements compared to Stage 1.

Stage 1 is estimated to generate the highest number of truck movements and therefore forms the basis of the following effects assessment.

The following scenarios have been modelled to ascertain the effect of construction traffic on road noise levels:

- Existing Baseline: Existing traffic counts and heavy vehicle volumes
- Scenario 1: An additional 64 heavy vehicle movements per day on each of the roads

Using traffic count data and trip generation estimates sourced from Commute, MDA has predicted traffic noise levels for two scenarios using the CoRTN algorithm[9]. The resulting change in traffic noise level for a receiver nominally located at 15m from road's edge is set out in the following table. Comparison to the Existing Baseline scenario indicates the change in noise level resulting from project construction traffic operating on surrounding roads.

Table 6: Predicted Change in Traffic Noise on Road Network

	AADT / HCV % / Predicted Road Traffic Noise Level (dB $L_{Aeq\ 1\text{-}hour}$)$^{1, 2,3}$		Change in Level
Road	Existing Baseline	Scenario 1	
Tawariki Street	208 / 3% / 50	264 / 25.5% / 55	+5dB
Parawai Crescent	7,132 / 3% / 63	7,188 / 3.8% / 63	No change
Richmond Road	11,748 / 4% / 66	11,804 / 4.5% / 66	No change
Surrey Crescent	11,200 / 4.8% / 70	11,256 / 5.3% / 70	No change
Great North Road	24,420 / 5.5% / 69	24,476 / 5.7% / 69	No change

Notes to table:

(1) Predictions are based on a nominal receiver distance from the road of 15m and a speed of 50km/h

(2) AADT = Annual Average Daily Traffic; HCV % = Heavy Commercial Vehicle (expressed as a percentage of total daily flow)

(3) Data sourced from mobileroad.org

The results in the table indicate that the increased truck movements and number of heavy vehicles on the identified roads would result in an imperceptible increase in noise when assessed over a daytime hour for all roads except for Tawariki Street. For Tawariki Street, noise levels are predicted to increase by approximately 5 decibels. A 5-decibel increase is an appreciable change in noise level.

MDA concludes that, given the relatively moderate number of trips generated during construction works and where these movements occur during normal construction hours of 7.30am to 6.00pm, no adverse traffic noise effects are anticipated.

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[8] Commute Transportation Assessment Section 3.7

[9] An adjustment has been applied to the output to convert from L_{10} to L_{eq} descriptor

The inclusion of the management of truck traffic should be included in the CNVMP to avoid trucks sitting outside the site for extended periods with engines running. This should also consider addressing and mitigating truck reverse beeper noise.

6.0 OPERATION NOISE

6.1 Noise Prediction Methodology

Operation noise has been predicted in general accordance with the algorithm detailed in ISO 9613-2:1996[10] as implemented in SoundPLAN® environmental noise modelling software.

ISO 9613 considers a range of frequency dependent attenuation factors, including spherical spreading, atmospheric absorption, ground effect and acoustic screening.

6.2 Noise Modelling Inputs, Assumptions and Proposed Mitigation

The operational noise emission from the project will be minimal. An above-ground single-storey plant room will house the power supply and controls for the penstocks. A passive air vent will be required for continuous air entry into the tunnel for ventilation purposes. Air exhaust may be passive or mechanical although noise from this event would occur infrequently i.e. only when the tunnel is nearly full during severe wet weather events.

The noise source sound power levels used in the assessment are listed in Appendix H.

The following details the assumed conceptual mitigation measures needed to ensure that operation noise complies with the relevant AUP (OP) limits and to ensure that noise remains reasonable.

Above Ground Plant Room

- Walls facing dwellings assumed to be constructed from precast concrete or an alternative material / design giving equivalent performance;
- Ceiling lined with an absorptive product;
- Plant room roof constructed from an insulated roofing product with a minimum performance of R_w 24dB;
- Roller door (where it faces a dwelling) to be acoustic with a minimum performance of R_w 24dB;
- Solid core access doors facing dwellings; and
- Where the air exhaust is by mechanical means the outlet-stack should be fitted with an attenuator to limit the sound power level leaving to no more than 78dB L_{WA}.

The above measures would be confirmed during the detailed design stage.

6.3 Operation Noise Predictions

Noise emissions from the proposed plant room and shaft ventilation system have been predicted to adjacent receiver locations and assessed against the relevant night-time noise limit.

The following table sets out the predicted operation noise levels. Refer to Appendix I for the predicted night-time noise contour.

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[10] ISO 9613-2: 1996 "Acoustics – Attenuation of sound during propagation outdoors – Part 2: General method of calculation"

Table 7: Plant Room Noise Levels

Receiver Location	Zone/AUP Night-time Limit [dB L_{Aeq}]	Predicted Noise Level (dB L_{Aeq})
33 Tawariki Street	Residential [40]	33
35 Tawariki Street	Residential [40]	34
37 Tawariki Street	Residential [40]	36
39 Tawariki Street	Residential [40]	38
41 Tawariki Street	Residential [40]	38
42 Tawariki Street	Residential [40]	38

Notes to table:

(1) An explanation of technical terms is provided in Appendix A

Based on the levels in the table, operation noise is predicted to comply with the AUP (OP) night-time noise limit at the closest dwellings, with the conceptual acoustic mitigation measures in place.

The noise levels generated by the plant room are predicted to be similar to the existing background noise level (refer to Table 2). As such, no adverse noise effects are anticipated from its operation.

7.0 MITIGATION AND MANAGEMENT OF CONSTRUCTION NOISE

Potential management and mitigation measures are discussed below.

7.1 Communication and Consultation

The most important tool for managing construction noise is consultation and communication. For the Grey Lynn Tunnel, the recommended daytime criterion is predicted to generally be achieved at dwellings which are located 20m distance and screened from general works.

Communication is needed in relation to the proposed works and their timing with any stakeholders potentially affected by noise levels higher than specified in the AUP (OP). Communication should occur with stakeholders prior to works being carried out, by means of letter drop or face-to-face contact.

7.2 Timing of activities

It is noted that general construction hours may span two periods, namely 06:30am to 07:30am and 07:30am to 6:00pm. Of these periods, the 06:30am to 07:30am period, often termed the ‘morning shoulder’, has a significantly lower noise limit than the daytime period. Therefore, a potential risk exists for construction activities to exceed the morning shoulder criterion by a significant margin, unless early morning site activities are appropriately managed. Two examples would be where trucks with engines running queue up outside the site gates prior to site opening, and crane lift of heavy items delivered by truck during this period.

The management of these issues could take the form of preventing trucks from queuing/idling adjacent to dwellings, prohibiting the use of tonal reverse beepers, and scheduling heavy deliveries to occur after 07:30am. These and others would be addressed via the CNVMP.

7.3 Noise Barriers

In general, placing temporary noise barriers, such as plywood sheets or proprietary ‘noise curtains’, between dwellings and the construction activities can reduce noise levels by up to 10 decibels. It is considered that 3-metre-high site hoardings are sufficient to act as effective noise barriers for ground-based receivers. The barriers should be placed as close as practicably possible to noise sources.

7.4 Avoidance of Unnecessary Noise

At many construction sites it can be observed that some construction practices unnecessarily increase noise levels. Those include the sounding of horns when a truck is fully laden, truck air-brake release and the use of audible, often tonal, reversing alarms.

Those issues can be avoided, or noise levels reduced, by means of changed construction site management; fitting of mufflers to trucks; maintenance of equipment to a high standard and the replacement of audible reversing alarms with visual or lower noise broadband audible reversing alarms. Where these measures are implemented they would form part of best practice management and mitigation of construction noise.

Other unnecessary noise may include shouting, loose tail gates and noise from music / radios played loudly. These can be avoided with good site management and are generally addressed in any CNVMP.

7.5 Construction Noise and Vibration Management Plan

It is common practice for infrastructure projects of significant size to implement a CNVMP as part of the construction management plan. These contain information on site management, mitigation, communication, complaints procedures and similar issues.

The purpose of such a plan is to reduce construction noise and vibration effects through selecting the best practicable option in terms of timing of activities, equipment selection and mitigation measures (or a combination thereof).

The minimum requirements of a CNVMP are set out in NZS6803:1999 Section 8 and Annex E.

The CNVMP should contain, but not be limited to:

- A summary of the project noise criteria;
- A summary of construction noise assessments/predictions;
- General construction practices, management and mitigation;
- Noise management and mitigation measures specific to activities and/or receiving environments;
- The requirement for pre and post-construction building condition surveys;
- Monitoring and reporting requirements;
- Procedures for handling complaints; and
- Procedures for review of the CNVMP throughout the project.

A CNVMP would be implemented for the work site and ASCNVMPs for some specific activities where exceedance of the AUP (OP) limits is predicted and will be kept up-to-date regarding actual timing/equipment use and methodologies, should these change at any point during the construction process.

8.0 SUMMARY AND CONCLUSIONS

MDA has undertaken an assessment of construction and operation noise effects for the Grey Lynn Tunnel.

The relevant acoustic performance standards in the AUP (OP) have been used in the assessment.

The works described in this report are typical construction works in an urban area and are carried out almost daily within Auckland. Construction noise (and vibration, assessed separately) are the principal acoustic issues that may result in potential effects. These effects have been successfully mitigated and managed on many other comparable construction projects, and the Grey Lynn Tunnel will adopt similar management and mitigation measures to ensure a similar outcome.

Noise from the proposed construction activities has been predicted at nominal setback distances from works. Predictions show that certain activities such as sheet piling will temporarily exceed the construction noise limits. The best practicable option (for noise) for this project is to ensure that construction noise effects are managed with the aim of meeting the relevant noise limits and any potential exceedances are identified and addressed through noise management and mitigation.

A project CNVMP is recommended which would be formulated and submitted to Council for certification prior to construction starting. Some activities, such as sheeting piling, would likely require an ASCNVMP.

MDA concludes that construction noise can be controlled to acceptable levels with appropriate mitigation and management measures in place. Communication with receivers located adjacent to the site is recommended so that they are kept informed of the project’s progress.

The operation of the plant room is predicted to comply with the relevant noise criteria at all times with the recommended conceptual acoustic measures in place. Any residual noise effects from its operation would be less than minor.

APPENDIX A GLOSSARY OF TERMINOLOGY

A-weighting	The process by which noise levels are corrected to account for the non-linear frequency response of the human ear. All noise levels are quoted relative to a sound pressure of $2x10^{-5}$Pa
dB	Decibel. The unit of sound level. Expressed as a logarithmic ratio of sound pressure P relative to a reference pressure of Pr=20 mPa i.e. dB = 20 x log(P/Pr)
dBA	The unit of sound level, which has its frequency characteristics modified by a filter (A-weighted) to approximate the frequency bias of the human ear.
$L_{Aeq\ (t)}$	The equivalent continuous (time-averaged) A-weighted sound level. This is commonly referred to as the average noise level. The suffix "t" represents the measurement time interval to which the noise level relates, e.g. (8 h) would represent a period of 8 hours, (15 min) would represent a period of 15 minutes and (2200-0700) would represent a measurement time between 10 pm and 7 am.
$L_{A10\ (t)}$	The A-weighted noise level equalled or exceeded for 10% of the measurement period. This is commonly referred to as the average maximum noise level.
$L_{A90\ (t)}$	The A-weighted noise level equalled or exceeded for 90% of the measurement period. This is commonly referred to as the background noise level.
L_{AFmax}	The A-weighted maximum noise level. The highest noise level that occurs during the measurement period.
NZS 6801:2008	New Zealand Standard NZS 6801:2008 "Acoustics – Measurement of environmental sound"
NZS 6802:2008	New Zealand Standard NZS 6802:2008 *"Acoustics - Environmental Noise"*
NZS 6803:1999	New Zealand Standard NZS 6803: 1999 *"Acoustics - Construction Noise"*
SWL or L_W	Sound Power Level A logarithmic ratio of the acoustic power output of a source relative to 10^{-12} watts and expressed in decibels. Sound power level is calculated from measured sound pressure levels and represents the level of total sound power radiated by a sound source.

APPENDIX B SHAFT SITE GENERAL LAYOUT

PLAN
SCALE 1:250 (A1)

SCALE 1:250 (A1) 0 5 10 15 20 25m 25.0 m

NOTES:

1. CO-ORDINATES ARE IN NZTM AND LEVELS ARE TO AUCKLAND L&S 1946 DATUM.
2. LOCATION OF EXISTING SERVICES HAVE BEEN EXTRACTED FROM AUCKLAND COUNCIL GIS AND UTILITIES PLANS AND ARE INDICATIVE.
3. SITE SURFACE FORMATION MAY REQUIRE RELOCATION OF UTILITIES.

LEGEND

PROPOSED MAIN TUNNEL
PROPOSED LINK SEWER
FUTURE CSO COLLECTOR SEWER AND MANHOLE
PROPOSED SEWERAGE
PROPOSED STORMWATER
PROPOSED POWER/CONTROL DUCT
SW EXISTING STORMWATER
EXISTING STORMWATER STREAM
SS EXISTING NETWORK WASTEWATER
SS EXISTING TRANSMISSION WASTEWATER
W EXISTING WATERMAIN
LV LV POWER CABLE
HV HV POWER CABLE
MV MV POWER CABLE
FIBRE OPTIC CABLE
TRANSPOWER LINE
VECTOR TRANSMISSION LINE
COMM CABLES
T TELECOM
GAS GAS LINE
X X X TO BE ABANDONED
EXISTING SEWER PREVIOUSLY ABANDONED
DESIGNATION BOUNDARY
① SITE OFFICE
② WORKSHOP
③ PARKING
④ SPOIL STORAGE
⑤ WHEEL WASH
⑥ CRAWLER CRANE PAD (MIN. DIMENSION)
⑦ LAYDOWN TEMP. MATERIAL STORAGE

DRAFT

JACOBS AECOM

ISSUE	DATE	AMENDMENT	BY	APPD.
		DRAFT FOR DISCUSSION		

DESIGNED
DES. CHECKED
DRAWN
DWG. CHECKED
PROJECT LEADER
INFRAST'R APP'D
BY DATE

OPERATIONS
INFRASTRUCTURE

Watercare
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DSCIN010 – TAWARIKI STREET
GENERAL ARRANGEMENT
SITE LAYOUT PLAN

CAD FILE 2012917.100
DATE
ORIGINAL SCALE A1 AS SHOWN
CONTRACT No.
REF. No. CI–CIVIL
ISSUE
DWG. No. 2012917 .100

APPENDIX C AUP CONSTRUCTION NOISE LIMITS

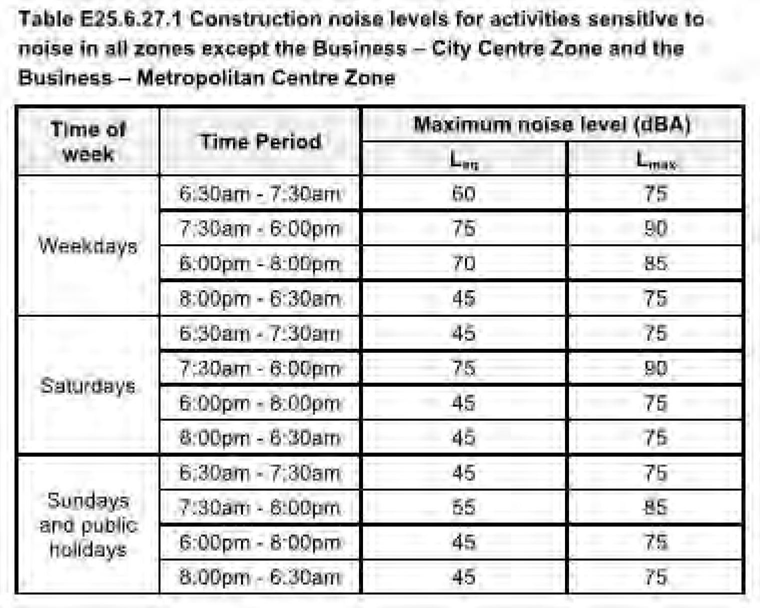


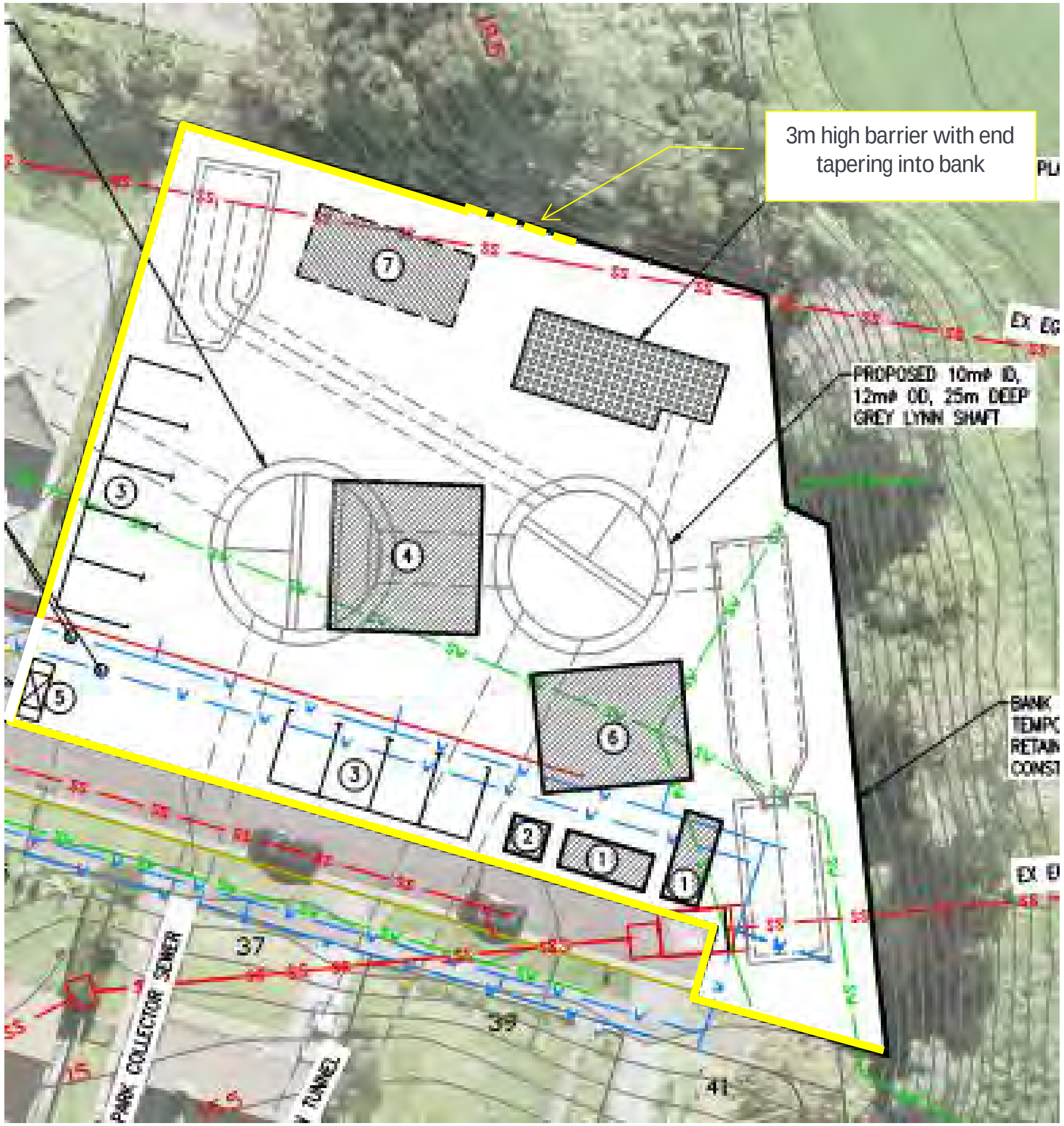
Table E25.6.27.1 Construction noise levels for activities sensitive to noise in all zones except the Business – City Centre Zone and the Business – Metropolitan Centre Zone

Time of week	Time Period	Maximum noise level (dBA)	
		L_{eq}	L_{max}
Weekdays	6:30am - 7:30am	60	75
	7:30am - 6:00pm	75	90
	6:00pm - 8:00pm	70	85
	8:00pm - 6:30am	45	75
Saturdays	6:30am - 7:30am	45	75
	7:30am - 6:00pm	75	90
	6:00pm - 8:00pm	45	75
	8:00pm - 6:30am	45	75
Sundays and public holidays	6:30am - 7:30am	45	75
	7:30am - 6:00pm	55	85
	6:00pm - 8:00pm	45	75
	8:00pm - 6:30am	45	75

APPENDIX D OUTLINE CONSTRUCTION METHODOLOGY

Tawariki Street	
Construction site	44-48 Tawariki St
Anticipated construction access	From Richmond Rd, via Mokau St and Moira St into Tawariki St.
Earthworks	10,000 – 15,000 m^3
Duration of construction	Stage 1: 2.5 years Stage 2 (secondary shaft): 2 years
Principal temporary construction activities	· Shaft excavation and construction – 26-27 m deep shaft, 12m diameter · Shaft excavation support - either secant piles, sheet piles, ring beams with lagging, steel liner plate, precast segmental rings, caisson or similar · TBM retrieval · Excavations for underground permanent works · Blasting will not be used for construction of the shaft as basalt is not anticipated in the shaft excavation · Construction of connections to Orakei Main Sewer and Tawariki CSO (likely trenchless methods)
Key features/equipment	· Shaft excavation with mechanical equipment e.g. CAT 330 medium hydraulic excavator or similar) through overburden soils and East Coast Bay Formation (ECBF) bedrock · One or more cranes · Blasting will not generally be used for construction of the shaft as basalt is not anticipated in the shaft excavation · Water treatment equipment · Storage areas for construction materials · Construction base, including: site access roading, security fencing, site offices · Wheel wash · Grout equipment · Materials storage area · Ventilation equipment · Compressor/generator · Site lighting
Permanent works	· Site to be reinstated upon completion of construction and surfaced with permeable paving (“Surepave” or similar) in the vicinity of shafts/chambers/accessways and grass for the remainder of the site. · The shaft roof slabs (i.e., lids) will be buried except for manholes and hatches at the ground surface which will be secured from public entry. At the completion of construction, the ground surface will be restored to the pre-existing conditions. · Connection to Orakei Main Sewer and Tawariki CSO. · Underground chambers fitted with penstocks · Above-ground plant room to house power supplies and controls for penstocks (90m^2, single-storey) · Air vent –an underground 1.5 m diameter air duct from the shaft to an air intake/exhaust ranging from a vent about 3m high integrated with the plant room, to a 1.5 m diameter 8 m high stack.
Future works	· Combined Sewers Overflow shaft (constructed adjacent to the tunnel shaft at a later date; approx. 10 m diameter and 25 m deep.

APPENDIX E CONSTRUCTION NOISE ACOUSTIC BARRIER LOCATION



APPENDIX F ACOUSTIC SCREEN CONSTRUCTION OPTIONS

Type	Constructions [Refer Notes (1) to (4) below]	
Timber (6)	Supporting Structure:	Timber, steel or aluminium posts and rails.
	Cladding Option 1:	*Plywood* panelling (5) with a minimum surface mass of 10 kg/m^2 (18mm minimum thickness).
	Cladding Option 2:	*Timber Palings* (minimum thickness of 20-25mm) either overlapped or close-boarded with battens over gaps between palings (6).
Fibre Cement	Supporting Structure:	Timber, steel or aluminium.
	Cladding Option 1:	9mm (min. thickness) *Fibre Cement* sheet (1 layer)
	Cladding Option 2:	7mm (min. thickness) *Compressed Fibre Cement* sheet (1 layer)
Acrylic	Supporting Structure:	Steel, aluminium or concrete.
	Infill panels:	12mm thick *Acrylic panels*.
Glass	Supporting Structure:	Steel, aluminium or concrete.
	Infill Panels:	Laminated glass (6mm minimum thickness).
Brick	Supporting Structure:	Concrete footing.
	Infill:	70mm mortared brick
Concrete	Supporting Structure:	Concrete footing.
	Infill:	Reinforced concrete or mortared concrete block (filled or unfilled).
Earth Bund		Earth or suitable fill material.

Notes:

(1). Any proposed acoustic screen shall be designed and certified by a suitably qualified structural engineer and relevant consents sought from the local council and other interested parties prior to its construction

(2). Acrylic and glass sections can be used to provide an acoustic screen while retaining visual transparency

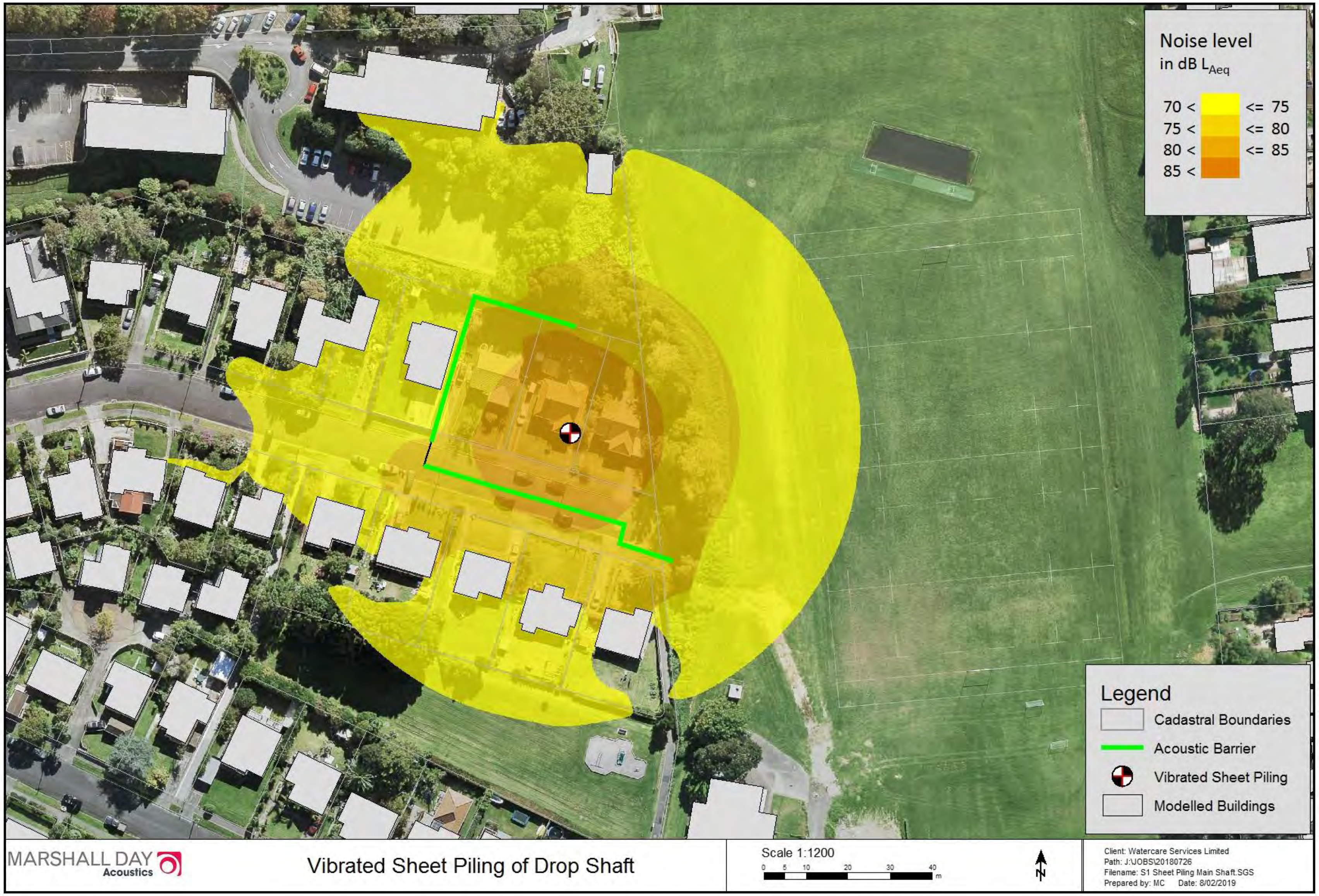
(3). For all fence constructions, ensure that there are no gaps in the screen or between the ground and the bottom of the screen

(4). Any proposed acoustic screen shall be designed with input from a suitably qualified acoustic consultant

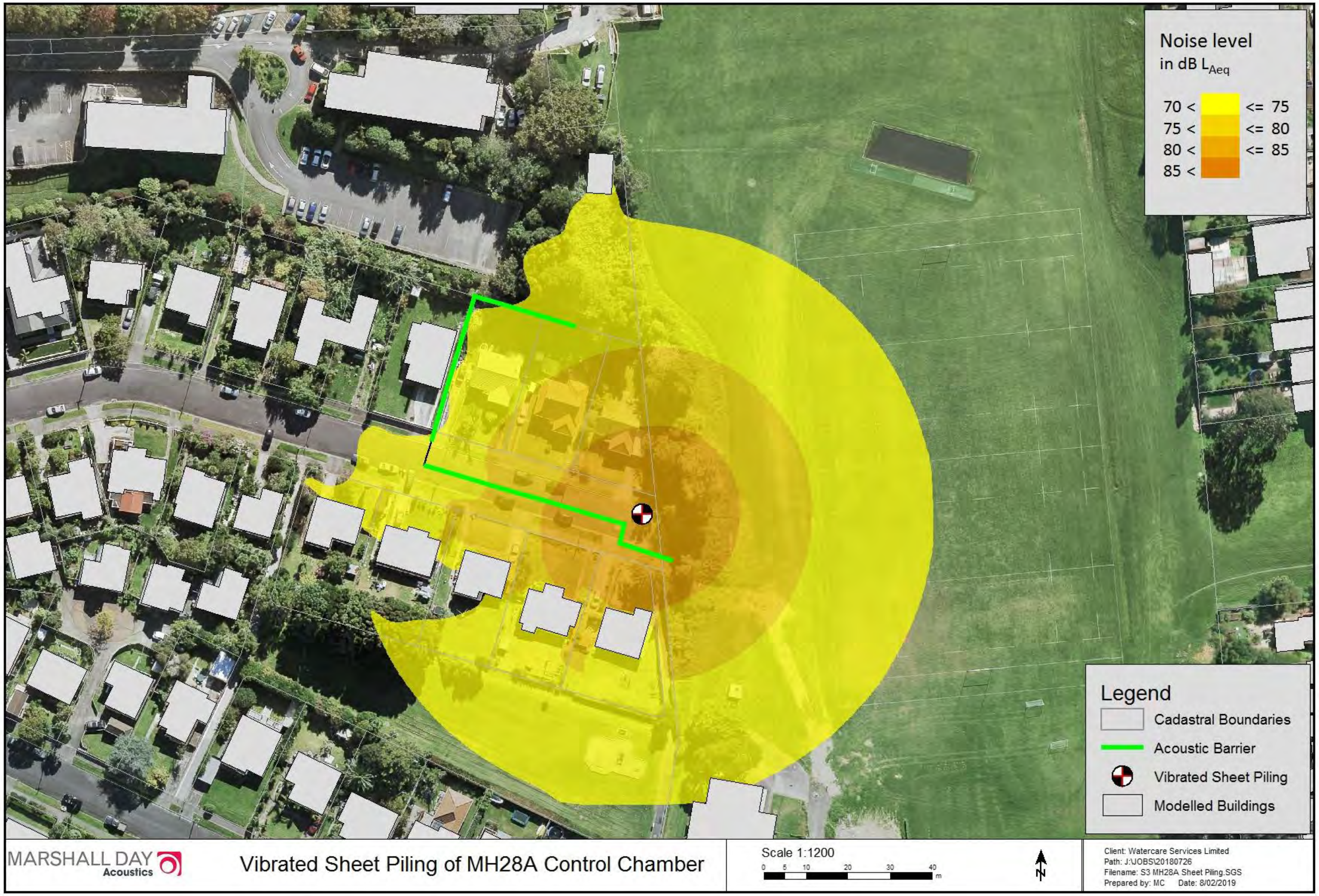
(5). Grooved plywood, manufactured to resemble a timber paling fence design, can be used to achieve a similar look to a close boarded fence design

(6). Plywood panelling is preferred to a close boarded fence design for long-term durability

APPENDIX G WORST CASE CONSTRUCTION NOISE CONTOUR PREDICTIONS



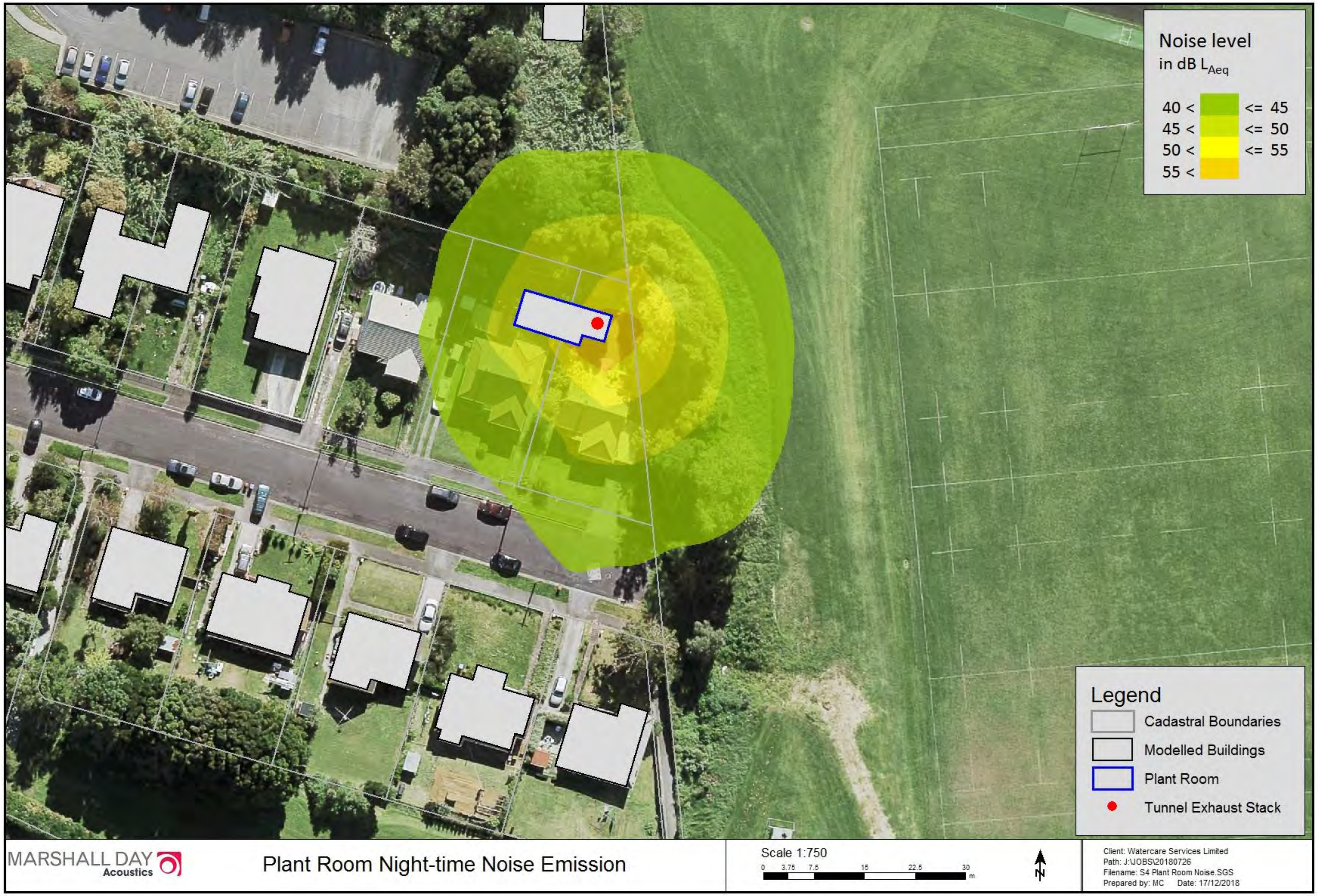
Vibrated Sheet Piling of MH01 Control Chamber


Noise level
in dB L_{Aeq}
70 < <= 75
75 < <= 80
80 < <= 85
85 <
Legend
Cadastral Boundaries
Acoustic Barrier
Vibrated Sheet Piling
Modelled Buildings
MARSHALL DAY Acoustics
Vibrated Sheet Piling of MH28A Control Chamber
Scale 1:1200
0 5 10 20 30 40 m
N
Client: Watercare Services Limited
Path: J:\JOBS\20180726
Filename: S3 MH28A Sheet Piling.SGS
Prepared by: MC Date: 8/02/2019

APPENDIX H OPERATION NOISE SOURCE SOUND POWER LEVELS

	Octave Band Centre Frequency (Hz)							
Source	63	125	250	500	1000	2000	4000	dBA
Odour Control Fan	106	96	94	92	92	89	85	96
Exhaust Stack (attenuated)	88	78	76	74	74	71	67	78
Plant Room (L_{prev})	90	89	83	81	78	74	70	83

APPENDIX I NIGHT-TIME OPERATION NOISE CONTOUR PREDICTION (WITH MITIGATION)



Appendix M

Grey Lynn Tunnel

Watercare Services Limited

Vibration Assessment of Grey Lynn Tunnel and Tawariki Street Shafts

Revision No. 2

21st December 2018

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Table of Contents

List of Tables

List of Figures

Distribution

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Reviewed By:	Victor Romero McMillen Jacobs Associates

Revision Log

Revision No.	Date	Revision Description
0	8 Nov 2018	Draft for internal review
1	20 Feb 2019	Internal Review comments incorporated
2	21 Feb 2019	For AEE lodgement

Executive Summary

The purpose of this report is to assess potential vibration impacts associated with the construction of the Grey Lynn Tunnel and Tawariki Street Shafts. The report has been prepared by McMillen Jacobs Associates for Watercare Services Ltd (Watercare). Watercare is proposing to construct a wastewater interceptor from Western Springs to Tawariki Street, Grey Lynn (Grey Lynn Tunnel). The Tawariki St shafts includes the main shaft that the Grey Lynn Tunnel connects to and a secondary shaft to enable the connection of future sewers. The Grey Lynn Tunnel will connect to the Central Interceptor at Western Springs via the Western Springs shaft site.

The Grey Lynn Tunnel will be excavated using an earth pressure balance (EPB) tunnel boring machine (TBM), which will be the primary source of vibration for tunnel construction. The Tawariki Street Shafts and near-surface sewer collection chambers will be constructed using conventional mechanical equipment (e.g. CAT 330 medium hydraulics excavator or similar). Shaft walls will be supported with either secant piles, sheet piles, ring beams with lagging, steel liner plate, precast segmental rings, caisson or similar installed by pile boring drilling rig or vibratory hammer.

This report assesses the potential construction vibration effects associated with the Grey Lynn Tunnel and shaft construction in accordance with the relevant Auckland Unitary Plan (Operative in Part) (AUP (OP)) rules (2016) and German Industrial Standard (DIN) 4150-3 (1999): Structural vibration – Part 3: Effects of vibration on structures. For human response, the report follows vibration guidelines that are outlined in the British Standard (BS) 5228-2:2009: Human Response.

The U.S. Federal Transit Administration (2006) method was used to assess vibration from various construction activities. This standard method has been adopted as a standard approach by various agencies and governments in the United States and has been adopted by a number of overseas jurisdictions, including Australia (Melbourne Metro, 2016), and elsewhere, to quantitatively assess vibration from construction activities.

There are no commercial and industrial buildings, listed historical structures or sensitive structures (such as laboratories or healthcare facilities with instruments and/or diagnostic equipment) along the alignment of the Grey Lynn Tunnel. Vibration levels during construction of the Grey Lynn Tunnel and Tawariki Street Shafts are predicted to be within limits and guidelines outlined in the AUP (OP) rules, DIN 4150-3, and BS 5228-2. The results of our assessment are tabulated in this report. This report recommends establishing seismograph monitoring stations at key receivers (identified under monitoring) along the Grey Lynn Tunnel alignment and at the Tawariki Street Shafts to record both background vibration and vibrations during construction to verify compliance with the guidelines outlined in this report. In addition, this report provides standard recommended and suggested methods to mitigate construction vibration, if required. The recommended methods are good practice for all construction sites. The suggested methods are additional means to mitigate construction impacts as needed.

This report provides an outline for the vibration sections of Construction Noise and Vibration Management Plan (CNVMP) identifying the minimum standards to be complied with during the construction of the Grey Lynn Tunnel and Tawariki Street Shaft. The CNVMP will be prepared by the construction contractor. The purpose of the CNVMP is to minimise the vibration effects on health and limit discomfort to people as well as minimise the risk of damage to structures.

1.0 Introduction

1.1 Project Description

Watercare Services Limited ("Watercare") is the water and wastewater service provider for Auckland. Watercare is proposing to construct a wastewater interceptor from Tawariki Street, Grey Lynn to Western Springs ("Grey Lynn Tunnel"). The Grey Lynn Tunnel will connect to the Central Interceptor at Western Springs.

1.2 Project Overview

The Grey Lynn Tunnel involves the elements shown in the drawings and outlined in more detail in the reports which form part of the application. These elements are summarised as follows.

1.2.1 Grey Lynn Tunnel

The Grey Lynn Tunnel involves construction, operation and maintenance of a 1.6km gravity tunnel from Western Springs to Tawariki Street, Grey Lynn with a 4.5m internal diameter, at an approximate depth of between 15 to 62m below ground surface, depending on local topography. The tunnel will be constructed northwards from Western Springs using a Tunnel Boring Machine ("TBM"). The Grey Lynn Tunnel will connect to the Central Interceptor at Western Springs via the Western Springs shaft site.

1.2.2 Tawariki Street Shaft Site

The Grey Lynn Tunnel also involves construction, operation and maintenance of two shafts and associated structures at Tawariki Street, Grey Lynn ("Tawariki Street Shaft Site").

The Tawariki Street Shaft Site will be located at 44-48 Tawariki Street where the majority of the construction works will take place. Construction works will also take place within the road reserve at the eastern end of Tawariki Street and a small area of school land (St Paul’s College) bordering the end of Tawariki Street (approximately 150m2).

The Tawariki Street Shaft Site will involve the following components:

1.2.2.1 Main Shaft

- A 25m deep shaft, with an internal diameter of approximately 10.8m, to drop flow from the existing sewers into the Grey Lynn Tunnel;

- Diversion of the Tawariki Local Sewer to a chamber to the north of the shaft. This chamber will be approximately 12m long, 5m wide and 5m deep below ground, and will connect to the shaft via a trenched sewer;

- Diversion of the Orakei Main Sewer to a chamber to the south of the shaft. This chamber will be approximately 10m long, 5m wide and 11m deep below ground;

- Construction of a stub pipe on the western edge of the shaft to enable future connections (that are not part of this proposal) from the CSO network;

- Construction of a grit trap within the property at 48 Tawariki St to replace the existing grit trap located within the Tawariki Street road reserve. The replacement grit trap will be approximately 16m long, 5m wide and 13m deep below ground;

- Permanent retaining of the bank at the end of Tawariki Street to enable the construction of the chamber for the Orakei Main Sewer. The area of the bank requiring retaining will be approximately 44m long, 3m wide and 2m high; and

- An above ground plant and ventilation building that is approximately 14m long, 6m wide and 4m high. An air vent in a form of a stack will be incorporated into the plant and ventilation building and discharge air vertically via a roof vent. The vent stack will be designed with a flange to allow future extension of up to 8m in total height and approximately 1m in diameter in the unexpected event of odour issues.

1.2.2.2 Tawariki Connection Sewer Shaft – Secondary Shaft

A secondary shaft will be constructed at the Tawariki Street Shaft Site to enable the connection of future sewers (that are not part of this proposal) from the Combined Sewers Overflows ("CSO") network. This will involve the following components:

- A 25m deep drop shaft with an internal diameter of approximately 10.2m; and

- A sewer pipe constructed by pipe-jacking to connect the secondary shaft to the main shaft.

1.3 Construction Timeframe

The construction works for the main shaft, chambers and tunnel will occur at the same time as works for the Central Interceptor. Construction will be up to 2 ½ years total duration. The construction of the main shaft and chambers is estimated to take approximately 12 months initially, followed by a hiatus of several months waiting for the TBM to arrive at Tawariki Street Shaft Site. This will be followed by approximately 9 months of activity to remove the TBM and complete the internal structure of the main shaft.

The secondary shaft will be constructed in conjunction with the future sewers at a later date but (subject to need) within a 10-year period following construction of the main shaft and tunnel. The construction period for the secondary shaft and future sewer connections is estimated to be up to 2 years total duration.

1.4 Assessment

This report provides technical input to supplement the Assessment of Environmental Effects (AEE) Report addressing the effects of vibrations from construction of the Grey Lynn Tunnel and Tawariki Street Shafts in Tawariki Street. Construction vibration effects associated with the Grey Lynn Tunnel and Tawariki Street Shafts are assessed in accordance with the AUP (OP) rules (2016) and (DIN) 4150-3 (1999). For human response, the report follows vibration guidelines in BS 5228-2:2009: Human Response.

This report has been developed based on input from the following:

- Briefing and site walk of the project area by the Grey Lynn Tunnel project team.

- Review of concept designs for Grey Lynn Tunnel and Tawariki Street Shafts, including construction methodology and layouts.
- Review of geotechnical information and assessment of ground conditions for excavation of the tunnel and shafts.
- Discussions with the project team to identify anticipated construction equipment and methods.
- Review of vibration source levels for various construction equipment and methods to evaluate the vibration.
- Review of vibration standards and guidelines and development of assessment criteria.
- Identification of structures and sensitive receivers along the tunnel alignment.
- Assessment of likely effects on receivers including effects of distance from vibration sources.
- Discussions with the project team regarding anticipated programme for the works.
- Assessment of mitigation measures (as needed).
- Outline of Construction Noise and Vibration Management Plan for construction.

2.0 Existing Conditions

2.1 Geology and Construction Ground Conditions

A geotechnical investigation has been conducted for the Grey Lynn Tunnel alignment and Tawariki Street Shaft site, which is summarised in an addendum to the project Geotechnical Factual Report (Jacobs/AECOM/McMillen Jacobs Associates 2018). The Grey Lynn Tunnel will transect rock units of the Waitemata Group (Miocene) and specifically the East Coast Bay Formation (ECBF) to the junction with the bottom of the Tawariki Street shafts. From the surface, the shafts will be excavated through units of the Tauranga Group (TG) and the upper weathered rock units of the Waitemata Group (RS) and into the ECBF rock.

The rock units of the ECBF have been characterised as moderately weathered to unweathered rock. The lithology of the ECBF has been characterised by alternating strata of very weak to weak (3.0–9.5 MPa), graded, bedded, silty, muddy sandstones and laminated mudstones. The upper weathered rock units of the RS consist of very stiff to hard, residually to highly weathered cohesive soil (silt and clay) and dense to very dense, residually to highly weathered sand. The undifferentiated TG consists of soft to firm cohesive clays, silts and loose to medium dense sands.

2.2 Existing Structures and Utilities

2.2.1 Existing Structures

The alignment of the Grey Lynn Tunnel is primarily through residential areas. There are no major existing commercial or industrial structures. The largest structures near the Tunnel alignment appear to be the Church of Jesus Christ of Latter-Day Saints (LDS) near the junction of Surrey Crescent and Old Mill Road (at approximately Chainage 233350) and the government offices for the Ministry of Social Development at junction of Surrey Crescent and Richmond Road (at approximately Chainage 23775). The Grey Lynn Tunnel will pass approximately 58 m directly beneath the LDS church. At the Ministry of Social Development, the Tunnel alignment will be about 100 m northwest and 55 m below grade with diagonal offset of about 115 m. In both areas, the Grey Lynn Tunnel will transect very weak to weak, moderately weathered to weathered units of the ECBF rock, which will attenuate TBM vibrations.

At the Tawariki Street Shaft site, the closest residence (42 Tawariki St) is approximately 15 m from the shafts. The residences across the street (35, 37, 39, and 41 Tawariki St) are approximately 20 m to 40 m from the shafts. The shafts will be excavated through the undifferentiated TG, upper weathered rock units of the RS and into the very weak to weak, moderately weathered to weathered units of the ECBF rock, all of which will attenuate vibration.

2.2.2 Utilities

The utilities generally follow streets, consisting of buried power lines, water lines and sewer lines. We have assumed the utilities cross very weak and highly weathered units of the undifferentiated TG and upper weathered rock units of the RS, all of which will attenuate vibration.

3.0 Construction Methodology and Sources of Vibration

The methods of construction and the types of equipment utilised determine the level of vibration generated. The following sections discuss the anticipated methods of construction for the two main components of the Grey Lynn Tunnel and Tawariki Street Shafts and the equipment required with regards to vibration impacts.

3.1 Grey Lynn Tunnel

The Grey Lynn Tunnel is approximately 1.6 km in length with an internal diameter of 4.5 m. The depth of overburden above the Tunnel may be as shallow as 15 m near the valleys and over 60 m in the uplands. The Tunnel will be excavated using an earth pressure balance (EPB) TBM. The TBM will be a primary source of vibration. An EPB TBM is a mechanised tunnelling method in which the excavated material is used to support the tunnel face while it is being plasticised using foams, polymers and other additives to make it transportable and impermeable. The spoils are admitted into the TBM via a screw conveyor arrangement, which allows the pressure at the face of the TBM to remain balanced without the use of slurry. Vibration source levels and spectral characteristics are dependent on the machine type, machine size, and ground conditions through which the tunnelling will occur.

TBM require transport of people, materials, and equipment into and out of the machine. Transport methods could include rolling stock, conveyors, or rubber-tired vehicles. Steel wheeled vehicles on rails will have a higher vibration source level than rubber-tired vehicles. The engine pulling the train may create a significant vibration source. In addition, vibration typically occurs at the juncture of the rails or at areas where there are voids beneath the rail resulting in bumps when a car goes over them.

3.2 Tawariki Street Shafts and Near-Surface Chambers

The Tawariki Street Shafts and near-surface sewer collection chambers will be constructed using an excavator with rippers. Because of the softer deposits, construction of the shaft walls will be supported with either secant piles, typically installed using a pile boring drilling rig, or sheet pile retention rings, installed by pile boring vibratory hammer.

Alternatively, sheet piles may be installed to support the shaft walls and the near-surface chambers, instead of secant piles or sheet pile retention rings. Vibratory hammers are widely used to drive and extract sheet piles. However, pile-driving is one of the greatest sources of vibration associated with equipment used during construction of a project.

Because of the soft, weak upper weathered rock units of the RS rock and weathered units of the ECBF rock, the Tawariki Street Shafts and near-surface chambers will likely be excavated using either a 320 or 330 excavator with an attached ripper.

4.0 Guideline Targets for Construction Vibration

Construction vibration effects associated with the Grey Lynn Tunnel and Tawariki Street Shafts are assessed in accordance with the AUP (OP) rules and DIN 4150-3 (2016). For human response, the report follows vibration guidelines outlined in BS 5228-2:2009.

4.1 Effects of Vibration on Structures

4.1.1 AUP(OP): Building Damage

Standard E25.6.30 Vibration of the AUP (OP) (2016) sets out requirements relating to construction vibration activities that address building damage and comfort to occupants. Standard E25.6.30 provides that construction activities must be controlled to ensure vibration does not exceed the following limits:

1. The limits outlined in DIN 4150-3 (1999) when measured in accordance with that standard on any structure not on the same site, and
2. The limits outlined in Table 1 in any axis when measured in the corner of the floor of the storey of interest for multi-storey buildings or within 500 mm of ground level at the foundation of a single-story building.

Table 1: Vibration Limits in Buildings (AUP (OP)Table E25.6.30.1)

Receiver	Period	Peak Particle Velocity (PPV) Limit (mm/sec)
Occupied activity sensitive to noise or vibration	Night-time 10 pm to 7 am	0.3
	Daytime 7 am to 10 pm	2.0
Other occupied buildings	At all times	2.0

According to Standard E25.6.30, works generating vibration for three days or less between the hours of 7 am and 6 pm may exceed the limits reproduced in Table 1 above, but must comply with a limit of 5 mm/sec peak particle velocity (PPV) in any axis when measured in the corner of the floor of the storey of interest for multi-storey buildings or within 500 mm of ground level at the foundation of a single storey building. Additional requirements to meet this standard are:

1. All occupied buildings within 50 m from of the extent the works generating vibration must be advised in writing no less than three days prior to the vibration-generating works commencing; and
2. The written advice must include details of the location of the works, the duration of the works, a phone number for complaints, and the name of the site manager.

4.1.2 DIN 4150-3 (1999)

4.1.2.1 Damage to Structures

DIN 4150-3 contains vibration limits for buildings that, when complied with, “will not result in damage that will have an adverse effect on the structure’s serviceability”. Table 2 outlines the limits from DIN 4150-3. Different criteria are provided for “short-term” or transient vibration sources such as blasting and pile driving and “long-term” or continuous vibration sources such as vibro-compaction or sheet piling. In addition to providing the guidelines summarised in Table 2, Clause 5.1 of DIN 4150-3 notes that a vibration level greater than the DIN criteria does not necessarily result in building damage.

Table 2: Vibration velocity guideline values for peak particle velocity (PPV) for structures (DIN 4150-3)

Category of Structure	Short-Term Vibration				Long-Term Vibration[b]
	PPV (mm/sec) at Foundation Frequency of:			PPV at Horizontal Plane of Highest Floor at All Frequencies (mm/sec)	PPV at Horizontal Plane of Highest Floor (mm/sec)
	1 Hz to 10 Hz	10 Hz to 50 Hz	50 Hz to 100 Hz[a]		
Commercial/Industrial	20	20 to 40	40 to 50	40	10
Residential/School	5	5 to 15	15 to 20	15	5
Historic or sensitive structures	3	3 to 8	8 to 10	8	2.5

a) At frequencies above 100 Hz, the values in this column may be used as minimum values.

b) Standard defines short-term vibration as “vibration which does not occur often enough to cause structural fatigue, and which does not produce resonance in the structure being evaluated”. Long-term is defined as all other vibration types not covered by the short-term definition.

As reflected in Table 2, the vibration guidelines in DIN 4150-3 include three categories of building structures with increasing levels of protection. The significant margins displayed in Table 2 and Figure 1 reflect higher level of protection provided by this standard to residential structures (Category 2) compared to commercial structures (Category 1). Similarly, a higher level of protection is provided to sensitive structures to include but not limited to laboratories or healthcare facilities with instruments and/or diagnostic equipment (Category 3).

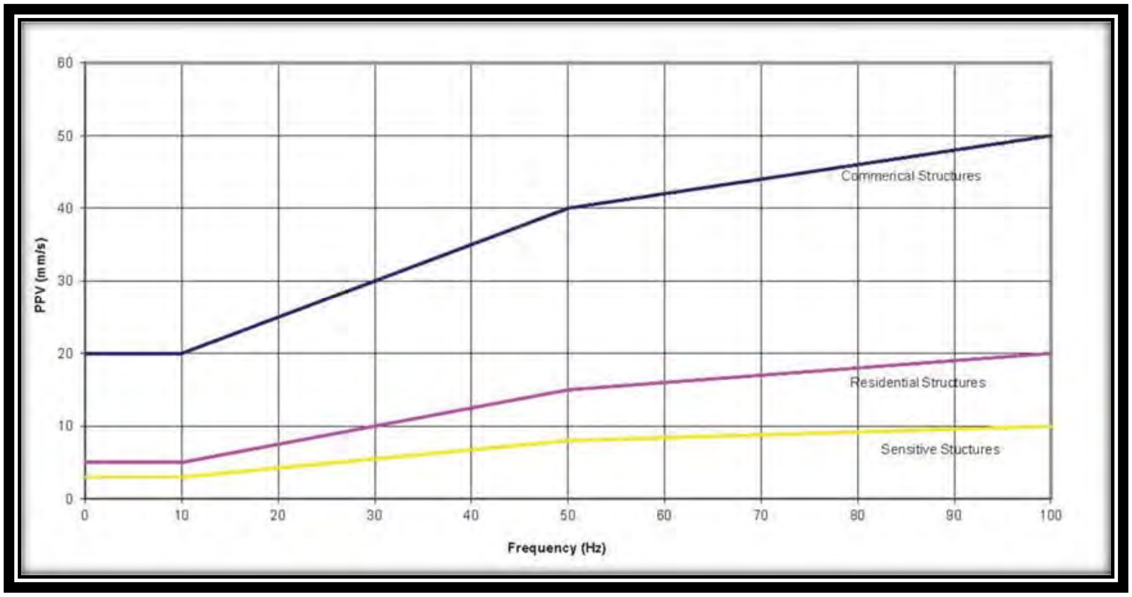

Figure 1: Baseline curves representing short-term vibration effects on structures in relation to recorded data (Marshal Day Acoustics 2010).

Category 3 in Table 1 outlines guidelines for vibration velocity values attendant to short-term vibration effects on historical and sensitive structures and equipment. There are no sensitive or historical structures on the Heritage New Zealand list that are on the Grey Lynn Tunnel alignment. Moreover, there are no hospitals, laboratories, or research institutions on the Tunnel alignment that employ sensitive equipment such as MRI machines or microscopes that are highly sensitive to vibration. As such, vibration criteria used in this assessment are based on Category 2 structures because there limited commercial and industrial structures on the alignment.

4.1.2.1.1 Damage to Utilities

There is no guideline for vibration limits for utilities and services listed in the AUP (OP) or DIN 4150-3. However, a rule-of-thumb for long-term vibrations on utility structures is to limit the PPV to 20 mm/sec at all frequencies. This guideline was used for the Melbourne Metro Rail Project (Melbourne Metro 2016) and is utilised in this assessment, as there was no other apparent guidance established for utilities in New Zealand.

4.2 Human Response Standard

4.2.1 BS 5228-2:2009 – Human Response

Appendix B.2 of BS 5228-2:2009 provides guidance for people's expectations and responses to construction vibration. Table 3 summarises the vibration criteria from this standard. Comparing these criteria to those by DIN 4150-3, as summarised in Table 2, it appears that people are likely to complain at vibration levels significantly below those that cause building and structural damage. Moreover, construction activities generate vibration at a wide range of frequencies, and peoples' sensitivity to the higher frequencies in this range exacerbates their perception of the potential for building damage.

Table 3: Criteria for human response to construction vibration (BS 5228-2:2009, Annex B)

Peak Particle Vibration level (mm/sec)	Effect
0.14	Vibration might be just perceptible in most sensitive situations for most vibration frequencies associated with construction. At lower frequencies, people are less sensitive to vibration.
0.3	Vibration might be just perceptible in residential environments.
1.0	It is likely that the vibration at this level in residential environments will cause complaint but can be tolerated if prior warning and explanation have been provided to the residents.
10	Vibration is likely to be intolerable for any more than a very brief exposure to this level.

5.0 Construction Vibration Assessment Methods

Vibration source levels and spectral characteristics are dependent on machine type and size and the ground conditions in which the construction occurs. Operation of construction equipment causes ground vibrations recorded as peak particle velocity (PPV) that spread through the ground and diminish in strength with distance. Structures founded on soil or rock near the construction site respond to these vibrations, with varying results ranging from no perceptible effects at the lowest levels, low rumbling sounds and perceptible vibration at moderate levels, and slight damage at the highest levels. Strong competent rock such as basalt tends to transmit ground vibration with ease. However, weak weathered rock and soil tend to attenuate the ground vibration within a relatively short distance.

The FTA (2006) method outlines a method to assess vibration from various construction activities in various ground conditions. This method has been adopted as a standard approach by various agencies and governments in the United States, such as Caltrans (2013), by overseas jurisdictions, such as Australia (Melbourne Metro 2016), and elsewhere to quantitatively assess vibration from construction activities. As part of this study, the FTA measured ground vibration source levels as PPV for various types of construction equipment in a multitude of ground conditions (rock and soil) and developed a regression curve comparing distance in metres from the vibration source to PPV in mm/sec.

Table 4 is a summary of the vibration source levels for various machines and other vibration sources compiled from FTA research (2006), Melbourne Metro (2016), and Caltrans (2013). The PPV produced by each piece of equipment is normalised at 7.6 m.

5.1 Vibration Assessment Steps

To conduct the vibration assessment, FTA (2006) recommends the following steps:

- Select equipment and associated source levels at the reference distance of 7.6 m listed on Table 4
- Use the following propagation equation to assess the ground vibrations as PPV:

$$PPV_{equip} = PPV_{ref}\left(\frac{7.6m}{D}\right)^{1.5}$$

Where:

PPV $_{(equip)}$ is the PPV in mm/sec of the equipment adjusted for distance;

PPV $_{(ref)}$ is the reference vibration level in mm/sec of the equipment in

Table 4;
D is the distance in metres from the equipment to the receiver at the structure.

Compare estimated results to vibration criteria tabulated in Table 1, Table 2, and Table 3.

- Note: Assessing construction vibration is an inexact science, computed vibration results are estimates and may vary slightly by a few mm/sec based on machine type and size and the ground conditions in which the construction occurs.

Table 4: Vibration source levels for construction equipment (modified from FTA 2006, Table 12-2; Melbourne Metro 2016)

Equipment		PPV at 7.6 m (mm/sec)	Reference Comments
Pile Driver (impact)	Upper range	38.6	FTA
	Typical	16.4	FTA
Pile Driver (Sonic)	Upper range	18.7	FTA
	Typical	4.3	FTA
Pile driver (vibratory)		16.5	Caltrans
Vibratory roller		5.3	FTA
Clam shovel drop (slurry wall)		5.1	FTA
EPB TBM		4.3	Melbourne Metro
20t excavator & hydraulic rock breaker		4.7	Melbourne Metro
12–15t excavator & hydraulic rock breaker		3.3	Melbourne Metro
7t excavator & hydraulic rock breaker		2.4	Melbourne Metro
Hoe ram		2.3	FTA
Large bulldozer		2.3	FTA
Caisson drilling (auger)		2.3	FTA
Crane, wheel-mounted with outriggers, 450t		2.3	Based on similar data
Excavator with ripper		1.3	Melbourne Metro
Hydromill (slurry wall)	In soil	0.2	FTA
	In rock	0.4	FTA
Piling drilling (bored)		1.0	BS 5228-2
Loaded trucks & traffic		1.9	FTA
Crane, track-mounted, 120t		1.9	Based on similar data
Fixed plant		1.9	Melbourne Metro
Jackhammer		0.9	FTA
Small bulldozer		0.1	FTA

6.0 Construction Vibration Estimates

6.1 Grey Lynn Tunnel

6.1.1 TBM: EPB

Overburden (OB) above the Grey Lynn Tunnel ranges from about 17 m where it runs under Sackville Street, Chainage 24300 to over 58 m (and ranges from 15 to 62m when applying a vertical envelope of -2m/+2m). The Tunnel will be excavated using an EPB TBM. The frequency spectra for TBM range between 16 and 80 Hz (Melbourne Metro 2016). We have assumed the vibration source level for the EPB is 4.3 mm/sec at 7.6 m (

Table 4). Using the propagation equation in Section 5.1, we can predict the following ground vibrations along the Grey Lynn Tunnel alignment due to operation of the EPB TBM.

- Daycare facility at 38 Sackville Street, near Chainage 24,300: 17.9 m OB ~1.19 mm/sec.
- Sackville Reserve at 36 Sackville Street, near Chainage 24,300: 17.3 m OB ~1.25 mm/sec.
- Residential house at 34 Sackville Street, near Chainage 24,300: 17.5 m OB ~1.23 mm/sec.
- Residential house at 32 Sackville Street, near Chainage 24,300: 18.9 m OB ~1.10 mm/sec.
- Residential house at 2/30 Sackville Street, near Chainage 24,300: 20.0 m OB ~1.00 mm/sec.
- Residential house at 30 Sackville Street, near Chainage 24,300: 20.0 m OB ~1.00 mm/sec.
- Residential house at 39 Tawariki Street, near Chainage 24,600: 20.5 m OB ~0.97 mm/sec.
- Residential house at 37 Tawariki Street, near Chainage 24,600: 20.4 m OB ~0.98 mm/sec.
- Old stream valley (Paleo Valley) near Chainage 24250 (length ± 50 m): 20 m OB ~1.00 mm/sec.
- LDS church near Chainage 23350: 58 m OB ~0.20 mm/sec.
- Government offices near Chainage 23775: 115 m OB ~0.07 mm/sec.
- For all utilities at all frequencies assume a buffer of 3 m offset around the tunnel periphery to maintain a PPV of < 20 mm/sec.

Based on our assessment, the predicted ground vibrations generated from the EPB TBM are just above the suggested vibration limits outlined by the AUP (OP) for night-time operations between 10:00 PM and 7:00 AM (see Table 1). Similarly, the predicted ground vibrations and frequency spectra are below the suggested guidelines in DIN 4150-3 (see Table 2) for structures in the following categories: commercial and industrial, residential and schools, and historic or sensitive structures. Referencing Table 3, vibrations from the EPB TBM along the alignment where the overburden is less than 17 (15m at worst case scenario) may be just perceptible. However, these low-level vibrations are considered acceptable provided that prior warning and an explanation of the drilling operations is provided to residents. A procedure for this prior warning and explanation of the vibrations from the drilling operations will be included in the CNVMP.

6.1.2 Rolling Stock and Conveyors

The same conditions for the OB apply for the rolling stock and conveyors in the Tunnel. We have assumed that rolling stock will be on rubber-tired vehicles and the vibration source level for heavy vehicle travel and conveyors is 1.9 mm/sec at 7.6 m (

Table 4). Using the propagation equation in Section 5.1, we can predict the following ground vibrations along the Tunnel alignment due to rolling stock and conveyors.

- Daycare facility at 38 Sackville Street, near Chainage 24,300: 17.9 m OB ~0.53 mm/sec.
- Sackville Reserve at 36 Sackville Street, near Chainage 24,300: 17.3 m OB ~0.55 mm/sec.
- Residential house at 34 Sackville Street, near Chainage 24,300: 17.5 m OB ~0.54 mm/sec.
- Residential house at 32 Sackville Street, near Chainage 24,300: 18.9 m OB ~0.48 mm/sec.
- Residential house at 2/30 Sackville Street, near Chainage 24,300: 20.0 m OB ~0.45 mm/sec.

- Residential house at 30 Sackville Street, near Chainage 24,300: 20.0 m OB ~0.45 mm/sec.
- Residential house at 39 Tawariki Street, near Chainage 24,600: 20.5 m OB ~0.43 mm/sec.
- Residential house at 37 Tawariki Street, near Chainage 24,600: 20.4 m OB ~0.43 mm/sec.
- Old stream valley near Chainage 24250 (length ± 50 m): 20 m OB ~0.45 mm/sec.
- LDS church near Chainage 23350: 58 m OB ~0.10 mm/sec.
- Government offices near Chainage 23775: 115 m OB ~0.03 mm/sec.

Based on our assessment, the predicted ground vibrations generated from the rolling stock fall below the suggested vibration limits outlined by the AUP (OP) (see Table 1) and DIN 4150-3 (see Table 2) for structures in the following categories: commercial and industrial, residential and schools, and historic or sensitive structures. In addition, the expected vibration levels may be just perceptible in areas where the OB is less than 17 in thickness (See Table 3).

6.2 Tawariki Street Shafts and Near-Surface Chambers

At the Tawariki Street Shaft area, the closest residence (42 Tawariki Street) is about 15 m from the shafts. Residences 44, 46 and 48 on Tawariki Street were excluded from this assessment as they are within the Tawariki Street Shaft construction site. The residences across the street (35, 37, 39, and 41 Tawariki Street) are about 20 m to 40 m from the shafts. The shafts will be excavated through the undifferentiated TG, upper weathered rock units of RS and into the very weak to weak, moderately weathered to weathered units of the ECBF rock, all of which will attenuate vibration. In addition, the near-surface chambers or vaults will probably be excavated in very weak and highly weathered undifferentiated TG.

6.2.1 Cranes: 120t and 450t Crane

To support construction of the Tawariki Street Shafts, up to two cranes may be required on site at any one time. The crane for the shaft construction will be a typical crawler crane, which is a 120 t crane, 7x5 m footprint with extended belts. The crane for the TBM recovery will be a 450 t crane, 16x9 m footprint with extended outriggers.

The nearest horizontal distance from a shaft to the residence at Tawariki Street is about 15 m. The vibration source level for the 120 t crane is about 1.9 mm/sec at 7.6 m (Table 4). Using the propagation equation in Section 5.1, we can predict the ground vibrations as PPV of 0.67 mm/sec at the closest residence (42 Tawariki Street).

Similarly, the vibration source level for 450 t crane is about 2.3 mm/sec at 7.6 m (Table 4). Using the propagation equation in Section 5.1, we can predict the ground vibrations as PPV of 0.82 mm/sec at the closest residence at 42 Tawariki Street.

Based on our assessment, the predicted ground vibrations of 0.67 mm/sec and 0.82 mm/sec generated from the 120 t crane and/or the 450 t crane are within limits for DIN 4150-3 (see Table 2). However, the vibrations are slightly above the suggested vibration limits outlined by the AUP (OP) (see Table 1) for the residential structure near 42 Tawariki Street for night-time hours of 10 pm to 7 am; and, the expected vibration levels may exceed the recommended criteria for human response to construction vibration as presented in Table 3. In both cases the vibrations may exceed the recommended guidelines by 0.37 mm/sec and 0.52 mm/sec and might be just perceptible to the residents in the vicinity of 35, 37, 39 and 42 Tawariki Street. However, these low-level vibrations are considered acceptable provided that prior warning and explanation of the construction operations are provided to the residents. A procedure for this prior warning and explanation of the vibrations from the shaft construction operations will be included in the CNVMP. Moreover, excavation of the shafts will not occur during the night-time hours of 10 pm to 7 am

6.2.2 Excavators: 320 or 330 Excavator with Ripper

The Tawariki Street Shafts and near-surface chambers will be excavated with a 320 or 330 excavator with a ripper. The nearest horizontal distance to the residence at Tawariki Street is about 15 m. The vibration source level for an excavator with a ripper is 1.3 mm/sec at 7.6 m (

Table 4). Using the propagation equation in Section 5.1, we can predict the ground vibrations as PPV of 0.47 mm/sec at the closest residence, 42 Tawariki Street.

Based on our assessment, the predicted ground vibrations of 0.47 mm/sec generated from the 320 or 330 excavators with a ripper are within limits for DIN 4150-3 (see Table 2). However, the vibrations are slightly above the suggested vibration limits outlined by the AUP (OP) (see Table 1) for residential structures for night-time hours of 10 pm to 7 am; and, the expected vibration levels may exceed the recommended criteria for human response to construction vibration as presented in Table 3. In both cases the vibrations may exceed the recommended guidelines by 0.17 mm/sec and might be just perceptible to the residents in the vicinity 42 Tawariki Street. Vibrations at residential areas 35, 37, 39 and 46 Tawariki Street are expected to be below 0.3 mm/sec. However, these low-level vibrations are considered acceptable provided that prior warning and explanation of the construction operations are provided to the residents. A procedure for this prior warning and explanation of the vibrations from the shaft construction operations will be included in the CNVMP. Moreover, excavation of the shafts will not occur during the night-time hours of 10 pm to 7 am.

6.2.3 Secant Pile Drill Rigs

Secant piles may be installed to support the shaft walls and the near-surface chambers. The secant piles would typically be installed using a pile boring drilling rig. The vibration source level as PPV at 7.6 m is approximately 1.0 mm/sec for boring pilling drill rigs (

Table 4). Using the propagation equation in Section 5.1, we can predict the ground vibrations as PPV of 0.36 mm/sec at the closest residence, 42 Tawariki Street.

Based on our assessment, the predicted ground vibrations of 0.36 mm/sec generated from the pile boring drilling rig in the residences near the shafts are within limits for DIN 4150-3 (see Table 2). However, the vibrations are slightly above the suggested vibration limits outlined by the AUP (OP) (see Table 1) for the residential structure near 42 Tawariki Street for night-time 10 pm to 7 am; and, the expected vibration levels may exceed the recommended criteria for human response to construction vibration as presented in Table 3. In both cases the vibrations may exceed the recommended guidelines by 0.06 mm/sec and might be just perceptible to the residents. Vibrations at residential areas 35, 37, 39 and 46 Tawariki Street are expected to be below 0.3 mm/sec. Moreover, these low-level vibrations are considered acceptable provided that prior warning and explanation of the construction operations are provided to the residents. A procedure for this prior warning and explanation of the vibrations from the shaft construction operations will be included in the CNVMP. Furthermore, installation of the secant piles for shoring will not occur during the night-time hours of 10 pm to 7 am.

6.2.4 Sheet Piles Vibratory Hammer

Alternatively, sheet piles may be installed to support the shaft walls and the near-surface chambers using a vibratory hammer. Pile-driving is one of the greatest sources of vibration associated with equipment used during construction of a project. According to literature research, the frequency spectra for the vibratory hammer range is between 20 and 50 Hz (DFI) and the typical vibration source level as PPV at 7.6 m is approximately 16.51 mm/sec for vibratory pile drivers (

Table 4). Using the propagation equation in Section 5.1, we can predict the ground vibrations as PPV of 5.95 mm/sec at the closest residence, 42 Tawariki Street.

Based on our assessment, the predicted ground vibrations of 5.95 mm/sec and frequency spectra of 20 to 50 Hz generated from the vibratory hammer around the shafts for expected short-term vibrations fall below the limits based on DIN 4150-3 (see Table 2) for the nearest residence (about 15 m) at 42 Tawariki Street around the shafts. However, for the same residence, the long-term PPV may be exceeded by 0.95 mm/sec. Vibrations at residential areas 35, 37, 39 and 46 Tawariki Street are expected to be below 3.9 mm/sec and within limits. On the other hand, the vibrations may exceed the suggested vibration limits outlined by the AUP (OP) (see Table 1) by at most 5.65 mm/sec near 42 Tawariki Street and about 3.6 mm/sec near residences 35, 37, 39 and 46 Tawariki Street. Referring to Table 3, vibrations from the vibratory hammer exceeding the guidance near the shafts will be perceptible and residents may complain. It should be noted that these low-level vibrations are considered acceptable provided that prior warning and explanation of the construction operations are provided to the residents. A procedure for this prior warning and explanation of the vibrations from the shaft construction operations will be included in the CNVMP. Furthermore, installation of the sheet pile shoring (if chosen) will not occur during the night-time hours of 10 pm to 7 am

6.2.5 Utilities

For utilities, we have assumed either:

- An in situ earthen buffer between the utility and the secant pile or sheet pile wall of at least 7 m at all frequencies to maintain PPV below 20 mm/sec; or

- Utilities within site boundaries will be connected (e.g. sanitary and storm sewers) into the chambers/drop shafts, or will be temporarily diverted and replaced (e.g. potable water).

6.3 Summary of Construction Vibration Estimates

In general, we expect construction of the Grey Lynn Tunnel will comply with vibration limit guidelines in the AUP (OP), DIN 4150-3, and BS 5228-2 (as outlined in Tables 1, 2, and 3, respectively). Vibrations from the TBM may be just above the suggested vibration limits outlined by the AUP (OP) for night-time operations between 10 pm and 7 am (see Table 1). In addition, vibrations may be at or exceed the BS 5228-2 guidelines (see Table 3) and may be just perceptible. However, these low-level vibrations are considered acceptable and can be tolerated provided that prior warning and explanation of the drilling operations are provided to the residents.

Use of 120t to support construction of the Tawariki Street Shafts, and use of a 450t crane for TBM retrieval, will comply with vibration limit guidelines of DIN 4150-3 (Table 2). However, because of the proximity of the residential structure at 42 Tawariki Street (15 m from a shaft) and 35, 37, 39 and 41 Tawariki Street, vibrations may exceed guidelines AUP (OP) (Table 1) for night-time hours 10 pm to 7 am and the BS 5228-2 guidelines (see Table 3) where vibrations may be just perceptible. However, these low-level vibrations are considered acceptable and could be tolerated provided that prior warning and explanation of the drilling operations was provided to the residents. Moreover, excavation and shoring of the shafts will not occur during the night-time hours of 10 pm to 7 am, therefore there is no effect. Similarly, TBM retrieval could be limited to day-time hours.

Excavation of the Tawariki Street Shafts using 320 or 330 excavators with a ripper and installation of shoring using a secant pile drilling rig will comply with vibration limit guidelines of DIN 4150-3 (Table 2). However, because of the proximity of the residential structure at 42 Tawariki Street (15 m from a shaft), vibrations will exceed guidelines AUP (OP) (Table 1) for night-time hours 10 pm to 7 am and the BS 5228-2 guidelines (see Table 3) where vibrations may be just perceptible. Vibrations at residential areas at 35, 37, 39 and 46 Tawariki Street are expected to be within limits. It should be noted that these low-level vibrations are considered acceptable and can be tolerated provided that prior warning and explanation of the drilling operations are provided to the residents. Moreover, excavation and shoring of the shafts will not occur during the night-time hours of 10 pm to 7 am, therefore there is no effect.

If the contractor elects to install sheet pile shoring with a vibratory hammer, short-term vibrations fall below the limits based on DIN 4150-3 (see Table 2) for residence at 42 Tawariki Street about 15 m from the shafts, but the long-term vibrations may be exceeded. In addition, vibrations will exceed guidelines AUP (OP) (Table 1) for night-time hours 10 pm to 7 am, including residential areas at 35, 37, 39 and 46 Tawariki Street. Referring to Table 3, vibrations from the vibratory hammer exceeding the guidance near the shafts will be perceptible and residents may complain. However, these low-level vibrations could be acceptable provided that prior warning and explanation of the construction operations was provided to the residents. Furthermore, shoring of the shafts with sheet piles will not occur during the night-time hours of 10 pm to 7 am.

7.0 Mitigation Options

The following are recommended and suggested methods to mitigate construction vibration. The recommended methods are good practice for all construction sites. The suggested methods are additional means to mitigate construction methods as needed.

7.1 Recommended Mitigation Options

- Communicate with adjacent residents along the Grey Lynn Tunnel alignment specifically at Sackville Street (residences 30, 2/30, 32, 34, and 38) and residences 35, 37, 39, 41 and 42 around the Tawariki Street Shaft site about the different vibrations and what to expect.
- Manage construction hours as follows:
    - Tunnelling activities – 24 hours a day, 7 days a week will occur for all tunnelling activities;
    - Shaft site construction activities – 7 am to 6 pm Monday to Friday, 8 am to 6 pm Saturday; and
    - Truck movements – 7 am to 6 pm Monday to Friday, 8 am to 6 pm Saturday.

- Conduct pre-condition surveys at the LDS Church on Surrey Crescent Street, the government buildings near Richmond Road and residences at 30,2/30, 32, 34, 38 Sackville Street along the alignment and residences 35, 37, 39, 41 and 42 Tawariki Street at the shaft site.
- Enact a warning procedure when construction vibration will occur.
- Use alternative design, construction methods, and equipment to mitigate construction vibration. For instance, for support of the shaft walls install secant piles with a pile drilling rig as opposed to installing sheet piles with a vibratory hammer, where the geologic conditions permit.
- Ensure ground vibrations are kept below 20 mm/sec at all frequencies at utilities lines adjacent to the work site by maintaining an in-situ buffer of earth of about 3 m between the tunnel work and utility and 7 m between the shaft work and utility.
- Institute a good maintenance program for construction equipment and vehicles to minimise vibration.

7.2 Suggested Mitigation Options (if required)

- Isolate vibration source, such as installing cushions below the rails at vibration points or using rubber-tired vehicles to transport rolling stock to and from the EPB TBM.
- Construct a vibration attenuation barrier between source and receiver.
- Consider, where mitigation is not feasible, possible temporary relocation of residents during activities that are close to the affected structures.
- Modify affected building structures to change the response characteristics by, for example, installing bracing to modify building response frequency.
- Isolate very sensitive equipment by use of, for example, airbags or floating slabs.

8.0 Consideration of Sensitive Receivers and Potential for Damage to Neighbouring Properties

We have conducted an initial assessment of the potential vibrations during construction operations along the Grey Lynn Tunnel and Tawariki Street Shafts. Our assessment included review of the likely construction methods, the levels of vibration that they will generate, and estimation of the distances at which vibration levels will exceed the proposed limits for both structural damage and sensitive receivers.

Section 6.0 summarises the construction activity and attendant vibration for the expected equipment. The summary was based on vibration source levels for each piece of equipment and the general distance from the source to the receiver.

Based on our initial assessment, there are no commercial or industrial structures within the alignment of the Grey Lynn Tunnel. Moreover, there are no historical structures listed as New Zealand Heritage structures or sensitive structures and equipment on the alignment and there is sufficient OB to attenuate the vibrations. Three residential structures within Tawariki Street Shaft Site will be removed. In short, the risk for damage from construction vibration near the Grey Lynn Tunnel alignment and the Tawariki Street shafts is low. The construction vibrations may be perceptible to residents but should not cause disturbance, especially if residents are notified and informed on the construction methods. Moreover, we do not expect that vibrations will exceed 20 mm/sec at buried utilities along the tunnel alignment and at the shafts. Overall, the risk of damage is less than minor.

In general, any effects of vibration on the structures along the alignment and in the vicinity of the shafts can be managed by control of construction means and methods to limit vibration levels at the source. To minimize vibration complaints and affects to residential structures, excavation and shoring of the shafts should not occur during the night-time hours of 10 pm to 7 am. If necessary, other mitigation measures may also be considered, as discussed in Section 7.0.

9.0 Construction Noise and Vibration Management Plan

A CNVMP identifying the minimum standards to be complied with during the construction of the Grey Lynn Tunnel and Tawariki Street Shafts shall be prepared by the contractor. The purpose of the CNVMP is to minimise the vibration effects on health and limit discomfort to people as well as minimise the risk of damage to structures.

We recommend the CNVMP include the following items in the document:

- Vibration guidance criteria as outlined in the AUP (OP) (Table 1) and DIN 4150-3 (Table 2), and the criteria for human response as outlined in BS 5228-2 (Table 3) for the Grey Lynn Tunnel and Tawariki Street Shafts.
- Notification requirements and information to be provided to the community. The AUP (OP) requires the following:
    - Post signage notification with construction schedule at key locations within 50 m of the works generating vibration advising public on when vibration-generating works will commence; and
    - The written notification must include details of the location of the works, the duration of the works, a phone number for complaints and the name of the site manager.
- Consider a ground vibration PPV of 20 mm/sec at all frequencies for excavation next to utility lines; maintain an in-situ buffer of earth of about 3 m between the utilities along the tunnel alignment work and 7 m at the shafts between the works and the utilities.
- Manage construction hours as follows:
    - Tunnelling activities – 24 hours a day, 7 days a week will occur for all tunnelling activities;
    - Shaft site construction activities – 7 am to 6 pm Monday to Friday, 8 am to 6 pm Saturday; and
    - Truck movements – 7 am to 6 pm Monday to Friday, 8 am to 6 pm Saturday.
- List of equipment that is likely to generate significant levels of vibration.
- Requirements for vibration monitoring including trials for establishing attenuation characteristics and the associated statistical parameters for design of safe operating distances.
- Requirements for condition (dilapidation) surveys on potentially affected structures (such as residential structures adjacent to the shafts) prior to, during, and after completion of the works.
- Requirements for background vibration monitoring in advance of the project.
- Reporting requirements including response flow chart identifying actions and reporting protocols if vibrations exceed the criteria.
- Roles and responsibilities of key personnel on site including contact details.

- Construction operator training procedures for activities likely to generate significant levels of vibration.
- Construction vibration mitigation options.
- Recording system for receiving and handling of complaints.

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Appendix N

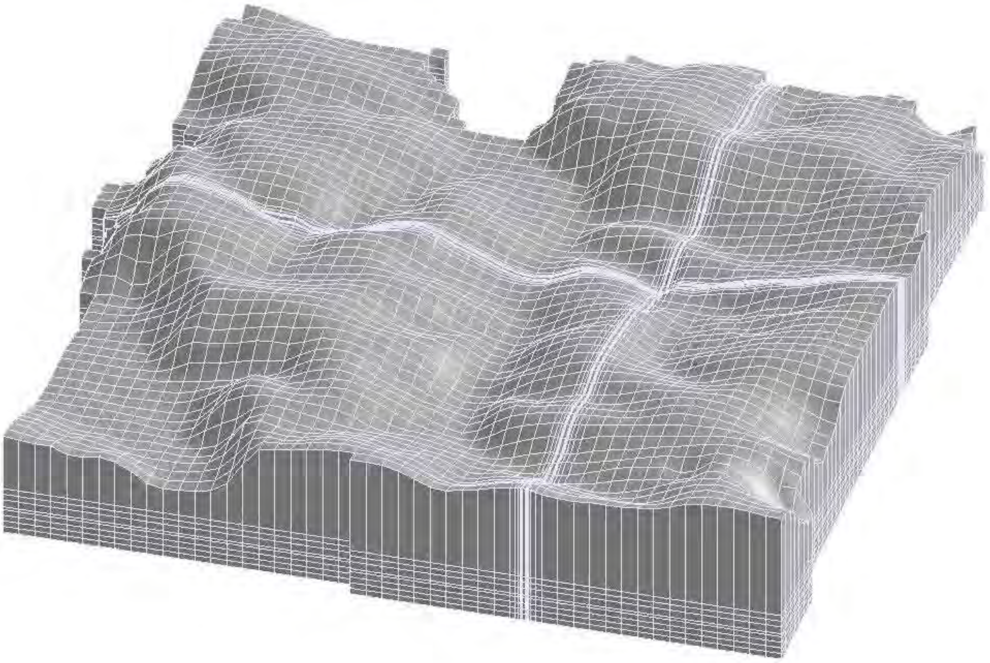
Grey Lynn Tunnel

Groundwater Effects Assessment

MCMILLEN JACOBS ASSOCIATES

WWA0047 | Rev. 11

19 February 2019



Grey Lynn Tunnel - Groundwater Effects Assessment

Project no:	WWA0047
Document title:	Grey Lynn Tunnel - Groundwater Effects Assessment
Revision:	11
Date:	19 February 2019
Client name:	McMillen Jacobs Associates
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Contents

Executive Summary

Project Overview

Williamson Water & Land Advisory (WWLA) has been commissioned by Watercare Services Limited (Watercare) to undertake a numerical modelling analysis and assessment of groundwater effects report for the proposed construction of new shafts at the termination of Tawariki Street in Grey Lynn, Auckland.

The groundwater effects assessment criteria for the proposed shafts used in this analysis are based on those included in Section E7.8 of the Auckland Unitary Plan (Operative in Part) (AUP(OP)). Additional consideration has been given to the potential impact of groundwater drainage on the construction process. These considerations can be summarised as follows:

- Estimation of seepage into the shaft during the construction process under various temporary lining conditions;
- An estimate of regional drawdown of groundwater levels during construction and groundwater recovery following project completion;
- Potential impact on surface water features, specifically streams; and
- Potential impact on neighbouring groundwater users.

The proposed Secondary Shaft will be constructed a minimum of 2.5 years after the competition of the Main Shaft. At this time any groundwater impacts from the Main Shaft would have fully recovered. The configuration of the Secondary Shaft is slightly smaller than the Main Shaft, so any new groundwater impacts of the Secondary Shaft will be less than for the Main Shaft. Therefore, separate modelling of the groundwater effects from the Secondary Shaft were not undertaken. However, the effects assessment is relevant to both shafts. Reference to “shaft” within this report is reference to the shaft under construction at the time.

The groundwater effects of the tunnel construction will be minimal as the construction proceeding involved an earth pressure balance tunnel boring machine, which limits groundwater ingress to the tunnel and prevents any groundwater related impacts from occurring.

Numerical Modelling

A calibrated numerical groundwater flow model was developed using MODFLOW to determine the potential impact of shaft construction on regional groundwater and to estimate the rate of groundwater drainage into the shaft during and following construction.

A two-year simulation was run to establish baseline conditions (**Scenario 1**). Six transient simulations were subsequently run, which included four simulations of various lining scenarios to the full depth of the shaft using different permeability assumptions. In the other two scenarios the shaft lining was simulated to extend to 7 m below ground level (mBGL), and a higher material conductivity was tested in Model Layer 1 in the final scenario:

- ***Scenario 2*** - no shaft lining;
- ***Scenario 3*** - 10^{-8} m/s shaft lining;
- ***Scenario 4*** - 10^{-9} m/s shaft lining;
- ***Scenario 5*** - 10^{-10} m/s shaft lining;
- ***Scenario 6*** - 10^{-9} m/s shaft lining extending to 7 m BGL; and
- ***Scenario 7*** - 10^{-9} m/s shaft lining extending to 7 m BGL and increased Layer 1 permeability.

Scenario 4 was considered to be the most representative of long-term conditions while Scenario 6 was considered to represent the temporary conditions during the construction period prior to the installation of the full shaft lining. Scenario 2 (unlined shaft) was considered to be the most conservative scenario from the perspective of demonstrating an upper envelope of potential effects.

Model Results

Drainage into the shaft is predicted to reach a peak 7 to 10 days into the shaft excavation process and rapidly decline at the end of the construction period, approaching a pseudo constant rate as conditions stabilize around the shaft. In Scenario 2, with no lining on the shaft wall, drainage is predicted to peak at 32 m^3/day. This reduces to 30, 24, and 23 m^3/day in Scenarios 3, 4, and 5 with progressively decreasing lining permeabilities, respectively. The rate of steady state drainage into the shaft after construction is 5.6 m^3/day in Scenario 2, 4.0 m^3/day in Scenario 3, and approximately 2 m^3/day in Scenario 4 and 5. Scenarios 6 and 7 are effectively the same as Scenario 2 in terms of peak and steady state drainage into the shaft.

The model results demonstrated that barrier permeability has a stronger influence on the long term rate drainage into the shaft, whereas the peak drainage includes a strong component of vertical inflow that limits the effect of the lining which is only applied on the sides of the shaft.

The predicted impact on surface drainage was minimal, with less than 0.07 m^3/day (0.0008 L/s[1]) of flow reduction predicted in Cox's Creek in the most extreme case (i.e. Scenario 2 – Unlined Shaft). Groundwater drawdown was greatest directly around the shaft location, but widespread impact on groundwater levels was not predicted as there are no current groundwater users within the range of impact. This is consistent with our expected result, due to the very low hydraulic conductivity of the rock formation, and the finite duration of dewatering.

Predicted drawdown in Scenario 2, with no shaft lining, was 5.5 m at a distance of 10 m from the shaft but only 0.6 m at 100 from the shaft. Measurable drawdown (>5 cm) was predicted to extend to approximately 420 m from the shaft location in the model layer corresponding to the bottom of the shaft. Drawdown in the shallow layer where settlement could occur was under 0.2 m in all scenarios other than Scenario 7.

Greater drawdown was predicted in Scenario 7 where a higher conductivity for the Tauranga Group/upper ECBF was assumed, however this Scenario was run to assess model sensitivity and did not apply calibrated parameters. In Scenario 7, approximately 7.2 m of drawdown was predicted at a distance of 10 m from the shaft and 2.9 m was predicted at 100 m from the shaft.

All other scenarios where the shaft wall was lined resulted in the prediction of significantly less drawdown than for Scenario 2. Drawdown was predicted to be under 0.5 m at 100 m distance from the shaft in Scenarios 3-5 and Scenario 6 was effectively the same as Scenario 2. Drawdown was not predicted to extend to the coast in any scenario, therefore shaft construction is therefore not predicted to induce saline intrusion into the aquifer. The model results all indicated a less than a minor impact on regional groundwater. Groundwater monitoring is recommended for a three-month period prior to construction and a maximum of one-year period following construction (with potential to reduce this period if the actual maximum drawdown level is less than predicted) to assure impacts are not beyond the expected levels.

Recommendations

The following is a list of recommendations based on model results and regional groundwater conditions:

1. The shaft should be lined to minimize the risk of impacting local groundwater levels and inducing ground settlement using a material, with a permeability of no greater than 1×10^{-8} m/s.
2. Monitoring existing boreholes at time periods and frequency as indicated in Recommendation 2-4 at CIE-BH04, CIE-BH05, and CIE-BH06 adjacent to the shaft, as well monitoring the borehole CIE-BH01 or CIE-BH02, located along the proposed route of the Grey Lynn Tunnel approximately 500 m from the shaft to confirm that the actual drawdown levels are not beyond the maximum expected levels.
3. Weekly monitoring of groundwater levels at all boreholes installed for the Grey Lynn Tunnel project is recommended for a three-month period prior to construction to document baseline conditions.

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[1] To place this in context, a garden hose has a typical peak flow rate of 0.2 L/s.

4. Weekly monitoring of all boreholes installed for the Grey Lynn Tunnel project in accordance with a Groundwater Monitoring and Contingency Plan is recommended during shaft construction to alert managers if there is any change in groundwater level that may incur risk to structures or the environment.
5. Monthly groundwater monitoring in accordance with a Groundwater Monitoring and Contingency Plan is recommended for a one-year period following construction to assure impacts are not beyond the expected levels and that groundwater levels recover to pre-construction conditions.

1 Introduction

Watercare Services Limited ("Watercare") is the water and wastewater service provider for Auckland. Watercare is proposing to construct a wastewater interceptor from Tawariki Street, Grey Lynn to Western Springs ("Grey Lynn Tunnel"). The Grey Lynn Tunnel will connect to the Central Interceptor at Western Springs.

1.1 Project Overview

The Grey Lynn Tunnel involves the elements shown in the drawings and outlined in more detail in the reports which form part of the application. These elements are summarised as follows.

1.1.1 Grey Lynn Tunnel

The Grey Lynn Tunnel involves construction, operation and maintenance of a 1.6km gravity tunnel from Western Springs to Tawariki Street, Grey Lynn with a 4.5m internal diameter, at an approximate depth of between 15 to 62m below ground surface, depending on local topography. The tunnel will be constructed north-wards from Western Springs using a Tunnel Boring Machine ("TBM"). The Grey Lynn Tunnel will connect to the Central Interceptor at Western Springs via the Western Springs Shaft Site.

1.1.2 Tawariki Street Shaft Site

The Grey Lynn Tunnel also involves construction, operation and maintenance of two shafts and associated structures at Tawariki Street, Grey Lynn ("Tawariki Street Shaft Site").

The Tawariki Street Shaft Site will be located at 44-48 Tawariki Street where the majority of the construction works will take place. Construction works will also take place within the road reserve at the eastern end of Tawariki Street and a small area of school land (St Paul’s College) bordering the end of Tawariki Street (approximately 150m^2).

The Tawariki Street Shaft Site will involve the following components:

1.1.3 Main Shaft

- A 25m deep shaft, with an internal diameter of approximately 10.8m, to drop flow from the existing sewers into the Grey Lynn Tunnel;
- Diversion of the Tawariki Local Sewer to a chamber to the north of the shaft. This chamber will be approximately 12m long, 5m wide and 5m deep below ground, and will connect to the shaft via a trenched sewer;
- Diversion of the Orakei Main Sewer to a chamber to the south of the shaft. This chamber will be approximately 10m long, 5m wide and 11m deep below ground;
- Construction of a stub pipe on the western edge of the shaft to enable future connections (that are not part of this proposal) from the CSO network;
- Construction of a grit trap within the property at 48 Tawariki St to replace the existing grit trap located within the Tawariki Street road reserve. The replacement grit trap will be approximately 16m long, 5m wide and 13m deep below ground;
- Permanent retaining of the bank at the end of Tawariki Street to enable the construction of the chamber for the Orakei Main Sewer. The area of the bank requiring retaining will be approximately 44m long, 3m wide and 2m high; and
- An above ground plant and ventilation building that is approximately 14m long, 6m wide and 4m high. An air vent in a form of a stack will be incorporated into the plant and ventilation building and discharge air vertically via a roof vent. The vent stack will be designed with a flange to allow future extension of up to 8m in total height and approximately 1m in diameter in the unexpected event of odour issues.

1.1.4 Tawariki Connection Sewer Shaft – Secondary Shaft

A secondary shaft will be constructed at the Tawariki Street Shaft Site to enable the connection of future sewers (that are not part of this proposal) from the Combined Sewers Overflows ("CSO") network. This will involve the following components:

- A 25m deep drop shaft with an internal diameter of approximately 10.2m; and
- A sewer pipe constructed by pipe-jacking to connect the secondary shaft to the main shaft.

1.2 Assessment

Williamson Water & Land Advisory (WWLA) has been commissioned by McMillan-Jacobs to undertake a numerical modelling analysis of the groundwater impact of constructing the proposed shaft at the Tawariki Street Shaft Site and of the effects of the tunnel construction. The shaft is to be used during construction as an access point for the machinery required to excavate the sewage tunnel and the shaft itself. Following construction, the shaft will remain in place as an access point for ongoing tunnel operation and maintenance.

Figure 1 shows the extent of the study area, defined as the model boundary, as well as the major features within the study area as related to this assessment.

Figure 1. Overview of study area (see A3 attachment at rear).

The primary components of this assessment are:

- Estimation of seepage into the shaft during the construction process under lined and unlined conditions
- An estimate of regional drawdown of groundwater levels during construction and groundwater recovery following project completion
- Potential impacts on surface water features, specifically streams
- Potential impacts on groundwater users
- Assessment of consolidation settlements resulting from groundwater drawdown is provided in a separate effects assessment report.

Report Structure

The report is divided into seven primary sections with each section sub-divided into specific topics to provide further detail as needed:

- **Considerations for Assessment:** Potential impacts on groundwater, relevant evaluation criteria, geological and hydro-geological setting (**Section 2**).
- **Conceptual Hydrogeological Model:** Regional geology and hydrogeology, hydraulic testing, groundwater recharge and flow characterisation (**Section 3**).
- **Groundwater Model development:** grid discretization, parameterization, conceptual model setup, boundary conditions (**Section 4**).
- **Model Calibration:** Observed groundwater conditions, calibrated model parameters, calibrated model groundwater budget (**Section 5**).
- **Predictive Simulations:** Scenario setup, transient model inputs, model results evaluated against baseline conditions **(Section 6).**

- **Assessment of Effects:** Model output evaluated against consent criteria, monitoring and reporting approach **(Section 7).**
- **Summary and conclusions:** Summary of predicted impact of shaft construction on groundwater conditions and groundwater flow into shaft, recommendations for groundwater management as related to shaft construction (**Section 8**).

2 Considerations for Assessment

2.1 Potential Effects of Shaft Construction

The construction of the proposed shafts brings several considerations for groundwater management during the construction process and for long-term impacts on local groundwater conditions. Groundwater inflows that occur during the shaft excavation process will require management throughout the construction period. Inflowing water will have to be removed by pumping and subsequent disposal into stormwater facilities provided the volume is manageable. The shafts will effectively act as a drain on local groundwater and an associated drawdown on local groundwater levels can be expected.

Groundwater drawdown has potential to deplete stream flows by reducing baseflow and initiate land settlement as underlying geologic material becomes desaturated. Land settlement is not in the scope of this study and is being evaluated separately, however the drawdown estimates derived from this study are used to inform land settlement calculations.

The development of a numerical model based on measured field hydraulic properties is used as a tool for estimating the rate of groundwater drainage into the shaft and the depth and extent of groundwater drawdown.

The shafts will be excavated and supported by a temporary system, consisting of secant piles, sheet piles or similar methods in thick soil layers above rockhead, and rockbolts, shotcrete and/or mesh in competent bedrock below overburden soils. Following shaft excavation, a concrete liner will be installed to support ground loads, house the sewer hydraulic drop structures and minimize groundwater leakage into the shafts.

The proposed Secondary Shaft will be constructed a minimum of 2.5 years after the completion of the Main Shaft. At this time any groundwater impacts from the Main Shaft would have fully recovered. The configuration of the Secondary Shaft is slightly smaller than the Main Shaft, so any new groundwater impacts of the Secondary Shaft will be less than for the Main Shaft. Therefore, separate modelling of the groundwater effects from the Secondary Shaft was not undertaken. However, the effects assessment is relevant to both shafts. Reference to “shaft” within this report is reference to the shaft under construction at the time.

2.2 Potential Impacts of Tunnel Construction

The proposed Grey Lynn Tunnel between Western Springs and the Tawariki Street Shaft Site will be constructed using the same tunnel boring machine (TBM) as the Central Interceptor mainline tunnel. Project specifications require that this tunnel must be constructed by an Earth Pressure Balance (EPB) TBM which limits groundwater ingress into the tunnel during construction. In the long-term, the precast tunnel lining limits long-term water ingress.

With the use of the EPB TBM construction method, the excavation is sealed from groundwater ingress, and minimal groundwater impacts are expected to occur. Nevertheless, an assessment of groundwater impacts due to EPB TBM tunnelling for the mainline tunnel are assessed in the Central Interceptor project report “Assessment of Potential Groundwater Drawdown due to Shaft Construction” (Ref. PWCIN-DEL-REP GT-J-100236). This report concluded that groundwater ingress to the tunnel was approximately 0.006 L/s per meter of tunnel. This is equivalent to a teaspoon of water per second, which is a very slow flow rate noting a garden hose has a typical flow of 0.2 L/s, which is 33 times greater. The same tunnel construction and control assumptions employed in the Central Interceptor mainline tunnel groundwater assessment apply to the Grey Lynn Tunnel, and the geological conditions are similar. Therefore, the potential groundwater impacts of the Grey Lynn Tunnel construction are considered to be negligible.

2.3 Relevant Statutory Provisions

Planning provisions related to the construction and potential groundwater impacts of the shaft are provided in the Auckland Unitary Plan – Operative in Part (AUP-OP) (Auckland Council, 2016). As explained in more detail in the Assessment of Effects, Section E7 (taking, using, damming and diversion of water), classifies the activity as restricted discretionary. Assessment criteria for groundwater impacts associated with restricted discretionary activities are addressed in Section E.7.8 of the AUP. **Table 1** summarises the specific matters of discretion considered for evaluating restricted discretionary activities with regard to groundwater impacts.

Table 1. AUP matters of discretion for evaluation of restricted discretionary activities with regard to groundwater impacts.

Criteria Number		**Matters of Discretion**	**Comment**
E7.8.1 (6a)	i)	How the proposal will avoid, remedy or mitigate adverse effects on the base flow of rivers and springs	Potential impacts on surface streams are addressed in **Section 6.2.3** and included in the assessment of effects provided in **Section 7.1**
	ii)	How the proposal will avoid, remedy or mitigate adverse effects on levels and flows in wetlands	Potential impacts on wetlands are addressed in **Section 6.2.3** and included in the assessment of effects provided in **Section 7.1.8**
	iii)	How the proposal will avoid, remedy or mitigate adverse effects on lake levels	Potential impacts on lakes are addressed in **Section 6.2.3** and included in the assessment of effects provided in **Section 7.1.2**
	iv)	How the proposal will avoid, remedy or mitigate adverse effects on existing lawful groundwater takes and diversions	Potential impacts on other groundwater takes are addressed in **Section 6.2.4** and included in the assessment of effects provided in **Section 7.1.3**
	v)	How the proposal will avoid, remedy or mitigate adverse effects on groundwater pressures, levels or flow paths and saline intrusion	Potential impacts along the coast are addressed in **Section 6.2.4** and included in the assessment of effects provided in **Section 7.1.4**
	vi)	How the proposal will avoid, remedy or mitigate adverse effects from ground settlement on existing buildings, structures and services including roads, pavements, power, gas, electricity, water mains, sewers and fibre optic cables	Not relevant to the technical scope of this report. Will be addressed in the Ground Settlement Report.
	vii)	How the proposal will avoid, remedy or mitigate adverse effects arising from surface flooding including any increase in frequency or magnitude of flood events	Not relevant to the technical scope of this activity (groundwater dewatering of an excavation)
	viii)	How the proposal will avoid, remedy or mitigate adverse effects from cumulative effects that may arise from the scale, location and/or number of groundwater diversions in the same general area	Potential cumulative impacts from groundwater extraction is addressed in **Section 6.2.4** and included in the assessment of effects provided in **Section 7.1.6**
	ix)	How the proposal will avoid, remedy or mitigate adverse effects from the discharge of groundwater containing sediment or other contaminants	Groundwater discharge into the shaft is addressed in **Section 6.2.1** and included in the assessment of effects provided in **Section 7.1.7**
	x	How the proposal will avoid, remedy or mitigate adverse effects on any scheduled historic heritage place	Not relevant to the technical scope of this report. May be addressed elsewhere.
	xi)	How the proposal will avoid, remedy or mitigate adverse effects on terrestrial and freshwater ecosystems and habitats	Not relevant to the technical scope of this report. Will be addressed in the ecology report
E7.8.1 (6c	i)	How the proposal will address monitoring and reporting requirements incorporating, but not limited to the measurement and recording of water levels and pressures	Recommendations for groundwater monitoring and reporting are provided in **Section 7.2**

3 Conceptual Hydrogeological Model

3.1 Regional Geology

The Grey Lynn Tunnel will be located within the Waitemata Basin, which formed between 24 and 18 million years ago as a subsiding shallow marine environment filled with sediments eroding from landforms. Sediments deposited in the Basin were predominantly interbedded silts and muddy sands with some coarser-grained volcaniclastic sands and conglomerates. Collectively, the sediments are known as the Waitemata Group.

Following deposition, the Waitemata Group sediments were unconformably overlain by Puketoka Formation sediments (2 million to 340,000 years ago) and undifferentiated alluvium (<14,000 years ago) of the Tauranga Group, and by basalt, scoria, lapilli and ash deposits belonging to the Auckland Volcanic Field (250,000 to 500 years ago) (Tuhono, 2011). The Regional geology of the Auckland area has been described in detail in the Groundwater and Surface Settlement report prepared by Tonkin and Taylor (2012).

The spatial distribution of geologic units in the study area is shown in **Figure 2**.

Figure 2. Study area geologic units (see A3 attachment at rear).

The primary materials present in the study area defined in **Section 1.2** are:

- **East Coast Bays Formation (ECBF)** – The primary geologic unit present around the shaft location and surrounding the tunnel alignment. The ECBF is a member of the Waitemata Group rocks characterised by alternating, graded sandstones, and siltstones with facies of volcanic-rich and volcanic-poor material. ECBF deposits are typically grey to greenish grey, very poorly-sorted to moderately-sorted materials with laminated or convoluted beds 0.1 to 1.4 m (median 0.5 m) thick (Tuhono, 2011). Within the ECBF there are zones of highly weathered material (wECBF) and a sub-unit recognized as the Parnell Grit (PG). The wECBF typically occurs in the upper five meters of the ECBF profile and is comprised of residual soils and weathered silts and clays from the ECBF with variable sand content. With depth, the relict structure of the original rock mass is evident.
- **Parnell Grit (PG)** - Volcanoclastic gravity flow deposits originating as submarine lahars. PG materials are comprised of a poorly sorted pebble to boulder size conglomerate in a compacted and cemented muddy to sandy matrix. PG units are difficult to predict the in location and extent because they are vertically and laterally variable, ranging from less than a meter to 20 meters in thickness and occurring at irregular intervals. Due to the units strength and lower clay content, joints can remain open and have a greater persistence than the ECBF allowing localised pathways for groundwater flow (Jacobs, 2016).
- **Auckland Volcanic Field Basalts (AVFB)** – Located to south and southwest of the tunnel alignment and Tawariki Street Shaft Site abutting the ECBF outside of the model boundary. AVFB consist of basalt, scoria, lapilli and ash deposits typically associated with volcanic cones. The basalt is described as grey to very dark grey, dense, fine-grained. Scoria deposits consist of red or red-brown to dark grey or black, angular to sub-rounded, poorly-sorted, and vesicular to very vesicular pebble to boulder size ejecta of basalt composition. Ash and lapilli deposits consist of unconsolidated beds of dark grey to black, very angular to rounded, well-sorted, dense to very vesicular, basalt fragments.
- **Tauranga Group Alluvium (TGA)** – Collectively the Puketoka Formation and recent alluvium and colluvium make-up the TGA. The recent TGA deposits are late Pleistocene to Holocene in age, having been deposited within low lying drainage channels and topography. These deposits are comprised of light grey to orange-brown, well sorted, bedded (2 to 20 mm) silts or clays with variable sand and gravel content and clasts of rhyolite pumice and weathered rock. On the Auckland Isthmus the alluvium is typically derived directly from the weathering and erosion of ECBF (Institute of Geological and Nuclear Sciences, 2001).

3.1.1 Material Hydrogeological Characteristics

The shaft will be situated primarily within the ECBF formation with thin wECBF or TG deposits overlaying at the land surface. There are thin deposits of the TGA material at the land surface adjacent along Motions Creek, a stream which forms the western model boundary. These deposits are considered to have negligible influence on groundwater impacts from shaft construction because they are hydraulically similar to the ECBF (i.e. both of low permeability) and only occur near the land surface. Therefore, only the ECBF was considered for groundwater dewatering modelling purposes.

Geological evolution, including both depositional environment and subsequent morphological processes have a strong influence on the hydrological characteristics of materials. The primary aspects for hydrogeological assessment include the lateral and vertical distribution of materials. Hydrogeological characteristics of these materials have been documented in previous studies and are summarised in **Table 2** and as follows:

- **ECBF:** Typically, low permeability in the range from $1x10^{-8}$ to $3x10^{-6}$ m/s, with an average across Auckland Isthmus of approximately $2.3x10^{-7}$ m/s. Hydraulic conductivity can be greater in areas where fracture zones are present. Strong anisotropy with horizontal conductivity 40 to 250 times greater than vertical conductivity.
- **wECBF:** Lower hydraulic conductivity relative to ECBF due to the influence of colloidal clay from weathering, with a range from $1x10^{-8}$ to $8x10^{-8}$.
- **TGA:** Low to moderate hydraulic conductivity, ranging from $5x10^{-8}$ to $2.5x10^{-5}$ m/s with somewhat greater storage characteristics with specific yields <0.1 in the unconfined areas, and storativity typically found to be around $1x10^{-3}$.

Table 2. Hydraulic parameters within the Auckland Isthmus.

Material	Parameter	Watercare[1.]	Tuhono[2.]	PDP[3.]	Tonkin & Taylor[4.]
		Central Interceptor Phase 1	**Waterview Connection**	**St Marys Bay & Mansfield Beach WQ Improvement Project**	**Central Interceptor Project Effect on GW and Surface Settlement**
ECBF	K_h (m/s)	$7.5x10^{-6}$	$2.3x10^{-7}$	$2.6x10^{-6}$	$2.0x10^{-7}$
	Storativity (1/m)	$1.9x10^{-3}$	$9.0x10^{-6}$	NA	NA
ECBF-Weathered	K_h (m/s)	$8.3x10^{-7}$	$1.0x10^{-8}$	$5.3x10^{-7}$	$2.0x10^{-7}$
	Storativity (1/m)	$3.8x10^{-3}$	$1.0x10^{-3}$	$2.5x10^{-3}$	NA
TGA	K_h (m/s)	$1.3x10^{-7}$	$5.0x10^{-8}$	$2.5x10^{-5}$	$2.0x10^{-7}$
	Storativity (1/m)	NA	$1.0x10^{-3}$	NA	NA
	Specific Yield (m)	$8x10^{-1}$	$1.0x10^{-2}$	$1.3x10^{-1}$	NA

Notes: Table states mean value where reported values were a range. NA = Not Available.

References. 1. Watercare Services LTD, 2013. 2. Tuhono Consortium, 2011. 3. Pattle Delamore Partners LTD, 2018. 4. Tonkin & Taylor, 2012.

3.1.2 Hydraulic Testing

Site specific investigations were performed as a part of the development and planning process for the Grey Lynn Tunnel. Six bores (CIE-BH1 to CIE-BH6) were drilled for the purpose of installing monitoring piezometers and are shown in **Figure 3**. Bore logs documenting geological materials encountered in the drilling process are presented in **Appendix A**. Vibrating wire piezometers were installed in CIE-BH04 and CIE-BH05 and a standpipe monitoring piezometer was installed in CIE-BH06.

Figure 3. Location of Phase 1 and Phase 2 monitoring bores (see A3 attachment at rear).

Hydraulic testing was performed by WWA in all monitoring boreholes. Three slug tests were performed at CIE-BH04, CIE-BH05, and CIE-BH06, respectively, where a volume of water was removed from the open borehole (CIE-BH4 and CIE-BH05) or piezometer (CIE-BH6) using a 2.1 m pipe sealed on one end. Water level recovery was monitored with a data logger.

The rate of water level recovery was evaluated using the Hvorslev method, which entails fitting the slope and offset parameters of a best-fit line to normalised drawdown data over one log interval of time to calculate an estimate of hydraulic conductivity within the test interval of the bore. **Table 3** provides a summary of the slug tests performed and the estimated hydraulic conductivity as determined by water level recovery. Data and analysis details are provided in **Appendix B**.

Table 3. Slug test results.

Borehole ID	Location	Slug Test-Hydraulic Conductivity (m/s)		
		Test 1	Test 2	Test 3
CIE-BH04	46 Tawariki St.	$1.10\text{x}10^{-6}$	$1.07\text{x}10^{-6}$	$1.04\text{x}10^{-6}$
CIE-BH05	44 Tawariki St.	$1.84\text{x}10^{-7}$	$4.01\text{x}10^{-7}$	$3.92\text{x}10^{-7}$
CIE-BH06	Fisherton/Richmond St.	$1.05\text{x}10^{-7}$	$1.49\text{x}10^{-7}$	$1.06\text{x}10^{-7}$

Packer tests (aka Lugeon tests) were performed at all boreholes. These tests involve isolating a section of the borehole using an inflatable packer and then pumping clean water into the bore for five-minute intervals at increasing, and then decreasing pressures, with flow rate monitored during each interval.

Data was subsequently analysed by WWA using the Richter and Lillich (1975) method as described in NZTA (2016) to classify the flow response and estimate hydraulic conductivity. **Table 4** summarises the packer tests performed, testing intervals, and resulting hydraulic conductivity. Estimated hydraulic conductivities derived from packer tests were generally low when compared to slug test and findings from other studies. Testing details, results, and complete analysis are provided in **Appendix C**.

Table 4. Packer (Lugeon) test results.

Borehole	Test Interval		Test result	Permeability (m/s)
	Top (mBGL)	Bottom (mBGL)		
CIE-BH01	17.0	21.5	Void Filling	9.7x10-8
CIE-BH02	18.7	21.5	Laminar	7.3x10-8
CIE-BH03	20.0	24.5	Dilation	5.3x10-8
CIE-BH04	9.8	12.0	Dilation	2.9x10-8
CIE-BH04	19.5	22.5	Laminar	6.5x10-8
CIE-BH04	28.5	31.5	Laminar	1.1x10-7
CIE-BH05	11.0	13.5	Dilation	9.0x10-8
CIE-BH05	19.0	21.0	Wash out	7.1x10-8
CIE-BH05	28.5	31.5	Dilation	2.6x10-7
CIE-BH06	27.0	30.0	Dilation	2.8x10-8
CIE-BH06	50.3	52.5	No Flow	NA
CIE-BH06	56.25	58.5	Dilation	4.2x10-8
CIE-BH06-High pressure	54.5	63.5	Dilation	2.9x10-8

3.2 Groundwater Recharge

The aquifer system in the study area is recharged by rainfall. Recharge along with material characteristics drives the development of hydraulic gradients and head elevations, hence an understanding of the rate and distribution of recharge is essential for estimating groundwater flow rate and volume.

Annual recharge volume varies depending on climate and geology. Geologic parent material governs soil infiltration rate, which in turn controls the partitioning of rainfall into surface runoff and groundwater recharge. Geology also determines the rate of percolation of soil water to groundwater.

Ground surface recharge is relatively high in areas where high permeability basalt is present, estimated to be 15-20% of mean annual precipitation (approximately 190 to 250 mm/year). Recharge is comparatively low in areas where ECBF is the dominant material, ranging from 25-50 mm/year or 2 to 4 % of mean annual rainfall (Tuhono, 2011).

For this study groundwater recharge in the ECBF has been assumed to be 3% of mean annual precipitation.

3.3 Groundwater Flow Direction

Monitoring bore data from Auckland Council and bore installation records from the Grey Lynn Tunnel were assessed and an estimated piezometric surface for the shallow aquifer is presented in **Figure 4**. Based on this analysis, groundwater is presumed to flow from southeast to northwest with an average gradient of approximately 1.5 percent. The groundwater table (shallow aquifer) geometry generally mimics regional topography, with areas of localized perching likely along ridge lines.

Groundwater discharges to surface water at several locations within or adjacent to the study area including Western Springs, Meola Creek, Motions Creek, and Cox's Creek, and is likely drained into local stormwater facilities in several additional locations where local drainage is concentrated.

Figure 4. Estimated piezometric surface (see A3 attachment at rear)

4 Groundwater Modelling Methodology

The MODFLOW (2005) Regular Grid, developed by the United States Geological Survey (USGS), was utilised within the GMS10.2 modelling platform to construct the groundwater flow model for the Tawariki Street Shaft. The discretisation of the model domain with decreasing cell size around the shaft area provides increases the resolution for areas of maximum interest (the shaft) and decreases resolution in other areas, thereby increasing the efficiency in model computation compared to a similarly constructed structured MODFLOW grid.

4.1 Model Domain

The study area, as defined by the model boundary, covers an area of 6.4 km^2 and was constructed based on nine layers, with a total of 35,028 active cells. The model was discretised using a global grid spacing of 50 m with a finer resolution grid spacing of down to 1.5 m in the shaft area. The same grid layout was used for each of the model layers. This spatially varying discretisation approach reduced model computational time while improving model resolution in the area of interest (**Figure 5**).

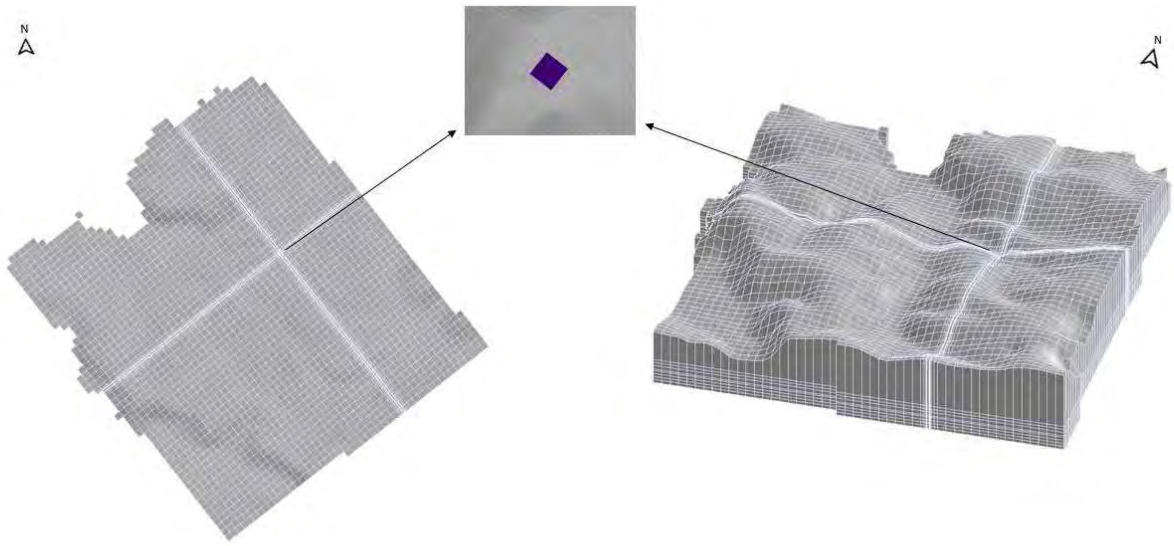

Figure 5. MODFLOW-USG grid in plan view with shaft area detail and orthogonal view with vertical magnification of 5.

The surface elevation used for the model was determined using the 1 m Lidar digital elevation model data available through the Land Information New Zealand (LINZ) service. Surface elevation for the model area is shown in **Figure 6**.

Figure 6. Model area ground surface elevation (see A3 attachment at rear).

4.1.1 Constant Head Boundaries

The northwest model boundary follows the coastline and was assigned a constant head boundary condition (CHB) of 0 m AMSL for model Layer 1 to represent the mean hydraulic head of the ocean at these locations.

4.1.2 General Head Boundaries

A general head boundary (GHB) is typically used to simulate the flow interaction between groundwater and external water sources to the model domain. The cells along the coastline from Layer 2 through 9 were also assigned with GHBs. The head values for all the cells were assigned as 0 mAMSL and the conductance value of each layer decreasing with the depth to reflect the progressively increasing disconnection with the free water surface of the ocean (i.e. the impedance of flow to the ocean floor increases with depth) and also the resistance of higher-density seawater offshore.

4.1.3 No-Flow Boundaries

No-flow boundaries were assigned to cells located on the northeast, southwest, and southeast boundaries of the model domain. Ridgelines along the northeast and southeast boundaries are expected to act as local groundwater divides with recharging water following local topography down slope.

Shallow groundwater along the southwest model boundary discharges into Motions Creek while deeper groundwater flows parallel to groundwater in the model area toward the Waitemata Harbour.

The base of the model was set significantly below the depth of the Tawariki Street Shaft or Grey Lynn Tunnel so that lower boundary conditions would not impact the simulation. A no-flow boundary condition was then assigned to the lower model boundary on the basis that groundwater at this depth has negligible bearing on the overall flow budget of the portion of the aquifer system impacted by the Tawariki Street Shaft.

4.1.4 Drain Boundaries

Drains in the model area were identified from the River Environment Classification (REC) database New Zealand.

The primary surface drains are Motions Creek, which forms the western model boundary discharging into Waitemata Harbour and Cox's Creek, which drains the central portion of the model area discharges into Cox's Bay. A subsurface drain passes below Tawariki Street, adjacent to the shaft location and discharging into Cox's Creek. The surface and sub-surface drains in the model area are included in **Figure 1** (attached at rear).

Drain boundaries were assigned in the model to simulate the groundwater discharged to the streams within the model area, subsurface drains, and perennial wet areas where they occur within the model area. The drain bed elevations were derived from the Digital Elevation Model (DEM) generated from LiDAR data, with specific depth determined through the model calibration process and based on the type of drain feature. Cells within the Tawariki Street Shaft were also assigned as drain boundaries with drain elevations decreasing with time over the construction period to simulate the increasing depth of the shaft. Following the construction period, the shaft drain elevations remain level with the bottom of the shaft.

- ***Surface streams*** – DEM minus 2.0 m;
- ***Subsurface drains*** – DEM minus 2.0 m;
- ***Inundated areas*** – Equal to DEM elevation
- ***Shaft Drains*** – Increasing depth to -13 mAMSL

The conductance value of the drains was set relatively high to reflect limited impedance to water removal (or drain functionality) where surface discharge was expected.

4.1.5 Horizontal Flow Barrier

A horizontal flow barrier (HFB) was assigned to the cells around the Tawariki Street Shaft location for model layers one through four encompassing the vertical extent of the completed shaft. The conductance of the barrier was varied to simulate a range of liner permeabilities. The HFB was only used in the transient simulations and was not included in the 'No-Barrier' scenario.

4.1.6 Well Boundaries

No wells were simulated in the model as there are no major groundwater users within the model area.

4.1.7 Sparse Matrix Solver

The Sparse Matrix Solver (SMS) package was utilised to solve linear and non-linear equations. A maximum head change of 0.01 m between iterations was set as the model convergence criteria. Default values were used for the maximum number of iterations for linear and non-linear equations.

4.2 Model Layer Configuration

4.2.1 Layer Geology

The model comprises nine layers that are used to represent the geologic strata and allow for the flow restrictions that would naturally occur in a stratified and vertically variable formation such as the ECBF. The ECBF material type was assigned for each model layer based on a review of the borelogs included in **Appendix A** and the findings of other geologic investigations within the Auckland Isthmus. TGA deposits in the model area were lumped with ECBF because the two materials have a largely overlapping range of hydraulic parameters and are therefore functionally the same for modelling purposes.

Model Layer 1 encompasses all the material within the model area from the ground surface to 1.0 m below mean sea level (-1 mAMSL). This value was selected to avoid numerical errors that can occur along the coastal margins where surface elevations were approximately 0 mAMSL.

The bottom elevation for each model layer was assigned as a uniform elevation with the specific elevation of Layer 4 determined to be 1 m below the bottom elevation of the Tawariki Street Shaft. The elevation configuration of the model layers is shown in **Table 5**.

Table 5. Model layer elevation configuration.

Model Layer	Top Elevation (mAMSL)	Bottom Elevation (mAMSL)
1	LINZ LiDAR Elevation	-1
2	-1	-4
3	-4	-9
4	-9	-14
5	-14	-16
6	-16	-20
7	-20	-24
8	-24	-28
9	-28	-32

5 Model Calibration

The model calibration was primarily conducted by manually changing the model hydraulic parameters to achieve an acceptable fit to measured groundwater levels. Drain elevation for surface streams relative to the DEM were tested at several levels and specific adjustments were made to match groundwater level observations. Groundwater recharge was not considered a calibration parameter.

5.1 Observation Points

Water level measurements obtained from six boreholes installed in preparation for shaft and tunnel development were used to guide model calibration. The boreholes used for calibration of the model are as shown in **Figure 3** and the key properties of the boreholes relevant to model calibration are summarised in **Table 6**.

Three of the boreholes are located directly around the planned shaft location on Tawariki Street. All but one of the boreholes are constructed on relatively low-lying areas situated between 9 and 13 mAMSL with the exception being the CIE-BH06 which is on a ridge at 48 mAMSL. It is notable that this borehole had the lowest conductivity of those tested.

The borehole screen intervals ranged from approximately -7 to -20 mAMSL corresponding to model Layers 3 through 6. Vibrating wire piezometers were installed in CIE-BH04 and CIE-BH05 however the water levels used for these boreholes were obtained prior to piezometer installation when the boreholes were uncased therefore the water levels were considered to be representative of the bottom elevation of the borehole.

Table 6. Summary of borehole information used in calibration.

Borehole ID	Location	Surface Elevation (mAMSL)	Borehole Depth (m)	Bottom Elevation (mAMSL)	Top of Screen (mAMSL)	Bottom of Screen (mAMSL)	Model Layer	Water Level (mAMSL)
CIE-BH01	28 Cockburn St.	13.31	25.5	-12.19	-3.19	-8.69	3	12.45
CIE-BH02	Hakanoa Reserve	9.68	25.5	-15.82	-7.32	-12.32	4	11.82
CIE-BH03	41 Tawariki St.	13.00	27.5	-14.50	-6.51	-11.51	4	15.04
CIE-BH04	46 Tawariki St.	11.94	31.5	-19.56	Vibrating wire piezometer (26 mBGL)		5	10.91
CIE-BH05	44 Tawariki St.	11.02	31.5	-20.48	Vibrating wire piezometer (26 mBGL)		6	13.79
CIE-BH06	Fisherton/Richmond St.	47.55	63.5	-15.95	0.35	-6.95	3	44.39

5.2 Steady-State Calibration

A steady-state model was developed and calibrated to validate the conceptualisation of the groundwater flow model. The objective of the calibration was to determine hydraulic parameters such that simulated groundwater head matched observations as accurately as possible, and to obtain initial heads for transient model simulation.

The six water level observations were used as the calibration targets. The simulated head is plotted against observations in **Figure 7**. The steady-state simulation has a mean head residual of 1.19 m, and root mean square error (RMSE) of 2.4 m, which is approximately 7% of the range of observations. A simulated RMSE of less than 10% of the measured range is considered a good calibration.

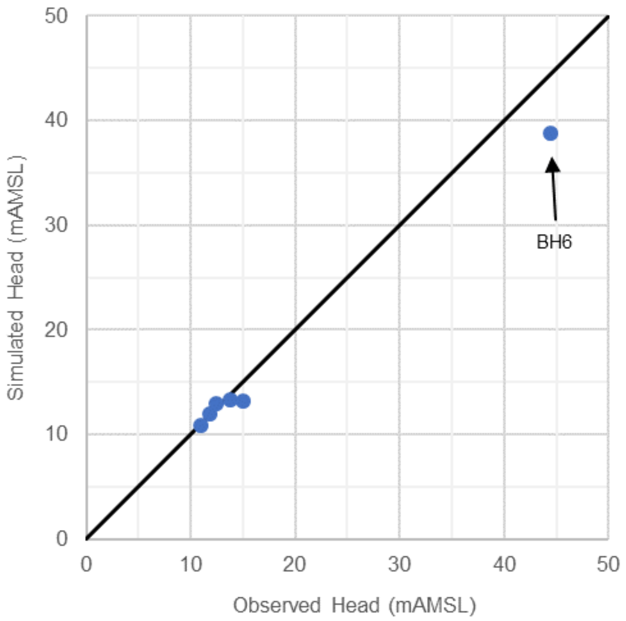

Figure 7. Simulated head versus observed head

The RMSE is strongly influenced by the observation at CIE-BH06. This bore is a relative anomaly compared to the other bores as it is the only observation obtained from a bore located on a ridge. The observed water level at CIE-BH06 was under simulated by the model by 5.6 m. Several methods were attempted to obtain a calibration that would match CIE-BH06 without losing the calibration at the other boreholes. This included varying hydraulic conductivity and vertical anisotropy within the range considered appropriate for ECBF material. It was apparent that reducing conductivity sufficiently to match the observed water level at CIE-BH06 generated an error over 10 m at the other boreholes.

Another approach was to vary conductivity with elevation based on the concept consistent with the geologic evolution of the landscape that ridges tend to be composed of more resistant material than valleys. This approach generated a good match for computed versus observed head at all boreholes for the model layers corresponding to the respective observations however the simulated head in Model Layer 1 was far above realistic values indicating widespread flooding over the model area.

Finally, to partially compensate for the high observed head at CIE-BH06 a low permeability zone was incorporated into the model over the ridge where CIE-BH06 is located. This represents an area where the parent rock is more resistant and less permeable that than what is present at the other boreholes and is supported by the low conductivity measured during hydraulic testing at CIE-BH06.

The simulated water levels obtained through the model calibration process are presented in **Table 7**.

If the water level observation at CIE-BH06 is disregarded, the RMSE is reduced to 0.89 m, representing 1% of the range of observations and mean head residual is reduced to 0.32 m.

Table 7. Observed and simulated water levels from steady state calibrated model

Borehole ID	Model Layer	Observed Water Level (mAMSL)	Simulated Water Level (mAMSL)	Residual (m)
CIE-BH0 01	3	12.45	12.92	-0.46
CIE-BH0 02	4	11.82	12.03	-0.21
CIE-BH0 03	4	15.04	13.15	1.88
CIE-BH0 04	5	10.91	10.92	-0.02
CIE-BH0 05	6	13.79	13.38	0.41
CIE-BH0 06	3	44.39	38.83	5.56

5.2.1 Calibrated Model Parameters

The calibrated model parameters are shown in **Table 8**. The calibrated model parameters are consistent with hydraulic parameters obtained in other investigations of ECBF material as shown in **Table 2**.

Table 8. Calibrated model hydraulic parameters

Material	Hydraulic Conductivity (m/s)	Vertical Anisotropy	Specific Storage (Layers 2-9)	Specific Yield (Layer 1)
ECBF	$3.0x10^{-7}$	30	0.0005	0.25
ECBF-Low permeability zone	$1.0x10^{-8}$	10	0.0005	0.25

The calibrated model hydraulic conductivity for the ECBF was $3.0x10^{-7}$ m/s with a vertical anisotropy of 30. Calibrated conductivity in the low permeability zone was over an order of magnitude lower at $1.0x10^{-8}$ m/s possibly indicating a highly compacted, unstratified area within the formation.

5.2.2 Model Flow Budget

Table 9 provides the long-term average water budget for the steady state calibration model. Groundwater recharge accounts for the entire model inflow.

The predominant discharge components from the model are the combined stream baseflow, which accounts for 61% of the model outflow. Coastal boundary outflows comprise 21% of the total model outflow with the majority occurring below the surface layer; largely because Layer 1 is very thin along the coastal margin so there is little material available through which outflow can occur. Approximately 19% of the model area groundwater outflow is predicted to occur at Western Springs in the southwest portion of the model area.

Table 9. Calibrated model groundwater flow budget

Mass balance	Components	Flow (m^3/d)	Percentage of Flow (%)
Inflow	Recharge	633	100
	Total inflow	633	100
Outflow	Shallow Coastal Discharge (CH)	-10	-1.6
	Deep Coastal Discharge (GHB)	-103	-16.2
	Stream Baseflow (Drain)	-520	-82.2
	Total outflow	633	100
Percentage discrepancy		0.03%	

6 Predictive Simulations

6.1 Scenario Setup

The numerical groundwater model was developed to assess the effect of construction of the shaft on local groundwater conditions. This assessment included a range of construction alternatives in the form of differing shaft liner permeability. In testing a range of liner permeabilities the model results can also be interpreted as a sensitivity analysis for liner permeability on groundwater impact. Aside from incorporating the Tawariki Street Shaft, all transient model variations applied the same boundary conditions as were used in the steady state calibration model.

The specified construction approach to the shaft is as follows:

- Temporary excavation support through soil materials and ECBF material shall consist of either secant piles, sheet piles, ring beams with lagging, steel liner plate, precast segmental rings, caisson or similar, and will be designed to be near-watertight to limit groundwater drawdown.

- Linings constructed of permanent concrete (precast or cast -in-situ), or potentially other corrosion resistant materials will be installed to support ground and groundwater loads in the long-term, provide a conduit for sewer hydraulic drop structures, and limit groundwater infiltration per to NZS 3106, *Design of Concrete Structures for the Storage of Liquids*, (tightness class 2).

The seven predictive model scenarios can be summarised as follows:

- ***Scenario 1: Basecase*** – The steady state calibration model was run as a transient model for the same time period as other scenarios. The shaft was not included in the model.
- ***Scenario 2: No Barrier*** – The shaft was incorporated into the steady state calibration model. Construction of the shaft proceeded at a rate of 2 m/day, reaching completion at 25 mBGL after 13 days. The model was run for a one year time period. No HFB was applied around the shaft.
- ***Scenario 3: Moderate Permeability Flow Barrier*** (***10^{-8} m/s)*** – The simulation was set up identically to Scenario 1 with the inclusion of the shaft and the addition of a HFB boundary applied around the shaft. The permeability of the HFB was assumed to be 1x10^{-8} m/s.
- ***Scenario 4: Low Permeability Flow Barrier*** (***10^{-9} m/s)*** – The simulation was set up identically to Scenario 1 with the inclusion of the shaft and the addition of a HFB boundary applied around the shaft. The permeability of the HFB was assumed to be 1x10^{-9} m/s.
- ***Scenario 5: Extra Low Permeability Flow Barrier*** (***10^{-10} m/s)*** – The simulation was set up identically to Scenario 1 with the inclusion of the shaft and the addition of a HFB boundary applied around the shaft. The permeability of the HFB was assumed to be 1x10^{-10} m/s.
- ***Scenario 6: Low Permeability Flow Barrier to 7 m BGL*** – The simulation was set up identically to Scenario 1 with the inclusion of the shaft and the addition of a HFB boundary applied around the shaft extending to 7 m BGL. The permeability of the HFB was assumed to be equal to Scenario 4 (1x10^{-9} m/s).
- ***Scenario 7: Low Permeability Flow Barrier to 7 m BGL-High Conductivity Material*** – The simulation was set up identically to Scenario 6; however, conductivity of the upper model layer was increased to 1x10^{-6} m/s to evaluate the sensitivity of predicted shaft drainage and drawdown to material conductivity
- Based on the specified construction methods for the shafts, Scenario 6 best approximates the temporary condition during construction, while Scenario 4 approximates the long-term condition during operations.

6.1.1 Construction Sequence

The shaft was excavated to a depth of 1 m on the first day of the simulation and then proceeded at a rate of 2 m/day thereafter until the terminal depth of 25 m was reached. The construction period totalled 13 days and the shaft depth remained constant for the rest of the simulation (**Figure 8**).

The simulation was run with a daily time step for the first month, after which it converted to a weekly time step as model input conditions were constant and simulated conditions approached steady state.

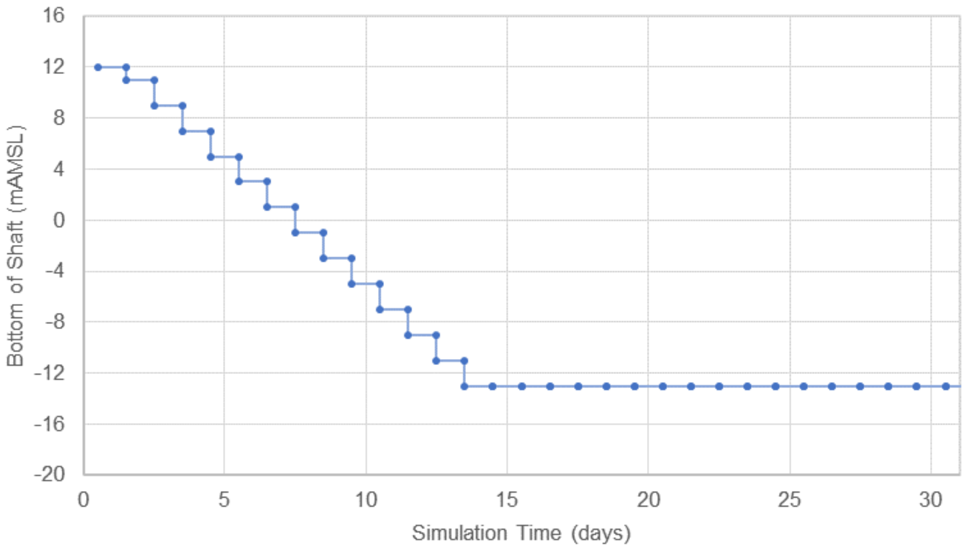

Figure 8. Elevation at bottom of shaft for first month of simulation.

6.1.2 Shaft and Liner Details

The thickness and depth of the flow barrier was 0.5 m and 25 m, respectively. This depth equates to an RL at the shaft of -13 mAMSL, which is 1 m above the bottom of Layer 4 in the model.

6.1.3 Boundary Conditions

An HFB was assigned to the cells around the shaft location for model Layers 1 through 4 encompassing the vertical extent of the completed shaft. The conductance of the barrier was varied to simulate the range of liner permeabilities tested in Scenario 1 through Scenario 3. The HFB was not included in the Scenario 1 (Baseline) or Scenario 2 (No Barrier).

The cells inside of the HFB and on the bottom of the shaft were assigned as drains where drainage was tracked over the course of the simulation with the resulting values representing drainage into the shaft.

6.1.4 Stress Periods and Time Steps

The model was simulated in transient mode for two years from 1 October 2018 to 30 September 2020. The simulation was subdivided into 131 stress periods where imposed stresses remain constant. Each day was considered a stress period for the first month of the simulation to capture the hydrologic changes that may occur during the shaft construction period. After the first simulation month weekly stress periods were applied as the rate of change in groundwater conditions was expected to decline and eventually approach steady state.

Each stress period consisted of ten time steps, with head and flow volume in each model cell evaluated at the end of each time step.

6.1.5 Initial Conditions

The transient model used the steady-state model heads as the starting condition.

6.1.6 Model Hydraulic Parameters

The calibrated model hydraulic parameters shown in **Table 8** were applied in all of the transient models.

6.2 Model Results

As described above, at completion of construction, the base of the shaft will be at -13 mAMSL (25 mBGL) corresponding to Layer 4, which is where the maximum impact on groundwater is expected to occur. For this reason, results are reported for Layer 4 to reflect the full impact of the shaft on groundwater conditions.

As previously stated, the shaft construction period was assumed to proceed at 2 m per day though 1 m was assumed for the first simulation day assuming some start up time. In all simulations a rapid change in groundwater level was predicted over the construction period and for the following days, however the rate of change slowed significantly by the end of the first month. After one year groundwater conditions had reached a quasi-steady state. Model results are reported for one month and one year after the initiation of shaft construction.

6.2.1 Drainage into Shaft

Simulated drainage into the shaft during and following construction for Scenarios 2 through 5 is presented in **Figure 9**. The greatest level of drainage is predicted to occur in scenarios where the shaft is unlined, i.e. Scenario 2, Scenario 6, and Scenario 7. The lining in Scenarios 6 and 7 had an impact when the shaft excavation was above the level of the liner material, producing the results virtually identical to Scenario 4 which had the same liner permeability. Once the excavation was below the liner level, seepage into the shaft increased in both scenarios relative to Scenario 2 where the liner was absent altogether. Once the additional seepage had drained the scenarios behaved identically to the Scenario 2 because conditions were the same. The higher permeability material tested in Scenario 7 had virtually no influence on drainage into the shaft as flow was controlled by the liner material.

The rate of seepage into the shaft is predicted to decline as the permeability of liner materials is decreased in Scenarios 3 through 5. However, there is negligible difference in the predicted drainage for Scenarios 4 and 5 indicating that the majority of flow in these scenarios is emerging through the floor of the shaft where there is no flow barrier. This indicates that a barrier with a permeability of 10^{-9} m/s, as is applied in Scenario 4, would be an effective barrier to prevent groundwater draining through the shaft walls.

Table 10 presents the predicted peak and steady state rate of drainage into the shaft. When no flow barrier is used in Scenario 2, a peak of 31.6 m^3/day is predicted, reducing to 30.3 m^3/day in Scenario 3. The more impermeable barriers used in Scenarios 4 and 5 reduce predicted peak flow into the shaft to 24.0 and 22.8 m^3/day, respectively.

A greater peak drainage is predicted in Scenario 6 and 7 because water that is initially detained by the flow barrier drains quickly after the excavation level falls below the barrier on day 5 of the simulation. In Scenario 6 the maximum drainage was 32.1 m^3/day and in Scenario 7 the peak flow was 32.0 m^3/day. Reducing barrier permeability has a limited impact on reducing drainage during excavation because of the limited penetration of the lining. Groundwater readily flows up through the bottom of the shaft where there is no lining.

Seepage increases sharply during the first week of shaft excavation in all scenarios, levelling off during the latter half of the excavations and then declining rapidly after the shaft excavation is complete and groundwater levels are reduced. As opposed to peak drainage, steady state drainage into the shaft is reduced significantly by decreasing the permeability of the liner.

The steady state drainage rate predicted follows a similar pattern with Scenario 2 generating 5.6 m^3/day of drainage into the shaft, which reduces to 4.0 m^3/day in Scenario 3. A drainage rate of approximately 2.2 m^3/day is predicted for Scenario 4 and 1.8 m^3/day for Scenario 5. This shows only a minor reduction in drainage is achieved by reducing the permeability of the barrier from 10^{-9} to 10^{-10} m/s. Scenarios 6 and 7 are essentially the same as Scenario 2 once the excavation level drops below the barrier.

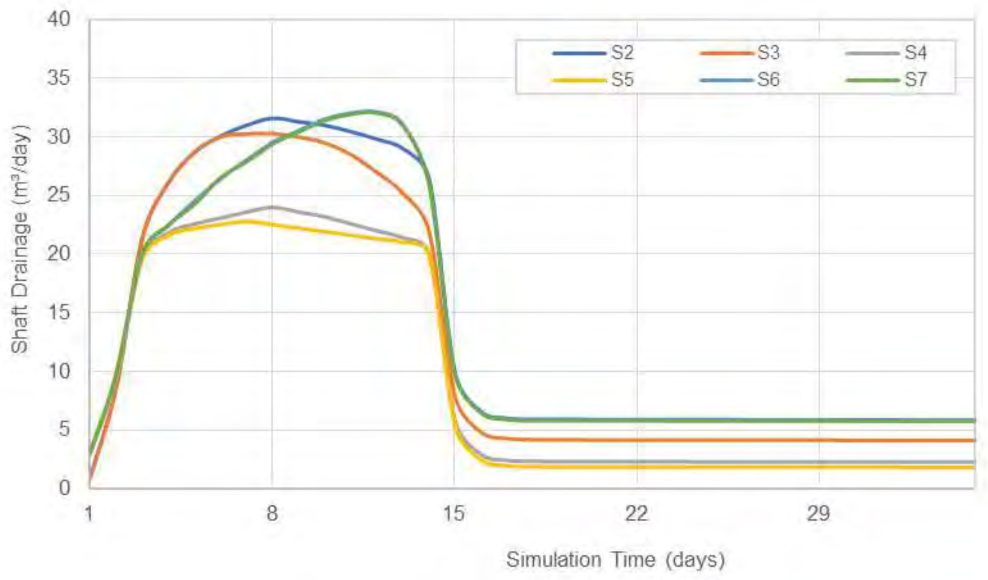

Figure 9. Simulated groundwater drainage into shaft during and immediately following construction.

Table 10. Model predicted peak and steady state groundwater drainage into shaft.

Scenario	Drainage Into Shaft-30 vertical anisotropy	
	Peak (m³/day)	Steady State (m³/day)
S2	31.6	5.6
S3	30.3	4.0
S4	24.0	2.2
S5	22.8	1.8
S6	32.1	5.6
S7	32.0	5.5

6.2.2 Mass Balance

A comparison of the average flow budget between the scenarios one year after the initiation of shaft construction is presented in **Table 11**. At this point in time the simulated groundwater conditions have reached steady state in all model scenarios. The purpose for providing this information is to demonstrate that the simulated water budget is internally balanced and reflects the expected hydrological conditions in the model area.

Table 11. Average flow budget one year after initiation of shaft construction.

Components	**Scenario 1**		**Scenario 2**		**Scenario 3**		**Scenario 4**		**Scenario 5**		**Scenario 6**		**Scenario 7**	
	Flow (m3/d)	**% of Flow**	**Flow (m3/d)**	**% of Flow**	**Flow (m3/d)**	**% of Flow**	**Flow (m3/d)**	**% of Flow**	**Flow (m3/d)**	**% of Flow**	**Flow (m3/d)**	**% of Flow**	**Flow (m3/d)**	**% of Flow**
Recharge	632.7	99.9	632.7	99.5	632.7	99.7	632.7	99.9	632.7	99.9	632.7	99.5	632.7	99.5
Storage	0.6	0.1	3.2	0.5	2.0	0.3	0.8	0.1	0.6	0.1	3.2	0.5	3.2	0.5
Total inflow	633.3	100	635.9	100	634.7	100	633.6	100	633.3	100	635.9	100	636.0	100
Storage	0.7	0.1	0.6	0.1	0.6	0.1	0.7	0.1	0.7	0.1	0.6	0.1	0.5	0.1
Shallow Coastal Discharge (CH)	9.8	1.6	9.8	1.5	9.8	1.5	9.8	1.5	9.8	1.6	9.8	1.5	26.7	4.2
Deep Coastal Discharge (GHB)	102.7	16.2	102.7	16.1	102.7	16.1	102.7	16.2	102.7	16.2	102.7	16.1	89.2	14.0
Surface/Sub-surface Drainage	521.6	82.2	518.8	81.4	519.3	81.6	519.8	81.8	520.0	81.9	518.8	81.4	515.2	80.9
Shaft Drainage	0.0	0.0	5.6	0.9	4.0	0.6	2.2	0.3	1.8	0.3	5.6	0.9	5.2	0.8
Total outflow	635	100	638	100	636	100	635	100	635	100	638	100	637	100

Key observations from **Table 11** include:

- Recharge accounts for virtually all of the model inflow in all scenarios though there is a small influx from groundwater storage predicted in the scenarios where the shaft is included.
- The influx from storage is a result of increased groundwater gradient where there is a cone of depression in the immediate vicinity of the shaft, increasing with the permeability of the barrier.
- Stream flow accounts for 82% of model outflow under baseline conditions (Scenario 1) with a significant portion of groundwater outflow emerging at Western Springs in the southwest part of the model area.
- With the shaft included in the model, a small amount of groundwater that would otherwise flow into surface streams or subsurface drains seeps into the shaft.
- The maximum amount of drainage into the shaft is in Scenario 2 where no barrier is applied.
- In Scenario 2, 1% (6 m³/day) of groundwater outflow in the model area is predicted to flow into the shaft. This declines to 0.6% (4 m³/day) in Scenario 3, and with more impermeable barriers in scenarios 4 and 5 the portion of groundwater outflow into the shaft falls to approximately 0.3% or 2 m³/day.
- Groundwater that drains into the shaft proportionally reduces the amount of groundwater discharging to surface water; however, the maximum reduction is 2.8 m³/day with the exception of Scenario 7 where a slightly greater reduction is predicted but this is due to the different material properties applied and does not signify an impact related to the shaft.
- The shaft is not predicted to impact coastal discharge.

6.2.3 Stream Flows

An analysis of the predicted impact of shaft construction on streamflow was undertaken. There was no measurable impact predicted for either Motions Creek or Cox's Creek (<0.01 L/s).

It should be noted that the model only reflects flow in Motions Creek originating from the east side of the stream as the area to the west is outside of the model boundary; therefore flow in Motions creek is underestimated and effects estimated here are conservative.

6.2.4 Aquifer Drawdown Effects

Groundwater drawdown within the aquifer adjacent to the shaft was calculated by subtracting predicted groundwater head for Scenarios 2 to 6 in Layers 1 to 4 from the corresponding head in the baseline model (Scenario 1). Layers beneath Layer 4 are not impacted by drawdown because they are below the bottom of the shaft. Model results from two years after the initiation of shaft construction were used for the calculations to allow groundwater conditions to reach steady state in all scenarios.

Predicted groundwater drawdown resulting from construction of the shaft are presented in **Table 12** using distances of 1, 10, and 100 m from the shaft for reference.

The greatest drawdown is predicted in Scenario 2 where no flow barrier is applied in the shaft and in Scenario 6 where the completed shaft extends 18 m below the lining. At a distance of 1 m from the shaft, drawdown in Layer 4 is 9.7 m whereas 5.5 m of drawdown is predicted 10 m away and 0.6 m is predicted 100 m away.

With a relatively permeable barrier installed, as in Scenario 3, the predicted drawdown in Layer 4 declines to 6.2 m, 3.7 m, and less than 0.5 m at distances of 1, 10, and 100 m from the shaft. The less permeable barriers used in Scenarios 4 and 5 decrease predicted Layer 4 drawdown at 10 m from the shaft to 1.6 and 1.1 m, respectively.

Figure 10 shows simulated groundwater head in Layer 4 at 10 and 100 m from the shaft for Scenario 2, where predicted drawdown is relatively high due to the unlined shaft, and Scenario 4 where a relatively impermeable liner is used as the expected long term condition. At 10 m from the shaft, groundwater head declines by approximately 5.5 m in Scenario 2. In Scenario 4 this impact is reduced to approximately 1.6 m. The simulated decline in groundwater head at 100 m form the shaft minimal in Scenario 2 and negligible in Scenario 4.

Table 12. Model predicted groundwater drawdown one year after the initiation of shaft construction.

Distance from Shaft (m)	Model Layer	Predicted Drawdown (m)				
		Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6
1	Layer 1	0.02	0.02	0.01	0.01	0.02
	Layer 2	1.35	0.93	0.46	0.34	1.35
	Layer 3	2.73	1.83	0.84	0.60	2.73
	Layer 4	9.71	6.19	2.30	1.36	9.71
10	Layer 1	0.02	0.02	0.01	0.01	0.02
	Layer 2	1.30	0.89	0.45	0.34	1.30
	Layer 3	2.45	1.66	0.79	0.58	2.45
	Layer 4	5.50	3.66	1.62	1.13	5.50
100	Layer 1	0.11	0.10	0.08	0.08	0.11
	Layer 2	0.42	0.32	0.20	0.17	0.42
	Layer 3	0.54	0.40	0.24	0.21	0.54
	Layer 4	0.64	0.47	0.29	0.24	0.64

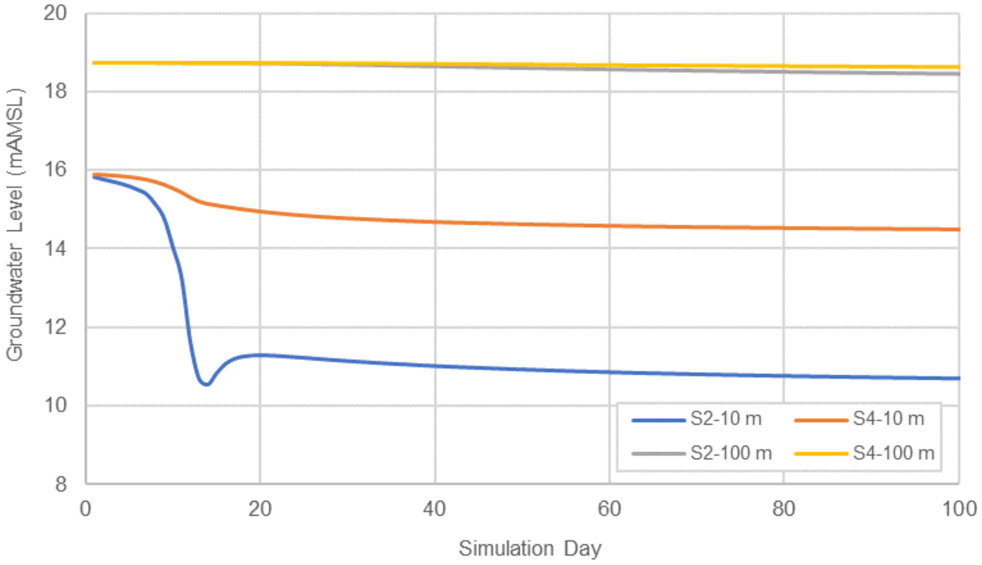

Figure 10. Simulated groundwater head in Layer 4 for the first 100 days in Scenario 2 and Scenario 4

Predicted drawdown in Scenario 4, considered to represent the long term condition after shaft construction, is shown for model Layer 2 in **Figure 11** and for model Layer 4 in **Figure 12**. Less than 0.2 m of drawdown was simulated in model Layer 1 throughout the model area. Model Layer 2 was selected to show expected drawdown at a level relatively near the surface. Model Layer 4 corresponds to where the bottom of the shaft is located and where maximum drawdown is expected to occur, though it is below the area where structures or infrastructure will be affected.

Scenario 6 is considered to represent the conditions during construction and prior to installation of the full shaft lining. Predicted drawdown for model Layer 2 is shown in **Figure 13** and for Layer 4 in **Figure 14**. The extent of drawdown in Scenario 6 is greater than in Scenario 4 because the temporary shaft lining only extends to 7 m BGL, allowing a greater cone of depression to form around the shaft prior to the full liner installation.

The lateral extent of predicted drawdown from Scenario 4 and Scenario 2 two years after the initiation of shaft construction is presented in **Table 13**. Scenario 2 represents the greatest potential drawdown among the scenarios that applied calibrated hydraulic parameters, while Scenario 4 is the most likely long-term condition.

Model results were assessed to determine the extent where drawdown was predicted to be 5 cm or more for each model layer and the maximum drawdown outside of the shaft. The maximum distance from the shaft where 5 cm of drawdown was predicted in Layer 1 was approximately 300 m, though the maximum drawdown in Layer 1 was 0.2 m in the unlined scenario and 0.1 m with a lined shaft. The maximum distance where 5 cm of drawdown was predicted was 420 m from the shaft in Scenario 2, Layer 4.

The maximum drawdown in Layer 4 (9.8 m in Scenario 2) was directly adjacent to the shaft and was reduced to 2.3 m in with a lined shaft as in Scenario 4. Maximum drawdown in shallower layers was significantly less than in Layer 4. It is evident that the impact of drawdown is relatively limited and does not reach the coast or any significant surface water features.

Table 13. Lateral extent and maximum predicted drawdown in select model layers for Scenario 4 and Scenario 6.

Model Layer	**Extent of Drawdown (m)**		**Maximum drawdown outside of shaft (m)**	
	Scenario 4	**Scenario 2**	**Scenario 4**	**Scenario 2**
1	300	300	0.1	0.2
2	340	395	0.5	1.4
4	365	420	2.3	9.8

Figure 11. Predicted drawdown after one year in Layer 2 from Scenario 4 (see A3 attachment at rear).

Figure 12. Predicted drawdown after one year in Layer 4 from Scenario 4 (see A3 attachment at rear).

Figure 13. Predicted drawdown after one year in Layer 2 from Scenario 6 (see A3 attachment at rear).

Figure 14. Predicted drawdown after one year in Layer 4 from Scenario 6 (see A3 attachment at rear).

7 Assessment of Effects

The following discussion is an assessment of potential groundwater related effects from construction of the shaft and with consideration for the relevant provisions of the Auckland Unitary Plan as referenced in **Section 2.3**.

Construction of the secondary shaft, to be built directly adjacent to the shaft, will be initiated a minimum of 2.5 years after the initial construction period for the shaft. This time frame will allow for a full recovery of groundwater levels following the construction of the shaft. The secondary shaft will be the same depth as the main shaft and slightly less in diameter, therefore groundwater effects from the secondary shaft will be within the envelope of (albeit slightly less than) the effects from construction of the main shaft, as described herein.

7.1 Potential Environmental Impacts

The following items are addressed based on the stated criteria for groundwater impacts related to restricted discretionary activities as defined in the AUP. The items addressed in the following sub-sections are those within the scope of this report considered relevant to construction of the proposed shaft, as defined in **Section 2.2**.

7.1.1 Stream Baseflow

A reduction of 0.6 m³/day (0.007 L/s) is predicted on baseflow for Cox's Creek of 211 m³/day. This represents an impact of 0.28% on baseflow and is considered less than minor. No impact on baseflow is predicted on Motions Creek.

7.1.2 Lake Levels

The closest lake is Western Springs, which is 1,800 m from the Tawariki Street Shaft Site. There are no adverse impacts predicted on Western Springs lake as the cone of depression does not extend to the lake. The shaft is predicted to only cause measurable drawdown (> 0.05 m) within 420 m of the Tawariki Street Shaft Site if unlined. With a lined shaft, similar to Scenario 4, this distance drops to under 350 m.

7.1.3 Existing Groundwater Takes

There are no groundwater takes in the area impacted by the shaft construction. The closest consented groundwater take is a 150 mm bore used for irrigating the sports ground at Eden Park, which is 2.5 km south of the Tawariki Street Shaft Site whereas the radius of worst case expected drawdown is 420 m.

7.1.4 Saline Intrusion

The reduction in groundwater level is, at worst, predicted to extend 420 m from the Tawariki Street Shaft Site. The area of anticipated reduction in groundwater level does not extend to the ocean, so there are no adverse effects related to saline intrusion predicted.

7.1.5 Surface Flooding

Changes in groundwater levels or flow patterns resulting from the shaft construction will not generate any increase in the frequency or magnitude of flood events. Depressurisation only serves to reduce moisture content of waterlogged materials and flooding.

7.1.6 Cumulative Effects of Groundwater Diversions

Cumulative effects are not applicable because there will not be any additional projects diverting groundwater within the study area.

7.1.7 Discharge of Contaminated Groundwater

Groundwater that drains into the shaft will be collected and routed to Watercare's own water treatment facilities.

7.1.8 Surface Water Effects

The Tawariki Street Shaft Site will be constructed on what is currently an urban residential street. Existing land and stormwater drainage is routed into subsurface pipes and diverted through the area. The anticipated hydrological flow regime impact from the proposed shaft construction is predicted to be less than minor.

Residual uncertainty regarding the potential impact of shaft construction on groundwater will be addressed in the following recommendations for the monitoring of, and reporting on, groundwater conditions.

7.1.9 Potential Settlement

Potential consolidation settlements due to groundwater drawdown are addressed in a separate assessment report.

7.2 Recommendations for Groundwater Monitoring and Reporting

Recommendations for groundwater monitoring prior to, during, and following shaft construction are based on the conditions stated in the consent for the Central Interceptor Main Works as provided by Watercare (2013) and consideration of specific site conditions at the proposed location of the shaft at the Tawariki Street Shaft Site. The monitoring protocol recommended below will provide information to confirm that the magnitude of impact, if any, associated with the development of the shaft is no greater than predicted in this AEE, and to inform management decisions should ground settlement triggers be reached where preventative action is required.

1. Groundwater monitoring boreholes shall be installed prior to construction to enable the establishment of baseline groundwater conditions. At least one of these boreholes shall be within 100 m of the shaft location and the another approximately 500 m from the shaft location adjacent to the proposed Grey Lynn Tunnel.

   Note: Six boreholes have been installed for monitoring groundwater along the proposed route of the Grey Lynn Tunnel and the two closest boreholes CIE-BH04 and CIE-BH05 located less than 10 and 29 m respectively from the shaft construction site have been outfitted with vibrating wire piezometers for high frequency data collection. CIE-BH01 and CIE-BH02 can be used as the monitoring boreholes 500 m from the shaft location.

2. To give effect to Recommendation 1, a monitoring program of at least three months in duration within boreholes CIE-BH04 and CIE-BH05 is recommended. Data shall be recorded to an accuracy of at least ±5 mm at an interval of no greater than one week during this time.

3. Groundwater monitoring records at CIE-BH04 and CIE-BH05 shall be collected from their respective vibrating wire piezometers and reviewed no less than weekly during shaft construction and no less than monthly for one year following shaft construction. Data records shall be compiled and submitted to Auckland Council Consents Manager.

4. In the event of land settlement reaching trigger levels defined in the Ground Settlement Report, the measured drawdown from the groundwater monitoring data should be compared to anticipated drawdown from the groundwater model. Any significant discrepancy shall be considered cause to review site management of groundwater pumping that is generating the drawdown.

5. After 12 months monitoring activities may cease in any borehole where water levels have recovered to within 2 m of pre-construction conditions. Monitoring activities shall continue if groundwater levels are not recovering from construction effects and there is a risk of adverse impacts related to dewatering.

6. Preparation of a Groundwater and Ground Settlement Monitoring and Contingency Plan that describes the monitoring suggested above, analysis of this data, and actions to be implemented should certain settlement outside of the anticipated range be triggered.

8 Summary and Conclusion

The Grey Lynn Tunnel is an infrastructure project being developed in Auckland to increase regional capacity for managing sewage flows and stormwater. The tunnel construction and subsequent operation and maintenance will require a shaft to be constructed on Tawariki Street in Grey Lynn.

A numerical groundwater flow model was developed to determine the potential impact of shaft construction on regional groundwater and estimate the rate of groundwater drainage into the shaft during and following construction. Regional geology around the shaft location is dominated by the ECBF formation which typically has permeability on the order of $3x10^{-7}$ m/s.

Site specific investigations found the geological and hydrogeological conditions to be typical for the area based on testing performed at six monitoring boreholes that were installed in preparation for shaft and tunnel construction. Three of these boreholes were located on Tawariki Street adjacent to the proposed shaft site.

Regional groundwater generally flows from higher elevation areas toward the Waitemata Harbour and the major surface drains are Motions Creek and Cox’s Creek.

Model Development and Calibration

A numerical groundwater model was developed using a MODFLOW unstructured grid with a 50 m grid spacing and enhanced resolution around the shaft location where grid spacing was reduced to under 0.5 m. The model was calibrated using water levels measured at the six monitoring boreholes. Accurate calibration of groundwater levels was achieved at four of the boreholes with a final calibrated hydraulic conductivity of $5x10^{-7}$ m/s.

Groundwater recharge in the model originates from rainfall. Based on calibrated model results 79% of groundwater outflows in the model area go to surface and subsurface drains with the remainder discharging into Waitemata Harbour.

Predictive Simulations and Results

A one-year simulation was run using calibrated parameters from the steady state model to establish baseline conditions (Scenario 1). Four transient simulations were then run which included the shaft being installed over a 13-day period at the beginning of the simulation. These scenarios simulated a range of construction alternatives by varying permeability of the shaft lining and testing an unlined shaft. The permeabilities tested were, no lining (Scenario 2), 10^{-8} m/s (Scenario 3), 10^{-9} m/s (Scenario 4), and 10^{-10} m/s (Scenario 5).

Two additional scenarios were devised where the shaft was lined to a depth of 7 m with a permeability equal to 10^{-9} m/s. In Scenario 6 model parameters were the same as for the other scenarios, whereas in Scenario 7 increased conductivity of the ECBF material was applied as a sensitivity test.

Scenario 4 was considered to be the most representative of long-term conditions while Scenario 6 was considered to represent the temporary conditions during the construction period prior to the installation of the full shaft lining. Scenario 2 (unlined shaft) was considered to be the most conservative scenario from the perspective of demonstrating an upper envelope of potential effects.

Drainage into the shaft was predicted peak during construction as the shaft was excavated below the pre-existing groundwater level and decline to a constant rate as groundwater conditions stabilized once the shaft was completed. In Scenario 2, with no lining, drainage into the shaft was predicted to peak at 32 m^3/day. This reduced to 30, 24, and 23 m^3/day in Scenarios 3, 4, and 5, respectively. Scenarios 6 and 7 where the shaft was only lined to 7m BGL were similar to Scenario 2 in terms of predicted drainage, though the peak occurred slightly later in the construction process after the excavation level had dropped below the liner. After construction drainage into the shaft dropped off significantly, approaching steady state in following weeks. The rate of steady state drainage into the shaft after construction is approximately 6 m^3/day in Scenario 2; 4 m^3/day in Scenario 3; and approximately 2 m^3/day in Scenario 4 and 5. Scenarios 6 and 7 were effectively equal to Scenario 2 in terms of the steady state drainage into the shaft.

The predicted impact on surface drainage was negligible. In the most extreme case, with an unlined shaft (Scenario 2), less than 0.01 L/s of flow reduction was predicted in Cox's Creek and no impact was predicted on Motions Creek.

Groundwater drawdown was significant directly around the shaft location, but widespread impact was not predicted. The greatest drawdown was predicted in Layer 4 of Scenario 2 where there was no shaft lining. In this case, 5.5 m of drawdown was predicted at 10 m from the shaft while 0.6 m was predicted 100 m from the shaft. Drawdown was significantly less in shallower model layers and below 0.2 m in Layer 1 which extends to -1 m AMSL, making damage to structures or other infrastructure unlikely.

In Scenario 2 measurable drawdown (5 cm) was predicted to extend to approximately 420 m from the shaft location. All scenarios where the shaft wall was lined yielded a lesser extent of drawdown and significantly lower maximum drawdown predictions. Drawdown was predicted to be under 0.5 m at 100 m distance from the shaft in scenarios 3 through 5. Drawdown was not predicted to extend to the coast in any scenario therefore shaft construction is not predicted to induce saline intrusion into the aquifer.

Model results indicate a less than a minor impact on regional groundwater. The following list of recommendations was developed based on the criteria for evaluating restricted discretionary activities outlined in the Auckland Unitary Plan and with consideration of model results.

The following is a list of recommendations based on model results and regional groundwater conditions:

1. The Tawariki Street Shaft shall be lined in the permanent case to minimize the risk of impacting local groundwater levels and inducing ground settlement using a material with a permeability of no greater than 1×10^{-8} m/s.
2. Monitoring existing boreholes CIE-BH04, CIE-BH05, and CIE-BH06 adjacent to the shaft as well monitoring the borehole CIE-BH01 or CIE-BH02, located along the proposed route of the Grey Lynn Tunnel approximately 500 m from the shaft to confirm that groundwater impacts are minimal, if any.
3. Weekly monitoring of groundwater levels at all boreholes installed for the Grey Lynn Tunnel project is recommended for a three month period prior to construction to document baseline conditions.
4. Weekly monitoring of all boreholes installed for the Grey Lynn Tunnel project in accordance with the Groundwater and Settlement Monitoring and Contingency Plan is recommended during shaft construction to alert managers if there is any change in groundwater level that may incur risk to structures or the environment.
5. Monthly groundwater monitoring in accordance with the Groundwater Monitoring and Contingency Plan is recommended for a one year period following construction to assure impacts are not beyond the expected levels and that groundwater levels recover to pre-construction conditions.

9 References

Auckland Unitary Plan (2016) Auckland Council.

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Appendix A. Borelogs

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Project: **Grey Lynn Tunnel CIGI5**

Borehole

Location: **29 Cockburn Street, Grey Lynn** Project No: **AE04725**

Hole ID: **CIE-BH01**

Client: **Watercare**

Date: **14/03/2018**

Started:	14/03/2018
Finished:	19/03/2018
Driller:	McMillan
Plant:	Rig N107 (McMillan)
Logged:	A. Coutts
Checked:	CS

Remarks
All hand vane results corrected, correction factor = 1.412
Packer Test at 17.00 - 21.50 m
Piezometer dipped 28/05/2018. Water level = 0.86m.
Hole location determined by Survey.

Co-ordinates:
5919643.48mN
1754735.36mE
Elevation: 13.68mRL
Inclination: -90°

Log cover page

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Version CI 1.10 09/07/2015 - R.Roberts

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Borehole

Project: **Grey Lynn Tunnel CIGI5**

Location: **29 Cockburn Street, Grey Lynn**   Project No: **AE04725**

Hole ID: **CIE-BH01**

Client: **Watercare**

Date: **14/03/2018**

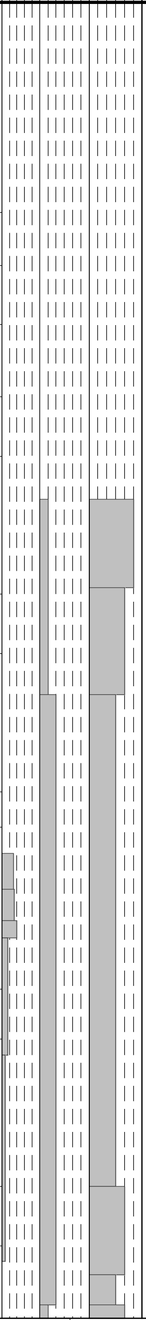
Columns: R.L. (m) | Depth (m) | *Shift Details & Standing Water Level* Drilling Method | Drilling Flush Return (%) 25 50 75 | TCR (SCR) [RQD] % | Spacing of Natural Defects (mm) 500 100 50 10 | Relative Strength EW VW W MS S VS | Weathering Grade UW SW MW HW CW RW | Sampling | In-Situ Testing | Geology Legend | GroundWater | Description of Strata | Defect Description | Comments | Geological Unit | Backfill / Installation

BOUNDARY
Well defined
Gradational
Poorly defined

TYPE		*SURFACE*		*APERTURE*	
CS	Clay seam	C	Clean	T	0mm
C	Clevage	Mc	Mineral coat	Vn	0-2mm
CR	Crushed zone	Si	Soil infill	N	2-6mm
DZ	Decomposed zone	Sn	Surface stain	Mn	6-20mm
DB	Drilling induced fracture	V	Veneer	Mw	20-60mm
FL	Foliation			W	60-200mm
FZ	Fracture zone	*PLANARITY*		Vw	>200mm
IF	Incipient fracture	P	Planar		
JT	Joint	St	Stepped		
SC	Schistosity	U	Undulating		
SH	Shear				
SZ	Shear Zone	*ROUGHNESS*			
SL	Sill	R	Rough		
VN	Vein	Ss	Slickensided		
VD	Void	Sm	Smooth		

R.L. (m): 13, 12, 11, 10, 9, 8, 7, 6, 5, 4
Depth (m): 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

Flush Type: **Water**
Flush Colour: **Grey**



Drilling Method	TCR (SCR) [RQD] %	In-Situ Testing	Description of Strata	Defect Description	Comments	Geological Unit
VAC EX	0		Vacuum Excavation: Infer fill material - gravelly SILT with some clay observed during vacuum excavation.			MG
13/03/2018 3:00:00 PM HQ3	100	I_{vp}40/I_{vr}13 (c)	SILT with some clay and minor gravel; dark greyish brown streaked dark brown. Firm, wet, high plasticity, moderately sensitive. Gravel is fine to coarse, subangular, basalt and sandstone.			
SPT	93	SPT_s 1,2,3 N=5	CLAY with some silt and trace gravel and rootlets; light grey streaked brownish orange. Firm, wet, high plasticity. Gravel is medium to coarse, subangular, basalt.			
HQ3	100	I_{vp}51/I_{vr}17 (c)	Interbedded silty fine SAND; yellowish orange, and silty CLAY; light grey streaked yellowish orange. Loose/Firm to stiff, moist, low plasticity. Beds are thin to moderately thin. *2.70m to 2.75m: Becomes wet.* *3.00m to 3.10m: Silty sand becomes brownish grey, wet.*			Tp
SPT	100	SPT_s 0,0,1 N=1	Silty fine SAND with minor clay and trace organics; dark grey mottled bluish black. Very loose, wet. Organics are amorphous. *3.72m to 3.78m: Becomes banded grey and yellowish orange. Laminated relict beds.*			
HQ3	57		Silty fine SAND with trace clay; grey. Medium dense, moist.			Wwc
		UTP_c	CORE LOSS.			
SPT	100	SPT_s 6,11,13 N=24	Interbedded clayey SILT; dark grey and silty fine SAND; dark grey. Stiff and medium dense, moist, clayey silt is non plastic. Beds are thin to moderately thin.			
HQ3	88 (61) [61]		Moderately weathered, dark grey, interbedded medium grained SANDSTONE and MUDSTONE. Very weak. Beds are laminated to moderately thin, steeply inclined.		5.36m: Core breaks along bedding planes on handling.	Wuw
SPT	0	SPT_c 29,50 N>50 50/120	CORE LOSS. *6.00m to 6.27m: Core loss due to solid cone SPT. Infer highly weathered mudstone and sandstone.*			
HQ3	100 (67) [62]	SPT_c 18,32,19 N>50 51/225	Moderately weathered, dark grey, interbedded medium grained SANDSTONE and MUDSTONE. Very weak. Beds are laminated to moderately thin, steeply inclined. *7.15m: Beds become moderately thin.*	6.47-6.56: Jt 65° R, P, Vn, C. 6.74-6.84: Jt 35° R, P, Mw, Si of silty fine to coarse gravel; sandstone. 6.98-7.04: Jt 35° Sm, P, Vn, Si of fine sand. 7.11-7.14: Jt 0° R, P, Mw, Si of silty fine to coarse gravel; sandstone.		
SPT	0		CORE LOSS - Solid cone SPT. Infer moderately weathered mudstone/sandstone.			
14/03/2018 5:00:00 PM HQ3 15/03/2018 7:45:00 AM(Water depth 2.7m)	94 (88) [81]		Moderately weathered, dark grey, interbedded medium grained SANDSTONE and MUDSTONE. Very weak. Beds moderately thin, steeply inclined. *8.50m: Very thin, steeply inclined, black carbonaceous bed.*	8.00-8.15: Jt 40° Sm, P, W, Si of fine to coarse gravel rock fragments.		
SPT	0	SPT_c 11,19,30 N=49	CORE LOSS - Solid cone SPT. Infer highly weathered mudstone and sandstone.			
			Highly weathered, dark grey, interbedded medium grained SANDSTONE and MUDSTONE. Very weak. Beds are moderately thin, steeply inclined. *9.67m to 9.90m: Becomes moderately weathered, SANDSTONE bed.*	9.57-9.61: Jt 40° Sm, P, Vn, Si of silty sand.		Wwnc

Started: 14/03/2018
Finished: 19/03/2018
Driller: McMillan
Plant: Rig N107 (McMillan)
Logged: A. Coutts
Checked: CS

Groundwater Observations

No.	Struck (m)	Date	Standing (m)	Observations

Remarks
All hand vane results corrected, correction factor = 1.412
Packer Test at 17.00 - 21.50 m
Piezometer dipped 28/05/2018. Water level = 0.86m.
Hole location determined by Survey.

Co-ordinates:
5919643.48mN
1754735.36mE
Elevation: 13.68mRL
Inclination: -90°

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Project: **Grey Lynn Tunnel CIGI5**

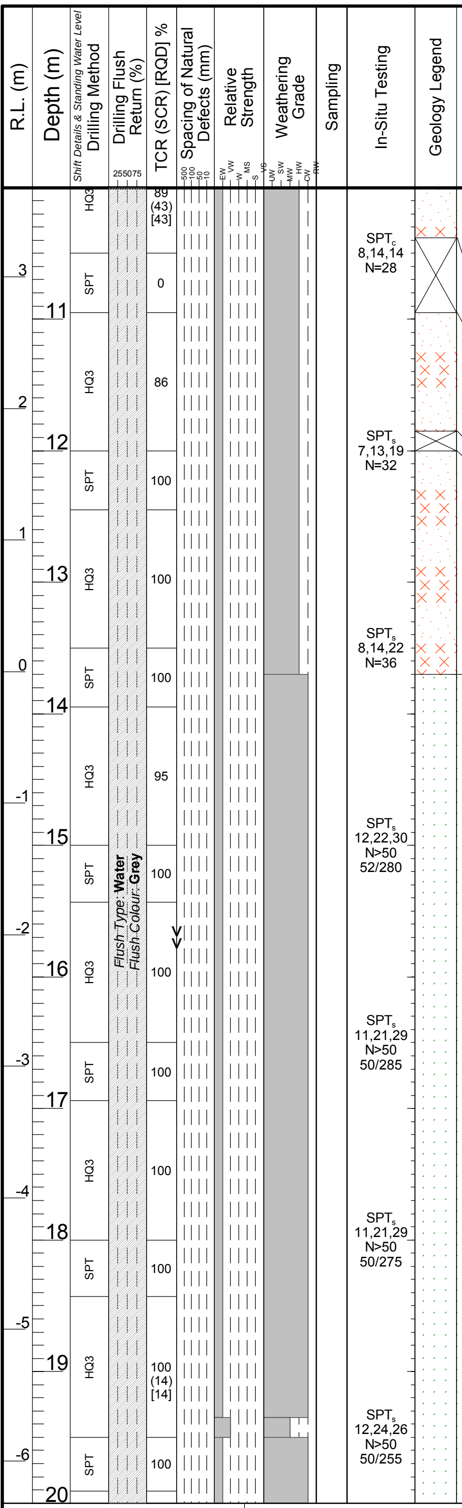
Borehole

Location: **29 Cockburn Street, Grey Lynn** Project No: **AE04725**

Hole ID: **CIE-BH01**

Client: **Watercare**

Date: **14/03/2018**



GroundWater

Description of Strata

BOUNDARY	
———	Well defined
— — —	Gradational
· · · · · ·	Poorly defined

Very weak.

Highly weathered, dark grey interbedded fine medium grained SANDSTONE and MUDSTONE. Extremely weak. Beds are thin to moderately thin, steeply inclined. Sandstone recovered as silty fine SAND; Medium dense, moist. Mudstone recovered as SILT with minor clay and sand; Firm, moist, low plasticity. Sand is fine.

CORE LOSS.

10.50m to 10.95m: Core loss due to solid cone SPT. Infer highly weathered interbedded mudstone and sandstone. Extremely weak.

Highly weathered, dark grey, interbedded fine grained SANDSTONE and MUDSTONE. Extremely weak. Sandstone beds are thin to moderately thick, steeply inclined. Mudstone beds are very thin to thin, steeply inclined. Sandstone recovered as fine to medium SAND with some silt; Medium dense, moist. Mudstone recovered as clayey SILT; Hard, moist.

CORE LOSS.

Highly weathered, dark grey, interbedded fine grained SANDSTONE and MUDSTONE. Extremely weak. Sandstone beds are thin to moderately thick, steeply inclined. Mudstone beds are very thin to thin, steeply inclined. Sandstone recovered as fine to medium SAND with some silt; Medium dense, moist. Mudstone recovered as clayey SILT; Hard, moist, silt has low plasticity.

12.22m to 12.26m: Laminated, steeply inclined, very dark brown carbonaceous bed.

Completely weathered, dark grey, massive, fine to medium grained SANDSTONE. Extremely weak. Recovered as fine to medium SAND with some silt. Dense, moist.

13.92m to 13.95m: Thin, steeply inclined, black carbonaceous bed.

14.42m: Thin, steeply inclined, black carbonaceous bed.

14.50m: Thin, steeply inclined, black carbonaceous bed.

14.70m: Becomes very dense.

15.97m: Very thin, steeply inclined, black carbonaceous bed.

16.94m to 16.97m: Becomes silty fine SAND with minor clay.

19.35m to 19.50m: Becomes moderately weathered, SANDSTONE. Very weak.

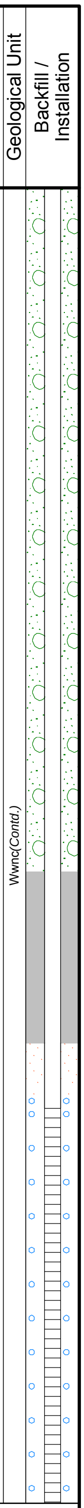
19.37m to 19.45m: Moderately thin, steeply inclined, grey speckled speckled black, discontinuous carbonaceous bed.

Defect Description

TYPE		***SURFACE***		***APERTURE***	
CS	Clay seam	C	Clean	T	0mm
C	Clevage	Mc	Mineral coat	Vn	0-2mm
CR	Crushed zone	Si	Soil infill	N	2-6mm
DZ	Decomposed zone	Sn	Surface stain	Mn	6-20mm
DB	Drilling induced fracture	V	Veneer	Mw	20-60mm
FL	Foliation			W	60-200mm
FZ	Fracture zone	***PLANARITY***		Vw	>200mm
IF	Incipient fracture	P	Planar		
JT	Joint	St	Stepped		
SC	Schistosity	U	Undulating		
SH	Shear				
SZ	Shear Zone	***ROUGHNESS***			
SL	Sill	R	Rough		
VN	Vein	Ss	Slickensided		
VD	Void	Sm	Smooth		

Comments

14.25m: Flush return turned black.



Started:	14/03/2018
Finished:	19/03/2018
Driller:	McMillan
Plant:	Rig N107 (McMillan)
Logged:	A. Coutts
Checked:	CS

Groundwater Observations

No.	*Struck (m)*	*Date*	*Standing (m)*	*Observations*

Remarks
All hand vane results corrected, correction factor = 1.412
Packer Test at 17.00 - 21.50 m
Piezometer dipped 28/05/2018. Water level = 0.86m.
Hole location determined by Survey.

Co-ordinates:
5919643.48mN
1754735.36mE
Elevation: 13.68mRL
Inclination: -90°

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Borehole

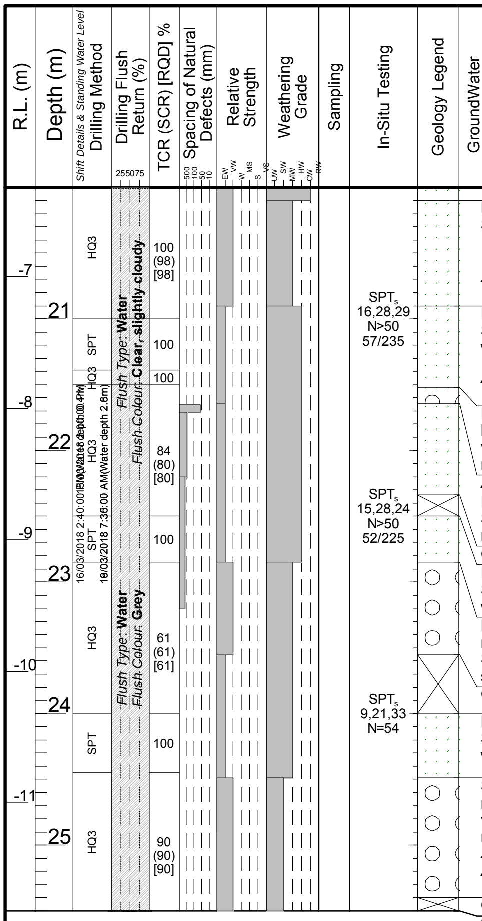
Project: **Grey Lynn Tunnel CIGI5**

Location: **29 Cockburn Street, Grey Lynn**   Project No: **AE04725**

Hole ID: **CIE-BH01**

Client: **Watercare**

Date: **14/03/2018**



Description of Strata

BOUNDARY
Well defined
Gradational
Poorly defined

Moderately weathered, dark grey, massive, fine to medium grained SANDSTONE. Very weak.

20.63m to 20.65m: Becomes silty fine SAND with minor clay.

Highly weathered, dark grey, massive, fine to medium grained SANDSTONE. Extremely weak. Recovered as fine to medium SAND with some silt. Very dense, moist.

21.39m to 21.42m: Becomes silty fine SAND with some clay; dark grey. Very dense, moist.

Highly weathered, grey speckled green, dark grey and trace reddish brown flecks, massive, fine to medium volcaniclastic SANDSTONE, very weak. With trace fine gravel sized, subrounded to subangular mudstone and sandstone clasts.

21.52m: Very thin, steeply inclined, black carbonaceous bed.

Highly weathered, dark grey, massive, fine to medium grained SANDSTONE. Extremely weak to very weak. Recovered as fine to medium SAND with some silt; Very dense, moist.

CORE LOSS.

Highly weathered, dark grey, massive, fine to medium grained SANDSTONE. Extremely weak. Recovered as fine to medium SAND with some silt; Very dense, moist.

Moderately weathered, grey speckled green, dark grey and trace reddish brown flecks, massive, medium to coarse volcaniclastic SANDSTONE. Very weak. With trace fine gravel sized, subrounded to subangular mudstone and sandstone clasts.

CORE LOSS.

Moderately weathered, dark grey, massive, fine to medium grained SANDSTONE. Extremely weak. Recovered as fine to medium SAND with some silt; Very dense, moist.

Slightly weathered, massive, grey speckled green, dark grey and trace reddish brown flecks, medium to coarse volcaniclastic SANDSTONE, very weak. With trace fine gravel sized, subrounded to subangular mudstone and sandstone clasts.

24.90m: Laminated, steeply inclined, black carbonaceous bed.

24.93m: Becomes fine to medium grained.

CORE LOSS.

CIE-BH01 terminated at 25.50m. Target Depth

Defect Description

TYPE		SURFACE		APERTURE	
CS	Clay seam	C	Clean	T	0mm
C	Clevage	Mc	Mineral coat	Vn	0-2mm
CR	Crushed zone	Si	Soil infill	N	2-6mm
DZ	Decomposed zone	Sn	Surface stain	Mn	6-20mm
DB	Drilling induced fracture	V	Veneer	Mw	20-60mm
FL	Foliation			W	60-200mm
FZ	Fracture zone	**PLANARITY**		Vw	>200mm
IF	Incipient fracture	P	Planar		
JT	Joint	St	Stepped		
SC	Schistosity	U	Undulating		
SH	Shear				
SZ	Shear Zone	**ROUGHNESS**			
SL	Sill	R	Rough		
VN	Vein	Ss	Slickensided		
VD	Void	Sm	Smooth		

21.65-21.68: Jt 40° Sm, P, N-Mn, Sn of silty sand.

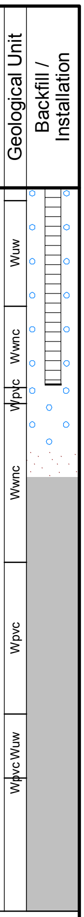
21.71-21.81: Jt 40° Sm, P, W, Si of clayey sand.

22.20-22.26: Jt 0° Sm, P, W, Si of clayey sand.

23.20: Jt 25° R, P, N, Si of clayey coarse sand.

Comments

21.39m: Short run for packer test.
21.5m: Pulled rods back to 16.5m for packer test.



Started:	14/03/2018
Finished:	19/03/2018
Driller:	McMillan
Plant:	Rig N107 (McMillan)
Logged:	A. Coutts
Checked:	CS

Groundwater Observations

No.	Struck (m)	Date	Standing (m)	Observations

Remarks
All hand vane results corrected, correction factor = 1.412
Packer Test at 17.00 - 21.50 m
Piezometer dipped 28/05/2018. Water level = 0.86m.
Hole location determined by Survey.

Co-ordinates:
5919643.48mN
1754735.36mE
Elevation: 13.68mRL
Inclination: -90°

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Project: **Grey Lynn Tunnel CIGI5**

Borehole

Location: **Hakanoa Reserve, Grey Lynn**
Project No: **AE04725**

Hole ID: **CIE-BH02**

Client: **Watercare**

Date: **19/03/2018**

Started: 19/03/2018
Finished: 22/03/2018
Driller: McMillan
Plant: Rig N107 (McMillan)
Logged: A. Coutts
Checked: CS

Remarks
All hand vane results corrected, correction factor = 1.412
Packer Test at 18.70-21.50 m
Artesian piezometer, low pressure gauge installed.
Pressure reading on 25/05/2018 was 20 kPa.
Hole location determined by Survey.

Co-ordinates:
5919692.99mN
1754644.56mE
Elevation: 10.11mRL
Inclination: -90°

Log cover page

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Data Template: AE04725 CI MASTER (NEW TEMPLATE).GPJ Output Form: BH COVER SHEET Project File Name: AE04725 CIGI4 ADDITIONAL INVESTIGATION.GPJ 7/9/18

Version CI 1.10 09/07/2015 - R.Roberts

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Project: **Grey Lynn Tunnel CIGI5**

Borehole

Location: **Hakanoa Reserve, Grey Lynn** Project No: **AE04725**

Hole ID: **CIE-BH02**

Client: **Watercare**

Date: **19/03/2018**

Depth (m)	Shift Details & Standing Water Level / Drilling Method	TCR (SCR) [RQD] %	Sampling	In-Situ Testing	Geological Unit
0 – 1.5	HA	100			MG (0 – 0.3) Tp
1.5 – 2.0	19/03/2018 3:30:00 PM HQ3 20/03/2018 8:00:00 AM	100			Tp
2.0 – 2.5	TBX	80	TW		Tp
2.5 – 3.0	HQ3	100		$I_{vp}59/I_{vr}14$	Tp
3.0 – 3.45	SPT	100		SPT_s 0,0,0 N<1	Tp
3.45 – 4.5	HQ3	43			Tp
4.5 – 4.95	SPT	100		SPT_s 0,0,3 N=3	Tp
4.95 – 6.0	HQ3	100			Wpvc
6.0 – 6.45	SPT	100		SPT_s 8,15,23 N=38	Wpvc
6.45 – 7.5	HQ3	95			Wpvc
7.5 – 7.95	SPT	100		SPT_s 12,24,26 N>50 50/250	Wpvc
7.95 – 9.0	HQ3	90			Wpvc
9.0 – 9.4	SPT	100		SPT_s 16,37,14 N>50 51/180	Wwhc Wpvc
9.4 – 10	HQ3	94			Wpvc

Drilling Flush Return (%): 25 50 75 — *Flush Type:* **Water**, *Flush Colour:* **Grey**

R.L. (m): 10 at 0 m to 0 at 10 m depth.

Description of Strata

BOUNDARY: Well defined; Gradational; Poorly defined

SILT with minor rootlets and clay; dark brown. Stiff, moist, low plasticity.

Silty CLAY with minor rootlets; brownish orange mottled dark orange and light greyish brown. Stiff, moist, low plasticity.

0.90m: Becomes light grey mottled dark orange.

1.20m: Trace rootlets.

1.40m: Becomes moderate plasticity.

PUSH TUBE: Material at top and bottom comprises: Silty CLAY; light grey mottled dark orange. Stiff, moist, low plasticity. 400mm recovered.

Silty CLAY with trace rootlets; light grey mottled dark orange. Firm, moist, high plasticity.

Silty CLAY with minor sand and trace rootlets; light greyish brown. Very soft, moist, high plasticity. Sand is fine.

3.45m: Minor organics. Brown mottled bluish black and yellowish brown. Organics are amorphous and fibrous decaying wood fragments.

CORE LOSS.

Silty CLAY with some organics and minor sand; dark greyish brown speckled black and light brown. Firm, wet, high plasticity. Organics are amorphous and fibrous decaying wood fragments. Sand is fine.

Silty fine to medium SAND with minor clay and trace fibrous decaying wood fragments; light grey. Loose, wet.

Highly weathered, grey speckled white, dark grey, green and trace reddish brown flecks, massive, fine to medium grained SANDSTONE. Extremely weak. Recovered as fine to medium SAND with some silt, trace gravel and clay; Medium dense, moist. Gravel is fine, subrounded mudstone and sandstone.

5.05m: Becomes medium dense, moist.

Moderately weathered, grey speckled white, dark grey, green and trace reddish brown flecks, massive, fine to medium grained SANDSTONE. Very weak. With trace gravel. Gravel is fine, subrounded mudstone and sandstone.

6.95m: Becomes very weak.

7.96m: Laminated, sub-horizontal, black carbonaceous bed.

CORE LOSS.

Highly weathered, grey speckled white, dark grey, green and trace reddish brown flecks, massive, fine to medium grained SANDSTONE. Extremely weak. Recovered as fine to medium SAND with some silt and trace gravel; Very dense, moist. Gravel is fine, subrounded mudstone and sandstone.

Moderately weathered, grey speckled white, dark grey, green and trace reddish brown flecks, massive, fine to medium grained SANDSTONE.

Defect Description

TYPE		*SURFACE*		*APERTURE*	
CS	Clay seam	C	Clean	T	0mm
C	Clevage	Mc	Mineral coat	Vn	0-2mm
CR	Crushed zone	Si	Soil infill	N	2-6mm
DZ	Decomposed zone	Sn	Surface stain	Mn	6-20mm
DB	Drilling induced fracture	V	Veneer	Mw	20-60mm
FL	Foliation			W	60-200mm
FZ	Fracture zone	*PLANARITY*		Vw	>200mm
IF	Incipient fracture	P	Planar		
JT	Joint	St	Stepped		
SC	Schistosity	U	Undulating		
SH	Shear				
SZ	Shear Zone	*ROUGHNESS*			
SL	Sill	R	Rough		
VN	Vein	Ss	Slickensided		
VD	Void	Sm	Smooth		

Comments

1.3m: Cased to 1.3 m.

3m: Difficulty with circulation.

3.45m: Drilled without water.

9.33m: Core oxidises to greenish

Started: 19/03/2018
Finished: 22/03/2018
Driller: McMillan
Plant: Rig N107 (McMillan)
Logged: A. Coutts
Checked: CS

Groundwater Observations

No.	Struck (m)	Date	Standing (m)	Observations

Remarks
All hand vane results corrected, correction factor = 1.412
Packer Test at 18.70-21.50 m
Artesian piezometer, low pressure gauge installed.
Pressure reading on 25/05/2018 was 20 kPa.
Hole location determined by Survey.

Co-ordinates:
5919692.99mN
1754644.56mE
Elevation: 10.11mRL
Inclination: -90°

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Data Template: AE04725 CI MASTER (NEW TEMPLATE).GPJ Output Form: COMPILATION BOREHOLE Project File Name: AE04725 CIGI4 ADDITIONAL INVESTIGATION.GPJ 7/9/18

Version CI 1.10 09/07/2015 - R.Roberts

Preliminary Log of Investigation

Project: **Grey Lynn Tunnel CIGI5**

Borehole

Location: **Hakanoa Reserve, Grey Lynn** Project No: **AE04725**

Hole ID: **CIE-BH02**

Client: **Watercare**

Date: **19/03/2018**

BOUNDARY: Well defined; Gradational; Poorly defined

TYPE	**SURFACE**	**APERTURE**
CS Clay seam	C Clean	T 0mm
C Clevage	Mc Mineral coat	Vn 0-2mm
CR Crushed zone	Si Soil infill	N 2-6mm
DZ Decomposed zone	Sn Surface stain	Mn 6-20mm
DB Drilling induced fracture	V Veneer	Mw 20-60mm
FL Foliation		W 60-200mm
FZ Fracture zone	**PLANARITY**	Vw >200mm
IF Incipient fracture	P Planar	
JT Joint	St Stepped	
SC Schistosity	U Undulating	
SH Shear		
SZ Shear Zone	**ROUGHNESS**	
SL Sill	R Rough	
VN Vein	Ss Slickensided	
VD Void	Sm Smooth	

Columns: R.L. (m) · Depth (m) · *Shift Details & Standing Water Level* Drilling Method · Drilling Flush Return (%) 25 50 75 · TCR (SCR) [RQD] % · Spacing of Natural Defects (mm) 500 100 50 10 · Relative Strength EW VW W MS S VS · Weathering Grade UW SW MW HW CW RW · Sampling · In-Situ Testing · Geology Legend · GroundWater · Description of Strata · Defect Description · Comments · Geological Unit · Backfill / Installation

*Flush Type: **Water*** · *Flush Colour: **Grey***

R.L. (m)	Depth (m)	Drilling Method	TCR (SCR) [RQD] %	In-Situ Testing
0			(92) [92]	
		SPT	0	SPT$_c$ 36 N>50 36/150 bouncing
-1	11	HQ3	87 (87) [87]	
-2	12	SPT	0	SPT$_c$ 48 N>50 48/150 bouncing
-3	13	HQ3	74 (74) [74]	
-4	14	HQ3	83 (83) [83]	
-5	15			
-6	16	HQ3	93 (87) [87]	
-7	17	HQ3	97	
-8	18			
-9	19	HQ3	90	
	20			

Description of Strata	Defect Description	Comments	Geological Unit
Very weak. Base of unit is steeply inclined. *9.42m: Laminated, steeply inclined, black carbonaceous bed.*		grey.	Wpvc(Contd.)
Moderately weathered, grey speckled white, dark grey, green and trace reddish brown flecks, massive, coarse volcaniclastic SANDSTONE. Very weak. With trace fine to medium gravel, subrounded mudstone and sandstone.	10.30-10.39: Jt 65° Sm, P, T-Vn, Si of clay.		
Slightly weathered, grey speckled white, dark grey, green and trace reddish brown flecks, massive, fine to medium grained SANDSTONE. Very weak.			
CORE LOSS - Solid cone SPT. Infer highly weathered sandstone.			
Slightly weathered, grey speckled white, dark grey, green and trace reddish brown flecks, massive, fine to medium grained SANDSTONE. Very weak. *10.86m to 11.22m: Becomes coarse grained.* *11.56m to 11.58m: Some white, subrounded fine gravel grains.*			
CORE LOSS. *12.00m to 12.15m: Core loss due to solid cone SPT. Infer highly weathered sandstone.*			
Slightly weathered, grey speckled white, dark grey, green and trace reddish brown flecks, massive, fine to medium grained SANDSTONE. Very weak.		12.6m: Core biscuiting with drilling.	
Slightly weathered, grey speckled white, dark grey, green and trace reddish brown flecks, massive, coarse volcaniclastic SANDSTONE. Very weak. With trace fine gravel, subrounded mudstone and sandstone.			Wwnc
Highly weathered, grey, massive, fine to medium grained SANDSTONE. Very weak.			
CORE LOSS.			Wuw
Slightly weathered, grey, massive, fine to medium grained SANDSTONE. Very weak.			Wpvc
Slightly weathered, grey speckled white, dark grey, green and trace reddish brown flecks, massive, coarse volcaniclastic SANDSTONE. Very weak. With trace fine gravel, subrounded mudstone and sandstone.			
Moderately weathered, grey, interbedded, fine grained SANDSTONE and MUDSTONE. Very weak. Beds are very thin to thin, sub-horizontal. *14.05m to 14.60m: Moderately thick, sandstone bed.* *14.46m: Laminated, sub-horizontal, black carbonaceous bed.*			
CORE LOSS.			
Slightly weathered, grey, massive, fine to medium grained SANDSTONE. Very weak. *15.54m to 15.70m: Becomes extremely weak. Recovered as fine to medium SAND with some silt; Very dense, moist.* *15.76m to 15.95m: Moderately thin, sub-horizontal, grey speckled black carbonaceous bed.* *16.12m to 16.17m: Thin, sub-horizontal, grey speckled black, discontinuous carbonaceous bed.*			Wuw
CORE LOSS.			
Silty SAND with minor clay; dark grey. Very dense, wet.			
Slightly weathered, grey, interbedded, fine to medium grained SANDSTONE and MUDSTONE. Extremely weak. Beds are very thin to moderately thin, sub-horizontal. Sandstone recovered as fine to medium SAND with minor silt; Very dense, Mudstone recovered as CLAY; Hard.			
Slightly weathered, massive, grey speckled white, dark grey, green and trace reddish brown flecks, fine to medium grained SANDSTONE. Extremely weak to very weak. Recovered as fine to medium SAND; Very dense. *18.23m: Laminated, sub-horizontal, black carbonaceous bed.* *18.32m to 18.40m: Becomes coarse grained. Trace coarse gravel, subrounded mudstone.* *18.33m: Laminated, sub-horizontal, black carbonaceous bed.* *18.68m to 18.77m: Moderately thin, sub-horizontal, grey speckled black, discontinuous carbonaceous bed.*			Wpvc
CORE LOSS.			
Slightly weathered, grey speckled white, dark grey, green and trace reddish brown flecks, massive fine to medium grained SANDSTONE. Very weak. With minor coarse sand, and trace fine gravel, subrounded, mudstone.			

Started: 19/03/2018
Finished: 22/03/2018
Driller: McMillan
Plant: Rig N107 (McMillan)
Logged: A. Coutts
Checked: CS

Groundwater Observations

No.	Struck (m)	Date	Standing (m)	Observations

Remarks
All hand vane results corrected, correction factor = 1.412
Packer Test at 18.70-21.50 m
Artesian piezometer, low pressure gauge installed.
Pressure reading on 25/05/2018 was 20 kPa.
Hole location determined by Survey.

Co-ordinates:
5919692.99mN
1754644.56mE
Elevation: 10.11mRL
Inclination: -90°

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Borehole

Project: **Grey Lynn Tunnel CIGI5**

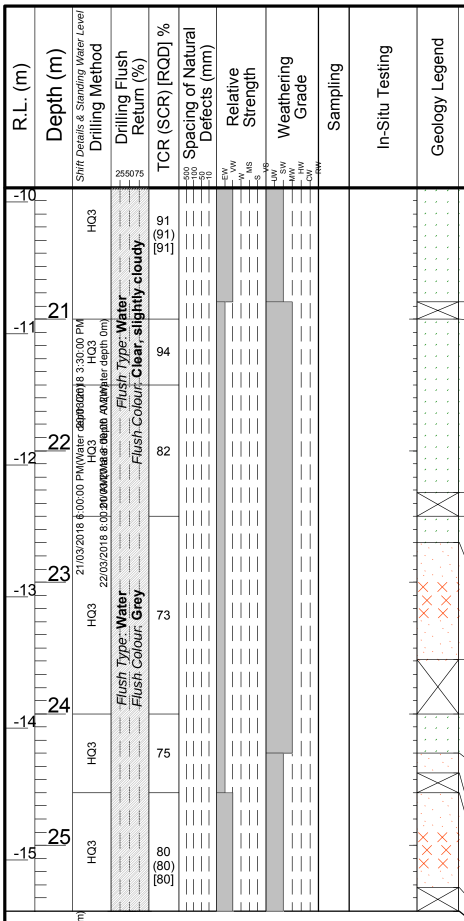
Location: **Hakanoa Reserve, Grey Lynn**

Project No: **AE04725**

Hole ID: **CIE-BH02**

Client: **Watercare**

Date: **19/03/2018**



GroundWater

Description of Strata

BOUNDARY
Well defined
Gradational
Poorly defined

CORE LOSS.

Moderately weathered, grey speckled white, dark grey, green and trace reddish brown flecks, massive, fine to medium grained SANDSTONE. Extremely weak. Recovered as fine to medium SAND with trace coarse sand and fine gravel; Very dense.
21.10m to 21.30m: Very thin, sub-horizontal, grey speckled black carbonaceous bed.
21.50m to 22.32m: Becomes slightly weathered.

CORE LOSS - infer sand washed away from drilling.

Moderately weathered, grey speckled white, dark grey, green and trace reddish brown flecks, massive, fine to medium grained SANDSTONE. Extremely weak. Recovered as fine to medium SAND with trace coarse sand and fine gravel; Very dense.

Moderately weathered, grey, interbedded, fine to medium grained SANDSTONE and MUDSTONE. Extremely weak. Beds are very thin to moderately thin, gently inclined. Sandstone recovered as silty fine to medium SAND; Very dense. Mudstone recovered as CLAY; Hard.

CORE LOSS - infer sand washed away from drilling.

Moderately weathered, grey, massive, fine to medium grained SANDSTONE. Extremely weak. Recovered as fine to medium SAND with some silt; very dense.

Slightly weathered, grey, interbedded medium grained SANDSTONE and MUDSTONE. Extremely weak. Beds are very thin, gently inclined. Sandstone recovered as silty fine to medium SAND; Very dense. Mudstone recovered as CLAY; Hard.

CORE LOSS.

Slightly weathered, grey, interbedded, medium grained SANDSTONE and MUDSTONE. Very weak. Beds are very thin to moderately thin, sub-horizontal.

CORE LOSS.

CIE-BH02 terminated at 25.50m. Target Depth

Defect Description

TYPE		*SURFACE*		*APERTURE*	
CS	Clay seam	C	Clean	T	0mm
C	Clevage	Mc	Mineral coat	Vn	0-2mm
CR	Crushed zone	Si	Soil infill	N	2-6mm
DZ	Decomposed zone	Sn	Surface stain	Mn	6-20mm
DB	Drilling induced fracture	V	Veneer	Mw	20-60mm
FL	Foliation			W	60-200mm
FZ	Fracture zone	*PLANARITY*		Vw	>200mm
IF	Incipient fracture	P	Planar		
JT	Joint	St	Stepped		
SC	Schistosity	U	Undulating		
SH	Shear				
SZ	Shear Zone	*ROUGHNESS*			
SL	Sill	R	Rough		
VN	Vein	Ss	Slickensided		
VD	Void	Sm	Smooth		

Comments

21.5m: Core biscuiting with drilling.

24m: Short run to improve recovery.

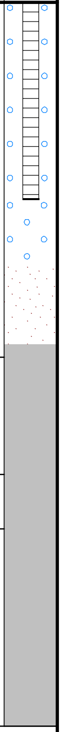
Geological Unit

Wpvc(Contd.)

Wuw

WuwWpvc

Backfill / Installation



Started:	19/03/2018
Finished:	22/03/2018
Driller:	McMillan
Plant:	Rig N107 (McMillan)
Logged:	A. Coutts
Checked:	CS

Groundwater Observations

No.	*Struck (m)*	*Date*	*Standing (m)*	*Observations*

Remarks
All hand vane results corrected, correction factor = 1.412
Packer Test at 18.70-21.50 m
Artesian piezometer, low pressure gauge installed.
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Hole location determined by Survey.

Co-ordinates:
5919692.99mN
1754644.56mE
Elevation: 10.11mRL
Inclination: -90°

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Project: **Grey Lynn Tunnel CIGI5**

Borehole

Location: **41 Tawariki Street, Grey Lynn**

Project No: **AE04725**

Hole ID: **CIE-BH03**

Client: **Watercare**

Date: **23/03/2018**

Started: 23/03/2018
Finished: 27/03/2018
Driller: McMillan
Plant: Rig N101 (McMillan)
Logged: A. Coutts
Checked: CS

Remarks
Packer Test at 20.00-24.50 m
Artesian piezometer, low pressure gauge installed..
Pressure reading on 25/05/2018 was 21 kPa.
Hole location determined by Survey.

Co-ordinates:
5920068.77mN
1754833.35mE
Elevation: 13.34mRL
Inclination: -90°

Log cover page

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Project: **Grey Lynn Tunnel CIGI5**

Borehole

Location: **41 Tawariki Street, Grey Lynn** Project No: **AE04725**

Hole ID: **CIE-BH03**

Client: **Watercare**

Date: **23/03/2018**

R.L. (m)	Depth (m)	Shift Details & Standing Water Level Drilling Method	Drilling Flush Return (%)	TCR (SCR) [RQD] %	Sampling	In-Situ Testing
13		VAC EX		0		
12	1					
		HQ3 23/03/2018 8:00:00 AM	Flush Type: **Water** Flush Colour: **Grey**	100		
11	2	TBX		70	TW	
		HQ3		100		SPT$_s$ 3,7,13 N=20
10	3	SPT		100	TW	
		TBX		100	TW	
9	4	HQ3		100		SPT$_s$ 13,14,33 N=47
		SPT		78		
8	5	HQ3		81 (74) [74]		SPT$_s$ 19,40 N>50 40/150 bouncing
7	6	SPT		100		
		HQ3		92 (92) [92]		SPT$_c$ 34,33 N>50 33/75 bouncing
6	7	SPT		0		
5	8	HQ3		75 (75) [75]		SPT$_c$ 44 N>50 44/150 bouncing
4	9	SPT (Water depth 1.8m)		0		
	10	23/03/2018 3:00:00 PM HQ3 26/03/2018 9:00:00 AM		100 (100)		

Drilling Flush Return (%): 25 50 75
Spacing of Natural Defects (mm): 500 100 50 10
Relative Strength: EW VW W MS S VS
Weathering Grade: UW SW MW HW CW RW

Description of Strata

BOUNDARY
Well defined
Gradational
Poorly defined

Vacuum Excavation.

Silty CLAY with minor rootlets; light grey mottled orange and dark brown. Very soft, saturated, high plasticity.
1.70m: Becomes minor sand. Sand is fine.
1.90m: Becomes dark grey.

PUSH TUBE: Material at top and bottom comprises: CLAY with minor silt and trace fine sand; dark grey mottled light brownish grey. Soft, saturated, low plasticity.

CORE LOSS.

CLAY with some silt; dark grey mottled light brownish grey. Soft, wet, high plasticity.

SILT with minor clay and trace sand and rootlets; dark grey mottled orange. Very stiff, moist, low plasticity. Sand is fine.

PUSH TUBE: Material at top is too deep in tube to obtain sample. Material at base is: Sandy SILT; dark grey. Hard, moist. Sand is fine.

Highly weathered, dark grey, massive, fine grained SANDSTONE. Extremely weak. Recovered as fine SAND with some silt; Dense, moist.

4.48m to 4.50m: Becomes silty CLAY. Hard, moist, low plasticity.

4.70m to 4.77m: Becomes SILT. Hard, moist, low plasticity.

CORE LOSS.

Highly weathered, dark grey, massive, fine grained SANDSTONE. Extremely weak. Recovered as fine SAND with some silt; Dense, wet.

Moderately weathered, interbedded, grey MUDSTONE and grey speckled white, green with trace red flecks SANDSTONE. Very weak. Mudstone beds are laminated to thin, sandstone beds are thin to moderately thin, sub-horizontal. With minor laminated to thin carbonaceous beds.
5.15m to 5.00m: Thin, sub-horizontal, grey speckled black, discontinuous carbonaceous bed.

Moderately weathered, grey speckled white, green with trace red flecks, massive, medium grained SANDSTONE. Very weak. With trace coarse sand and fine gravel, subrounded, mudstone.

CORE LOSS.

Moderately weathered, grey speckled white, green with trace red flecks, massive, medium grained SANDSTONE. Very weak. With trace coarse sand and fine gravel, subrounded, mudstone.
6.30m: Becomes slightly weathered.
6.40m to 6.50m: Very thin, moderately inclined carbonaceous bed.

Slightly weathered, grey, massive, fine grained SANDSTONE. Very weak.

Slightly weathered, grey, medium grained SANDSTONE. Extremely weak. Recovered as fine to medium SAND with some silt; Very dense.

CORE LOSS.

Slightly weathered, grey speckled white, green with trace red flecks, massive, medium grained SANDSTONE. Very weak. With minor coarse sand grains and trace fine gravel, subrounded, mudstone.

CORE LOSS.

Slightly weathered, grey speckled white, green with trace red flecks, medium grained SANDSTONE. Very weak. With minor coarse sand grains and trace fine gravel, subrounded, mudstone.

Slightly weathered, grey speckled white and green with red flecks, massive, fine to coarse volcaniclastic SANDSTONE, very weak. With trace fine gravel sized, subrounded to subangular mudstone and

Defect Description

TYPE		SURFACE		APERTURE	
CS	Clay seam	C	Clean	T	0mm
C	Clevage	Mc	Mineral coat	Vn	0-2mm
CR	Crushed zone	Si	Soil infill	N	2-6mm
DZ	Decomposed zone	Sn	Surface stain	Mn	6-20mm
DB	Drilling induced fracture	V	Veneer	Mw	20-60mm
FL	Foliation			W	60-200mm
FZ	Fracture zone	PLANARITY		Vw	>200mm
IF	Incipient fracture	P	Planar		
JT	Joint	St	Stepped		
SC	Schistosity	U	Undulating		
SH	Shear				
SZ	Shear Zone	ROUGHNESS			
SL	Sill	R	Rough		
VN	Vein	Ss	Slickensided		
VD	Void	Sm	Smooth		

Comments

2.7m: Rods sinking under own weight.
3.45m: Push tube could only be pushed 250mm, too hard.

6.3m: Core biscuiting with drilling.

Geological Unit

MG
Tp
Wwc
Wwmc
Wuw
Wpvc
Wuw
Wpvc
Wuw
Wpvc

Backfill / Installation

Started: 23/03/2018
Finished: 27/03/2018
Driller: McMillan
Plant: Rig N101 (McMillan)
Logged: A. Coutts
Checked: CS

Groundwater Observations

No.	Struck (m)	Date	Standing (m)	Observations

Remarks
Packer Test at 20.00-24.50 m
Artesian piezometer, low pressure gauge installed..
Pressure reading on 25/05/2018 was 21 kPa.
Hole location determined by Survey.

Co-ordinates:
5920068.77mN
1754833.35mE
Elevation: 13.34mRL
Inclination: -90°

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Project: **Grey Lynn Tunnel CIGI5**

Borehole

Location: **41 Tawariki Street, Grey Lynn** Project No: **AE04725**

Hole ID: **CIE-BH03**

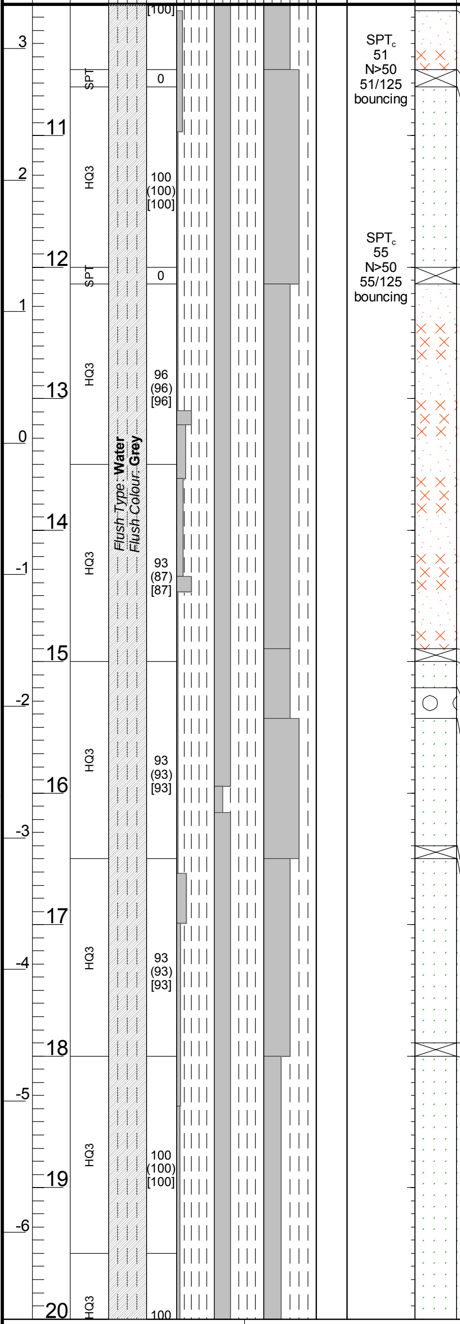
Client: **Watercare**

Date: **23/03/2018**

R.L. (m)	Depth (m)	Shift Details & Standing Water Level / Drilling Method	Drilling Flush Return (%) 25 50 75	TCR (SCR) [RQD] %	Spacing of Natural Defects (mm) 500 100 50 10	Relative Strength EW VW W MS S VS	Weathering Grade RS CW HW MW SW UW	Sampling	In-Situ Testing	Geology Legend	GroundWater	Description of Strata	Defect Description	Comments	Geological Unit	Backfill / Installation

BOUNDARY
- ——— Well defined
- — — Gradational
- · · · · Poorly defined

TYPE		SURFACE		APERTURE	
CS	Clay seam	C	Clean	T	0mm
C	Clevage	Mc	Mineral coat	Vn	0-2mm
CR	Crushed zone	Si	Soil infill	N	2-6mm
DZ	Decomposed zone	Sn	Surface stain	Mn	6-20mm
DB	Drilling induced fracture	V	Veneer	Mw	20-60mm
FL	Foliation			W	60-200mm
FZ	Fracture zone	**PLANARITY**		Vw	>200mm
IF	Incipient fracture	P	Planar		
JT	Joint	St	Stepped		
SC	Schistosity	U	Undulating		
SH	Shear				
SZ	Shear Zone	**ROUGHNESS**			
SL	Sill	R	Rough		
VN	Vein	Ss	Slickensided		
VD	Void	Sm	Smooth		



Drilling Method	TCR (SCR) [RQD] %	In-Situ Testing
	[100]	
SPT	0	SPT_c 51 N>50 51/125 bouncing
HQ3	100 (100) [100]	
SPT	0	SPT_c 55 N>50 55/125 bouncing
HQ3	96 (96) [96]	
HQ3	93 (87) [87]	
HQ3	93 (93) [93]	
HQ3	93 (93) [93]	
HQ3	100 (100) [100]	
HQ3	100	

Description of Strata

sandstone clasts.

Moderately weathered, grey, interbedded, fine grained SANDSTONE and MUDSTONE . Very weak. Sandstone beds are thin to moderately thin, gently inclined. Mudstone beds are laminated to thin, gently inclined.

10.05m to 10.15m: Moderately thin, sub-horizontal, grey speckled black, discontinuous carbonaceous bed.

CORE LOSS.

Slighlty weathered, grey, bedded, medium grained SANDSTONE. Very weak. Beds are thin to moderately thin, gently inclined. With trace laminated, gently inclined, black carbonaceous beds.

CORE LOSS.

Slightly weathered, grey, interbedded, medium grained SANDSTONE and MUDSTONE. Very weak. Sandstone beds are thin to moderately thin, gently inclined. Mudstone beds are laminated to thin, gently inclined.

13.92m: Very thin, gently inclined, black carbonaceous bed.

14.67m: Laminated, sub-horizontal, grey speckled black, discontinuous carbonaceous bed.

CORE LOSS.

Slightly weathered, grey speckled white, green with trace red flecks, massive, medium grained SANDSTONE. Very weak. With minor coarse sand grains and trace fine gravel, subrounded, mudstone.

Moderately weathered, grey speckled white and green with red flecks, massive, fine to coarse volcaniclastic SANDSTONE, very weak. With trace fine gravel sized, subrounded to subangular mudstone and sandstone clasts.

15.32m to 15.52m: Coarse gravel sized, subrounded mudstone clasts.

Moderately weathered, grey speckled white, green with trace red flecks, massive, medium grained SANDSTONE. Very weak. With minor coarse sand grains and trace fine gravel, subrounded, mudstone.

15.49m: Very thin, gently inclined, black carbonaceous bed.

15.95m to 16.15m: Becomes extremely weak. Recovered as fine to medium sand; Very dense.

16.20m: Very thin, moderately inclined carbonaceous bed.

CORE LOSS.

Highly weathered, grey speckled white and green with red flecks, massive, medium grained SANDSTONE, very weak. With trace fine gravel sized, subrounded to subangular mudstone and sandstone clasts and trace fine to medium gavel sized carbonaceous clasts.

CORE LOSS.

Slightly weathered, grey speckled white and green with red flecks, massive, fine to medium grained SANDSTONE, very weak. With trace fine to medium gravel sized, subrounded to subangular mudstone and sandstone clasts.

18.47m to 18.54m: Moderately thin, sub-horizontal, grey banded and speckled black, discontinuous carbonaceous bed.

18.90m to 19.00m: Becomes fine grained with carbonaceous clasts.

19.56m: Very thin, gently inclined, black carbonaceous bed.

Defect Description

- 10.05: Jt 15° R, P, Vn, C.
- 10.97: Jt 5° Sm, P, T-Vn, C.
- 13.09: Jt 10° R, U, Vn, C.
- 13.20: Jt 5° Sm, P, T-Vn, C.
- 13.61: Jt 0° Sm, P, N, Si of clay.
- 14.35-14.45: Jt 75° Sm, U, Vn, C.
- 14.47-14.51: Jt 75° Sm, St, Vn, C.
- 16.61: Jt 5° Sm, U, Vn, C.
- 16.99-17.03: Jt 85° Sm, P, T-Vn, C.
- 18.38: Jt 0° Sm, P, N, Si of clay.

Comments

10.63m: Very closely spaced to closely spaced driling induced fractures.

Geological Unit

Wuw

Wpvc

Started:	23/03/2018	Groundwater Observations	Co-ordinates:
Finished:	27/03/2018	*No. Struck (m) Date Standing (m) Observations*	5920068.77mN
Driller:	McMillan		1754833.35mE
Plant:	Rig N101 (McMillan)		Elevation: 13.34mRL
Logged:	A. Coutts		Inclination: -90°
Checked:	CS		

Remarks
Packer Test at 20.00-24.50 m
Artesian piezometer, low pressure gauge installed..
Pressure reading on 25/05/2018 was 21 kPa.
Hole location determined by Survey.

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Data Template: AE04725 CI MASTER (NEW TEMPLATE).GPJ Output Form: COMPILATION BOREHOLE Project File Name: AE04725 CIGI4 ADDITIONAL INVESTIGATION.GPJ 7/9/18

Version CI 1.10 09/07/2015 - R.Roberts

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Project: **Grey Lynn Tunnel CIGI5**

Borehole

Location: **41 Tawariki Street, Grey Lynn** Project No: **AE04725**

Hole ID: **CIE-BH03**

Client: **Watercare**

Date: **23/03/2018**

R.L. (m)	Depth (m)	Shift Details & Standing Water Level / Drilling Method	Drilling Flush Return (%)	TCR (SCR) [RQD] %
-7				100 (100) [100]
	21	HQ3		90 (70) [70]
-8	22	HQ3		94 (94) [94]
-9	23	HQ3	*Flush Type:* **Water** *Flush Colour:* **Clear, slightly cloudy**	100 (53) [53]
-10	24	26/03/2018 4:00:00 PM (Water depth 0.9m) HQ3		42 (26) [26]
-11	25	27/03/2018 8:00:00 AM(Water depth 0.9m) HQ3	*Flush Type:* **Water** *Flush Colour:* **Grey**	92 (92) [92]
-12	26			
-13	27	HQ3		100 (100) [100]
-14		27/03/2018 2:00:00 PM(Water depth 0.9m) 28/03/2018 9:15:00 AM(Water depth 0.7m)		

Drilling Flush Return (%): 25 50 75

Spacing of Natural Defects (mm): 500 100 50 10

Relative Strength: EW VW W MS S VS

Weathering Grade: UW SW MW HW CW RW

Sampling

In-Situ Testing

Geology Legend

GroundWater

Description of Strata

BOUNDARY
Well defined
Gradational
Poorly defined

CORE LOSS.

Slightly weathered, grey speckled white and green with red flecks, massive, fine to medium grained SANDSTONE, very weak. With trace fine to medium gravel sized, subrounded to subangular mudstone and sandstone clasts.

CORE LOSS.

Slightly weathered, grey speckled white and green with red flecks, massive, medium grained SANDSTONE, very weak. With trace fine to medium gravel sized, subrounded to subangular mudstone and sandstone clasts.

23.30m to 24.00m: Becomes extremely weak. Recovered as fine to medium SAND with minor silt; Very dense.

CORE LOSS. Infer sandstone broke and washed away while trying to recover run.

Slightly weathered, grey speckled white and green with red flecks, massive, fine to coarse grained SANDSTONE, very weak. With trace fine gravel sized, subrounded mudstone and sandstone clasts.

Slightly weathered, interbedded, medium grey speckled white, green and flecks of red, medium grained SANDSTONE and dark grey MUDSTONE . Very weak. Sandstone beds are thin to moderately thin, mudstone beds are laminated to thin, gently inclined.
25.30m: Very thin, sub-horizontal, black carbonaceous bed.
25.63m to 25.88m: Beds become steeply inclined.

CORE LOSS.

Slightly weathered, grey, massive, medium grained SANDSTONE. Very weak.

Slightly weathered, grey speckled white, green and flecks of red, massive, medium grained SANDSTONE. Very weak.

CIE-BH03 terminated at 27.50m. Target Depth

Defect Description

TYPE		*SURFACE*		*APERTURE*	
CS	Clay seam	C	Clean	T	0mm
C	Clevage	Mc	Mineral coat	Vn	0-2mm
CR	Crushed zone	Si	Soil infill	N	2-6mm
DZ	Decomposed zone	Sn	Surface stain	Mn	6-20mm
DB	Drilling induced fracture	V	Veneer	Mw	20-60mm
FL	Foliation			W	60-200mm
FZ	Fracture zone	*PLANARITY*		Vw	>200mm
IF	Incipient fracture	P	Planar		
JT	Joint	St	Stepped		
SC	Schistosity	U	Undulating		
SH	Shear				
SZ	Shear Zone	*ROUGHNESS*			
SL	Sill	R	Rough		
VN	Vein	Ss	Slickensided		
VD	Void	Sm	Smooth		

20.08: Jt 0° R, P, Vn, C.

20.43-20.53: Jt 85° R, P, Vn, C.
20.55-20.61: Jt 85° R, P, Vn, C.

21.14-21.19: Jt 0° R, P, Mw, Si of rock fragments.
21.22: Jt 0° R, P, Mn, Si of clay.

Comments

24m: Difficulty recovering run. Third attempted recovered with fingered catcher.
24.47m: Dipped hole for packer test. Replaced mud pit with fresh water.

Geological Unit: Wpvc(Contd.) / Wuw / Wpvc

Backfill / Installation

Started:	23/03/2018
Finished:	27/03/2018
Driller:	McMillan
Plant:	Rig N101 (McMillan)
Logged:	A. Coutts
Checked:	CS

Groundwater Observations

No.	Struck (m)	Date	Standing (m)	Observations

Remarks
Packer Test at 20.00-24.50 m
Artesian piezometer, low pressure gauge installed..
Pressure reading on 25/05/2018 was 21 kPa.
Hole location determined by Survey.

Co-ordinates:
5920068.77mN
1754833.35mE
Elevation: 13.34mRL
Inclination: -90°

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Data Template: AE04725 CI MASTER (NEW TEMPLATE).GPJ Output Form: COMPILATION BOREHOLE Project File Name: AE04725 CIGI4 ADDITIONAL INVESTIGATION.GPJ 7/9/18

Version CI 1.10 09/07/2015 - R.Roberts

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Project: **Grey Lynn Tunnel CIGI5**

Borehole

Location: **46 Tawariki Street, Grey Lynn**

Project No: **AE04725**

Hole ID: **CIE-BH04**

Client: **Watercare**

Date: **5/07/2018**

Started: 5/07/2018
Finished: 10/07/2018
Driller: McMillan
Plant: Rig N111 (McMillan)
Logged: S. Burgess
Checked: LD

Remarks
Packer Test 1 at 9.75-12.00 m, Packer Test 2 at 19.25-22.50 m, Packer Test 3 at 28.50-31.50 m. Vibrating wire piezometer installed with sensor at 26.0m. Water level = 16.1 m RL.
Joint angles are relative to the core axis. If a borehole is true vertical; horizontal=90, vertical=0.
Hole location is in NZTM projection. Elevation is relative to Auckland Vertical Datum 1946
Hole location determined by Survey.

Co-ordinates:
5920092.60mN
1754813.92mE
Elevation: 12.29mRL
Inclination: -90°

Log cover page

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Project: **Grey Lynn Tunnel CIGI5**

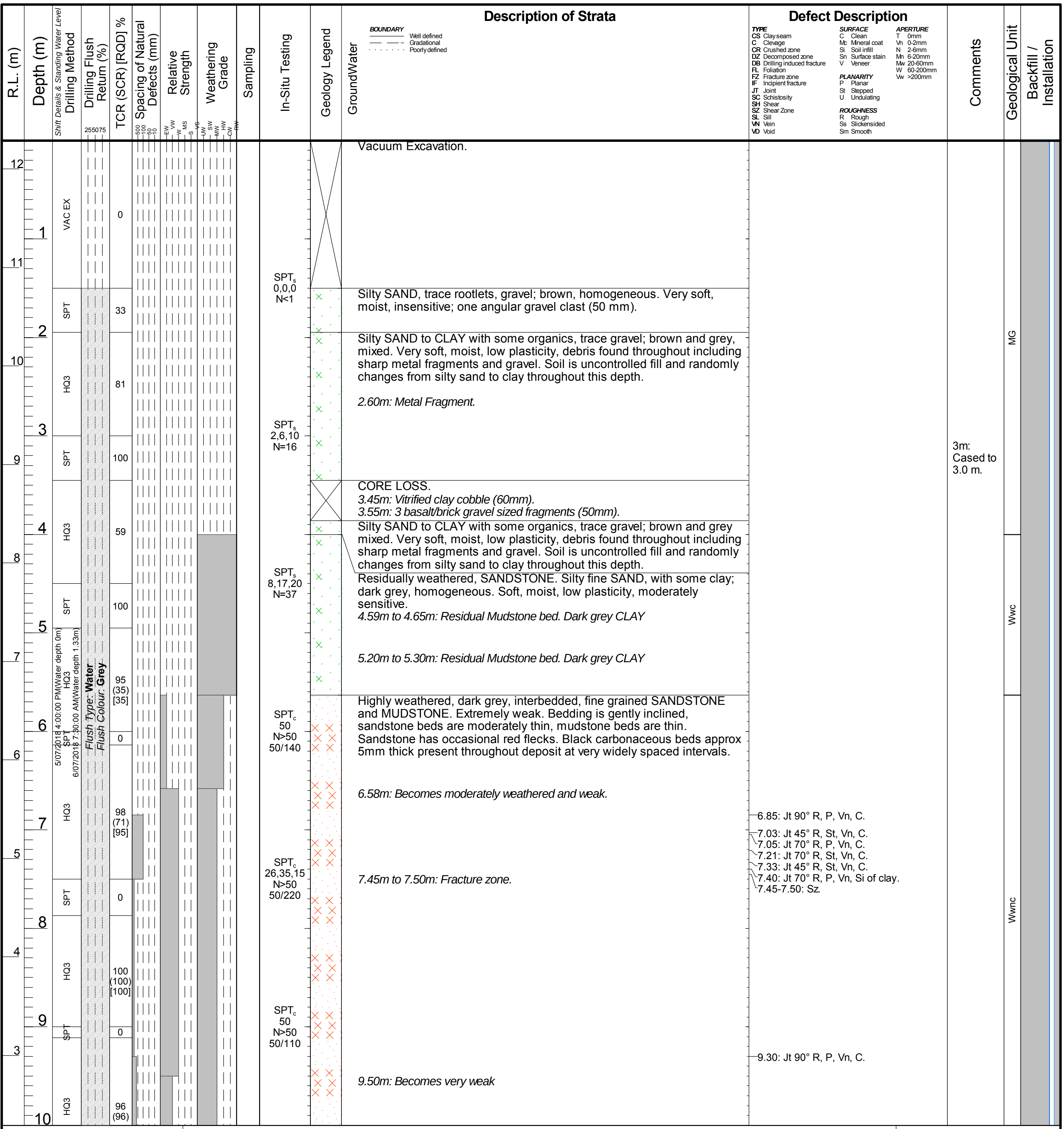
Borehole

Location: **46 Tawariki Street, Grey Lynn** Project No: **AE04725**

Hole ID: **CIE-BH04**

Client: **Watercare**

Date: **5/07/2018**



In-Situ Testing:

- SPT_s 0,0,0 N<1
- SPT_s 2,6,10 N=16
- SPT_s 8,17,20 N=37
- SPT_c 50 N>50 50/140
- SPT_c 26,35,15 N>50 50/220
- SPT_c 50 N>50 50/110

Description of Strata

Vacuum Excavation.

Silty SAND, trace rootlets, gravel; brown, homogeneous. Very soft, moist, insensitive; one angular gravel clast (50 mm).

Silty SAND to CLAY with some organics, trace gravel; brown and grey, mixed. Very soft, moist, low plasticity, debris found throughout including sharp metal fragments and gravel. Soil is uncontrolled fill and randomly changes from silty sand to clay throughout this depth.

2.60m: Metal Fragment.

CORE LOSS.
3.45m: Vitrified clay cobble (60mm).
3.55m: 3 basalt/brick gravel sized fragments (50mm).

Silty SAND to CLAY with some organics, trace gravel; brown and grey mixed. Very soft, moist, low plasticity, debris found throughout including sharp metal fragments and gravel. Soil is uncontrolled fill and randomly changes from silty sand to clay throughout this depth.

Residually weathered, SANDSTONE. Silty fine SAND, with some clay; dark grey, homogeneous. Soft, moist, low plasticity, moderately sensitive.
4.59m to 4.65m: Residual Mudstone bed. Dark grey CLAY

5.20m to 5.30m: Residual Mudstone bed. Dark grey CLAY

Highly weathered, dark grey, interbedded, fine grained SANDSTONE and MUDSTONE. Extremely weak. Bedding is gently inclined, sandstone beds are moderately thin, mudstone beds are thin. Sandstone has occasional red flecks. Black carbonaceous beds approx 5mm thick present throughout deposit at very widely spaced intervals.

6.58m: Becomes moderately weathered and weak.

7.45m to 7.50m: Fracture zone.

9.50m: Becomes very weak

Defect Description

TYPE		SURFACE		APERTURE	
CS	Clay seam	C	Clean	T	0mm
C	Clevage	Mc	Mineral coat	Vn	0-2mm
CR	Crushed zone	Si	Soil infill	N	2-6mm
DZ	Decomposed zone	Sn	Surface stain	Mn	6-20mm
DB	Drilling induced fracture	V	Veneer	Mw	20-60mm
FL	Foliation			W	60-200mm
FZ	Fracture zone	*PLANARITY*		Vw	>200mm
IF	Incipient fracture	P	Planar		
JT	Joint	St	Stepped		
SC	Schistosity	U	Undulating		
SH	Shear				
SZ	Shear Zone	*ROUGHNESS*			
SL	Sill	R	Rough		
VN	Vein	Ss	Slickensided		
VD	Void	Sm	Smooth		

6.85: Jt 90° R, P, Vn, C.
7.03: Jt 45° R, St, Vn, C.
7.05: Jt 70° R, P, Vn, C.
7.21: Jt 70° R, St, Vn, C.
7.33: Jt 45° R, St, Vn, C.
7.40: Jt 70° R, P, Vn, Si of clay.
7.45-7.50: Sz.
9.30: Jt 90° R, P, Vn, C.

Comments: 3m: Cased to 3.0 m.

Started:	5/07/2018
Finished:	10/07/2018
Driller:	McMillan
Plant:	Rig N111 (McMillan)
Logged:	S. Burgess
Checked:	LD

Groundwater Observations

No.	Struck (m)	Date	Standing (m)	Observations
1	-3.78	14/09/2018		Midday WL

Remarks
Packer Test 1 at 9.75-12.00 m, Packer Test 2 at 19.25-22.50 m, Packer Test 3 at 28.50-31.50 m. Vibrating wire piezometer installed with sensor at 26.0m. Water level = 16.1 m RL.
Joint angles are relative to the core axis. If a borehole is true vertical; horizontal=90, vertical=0.
Hole location is in NZTM projection. Elevation is relative to Auckland Vertical Datum 1946
Hole location determined by Survey.

Co-ordinates:
5920092.60mN
1754813.92mE
Elevation: 12.29mRL
Inclination: -90°

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Project: **Grey Lynn Tunnel CIGI5**

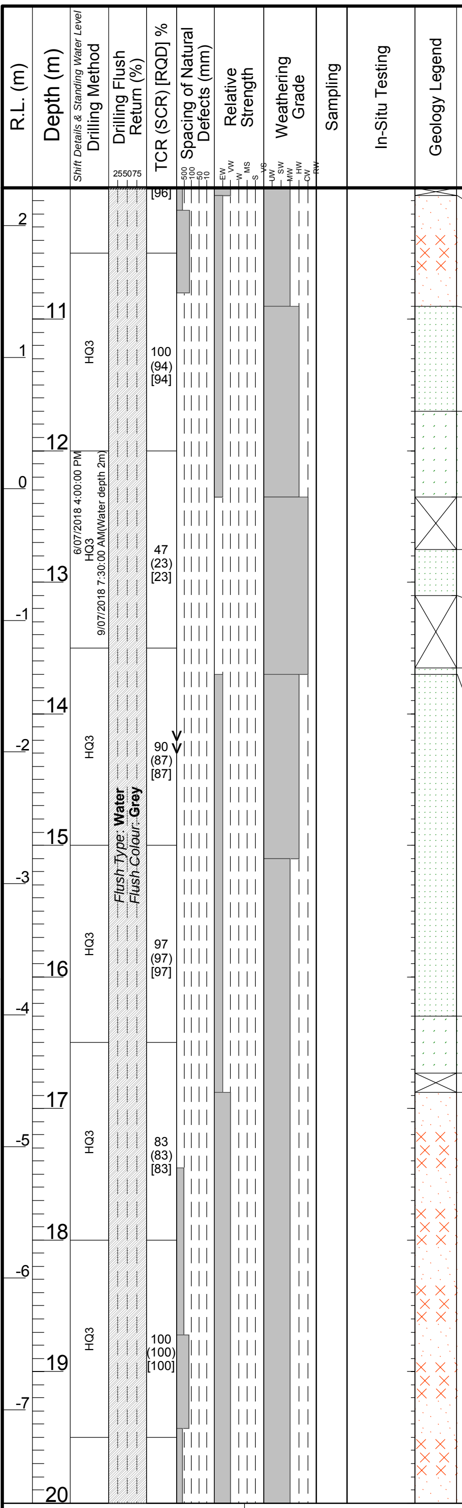
Borehole

Location: **46 Tawariki Street, Grey Lynn** Project No: **AE04725**

Hole ID: **CIE-BH04**

Client: **Watercare**

Date: **5/07/2018**



Description of Strata

BOUNDARY
Well defined
Gradational
Poorly defined

CORE LOSS.

Moderately weathered, dark grey, interbedded, fine grained SANDSTONE and MUDSTONE. Extremely weak. Bedding is sub-horizontal, sandstone beds are moderately thin, mudstone beds are thin. Sandstone has occasional red flecks. Black carbonaceous beds approx 5mm thick present throughout deposit at very widely spaced intervals.

Highly weathered, dark grey, homogeneous, fine grained SANDSTONE. Extremely weak.

Highly weathered, dark grey, homogeneous, medium grained SANDSTONE. Extremely weak.
11.83m to 11.85m: Very thin, sub-horizontal, black carbonaceous bed.

CORE LOSS.

Completely weathered, dark grey, homogeneous, fine grained SANDSTONE. Recovered as fine silty SAND, trace clay.Tightly packed, moist.

CORE LOSS.

Completely weathered, dark grey, homogeneous, fine grained SANDSTONE. Recovered as fine silty SAND, trace clay.Tightly packed, moist.

Highly weathered, dark grey, homogeneous, fine grained SANDSTONE. Extremely weak. Minor white clasts present throughout matrix (1mm). Silty SAND. Loosely packed, moist.

15.05m to 15.55m: Trace green clasts (1-3 mm)
15.10m: Becomes moderately weathered

15.45m to 15.50m: Thin, sub-horizontal, black carbonaceous bed.

Highly weathered, dark grey, homogeneous, fine to medium grained SANDSTONE. Extremely weak. Minor white clasts present throughout matrix (1mm). Silty SAND. Loosely packed, moist.

CORE LOSS.

Moderately weathered, dark grey, interbedded, fine to coarse grained SANDSTONE and MUDSTONE. Very weak. Bedding is sub-horizontal, sandstone beds are moderately thin, mudstone beds are thin. Black carbonaceous beds, laminated to thin, present throughout deposit at widely spaced intervals.

18.13m to 18.15m: Thin, sub-horizontal, black carbonaceous bed.

18.70m to 19.30m: Laminated to thin, sub-horizontal, black carbonaceous beds.

Defect Description

TYPE		*SURFACE*		*APERTURE*	
CS	Clay seam	C	Clean	T	0mm
C	Clevage	Mc	Mineral coat	Vn	0-2mm
CR	Crushed zone	Si	Soil infill	N	2-6mm
DZ	Decomposed zone	Sn	Surface stain	Mn	6-20mm
DB	Drilling induced fracture	V	Veneer	Mw	20-60mm
FL	Foliation			W	60-200mm
FZ	Fracture zone	*PLANARITY*		Vw	>200mm
IF	Incipient fracture	P	Planar		
JT	Joint	St	Stepped		
SC	Schistosity	U	Undulating		
SH	Shear				
SZ	Shear Zone	*ROUGHNESS*			
SL	Sill	R	Rough		
VN	Vein	Ss	Slickensided		
VD	Void	Sm	Smooth		

10.17: Jt 70° R, P, Vn, C.
10.68: Jt 85° R, U, Vn, C.
10.75: Jt 85° R, U, Vn, C.
10.77: Jt 85° R, U, Vn, C.
17.45: Jt 85° R, P, Vn, C.
18.00: Jt 85° R, St, Vn, C.
18.72: Jt 85° R, P, Vn, C.
18.94: Jt 85° R, P, Vn, C.
19.30: Jt 85° R, St, Vn, C.
19.43: Jt 85° R, St, Vn, C.

Comments

Geological Unit
Wwnc(Contd.)
Wpvc
Wwnc
Wpvc
Wwnc
Wuw

Backfill / Installation

Started: 5/07/2018
Finished: 10/07/2018
Driller: McMillan
Plant: Rig N111 (McMillan)
Logged: S. Burgess
Checked: LD

Groundwater Observations

No.	Struck (m)	Date	Standing (m)	Observations
1	-3.78	14/09/2018		Midday WL

Remarks
Packer Test 1 at 9.75-12.00 m, Packer Test 2 at 19.25-22.50 m, Packer Test 3 at 28.50-31.50 m. Vibrating wire piezometer installed with sensor at 26.0m. Water level = 16.1 m RL.
Joint angles are relative to the core axis. If a borehole is true vertical; horizontal=90, vertical=0.
Hole location is in NZTM projection. Elevation is relative to Auckland Vertical Datum 1946
Hole location determined by Survey.

Co-ordinates:
5920092.60mN
1754813.92mE
Elevation: 12.29mRL
Inclination: -90°

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Borehole

Project: **Grey Lynn Tunnel CIGI5**

Location: **46 Tawariki Street, Grey Lynn** Project No: **AE04725**

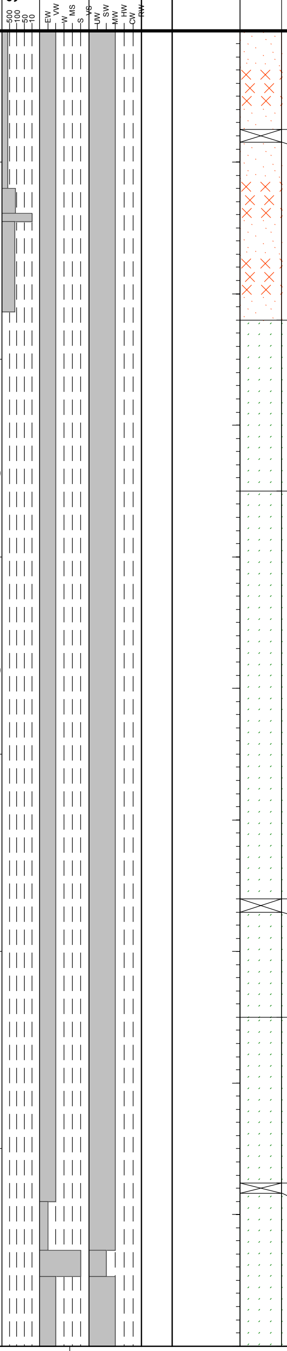
Hole ID: **CIE-BH04**

Client: **Watercare**

Date: **5/07/2018**

R.L. (m)	Depth (m)	Shift Details & Standing Water Level / Drilling Method	TCR (SCR) [RQD] %
-8	20–21	HQ3	93 (93) [93]
-9	21–22	HQ3	100 (96) [96]
-10 / -11	22–23 / 23–24	9/07/2018 1:45:00 PM(Water depth 0.97m) HQ3	100 (100) [100]
-12 / -13	24–25	HQ3	100 (100) [100]
-14	25–26 / 26–27	HQ3	93 (93) [93]
-15 / -16	27–28	9/07/2018 4:00:00 PM HQ3 10/07/2018 7:30:00 AM(Water depth 0.6m)	96 (96) [96]
-17	28–29 / 29–30	HQ3	95 (95) [95]

Drilling Flush Return (%): 25 50 75 — Flush Type: **Water** Flush Colour: **Grey**



Description of Strata

BOUNDARY: Well defined; Gradational; Poorly defined

CORE LOSS.

Moderately weathered, dark grey, interbedded, fine grained SANDSTONE and MUDSTONE. Very weak. Bedding is gently inclined, sandstone beds are moderately thin, mudstone beds are thin. Black carbonaceous beds, laminated to thin, present throughout deposit at widely spaced intervals.
21.45m to 21.48m: Fracture zone. Coal.

Moderately weathered, dark grey, homogeneous, medium grained SANDSTONE. Very weak. Minor white clasts present throughout matrix (1mm), trace dark brownish green mudstone clasts (2-6 mm). Local, very thin mudstone beds are present.

Moderately weathered, dark grey, homogeneous, fine grained SANDSTONE. Very weak. Minor white clasts present throughout matrix (1mm). Local, very thin mudstone beds are present.

24.30m: Becomes medium grained.

24.68m: Becomes fine grained.
24.82m: Thin, gently inclined, grey speckled black, discontinuous carbonaceous bed.
24.92m: Becomes medium grained.
25.11m to 25.22m: Mudstone bed.

25.68m to 25.71m: Mudstone bed.

CORE LOSS.

Moderately weathered, dark grey, homogeneous, medium grained SANDSTONE. Very weak. Minor white clasts present throughout matrix (1mm), trace dark brownish green mudstone clasts (2-6 mm).
27.16m to 27.18m: Mudstone bed.

Moderately weathered, dark grey, homogeneous, medium grained SANDSTONE. Very weak. Minor white clasts present throughout matrix (1mm), trace dark brownish green mudstone clasts (2-6 mm).

28.05m to 28.25m: Becomes coarse grained.

CORE LOSS.

Moderately weathered, dark grey, homogeneous, medium grained SANDSTONE. Very weak. Minor white clasts present throughout matrix (1mm), trace dark brownish green mudstone clasts (2-6 mm).
28.90m: Becomes extremely weak.
29.27m: Becomes slightly weathered and strong.
29.47m: Becomes moderately weathered and very weak.

Defect Description

TYPE		SURFACE		APERTURE	
CS	Clay seam	C	Clean	T	0mm
C	Clevage	Mc	Mineral coat	Vn	0-2mm
CR	Crushed zone	Si	Soil infill	N	2-6mm
DZ	Decomposed zone	Sn	Surface stain	Mn	6-20mm
DB	Drilling induced fracture	V	Veneer	Mw	20-60mm
FL	Foliation			W	60-200mm
FZ	Fracture zone	**PLANARITY**		Vw	>200mm
IF	Incipient fracture	P	Planar		
JT	Joint	St	Stepped		
SC	Schistosity	U	Undulating		
SH	Shear				
SZ	Shear Zone	**ROUGHNESS**			
SL	Sill	R	Rough		
VN	Vein	Ss	Slickensided		
VD	Void	Sm	Smooth		

20.30: Jt 75° R, P, Vn, C.
21.20: Jt 75° R, P, Vn, C.
21.39: Jt 75° R, P, Vn, C.
21.45-21.48: Sz Vn, C, Highly fractured coal.
21.58: Jt 50° R, U, Vn, C.
21.97: Jt 75° R, P, T, C.
22.14: Jt 75° R, U, Vn, C.
24.82: Jt 75° R, P, Vn, C.
29.37: Jt 30° R, P, T, C.

Comments

Geological Unit: Wuw(Contd.); Wpvc; Wuw; Wpvc

Backfill / Installation

Started:	5/07/2018
Finished:	10/07/2018
Driller:	McMillan
Plant:	Rig N111 (McMillan)
Logged:	S. Burgess
Checked:	LD

Groundwater Observations

No.	Struck (m)	Date	Standing (m)	Observations
1	-3.78	14/09/2018		Midday WL

Remarks
Packer Test 1 at 9.75-12.00 m, Packer Test 2 at 19.25-22.50 m, Packer Test 3 at 28.50-31.50 m. Vibrating wire piezometer installed with sensor at 26.0m. Water level = 16.1 m RL.
Joint angles are relative to the core axis. If a borehole is true vertical; horizontal=90, vertical=0.
Hole location is in NZTM projection. Elevation is relative to Auckland Vertical Datum 1946
Hole location determined by Survey.

Co-ordinates:
5920092.60mN
1754813.92mE
Elevation: 12.29mRL
Inclination: -90°

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Project:	**Grey Lynn Tunnel CIGI5**		**Borehole**
Location:	**46 Tawariki Street, Grey Lynn**	Project No: **AE04725**	Hole ID: **CIE-BH04**
Client:	**Watercare**		Date: **5/07/2018**

BOUNDARY
- Well defined
- Gradational
- Poorly defined

Defect Description

TYPE	SURFACE	APERTURE
CS Clay seam	C Clean	T 0mm
C Clevage	Mc Mineral coat	Vn 0-2mm
CR Crushed zone	Si Soil infill	N 2-6mm
DZ Decomposed zone	Sn Surface stain	Mn 6-20mm
DB Drilling induced fracture	V Veneer	Mw 20-60mm
FL Foliation		W 60-200mm
FZ Fracture zone	**PLANARITY**	Vw >200mm
IF Incipient fracture	P Planar	
JT Joint	St Stepped	
SC Schistosity	U Undulating	
SH Shear		
SZ Shear Zone	**ROUGHNESS**	
SL Sill	R Rough	
VN Vein	Ss Slickensided	
VD Void	Sm Smooth	

R.L. (m)	Depth (m)	Shift Details & Standing Water Level Drilling Method	Drilling Flush Return (%) 255075	TCR (SCR) [RQD] %	Spacing of Natural Defects (mm) 500 100 50 10	Relative Strength EW VW W MS S VS	Weathering Grade UW SW MW HW CW RW	Sampling	In-Situ Testing	Geology Legend	GroundWater	Description of Strata	Defect Description	Comments	Geological Unit	Backfill / Installation
-18 -19	31	HQ3	Flush Type: **Water** Flush Colour: **Grey**	94 (94) [94]								CORE LOSS. Moderately weathered, dark grey, homogeneous, medium grained SANDSTONE. Very weak. Minor white clasts present throughout matrix (1mm), trace dark brownish green mudstone clasts (2-6 mm). *31.06m to 31.11m: Mudstone bed.*	30.47: Jt 30° P, T, C. 30.54: Jt 30° P, T, C. 30.61: Jt 30° P, T, C. 30.68: Jt 30° P, T, C. 30.75: Jt 30° P, T, C.		Wuw	
		10/07/2018 4:00:00 PM 17/07/2018 7:30:00 AM(Water depth 0.8m)										CIE-BH04 terminated at 31.50m. Target Depth				

Started:	5/07/2018
Finished:	10/07/2018
Driller:	McMillan
Plant:	Rig N111 (McMillan)
Logged:	S. Burgess
Checked:	LD

Groundwater Observations

No.	*Struck (m)*	*Date*	*Standing (m)*	*Observations*
1	-3.78	14/09/2018		Midday WL

Remarks
Packer Test 1 at 9.75-12.00 m, Packer Test 2 at 19.25-22.50 m, Packer Test 3 at 28.50-31.50 m. Vibrating wire piezometer installed with sensor at 26.0m. Water level = 16.1 m RL.
Joint angles are relative to the core axis. If a borehole is true vertical; horizontal=90, vertical=0.
Hole location is in NZTM projection. Elevation is relative to Auckland Vertical Datum 1946
Hole location determined by Survey.

Co-ordinates:
5920092.60mN
1754813.92mE
Elevation: 12.29mRL
Inclination: -90°

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Project: **Grey Lynn Tunnel CIGI5**

Borehole

Location: **44 Tawariki Street, Grey Lynn**

Project No: **AE04725**

Hole ID: **CIE-BH05**

Client: **Watercare**

Date: **11/07/2018**

Started: 11/07/2018
Finished: 13/07/2018
Driller: McMillan
Plant: Rig N111 (McMillan)
Logged: S. Burgess
Checked: LD

Remarks
Packer Test 1: 11.00 - 13.50 m, Packer Test 2: 19.00 - 21.00 m, Packer Test 3: 28.50 - 31.50 m.
Vibrating wire piezometer installed with sensor at 26.0m. Water level = 16.0 m.
Joint angles are relative to the core axis. If a borehole is true vertical; horizontal=90, vertical=0.
Hole location is in NZTM projection. Elevation is relative to Auckland Vertical Datum 1946 Hole location determined by Survey.

Co-ordinates:
5920115.28mN
1754793.05mE
Elevation: 11.59mRL
Inclination: -90°

Log cover page

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Data Template: AE04725 CI MASTER (NEW TEMPLATE).GPJ Output Form: BH COVER SHEET Project File Name: AE04725 CIGI4 ADDITIONAL INVESTIGATION.GPJ 7/9/18

Version CI 1.10 09/07/2015 - R.Roberts

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Project: **Grey Lynn Tunnel CIGI5**	**Borehole**
Location: **44 Tawariki Street, Grey Lynn** Project No: **AE04725**	Hole ID: **CIE-BH05**
Client: **Watercare**	Date: **11/07/2018**

Columns: R.L. (m) | Depth (m) | *Shift Details & Standing Water Level* Drilling Method | Drilling Flush Return (%) 25 50 75 | TCR (SCR) [RQD] % | Spacing of Natural Defects (mm) 500 100 50 10 | Relative Strength EW VW W MS S VS | Weathering Grade UW SW MW HW CW RW | Sampling | In-Situ Testing | Geology Legend | GroundWater | Description of Strata | Defect Description | Comments | Geological Unit | Backfill / Installation

BOUNDARY
Well defined
Gradational
Poorly defined

Defect Description

TYPE		*SURFACE*		*APERTURE*	
CS	Clay seam	C	Clean	T	0mm
C	Clevage	Mc	Mineral coat	Vn	0-2mm
CR	Crushed zone	Si	Soil infill	N	2-6mm
DZ	Decomposed zone	Sn	Surface stain	Mn	6-20mm
DB	Drilling induced fracture	V	Veneer	Mw	20-60mm
FL	Foliation			W	60-200mm
FZ	Fracture zone	*PLANARITY*		Vw	>200mm
IF	Incipient fracture	P	Planar		
JT	Joint	St	Stepped		
SC	Schistosity	U	Undulating		
SH	Shear				
SZ	Shear Zone	*ROUGHNESS*			
SL	Sill	R	Rough		
VN	Vein	Ss	Slickensided		
VD	Void	Sm	Smooth		

R.L. (m): 11, 10, 9, 8, 7, 6, 5, 4, 3, 2
Depth (m): 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

Drilling Method	TCR (SCR) [RQD] %
VAC EX	0
SPT	100
HQ3	100
TBX	100
SPT	100
HQ3	100
TBX	100
SPT	100
HQ3	77 (77) [77]
SPT	100
HQ3	67 (33) [33]
SPT	100
HQ3	71 (65) [65]
SPT	100

Shift details: 11/07/2018 7:30:00 AM(Water depth 0.3m); 11/07/2018 4:00:00 PM; 12/07/2018 7:30:00 AM(Water depth 0.2m)

Flush Type: **Water**
Flush Colour: **Grey**

In-Situ Testing:
- SPT_s 1,2,2 N=4
- SPT_s 0,0,1 N=1
- SPT_s 6,11,12 N=23
- SPT_s 5,9,11 N=20
- SPT_s 8,12,19 N=31
- SPT_s 12,20,30 N>50 50/290

Description of Strata

Vacuum Excavation

Silty CLAY; light grey mottled orange. Very soft, moist, high plasticity, not dilatant.

2.25m: Becomes brown grey with some wood fragments. Soft.

2.70m: Becomes firm.

Push Tube. Material change at 3.1m from silty CLAY to silty clayey SAND.

Residually weathered, SANDSTONE. Clayey silty fine SAND; dark grey, homogeneous. Soft, moist, low plasticity, low dilatancy.

Push Tube.
4.50m: Becomes firm.

Residually weathered, SANDSTONE. Clayey silty fine SAND; dark grey, homogeneous. Firm, moist, low plasticity, Insensitive

CORE LOSS.

Highly weathered, fine grained SANDSTONE. Extremely weak. Clayey silty fine SAND; dark grey, homogeneous. Dense, moist.

6.45m to 6.80m: Recovered as Silty fine SAND

CORE LOSS.

Highly weathered, fine grained SANDSTONE. Extremely weak. Silty fine SAND, some clay; dark grey, homogeneous. Dense, moist.

Highly weathered, dark grey, BRECCIA with fine to medium gravel sized, angular to sub-rounded mudstone clasts in a well cemented fine sandstone matrix. Extremely weak.

Core loss. Infer BRECCIA and a mudstone clast blocked catcher.

Highly weathered, dark grey, BRECCIA with fine to medium gravel sized, angular to sub-rounded mudstone clasts in a well cemented fine sandstone matrix. Extremely weak.

Highly weathered, fine grained SANDSTONE. Extremely weak. Silty fine SAND, some clay; dark grey, homogeneous. Very dense, moist.

Highly weathered, dark grey, BRECCIA, fine to medium gravel sized, sub- rounded mudstone and fine grained sandstone clasts in a fine

Comments: 3m: Cased to 3.0 m.

Geological Unit: MG, T, Wwc, Wwnc, Wwc

Started:	11/07/2018
Finished:	13/07/2018
Driller:	McMillan
Plant:	Rig N111 (McMillan)
Logged:	S. Burgess
Checked:	LD

Groundwater Observations

No.	Struck (m)	Date	Standing (m)	Observations
1	-4.39	13/09/2018		End of Day WL

Remarks
Packer Test 1: 11.00 - 13.50 m, Packer Test 2: 19.00 - 21.00 m, Packer Test 3: 28.50 - 31.50 m.
Vibrating wire piezometer installed with sensor at 26.0m. Water level = 16.0 m RL.
Joint angles are relative to the core axis. If a borehole is true vertical; horizontal=90, vertical=0.
Hole location is in NZTM projection. Elevation is relative to Auckland Vertical Datum 1946 Hole location determined by Survey.

Co-ordinates:
5920115.28mN
1754793.05mE
Elevation: 11.59mRL
Inclination: -90°

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Data Template: AE04725 CI MASTER (NEW TEMPLATE).GPJ Output Form: COMPILATION BOREHOLE Project File Name: AE04725 CIGI4 ADDITIONAL INVESTIGATION.GPJ 7/9/18

Version CI 1.10 09/07/2015 - R.Roberts

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Project: **Grey Lynn Tunnel CIGI5**

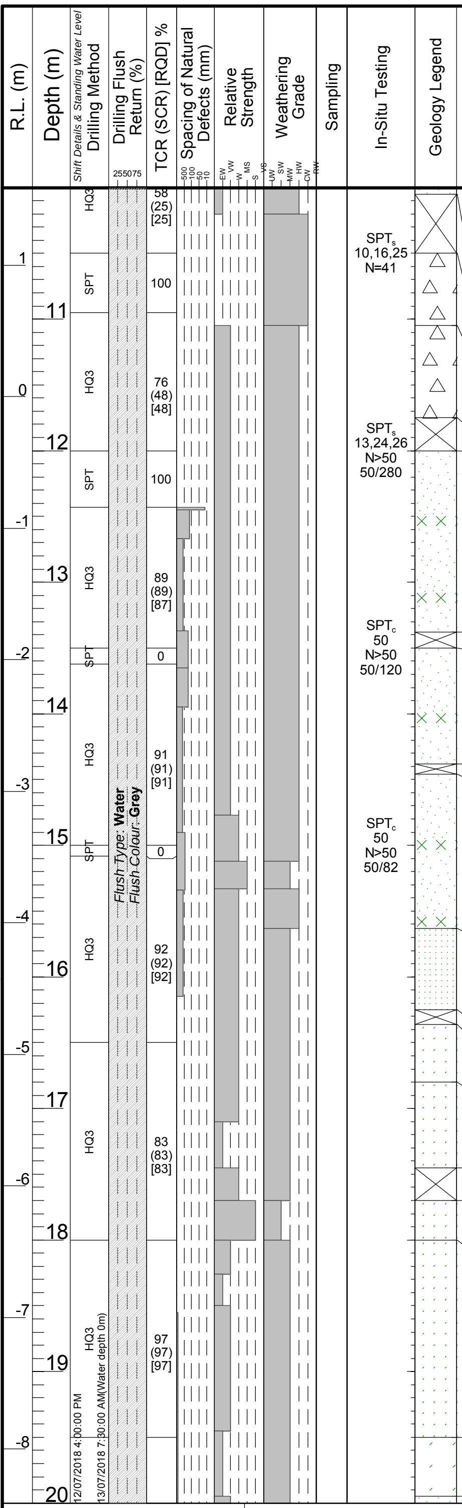
Borehole

Location: **44 Tawariki Street, Grey Lynn** Project No: **AE04725**

Hole ID: **CIE-BH05**

Client: **Watercare**

Date: **11/07/2018**



Description of Strata

BOUNDARY
- Well defined
- Gradational
- Poorly defined

grained sandstone matrix. Recovered as clasts with silty fine SAND; dense.

Highly weathered, dark grey, interbedded, moderately thinly bedded fine grained SANDSTONE and thinly bedded MUDSTONE. Sub-horizontal bedding. Extremely weak. Mudstone has trace black carbonaceous material.

CORE LOSS. Infer residual BRECCIA from 10.2m. Clast blocked catcher.

Completely weathered, dark grey, BRECCIA recovered as a silty CLAY with subangular mudstone clasts (5-10mm). Soft, moist, low plasticity.

Highly weathered, dark grey, BRECCIA with fine to coarse gravel sized, sub-angular to sub-rounded mudstone clasts and some wood fragments in a fine grained sandstone matrix. Very weak. *11.57m: Recovered as clasts only; infer matrix washed out.*

CORE LOSS.

Highly weathered, dark grey, interbedded, fine grained SANDSTONE and MUDSTONE. Sandstone beds are moderately thin, sub-horizontal. Mudstone beds are thin to moderately thin, sub-horizontal. Very weak.

CORE LOSS.

Highly weathered, dark grey, interbedded, fine grained SANDSTONE and MUDSTONE. Sandstone beds are moderately thin, moderately inclined. Mudstone beds are thin to moderately thin, moderately inclined. Very weak.

CORE LOSS.

Highly weathered, dark grey, interbedded, fine grained SANDSTONE and MUDSTONE. Sandstone beds are moderately thin, moderately inclined. Mudstone beds are thin to moderately thin, moderately inclined. Very weak. *14.70m to 15.00m: Becomes interbedded with laminated carbonaceous beds (black).* *14.77m: Becomes weak.* *15.12m to 15.33m: Moderately weathered, moderately thick SANDSTONE bed. Moderately strong.*

Moderately weathered, dark grey, homogeneous fine grained SANDSTONE.Weak, moderately inclined. *15.83m: Becomes medium grained with white and cream fine gravel sized clasts and black speckles inferred as trace carbonaceous material.*

CORE LOSS.

Moderately weathered, dark grey, homogeneous, fine grained SANDSTONE. Weak, moderately inclined. Minor white clasts present throughout matrix (1mm).

Moderately weathered, dark grey, homogeneous, fine to medium grained SANDSTONE. Weak. Minor white clasts present throughout matrix (1mm). *17.10m to 17.45m: Extremely weak.*

CORE LOSS.

Slightly weathered, dark grey, fine to medium grained SANDSTONE. Strong. Matrix has some sand to fine gravel sized white and cream clasts and discontinuous black carbonaceous beds.

Moderately weathered, dark grey, fine to medium grained SANDSTONE. Very weak. Minor white clasts present throughout matrix (1mm). *18.26m: Becomes extremely weak* *18.50m: Becomes very weak.*

Moderately weathered, dark grey, medium to coarse grained SANDSTONE. Extremly weak. Minor white clasts present throughout matrix (1mm).

Defect Description

TYPE		SURFACE		APERTURE	
CS	Clay seam	C	Clean	T	0mm
C	Clevage	Mc	Mineral coat	Vn	0-2mm
CR	Crushed zone	Si	Soil infill	N	2-6mm
DZ	Decomposed zone	Sn	Surface stain	Mn	6-20mm
DB	Drilling induced fracture	V	Veneer	Mw	20-60mm
FL	Foliation			W	60-200mm
FZ	Fracture zone	*PLANARITY*		Vw	>200mm
IF	Incipient fracture	P	Planar		
JT	Joint	St	Stepped		
SC	Schistosity	U	Undulating		
SH	Shear				
SZ	Shear Zone	*ROUGHNESS*			
SL	Sill	R	Rough		
VN	Vein	Ss	Slickensided		
VD	Void	Sm	Smooth		

12.43: Jt 90° Sm, P, Vn, C.
12.45: Jt 90° Sm, P, Vn, C.
12.67: Jt 90° R, St, Vn, C.
13.37: Jt 70° R, P, Vn, C.
13.65: Jt 70° Sm, U, Vn, Si of sandy clay.
13.95: Jt 70° R, P, Vn, C.
14.90: Jt 70° R, St, Vn, C.
15.34: Jt 70° R, P, Vn, C.
16.15: Jt 70° R, P, Vn, C.
18.55: Jt 10° Sm, P, T, C, joint is displaced by 10mm.

Comments

Geological Unit

Wwc(Contd.)
Wwnc
Wwc
Wuw
Wpvc

Backfill / Installation

Started:	11/07/2018
Finished:	13/07/2018
Driller:	McMillan
Plant:	Rig N111 (McMillan)
Logged:	S. Burgess
Checked:	LD

Groundwater Observations

No.	Struck (m)	Date	Standing (m)	Observations
1	-4.39	13/09/2018		End of Day WL

Remarks
Packer Test 1: 11.00 - 13.50 m, Packer Test 2: 19.00 - 21.00 m, Packer Test 3: 28.50 - 31.50 m.
Vibrating wire piezometer installed with sensor at 26.0m. Water level = 16.0 m RL.
Joint angles are relative to the core axis. If a borehole is true vertical; horizontal=90, vertical=0.
Hole location is in NZTM projection. Elevation is relative to Auckland Vertical Datum 1946 Hole location determined by Survey.

Co-ordinates:
5920115.28mN
1754793.05mE
Elevation: 11.59mRL
Inclination: -90°

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Borehole

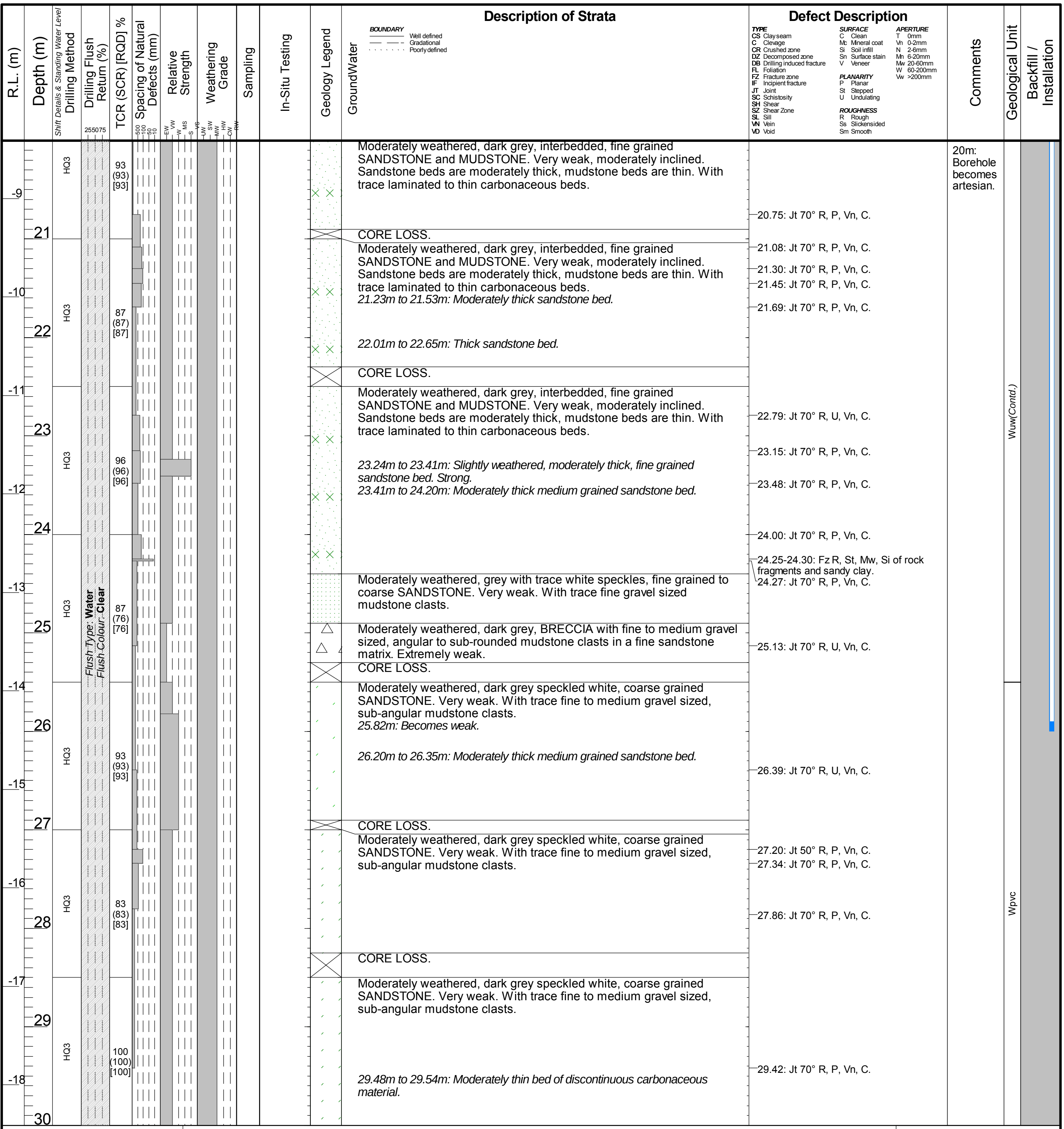
Project: **Grey Lynn Tunnel CIGI5**

Location: **44 Tawariki Street, Grey Lynn** Project No: **AE04725**

Hole ID: **CIE-BH05**

Client: **Watercare**

Date: **11/07/2018**



Core runs

Drilling Method	TCR (SCR) [RQD] %
HQ3	93 (93) [93]
HQ3	87 (87) [87]
HQ3	96 (96) [96]
HQ3	87 (76) [76]
HQ3	93 (93) [93]
HQ3	83 (83) [83]
HQ3	100 (100) [100]

Description of Strata

Moderately weathered, dark grey, interbedded, fine grained SANDSTONE and MUDSTONE. Very weak, moderately inclined. Sandstone beds are moderately thick, mudstone beds are thin. With trace laminated to thin carbonaceous beds.

CORE LOSS.

Moderately weathered, dark grey, interbedded, fine grained SANDSTONE and MUDSTONE. Very weak, moderately inclined. Sandstone beds are moderately thick, mudstone beds are thin. With trace laminated to thin carbonaceous beds.
21.23m to 21.53m: Moderately thick sandstone bed.

22.01m to 22.65m: Thick sandstone bed.

CORE LOSS.

Moderately weathered, dark grey, interbedded, fine grained SANDSTONE and MUDSTONE. Very weak, moderately inclined. Sandstone beds are moderately thick, mudstone beds are thin. With trace laminated to thin carbonaceous beds.

23.24m to 23.41m: Slightly weathered, moderately thick, fine grained sandstone bed. Strong.
23.41m to 24.20m: Moderately thick medium grained sandstone bed.

Moderately weathered, grey with trace white speckles, fine grained to coarse SANDSTONE. Very weak. With trace fine gravel sized mudstone clasts.

Moderately weathered, dark grey, BRECCIA with fine to medium gravel sized, angular to sub-rounded mudstone clasts in a fine sandstone matrix. Extremely weak.

CORE LOSS.

Moderately weathered, dark grey speckled white, coarse grained SANDSTONE. Very weak. With trace fine to medium gravel sized, sub-angular mudstone clasts.
25.82m: Becomes weak.

26.20m to 26.35m: Moderately thick medium grained sandstone bed.

CORE LOSS.

Moderately weathered, dark grey speckled white, coarse grained SANDSTONE. Very weak. With trace fine to medium gravel sized, sub-angular mudstone clasts.

CORE LOSS.

Moderately weathered, dark grey speckled white, coarse grained SANDSTONE. Very weak. With trace fine to medium gravel sized, sub-angular mudstone clasts.

29.48m to 29.54m: Moderately thin bed of discontinuous carbonaceous material.

Defect Description

TYPE		SURFACE		APERTURE	
CS	Clay seam	C	Clean	T	0mm
C	Clevage	Mc	Mineral coat	Vn	0-2mm
CR	Crushed zone	Si	Soil infill	N	2-6mm
DZ	Decomposed zone	Sn	Surface stain	Mn	6-20mm
DB	Drilling induced fracture	V	Veneer	Mw	20-60mm
FL	Foliation			W	60-200mm
FZ	Fracture zone	**PLANARITY**		Vw	>200mm
IF	Incipient fracture	P	Planar		
JT	Joint	St	Stepped		
SC	Schistosity	U	Undulating		
SH	Shear				
SZ	Shear Zone	**ROUGHNESS**			
SL	Sill	R	Rough		
VN	Vein	Ss	Slickensided		
VD	Void	Sm	Smooth		

20.75: Jt 70° R, P, Vn, C.
21.08: Jt 70° R, P, Vn, C.
21.30: Jt 70° R, P, Vn, C.
21.45: Jt 70° R, P, Vn, C.
21.69: Jt 70° R, P, Vn, C.
22.79: Jt 70° R, U, Vn, C.
23.15: Jt 70° R, P, Vn, C.
23.48: Jt 70° R, P, Vn, C.
24.00: Jt 70° R, P, Vn, C.
24.25-24.30: Fz R, St, Mw, Si of rock fragments and sandy clay.
24.27: Jt 70° R, P, Vn, C.
25.13: Jt 70° R, U, Vn, C.
26.39: Jt 70° R, U, Vn, C.
27.20: Jt 50° R, P, Vn, C.
27.34: Jt 70° R, P, Vn, C.
27.86: Jt 70° R, P, Vn, C.
29.42: Jt 70° R, P, Vn, C.

Comments

20m: Borehole becomes artesian.

Started:	11/07/2018
Finished:	13/07/2018
Driller:	McMillan
Plant:	Rig N111 (McMillan)
Logged:	S. Burgess
Checked:	LD

Groundwater Observations

No.	*Struck (m)*	*Date*	*Standing (m)*	*Observations*
1	-4.39	13/09/2018		End of Day WL

Remarks
Packer Test 1: 11.00 - 13.50 m, Packer Test 2: 19.00 - 21.00 m, Packer Test 3: 28.50 - 31.50 m.
Vibrating wire piezometer installed with sensor at 26.0m. Water level = 16.0 m RL.
Joint angles are relative to the core axis. If a borehole is true vertical; horizontal=90, vertical=0.
Hole location is in NZTM projection. Elevation is relative to Auckland Vertical Datum 1946 Hole location determined by Survey.

Co-ordinates:
5920115.28mN
1754793.05mE
Elevation: 11.59mRL
Inclination: -90°

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Project: **Grey Lynn Tunnel CIGI5**

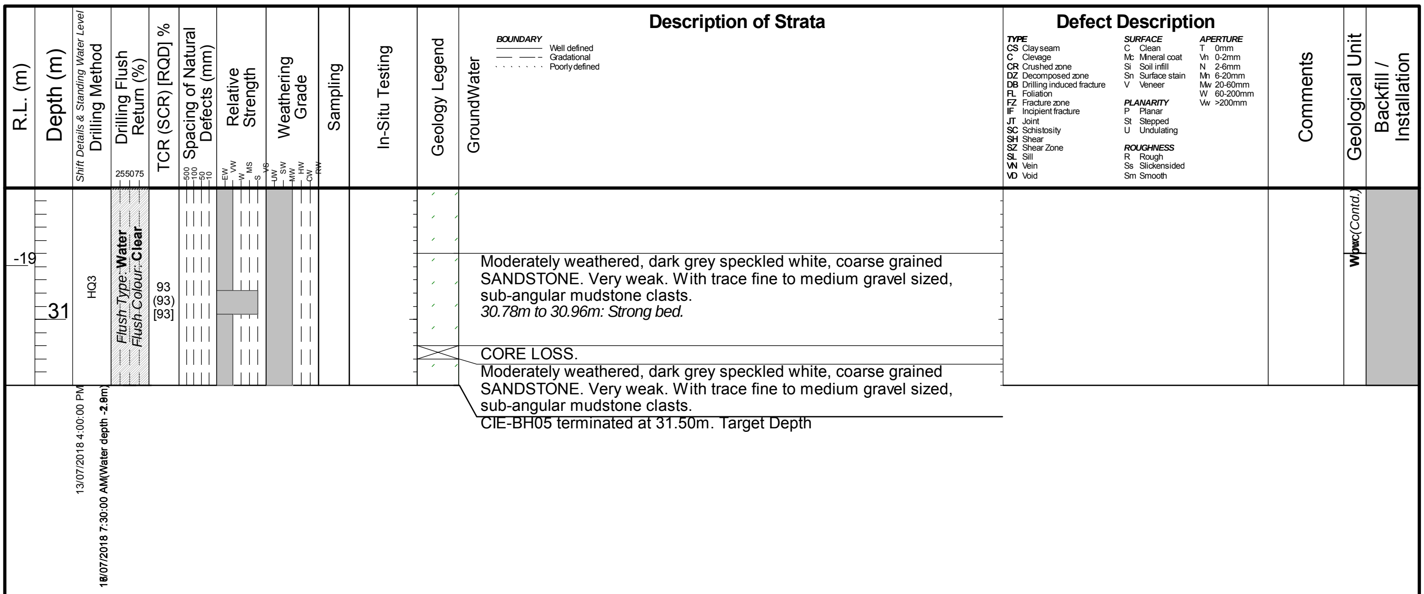
Borehole

Location: **44 Tawariki Street, Grey Lynn** Project No: **AE04725**

Hole ID: **CIE-BH05**

Client: **Watercare**

Date: **11/07/2018**



Description of Strata

BOUNDARY
- Well defined
- Gradational
- Poorly defined

Moderately weathered, dark grey speckled white, coarse grained SANDSTONE. Very weak. With trace fine to medium gravel sized, sub-angular mudstone clasts.
30.78m to 30.96m: Strong bed.

CORE LOSS.

Moderately weathered, dark grey speckled white, coarse grained SANDSTONE. Very weak. With trace fine to medium gravel sized, sub-angular mudstone clasts.

CIE-BH05 terminated at 31.50m. Target Depth

Defect Description

TYPE		SURFACE		APERTURE	
CS	Clay seam	C	Clean	T	0mm
C	Clevage	Mc	Mineral coat	Vn	0-2mm
CR	Crushed zone	Si	Soil infill	N	2-6mm
DZ	Decomposed zone	Sn	Surface stain	Mn	6-20mm
DB	Drilling induced fracture	V	Veneer	Mw	20-60mm
FL	Foliation			W	60-200mm
FZ	Fracture zone	PLANARITY		Vw	>200mm
IF	Incipient fracture	P	Planar		
JT	Joint	St	Stepped		
SC	Schistosity	U	Undulating		
SH	Shear				
SZ	Shear Zone	ROUGHNESS			
SL	Sill	R	Rough		
VN	Vein	Ss	Slickensided		
VD	Void	Sm	Smooth		

Started: 11/07/2018
Finished: 13/07/2018
Driller: McMillan
Plant: Rig N111 (McMillan)
Logged: S. Burgess
Checked: LD

Groundwater Observations

No.	Struck (m)	Date	Standing (m)	Observations
1	-4.39	13/09/2018		End of Day WL

Remarks
Packer Test 1: 11.00 - 13.50 m, Packer Test 2: 19.00 - 21.00 m, Packer Test 3: 28.50 - 31.50 m.
Vibrating wire piezometer installed with sensor at 26.0m. Water level = 16.0 m RL.
Joint angles are relative to the core axis. If a borehole is true vertical; horizontal=90, vertical=0.
Hole location is in NZTM projection. Elevation is relative to Auckland Vertical Datum 1946 Hole location determined by Survey.

Co-ordinates:
5920115.28mN
1754793.05mE
Elevation: 11.59mRL
Inclination: -90°

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Project: **Grey Lynn Tunnel CIGI5**

Borehole

Location: **1A Fisherton St, Grey Lynn**

Project No: **AE04725**

Hole ID: **CIE-BH06**

Client: **Watercare**

Date: **27/06/2018**

Started:	27/06/2018
Finished:	3/07/2018
Driller:	McMillan
Plant:	Rig N102 (McMillan)
Logged:	S. Burgess
Checked:	LD

Remarks
Packer Test 1 at 27.00-30.00m, Packer Test 2 at 50.25-52.50m, Packer Test 3 at 56.25-58.50m, Packer Test 4 at 54.5-63.50m.
Piezometer dipped 10/07/2018; water level 1.8mbgl.
Joint angles are relative to the core axis. If a borehole is true vertical; horizontal=90, vertical=0.
Hole location is in NZTM projection. Elevation is relative to Auckland Vertical Datum 1946
Hole location determined by Survey.

Co-ordinates:
5919179.64mN
1754450.92mE
Elevation: 48.00mRL
Inclination: -90°

Log cover page

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Data Template: AE04725 CI MASTER (NEW TEMPLATE).GPJ Output Form: BH COVER SHEET Project File Name: AE04725 CIGI4 ADDITIONAL INVESTIGATION.GPJ 7/9/18

Version CI 1.10 09/07/2015 - R.Roberts

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Borehole

Project: **Grey Lynn Tunnel CIGI5**

Location: **1A Fisherton St, Grey Lynn** Project No: **AE04725**

Hole ID: **CIE-BH06**

Client: **Watercare**

Date: **27/06/2018**

R.L. (m)	Depth (m)
47	1
46	2
45	3
44	4
43	5
42	6
41	7
40	8
39	9
38	10

Column headings: R.L. (m); Depth (m); *Shift Details & Standing Water Level* Drilling Method; Drilling Flush Return (%) 25 50 75; TCR (SCR) [RQD] %; Spacing of Natural Defects (mm) 500 100 50 10; Relative Strength EW VW W MS S VS; Weathering Grade UW SW MW HW CW RW; Sampling; In-Situ Testing; Geology Legend; GroundWater; Description of Strata; Defect Description; Comments; Geological Unit; Backfill / Installation

BOUNDARY
- Well defined
- Gradational
- Poorly defined

TYPE		*SURFACE*		*APERTURE*	
CS	Clay seam	C	Clean	T	0mm
C	Clevage	Mc	Mineral coat	Vn	0-2mm
CR	Crushed zone	Si	Soil infill	N	2-6mm
DZ	Decomposed zone	Sn	Surface stain	Mn	6-20mm
DB	Drilling induced fracture	V	Veneer	Mw	20-60mm
FL	Foliation			W	60-200mm
FZ	Fracture zone	*PLANARITY*		Vw	>200mm
IF	Incipient fracture	P	Planar		
JT	Joint	St	Stepped		
SC	Schistosity	U	Undulating		
SH	Shear				
SZ	Shear Zone	*ROUGHNESS*			
SL	Sill	R	Rough		
VN	Vein	Ss	Slickensided		
VD	Void	Sm	Smooth		

Drilling Method	TCR (SCR) [RQD] %	Sampling
VAC EX	0	
SPT	100	
HQ3	100	
TBX	100	TW
SPT	100	
HQ3	100	
SPT	100	
HQ3	76	
SPT	100	
HQ3	100	
SPT	100	
27/06/2018 4:00:00 PM(Water depth 1.9m) HQ3 28/06/2018 7:30:00 AM(Water depth 0.3m)	62	
SPT	100	

Flush Type: Water
Flush Colour: Grey

In-Situ Testing	Description of Strata	Comments	Geological Unit
	Vacuum Excavation.		MG
SPT_s 0,2,3 N=5	Silty CLAY; blueish grey with some mottled yellow (oxidation). Firm, moist, low plasticity. *2.55m to 2.70m: Changes to Silty CLAY with some well graded fine grained angular sand*		Wwc
	Push tube.		
SPT_s 2,2,4 N=6	Sandy Silty CLAY; blueish grey mottled heavily with yellow (oxidation). Soft, moist, low plasticity, Insensitive.		
SPT_s 1,2,4 N=6	SILT with minor clay and trace sand; brownish grey and brownish orange, mixed. Soft, moist, low plasticity, Insensitive, heavily oxidised. *4.50m to 4.95m: Changes to trace fine sand* *4.95m: Changes to some clay, no sand; grey.*	4m: Cased to 4.0 m.	
	CORE LOSS.		
SPT_s 3,8,9 N=17 SPT_s 2,4,4 N=8	Residually weathered SANDSTONE/SILTSTONE. SILT with minor clay and trace sand; dark grey, homogeneous. Firm, moist, low plasticity, Insensitive.		
	Completely weathered, SILTSTONE. SILT with some clay; dark grey, homogeneous. Very stiff, moist, low plasticity, Insensitive. Residual joints present. *8.40m to 8.60m: Becomes firm.*		
	CORE LOSS.		
SPT_s 3,5,8 N=13	Completely weathered, SILTSTONE. SILT with some clay; dark grey, homogeneous. Very stiff, moist, low plasticity, Insensitive. Residual joints present.		

Started: 27/06/2018

Finished: 3/07/2018

Driller: McMillan

Plant: Rig N102 (McMillan)

Logged: S. Burgess

Checked: LD

Groundwater Observations

No.	*Struck (m)*	*Date*	*Standing (m)*	*Observations*
1	1.8	10/07/2018		
2	2.1	28/08/2018		

Remarks
Packer Test 1 at 27.00-30.00m, Packer Test 2 at 50.25-52.50m, Packer Test 3 at 56.25-58.50m, Packer Test 4 at 54.5-63.50m.
Piezometer dipped 10/07/2018; water level 1.8 mbgl.
Joint angles are relative to the core axis. If a borehole is true vertical; horizontal=90, vertical=0.
Hole location is in NZTM projection. Elevation is relative to Auckland Vertical Datum 1946
Hole location determined by Survey.

Co-ordinates:
5919179.64mN
1754450.92mE
Elevation: 48.00mRL
Inclination: -90°

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Data Template: AE04725 CI MASTER (NEW TEMPLATE).GPJ Output Form: COMPILATION BOREHOLE Project File Name: AE04725 CIGI4 ADDITIONAL INVESTIGATION.GPJ 7/9/18

Version CI 1.10 09/07/2015 - R.Roberts

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Project: **Grey Lynn Tunnel CIGI5**

Borehole

Location: **1A Fisherton St, Grey Lynn** Project No: **AE04725**

Hole ID: **CIE-BH06**

Client: **Watercare**

Date: **27/06/2018**

R.L. (m) | Depth (m) | *Shift Details & Standing Water Level* Drilling Method | Drilling Flush Return (%) 25 50 75 | TCR (SCR) [RQD] % | Spacing of Natural Defects (mm) 500 100 50 10 | Relative Strength EW VW W MS S VS | Weathering Grade UW SW MW HW CW RW | Sampling | In-Situ Testing | Geology Legend | GroundWater | Description of Strata | Defect Description | Comments | Geological Unit | Backfill / Installation

Description of Strata

BOUNDARY
Well defined
Gradational
Poorly defined

Defect Description

TYPE		SURFACE		APERTURE	
CS	Clay seam	C	Clean	T	0mm
C	Clevage	Mc	Mineral coat	Vn	0-2mm
CR	Crushed zone	Si	Soil infill	N	2-6mm
DZ	Decomposed zone	Sn	Surface stain	Mn	6-20mm
DB	Drilling induced fracture	V	Veneer	Mw	20-60mm
FL	Foliation			W	60-200mm
FZ	Fracture zone	PLANARITY		Vw	>200mm
IF	Incipient fracture	P	Planar		
JT	Joint	St	Stepped		
SC	Schistosity	U	Undulating		
SH	Shear				
SZ	Shear Zone	ROUGHNESS			
SL	Sill	R	Rough		
VN	Vein	Ss	Slickensided		
VD	Void	Sm	Smooth		

R.L. (m)	Depth (m)	Drilling Method	TCR (SCR) [RQD] %	In-Situ Testing
		HQ3	100	
		SPT	100	SPT$_s$ 3,6,9 N=15
37	11	HQ3	95 (95) [95]	
36	12	SPT	100	SPT$_s$ 6,9,14 N=23
35	13	HQ3	100 (62) [62]	
		SPT	100	SPT$_s$ 7,12,16 N=28
34	14	HQ3	100 (100) [100]	
33	15	SPT	100	SPT$_s$ 8,13,19 N=32
32	16	HQ3	90 (90) [90]	
		SPT	100	SPT$_s$ 8,12,17 N=29
31	17	HQ3	90 (90) [90]	
30	18	SPT	100	SPT$_s$ 8,12,15 N=27
29	19	HQ3	86 (86) [75]	
		SPT	100	SPT$_s$ 7,10,14 N=24
28	20			

Drilling Flush Return: *Flush Type:* **Water**, *Flush Colour:* **Grey**

Highly weathered, dark grey, interbedded, fine grained SANDSTONE and MUDSTONE. Extremely weak, moderately inclined. Silty SAND, some clay. Medium dense, moist.

12.80m to 13.20m: Recovered as crushed zone.

14.10m: Thin, moderately inclined, black carbonaceous bed.

16.04m: Thin, moderately inclined, black carbonaceous bed.

CORE LOSS.

Highly weathered, dark grey, interbedded, fine grained SANDSTONE and MUDSTONE. Extremely weak, moderately inclined. Silty SAND, some clay. Medium dense, moist.

CORE LOSS.

Highly weathered, dark grey, interbedded, fine grained SANDSTONE and MUDSTONE. Extremely weak, moderately inclined. Silty SAND, some clay. Medium dense, moist.

CORE LOSS.

Highly weathered, dark grey, interbedded, fine grained SANDSTONE and MUDSTONE. Extremely weak, moderately inclined. Silty SAND, some clay. Medium dense, moist.

Defect Description:
14.70: Jt 70° Sm, P, Vn, C.
17.80: Jt 70° R, P, Vn, C.

Geological Unit: Wwc(Contd.)
Wwnc

Started:	27/06/2018
Finished:	3/07/2018
Driller:	McMillan
Plant:	Rig N102 (McMillan)
Logged:	S. Burgess
Checked:	LD

Groundwater Observations

No.	*Struck (m)*	*Date*	*Standing (m)*	*Observations*
1	1.8	10/07/2018		
2	2.1	28/08/2018		

Remarks
Packer Test 1 at 27.00-30.00m, Packer Test 2 at 50.25-52.50m, Packer Test 3 at 56.25-58.50m, Packer Test 4 at 54.5-63.50m.
Piezometer dipped 10/07/2018; water level 1.8mbgl.
Joint angles are relative to the core axis. If a borehole is true vertical; horizontal=90, vertical=0.
Hole location is in NZTM projection. Elevation is relative to Auckland Vertical Datum 1946
Hole location determined by Survey.

Co-ordinates:
5919179.64mN
1754450.92mE
Elevation: 48.00mRL
Inclination: -90°

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

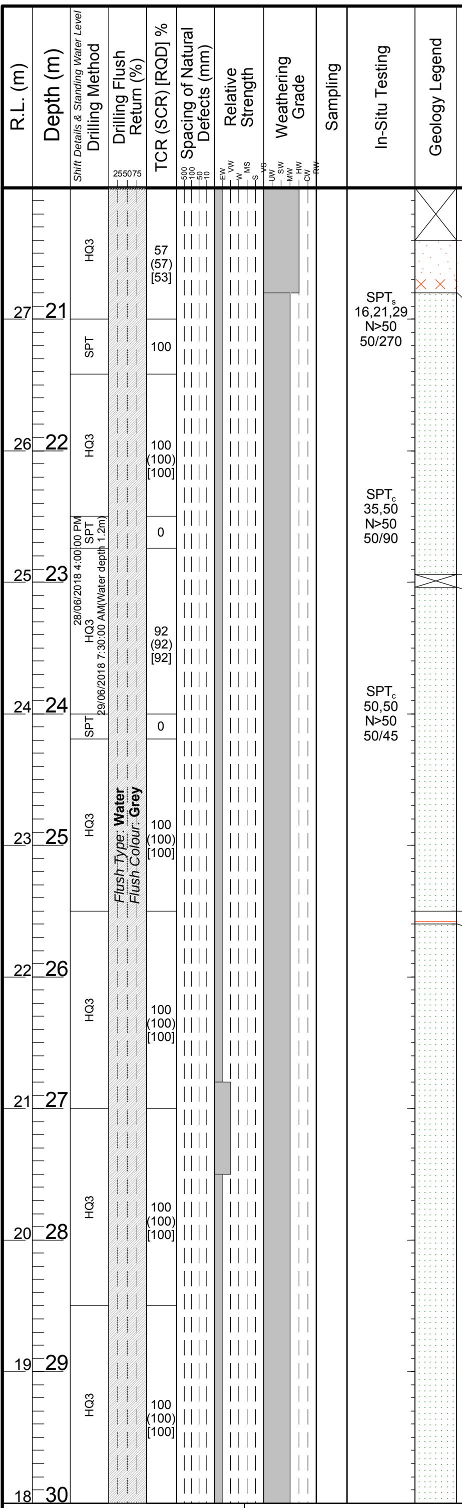
Data Template: AE04725 CI MASTER (NEW TEMPLATE).GPJ Output Form: COMPILATION BOREHOLE Project File Name: AE04725 CIGI4 ADDITIONAL INVESTIGATION.GPJ 7/9/18

Version CI 1.10 09/07/2015 - R.Roberts

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Project: **Grey Lynn Tunnel CIGI5**	**Borehole**
Location: **1A Fisherton St, Grey Lynn** Project No: **AE04725**	Hole ID: **CIE-BH06**
Client: **Watercare**	Date: **27/06/2018**



Description of Strata

BOUNDARY
——— Well defined
— — — Gradational
· · · · · · Poorly defined

CORE LOSS.

Highly weathered, dark grey, interbedded, fine grained SANDSTONE and MUDSTONE. Extremely weak, moderately inclined. Silty SAND, some clay. Very dense, moist.
20.45m to 20.50m: Thin, black carbonaceous bed.

Moderately weathered, dark grey, indistinctly bedded, fine grained SANDSTONE. Extremely weak, moderately inclined. Silty SAND, some clay. Very dense, moist.
21.20m: Thin, black carbonaceous bed.

22.30m: Thin, black carbonaceous bed.

CORE LOSS.

Moderately weathered, dark grey, indistinctly bedded, fine grained SANDSTONE. Extremely weak, moderately inclined. Silty SAND, some clay. Very dense, moist.

24.59m: Thin, black carbonaceous bed.

Moderately weathered, dark grey, MUDSTONE. Extremely weak.

Moderately weathered, dark grey, indistinctly bedded, fine grained SANDSTONE. Extremely weak, steeply inclined. Trace red flecks. Silty SAND, some clay. Very dense, moist.

26.80m to 27.50m: Very weak.

Defect Description

TYPE		SURFACE		APERTURE	
CS	Clay seam	C	Clean	T	0mm
C	Clevage	Mc	Mineral coat	Vn	0-2mm
CR	Crushed zone	Si	Soil infill	N	2-6mm
DZ	Decomposed zone	Sn	Surface stain	Mn	6-20mm
DB	Drilling induced fracture	V	Veneer	Mw	20-60mm
FL	Foliation			W	60-200mm
FZ	Fracture zone	*PLANARITY*		Vw	>200mm
IF	Incipient fracture	P	Planar		
JT	Joint	St	Stepped		
SC	Schistosity	U	Undulating		
SH	Shear				
SZ	Shear Zone	*ROUGHNESS*			
SL	Sill	R	Rough		
VN	Vein	Ss	Slickensided		
VD	Void	Sm	Smooth		

20.55: Jt 30° R, U, Vn, C.

22.30: Jt 70° R, U, Vn, C.

Comments

Geological Unit: Wwnc(Contd.)
Wuw

Backfill / Installation

Started:	27/06/2018
Finished:	3/07/2018
Driller:	McMillan
Plant:	Rig N102 (McMillan)
Logged:	S. Burgess
Checked:	LD

Groundwater Observations

No.	Struck (m)	Date	Standing (m)	Observations
1	1.8	10/07/2018		
2	2.1	28/08/2018		

Remarks
Packer Test 1 at 27.00-30.00m, Packer Test 2 at 50.25-52.50m, Packer Test 3 at 56.25-58.50m, Packer Test 4 at 54.5-63.50m.
Piezometer dipped 10/07/2018; water level 1.8mbgl.
Joint angles are relative to the core axis. If a borehole is true vertical; horizontal=90, vertical=0.
Hole location is in NZTM projection. Elevation is relative to Auckland Vertical Datum 1946
Hole location determined by Survey.

Co-ordinates:
5919179.64mN
1754450.92mE
Elevation: 48.00mRL
Inclination: -90°

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Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Borehole

Project: **Grey Lynn Tunnel CIGI5**

Location: **1A Fisherton St, Grey Lynn**

Project No: **AE04725**

Hole ID: **CIE-BH06**

Client: **Watercare**

Date: **27/06/2018**

R.L. (m)	Depth (m)	Shift Details & Standing Water Level / Drilling Method	TCR (SCR) [RQD] %
17	31	29/06/2018 1:45:00 PM(Water depth 0.6m) HQ3 2/07/2018 7:30:00 AM(Water depth 1.05m)	100 (100) [100]
16	32	HQ3	100 (100) [100]
15	33		
14	34	HQ3	100 (100) [100]
13	35	HQ3	100 (100) [100]
12	36		
11	37	HQ3	100 (100) [100]
10	38	HQ3	100 (100) [100]
9	39		
8	40	HQ3	100 (100) [100]

Drilling Flush Return (%): 25 50 75 — *Flush Type: Water* / *Flush Colour: Grey*

Spacing of Natural Defects (mm): 500 100 50 10

Relative Strength: EW VW W MS S VS

Weathering Grade: UW SW MW HW CW RW

Sampling | In-Situ Testing | Geology Legend | GroundWater

BOUNDARY
- Well defined
- Gradational
- Poorly defined

Description of Strata	Geological Unit
30.00m: Becomes slightly weathered, very weak. Trace black clasts (2mm) *32.75m to 32.80m: Thin, steeply inclined, black carbonaceous bed.*	Wuw(Contd.)
Slightly weathered, dark grey, MUDSTONE. Very weak, steeply inclined. *34.66m to 34.76m: Laminated, steeply inclined, black carbonaceous beds.*	Wuw(Contd.)
Slightly weathered, dark grey, homogeneous, fine grained SANDSTONE. Very weak. Trace red flecks, trace black clasts (2mm)	Wuw(Contd.)
Slightly weathered, dark grey, MUDSTONE. Very weak, steeply inclined.	Wuw(Contd.)
Slightly weathered, dark grey, homogeneous, fine grained SANDSTONE. Very weak. Trace red flecks, trace black clasts (2mm)	Wuw(Contd.)
Slightly weathered, dark grey, homogeneous, fine to medium grained SANDSTONE. Very weak. Trace red flecks, trace light and dark grey clasts (2mm)	Wpvc
Slightly weathered, dark grey, homogeneous, fine grained SANDSTONE. Very weak. Trace red flecks, trace black clasts (2mm)	Wuw
Slightly weathered, dark grey, homogeneous, fine to medium grained SANDSTONE. Very weak. Trace red flecks, trace light and dark grey clasts (2mm)	Wpvc
Slightly weathered, dark grey, homogeneous, fine grained SANDSTONE. Very weak. Trace red flecks, trace black clasts (2mm)	Wuw

Defect Description

TYPE		*SURFACE*		*APERTURE*	
CS	Clay seam	C	Clean	T	0mm
C	Clevage	Mc	Mineral coat	Vn	0-2mm
CR	Crushed zone	Si	Soil infill	N	2-6mm
DZ	Decomposed zone	Sn	Surface stain	Mn	6-20mm
DB	Drilling induced fracture	V	Veneer	Mw	20-60mm
FL	Foliation			W	60-200mm
FZ	Fracture zone	*PLANARITY*		Vw	>200mm
IF	Incipient fracture	P	Planar		
JT	Joint	St	Stepped		
SC	Schistosity	U	Undulating		
SH	Shear				
SZ	Shear Zone	*ROUGHNESS*			
SL	Sill	R	Rough		
VN	Vein	Ss	Slickensided		
VD	Void	Sm	Smooth		

Comments | Geological Unit | Backfill / Installation

Started:	27/06/2018
Finished:	3/07/2018
Driller:	McMillan
Plant:	Rig N102 (McMillan)
Logged:	S. Burgess
Checked:	LD

Groundwater Observations

No.	*Struck (m)*	*Date*	*Standing (m)*	*Observations*
1	1.8	10/07/2018		
2	2.1	28/08/2018		

Remarks
Packer Test 1 at 27.00-30.00m, Packer Test 2 at 50.25-52.50m, Packer Test 3 at 56.25-58.50m, Packer Test 4 at 54.5-63.50m.
Piezometer dipped 10/07/2018; water level 1.8mbgl.
Joint angles are relative to the core axis. If a borehole is true vertical; horizontal=90, vertical=0.
Hole location is in NZTM projection. Elevation is relative to Auckland Vertical Datum 1946
Hole location determined by Survey.

Co-ordinates:
5919179.64mN
1754450.92mE
Elevation: 48.00mRL
Inclination: -90°

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Data Template: AE04725 CI MASTER (NEW TEMPLATE).GPJ Output Form: COMPILATION BOREHOLE Project File Name: AE04725 CIGI4 ADDITIONAL INVESTIGATION.GPJ 7/9/18

Version CI 1.10 09/07/2015 - R.Roberts

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Project: **Grey Lynn Tunnel CIGI5**

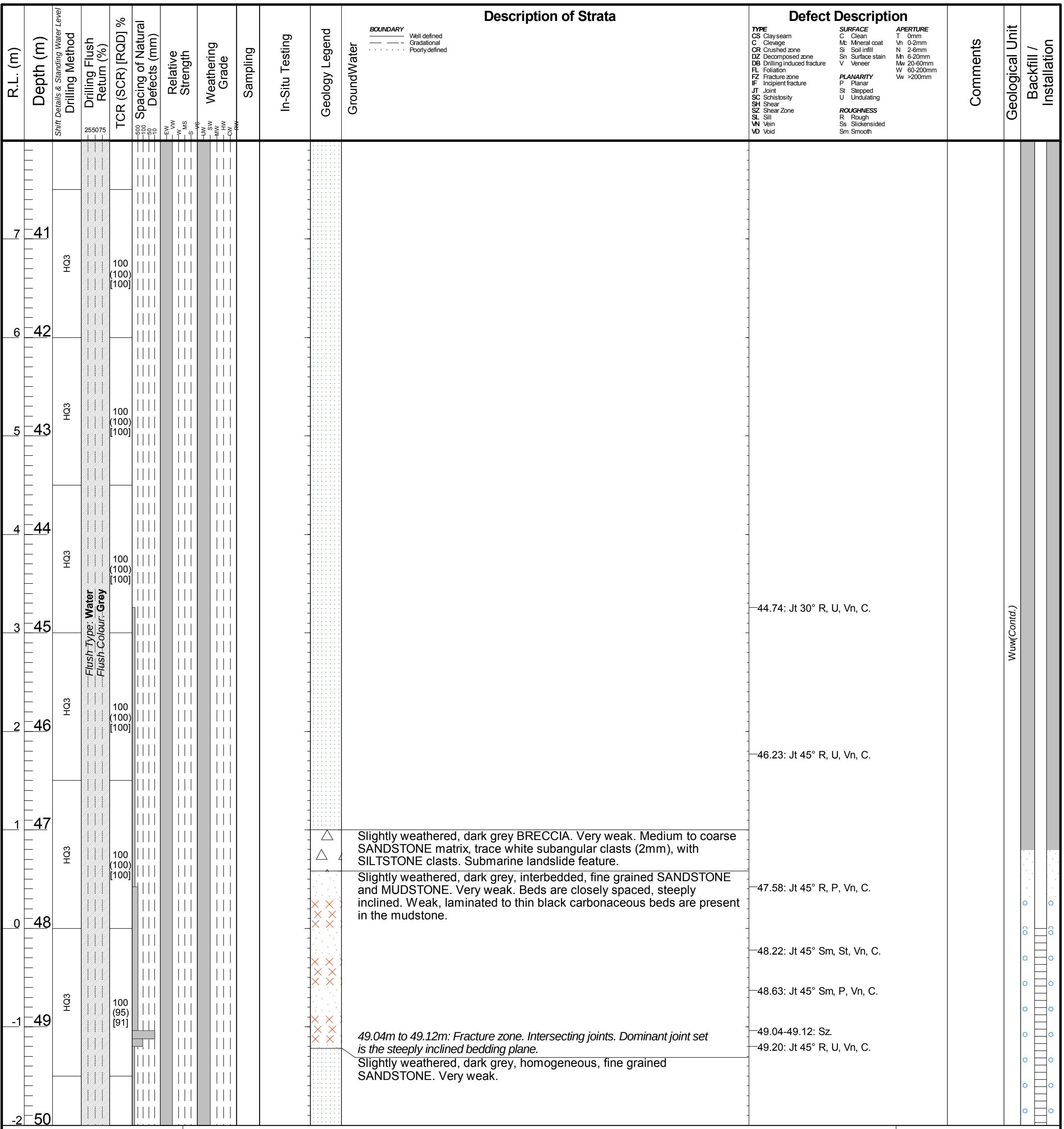
Borehole

Location: **1A Fisherton St, Grey Lynn** Project No: **AE04725**

Hole ID: **CIE-BH06**

Client: **Watercare**

Date: **27/06/2018**



Defect Description

TYPE		SURFACE		APERTURE	
CS	Clay seam	C	Clean	T	0mm
C	Clevage	Mc	Mineral coat	Vn	0-2mm
CR	Crushed zone	Si	Soil infill	N	2-6mm
DZ	Decomposed zone	Sn	Surface stain	Mn	6-20mm
DB	Drilling induced fracture	V	Veneer	Mw	20-60mm
FL	Foliation			W	60-200mm
FZ	Fracture zone	*PLANARITY*		Vw	>200mm
IF	Incipient fracture	P	Planar		
JT	Joint	St	Stepped		
SC	Schistosity	U	Undulating		
SH	Shear				
SZ	Shear Zone	*ROUGHNESS*			
SL	Sill	R	Rough		
VN	Vein	Ss	Slickensided		
VD	Void	Sm	Smooth		

Description of Strata

Slightly weathered, dark grey BRECCIA. Very weak. Medium to coarse SANDSTONE matrix, trace white subangular clasts (2mm), with SILTSTONE clasts. Submarine landslide feature.

Slightly weathered, dark grey, interbedded, fine grained SANDSTONE and MUDSTONE. Very weak. Beds are closely spaced, steeply inclined. Weak, laminated to thin black carbonaceous beds are present in the mudstone.

49.04m to 49.12m: Fracture zone. Intersecting joints. Dominant joint set is the steeply inclined bedding plane.

Slightly weathered, dark grey, homogeneous, fine grained SANDSTONE. Very weak.

Defect Description

44.74: Jt 30° R, U, Vn, C.
46.23: Jt 45° R, U, Vn, C.
47.58: Jt 45° R, P, Vn, C.
48.22: Jt 45° Sm, St, Vn, C.
48.63: Jt 45° Sm, P, Vn, C.
49.04-49.12: Sz.
49.20: Jt 45° R, U, Vn, C.

Started:	27/06/2018
Finished:	3/07/2018
Driller:	McMillan
Plant:	Rig N102 (McMillan)
Logged:	S. Burgess
Checked:	LD

Groundwater Observations

No.	Struck (m)	Date	Standing (m)	Observations
1	1.8	10/07/2018		
2	2.1	28/08/2018		

Remarks

Packer Test 1 at 27.00-30.00m, Packer Test 2 at 50.25-52.50m, Packer Test 3 at 56.25-58.50m, Packer Test 4 at 54.5-63.50m.
Piezometer dipped 10/07/2018; water level 1.8mbgl.
Joint angles are relative to the core axis. If a borehole is true vertical; horizontal=90, vertical=0.
Hole location is in NZTM projection. Elevation is relative to Auckland Vertical Datum 1946
Hole location determined by Survey.

Co-ordinates:
5919179.64mN
1754450.92mE
Elevation: 48.00mRL
Inclination: -90°

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Project: **Grey Lynn Tunnel CIGI5**

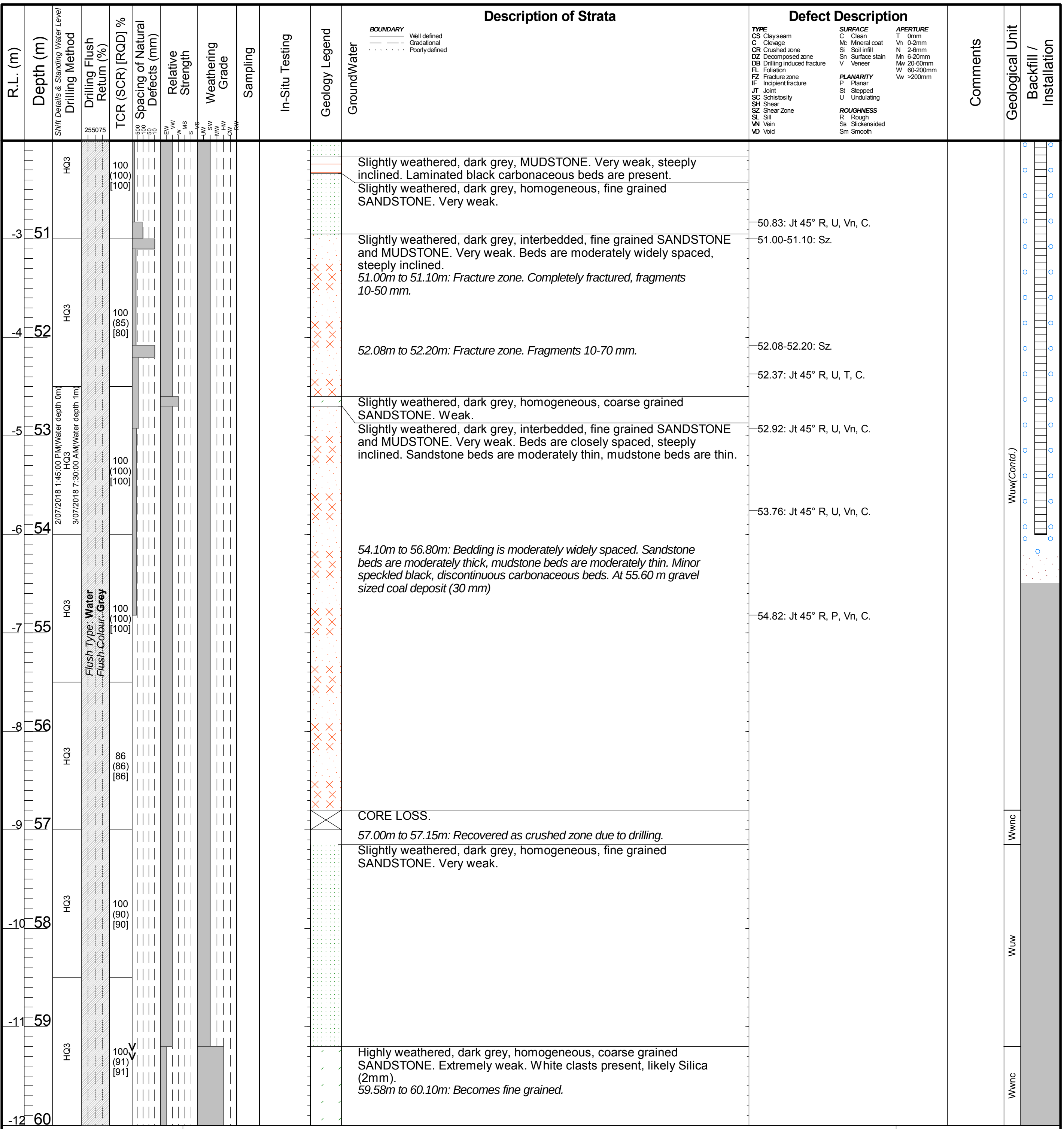
Borehole

Location: **1A Fisherton St, Grey Lynn** Project No: **AE04725**

Hole ID: **CIE-BH06**

Client: **Watercare**

Date: **27/06/2018**



Description of Strata

BOUNDARY
Well defined
Gradational
Poorly defined

Slightly weathered, dark grey, MUDSTONE. Very weak, steeply inclined. Laminated black carbonaceous beds are present.

Slightly weathered, dark grey, homogeneous, fine grained SANDSTONE. Very weak.

Slightly weathered, dark grey, interbedded, fine grained SANDSTONE and MUDSTONE. Very weak. Beds are moderately widely spaced, steeply inclined.
51.00m to 51.10m: Fracture zone. Completely fractured, fragments 10-50 mm.

52.08m to 52.20m: Fracture zone. Fragments 10-70 mm.

Slightly weathered, dark grey, homogeneous, coarse grained SANDSTONE. Weak.

Slightly weathered, dark grey, interbedded, fine grained SANDSTONE and MUDSTONE. Very weak. Beds are closely spaced, steeply inclined. Sandstone beds are moderately thin, mudstone beds are thin.

54.10m to 56.80m: Bedding is moderately widely spaced. Sandstone beds are moderately thick, mudstone beds are moderately thin. Minor speckled black, discontinuous carbonaceous beds. At 55.60 m gravel sized coal deposit (30 mm)

CORE LOSS.
57.00m to 57.15m: Recovered as crushed zone due to drilling.

Slightly weathered, dark grey, homogeneous, fine grained SANDSTONE. Very weak.

Highly weathered, dark grey, homogeneous, coarse grained SANDSTONE. Extremely weak. White clasts present, likely Silica (2mm).
59.58m to 60.10m: Becomes fine grained.

Defect Description

TYPE		SURFACE		APERTURE	
CS	Clay seam	C	Clean	T	0mm
C	Clevage	Mc	Mineral coat	Vn	0-2mm
CR	Crushed zone	Si	Soil infill	N	2-6mm
DZ	Decomposed zone	Sn	Surface stain	Mn	6-20mm
DB	Drilling induced fracture	V	Veneer	Mw	20-60mm
FL	Foliation			W	60-200mm
FZ	Fracture zone	**PLANARITY**		Vw	>200mm
IF	Incipient fracture	P	Planar		
JT	Joint	St	Stepped		
SC	Schistosity	U	Undulating		
SH	Shear				
SZ	Shear Zone	**ROUGHNESS**			
SL	Sill	R	Rough		
VN	Vein	Ss	Slickensided		
VD	Void	Sm	Smooth		

50.83: Jt 45° R, U, Vn, C.
51.00-51.10: Sz.
52.08-52.20: Sz.
52.37: Jt 45° R, U, T, C.
52.92: Jt 45° R, U, Vn, C.
53.76: Jt 45° R, U, Vn, C.
54.82: Jt 45° R, P, Vn, C.

Started:	27/06/2018
Finished:	3/07/2018
Driller:	McMillan
Plant:	Rig N102 (McMillan)
Logged:	S. Burgess
Checked:	LD

Groundwater Observations

No.	Struck (m)	Date	Standing (m)	Observations
1	1.8	10/07/2018		
2	2.1	28/08/2018		

Remarks
Packer Test 1 at 27.00-30.00m, Packer Test 2 at 50.25-52.50m, Packer Test 3 at 56.25-58.50m, Packer Test 4 at 54.5-63.50m.
Piezometer dipped 10/07/2018; water level 1.8mbgl.
Joint angles are relative to the core axis. If a borehole is true vertical; horizontal=90, vertical=0.
Hole location is in NZTM projection. Elevation is relative to Auckland Vertical Datum 1946
Hole location determined by Survey.

Co-ordinates:
5919179.64mN
1754450.92mE
Elevation: 48.00mRL
Inclination: -90°

See key sheet for an explanation of symbols and abbreviations. Material descriptions as per NZGS Guidelines - December 2005.

Jacobs in association with
AECOM and McMillen Jacobs Associates

Preliminary Log of Investigation

Project: **Grey Lynn Tunnel CIGI5**

Borehole

Location: **1A Fisherton St, Grey Lynn** Project No: **AE04725**

Hole ID: **CIE-BH06**

Client: **Watercare**

Date: **27/06/2018**

R.L. (m)	Depth (m)	Shift Details & Standing Water Level Drilling Method	Drilling Flush Return (%) 255075	TCR (SCR) [RQD] %	Spacing of Natural Defects (mm) 500 100 50 10	Relative Strength EW VW W MS S VS	Weathering Grade UW SW MW HW CW RW	Sampling	In-Situ Testing	Geology Legend	GroundWater	Description of Strata	Defect Description	Comments	Geological Unit	Backfill / Installation
		HQ3	*Flush Type:* **Water** *Flush Colour:* **Grey**	100 (100) [100]								Slightly weathered, dark grey, homogeneous, fine grained SANDSTONE. Very weak. Widely spaced thin black carbonaceous beds.			Ww	
-13	61															
		HQ3		100 (100) [100]												
-14	62															
-15	63															
		HQ3		100 (100) [100]												

3/07/2018 4:00:00 PM

[illegible] (Water depth [illegible]58m)

CIE-BH06 terminated at 63.50m. Target Depth

BOUNDARY
Well defined
Gradational
Poorly defined

TYPE		SURFACE		APERTURE	
CS	Clay seam	C	Clean	T	0mm
C	Clevage	Mc	Mineral coat	Vn	0-2mm
CR	Crushed zone	Si	Soil infill	N	2-6mm
DZ	Decomposed zone	Sn	Surface stain	Mn	6-20mm
DB	Drilling induced fracture	V	Veneer	Mw	20-60mm
FL	Foliation			W	60-200mm
FZ	Fracture zone	*PLANARITY*		Vw	>200mm
IF	Incipient fracture	P	Planar		
JT	Joint	St	Stepped		
SC	Schistosity	U	Undulating		
SH	Shear				
SZ	Shear Zone	*ROUGHNESS*			
SL	Sill	R	Rough		
VN	Vein	Ss	Slickensided		
VD	Void	Sm	Smooth		

Started:	27/06/2018
Finished:	3/07/2018
Driller:	McMillan
Plant:	Rig N102 (McMillan)
Logged:	S. Burgess
Checked:	LD

Groundwater Observations

No.	Struck (m)	Date	Standing (m)	Observations
1	1.8	10/07/2018		
2	2.1	28/08/2018		

Remarks

Packer Test 1 at 27.00-30.00m, Packer Test 2 at 50.25-52.50m, Packer Test 3 at 56.25-58.50m, Packer Test 4 at 54.5-63.50m.
Piezometer dipped 10/07/2018; water level 1.8mbgl.
Joint angles are relative to the core axis. If a borehole is true vertical; horizontal=90, vertical=0.
Hole location is in NZTM projection. Elevation is relative to Auckland Vertical Datum 1946
Hole location determined by Survey.

Co-ordinates:
5919179.64mN
1754450.92mE
Elevation: 48.00mRL
Inclination: -90°

Appendix B. Slug Test Analyses

Borehole Variable Head Permeability Test

Borehole ID

BH4_test 1

Project Details

Project Name	Grey Lynn Tunnel
Project Number	WWA047
Test Date	17/7/1018
Tested	JNS
Checked	

Test Parameters

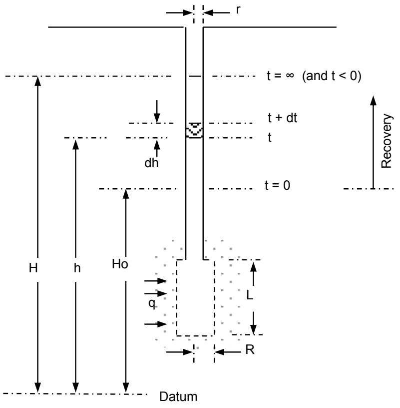
Top of screen	25.000 m
Bottom of Screen	31.500 m
Screen Length, *L*	6.500 m
Static Water Level, *H*	1.030 m
Initial Water Level, H_0	1.300 m
Hole Radius, *R*	0.060 m
Casing Radius, *r*	0.050 m

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

Result

Hydraulic Conductivity
***K* = 1.10E-06 m/s**

Test Schematic



Hvorslev (1951) method:

$$F = \frac{2\pi L}{\ln(L / R)}$$

$$K = \frac{\pi r^2}{FT_0}$$

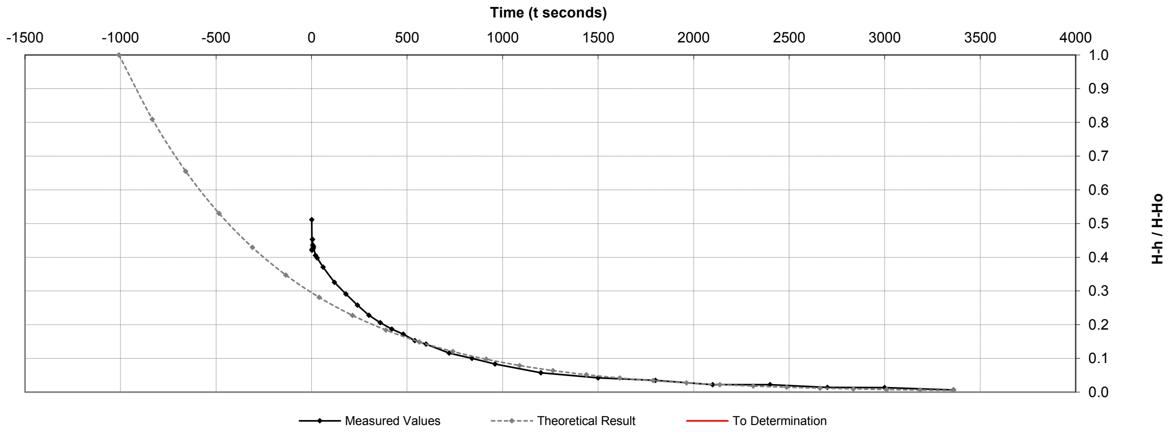
Calibrated Parameters

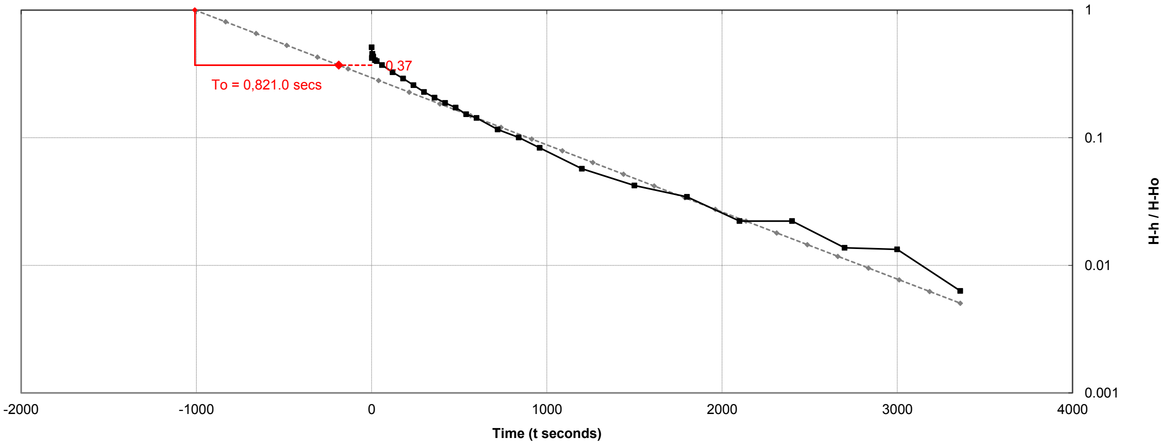
Intake factor, *F*	8.72
Time Factor, T_0	821.0

Measured Data

Time (Secs)	Depth (h) (m)	H - h (m)	H-h / H-H0
1	1.168	0.14	0.51
2	1.144	0.11	0.42
3	1.144	0.11	0.42
4	1.152	0.12	0.45
5	1.152	0.12	0.45
6	1.148	0.12	0.44
7	1.145	0.12	0.43
8	1.145	0.12	0.43
9	1.147	0.12	0.43
10	1.146	0.12	0.43
20	1.140	0.11	0.41
30	1.137	0.11	0.40
60	1.130	0.10	0.37
120	1.118	0.09	0.33
180	1.109	0.08	0.29
240	1.100	0.07	0.26
300	1.092	0.06	0.23
360	1.086	0.06	0.21
420	1.081	0.05	0.19
480	1.076	0.05	0.17
540	1.071	0.04	0.15
600	1.069	0.04	0.14
720	1.061	0.03	0.12
840	1.057	0.03	0.10
960	1.053	0.02	0.08
1200	1.045	0.02	0.06
1500	1.041	0.01	0.04
1800	1.039	0.01	0.03
2100	1.036	0.01	0.02
2400	1.036	0.01	0.02
2700	1.034	0.00	0.01
3000	1.034	0.00	0.01
3360	1.032	0.00	0.01

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





Note: Hvorslev method is based on the slope of the best-fit line. This is calculated by taking the elapsed time, T_0, over 1 natural log interval of the graph, i.e. normalised head between 1 and 0.37.

Borehole Variable Head Permeability Test

Borehole ID

BH4_test 2

Project Details

Project Name	Grey Lynn Tunnel
Project Number	WWA047
Test Date	17/7/1018
Tested	JNS
Checked	

Test Parameters

Top of screen	25.000 m
Bottom of Screen	31.500 m
Screen Length, *L*	6.500 m
Static Water Level, *H*	1.030 m
Initial Water Level, H_0	1.300 m
Hole Radius, *R*	0.060 m
Casing Radius, *r*	0.050 m

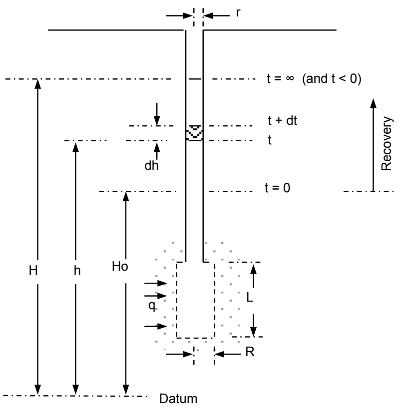
Note: If the datum is above the hole, the height/depth readings do not have to be negative numbers - as long as they are either all negative or all positive, the answer will be correct.

Result

Hydraulic Conductivity

***K* = 1.07E-06 m/s**

Test Schematic



Hvorslev (1951) method:

$$F = \frac{2\pi L}{\ln(L/R)}$$

$$K = \frac{\pi r^2}{F T_0}$$

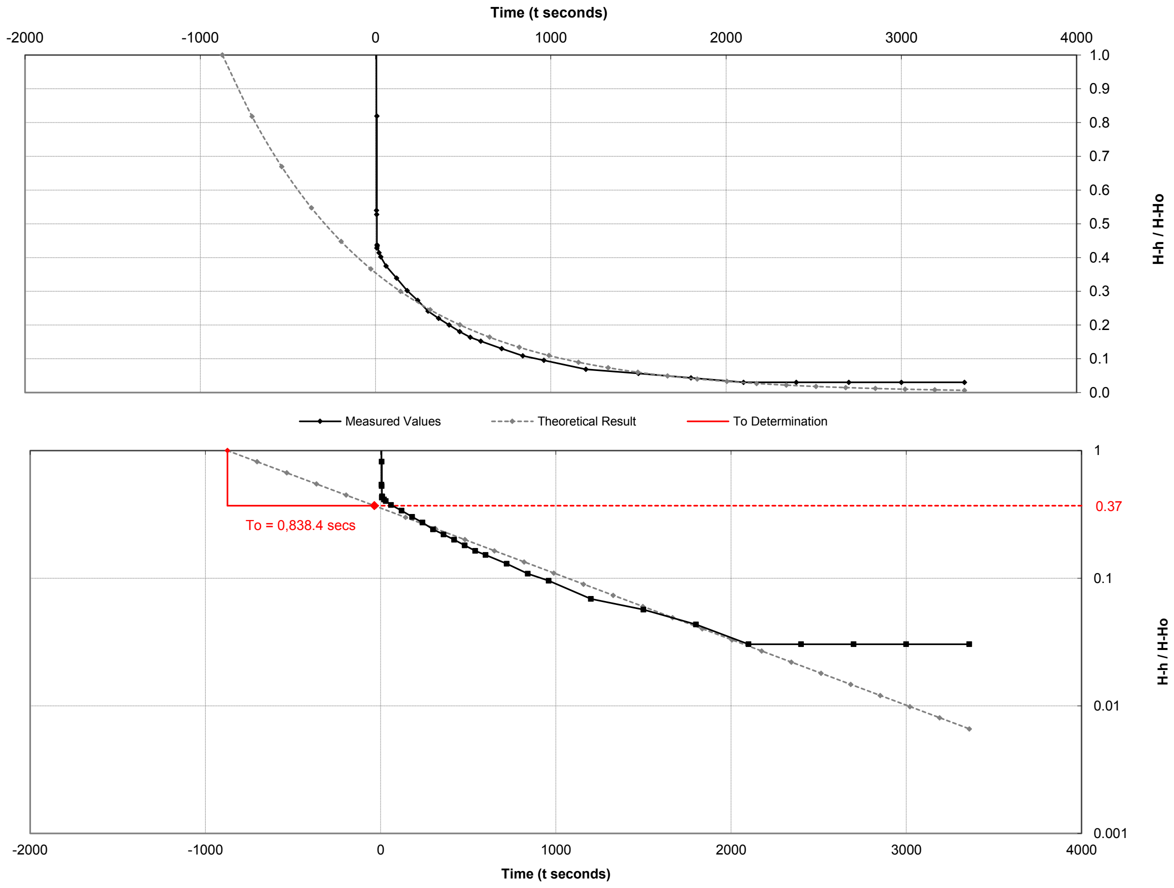
Calibrated Parameters

Intake factor, *F*	8.72
Time Factor, T_0	838.4

Measured Data

Time (Secs)	Depth (h) (m)	H - h (m)	H-h / H-H0
1	2.823	1.79	6.64
2	2.765	1.74	6.43
3	2.700	1.67	6.19
4	1.686	0.66	2.43
5	1.176	0.15	0.54
6	1.172	0.14	0.53
7	1.251	0.22	0.82
8	1.146	0.12	0.43
9	1.148	0.12	0.44
10	1.147	0.12	0.43
20	1.142	0.11	0.41
30	1.139	0.11	0.40
60	1.131	0.10	0.37
120	1.122	0.09	0.34
180	1.112	0.08	0.30
240	1.104	0.07	0.27
300	1.095	0.07	0.24
360	1.089	0.06	0.22
420	1.084	0.05	0.20
480	1.079	0.05	0.18
540	1.074	0.04	0.16
600	1.071	0.04	0.15
720	1.065	0.04	0.13
840	1.059	0.03	0.11
960	1.056	0.03	0.10
1200	1.049	0.02	0.07
1500	1.045	0.02	0.06
1800	1.042	0.01	0.04
2100	1.038	0.01	0.03
2400			
2700			
3000			
3360			

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



Note: Hvorslev method is based on the slope of the best-fit line. This is calculated by taking the elapsed time, T_0, over 1 natural log interval of the graph, i.e. normalised head between 1 and 0.37.

Borehole Variable Head Permeability Test

Borehole ID

BH4_test 3

Project Details

Project Name	Grey Lynn Tunnel
Project Number	WWA047
Test Date	17/7/1018
Tested	JNS
Checked	

Test Parameters

Top of screen	25.000 m
Bottom of Screen	31.500 m
Screen Length, *L*	6.500 m
Static Water Level, *H*	1.030 m
Initial Water Level, H_0	1.300 m
Hole Radius, *R*	0.060 m
Casing Radius, *r*	0.050 m

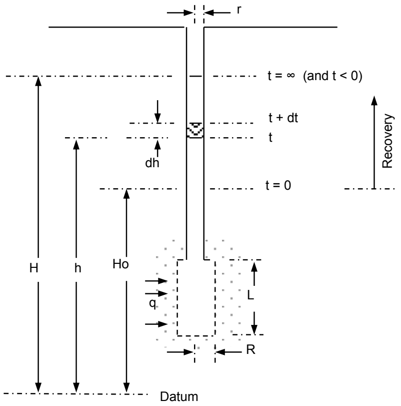
Note: If the datum is above the hole, the height/depth readings do not have to be negative numbers - as long as they are either all negative or all positive, the answer will be correct.

Result

Hydraulic Conductivity

***K* = 1.04E-06 m/s**

Test Schematic



Hvorslev (1951) method:

$$F = \frac{2\pi L}{\ln(L/R)}$$

$$K = \frac{\pi r^2}{FT_0}$$

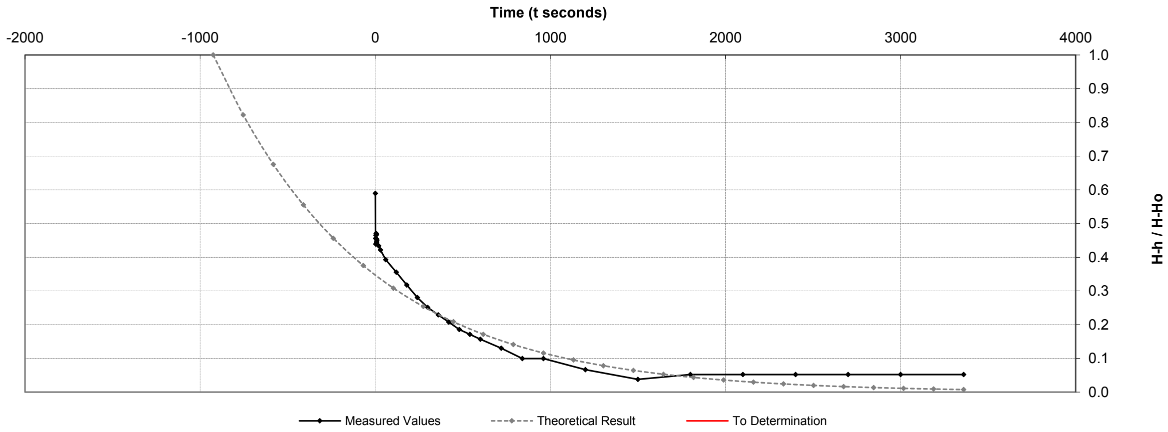
Calibrated Parameters

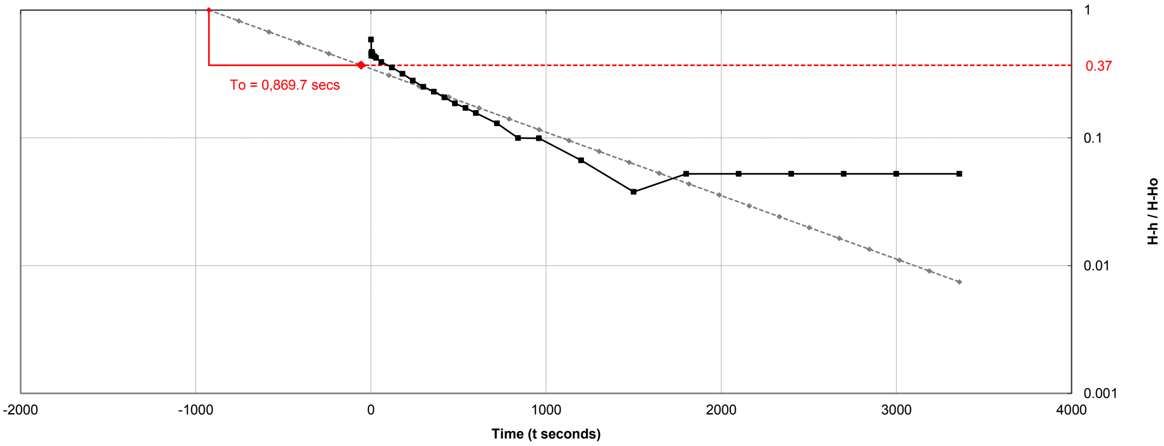
Intake factor, *F*	8.72
Time Factor, T_0	869.7

Measured Data

Time (Secs)	Depth (h) (m)	H - h (m)	H-h / H-H0
1	1.189	0.16	0.59
2	1.153	0.12	0.46
3	1.149	0.12	0.44
4	1.156	0.13	0.46
5	1.157	0.13	0.47
6	1.156	0.13	0.47
7	1.150	0.12	0.45
8	1.150	0.12	0.44
9	1.152	0.12	0.45
10	1.152	0.12	0.45
20	1.147	0.12	0.43
30	1.144	0.11	0.42
60	1.136	0.11	0.39
120	1.126	0.10	0.36
180	1.116	0.09	0.32
240	1.106	0.08	0.28
300	1.098	0.07	0.25
360	1.092	0.06	0.23
420	1.086	0.06	0.21
480	1.080	0.05	0.19
540	1.076	0.05	0.17
600	1.072	0.04	0.16
720	1.065	0.04	0.13
840	1.057	0.03	0.10
960	1.057	0.03	0.10
1200	1.048	0.02	0.07
1500	1.040	0.01	0.04
1800	1.044	0.01	0.05
2100			
2400			
2700			
3000			
3360			

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





Note: Hvorslev method is based on the slope of the best-fit line. This is calculated by taking the elapsed time, T_0, over 1 natural log interval of the graph, i.e. normalised head between 1 and 0.37.

Borehole Variable Head Permeability Test

Borehole ID

BH5_test 1

Project Details

Project Name	Grey Lynn Tunnel
Project Number	WWA047
Test Date	17/7/1018
Tested	JNS
Checked	

Test Parameters

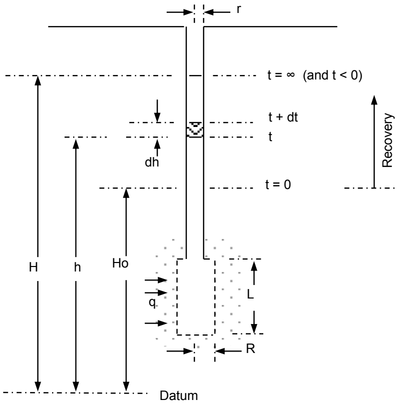
Top of screen	25.000 m
Bottom of Screen	31.500 m
Screen Length, *L*	6.500 m
Static Water Level, *H*	-2.773 m
Initial Water Level, H_0	-2.358 m
Hole Radius, *R*	0.060 m
Casing Radius, *r*	0.050 m

Note: If the datum is above the hole, the height/depth readings do not have to be negative numbers - as long as they are either all negative or all
\

Result

Hydraulic Conductivity
***K* = 1.84E-07 m/s**

Test Schematic



Hvorslev (1951) method:

$$F = \frac{2\pi L}{\ln(L/R)}$$

$$K = \frac{\pi r^2}{FT_0}$$

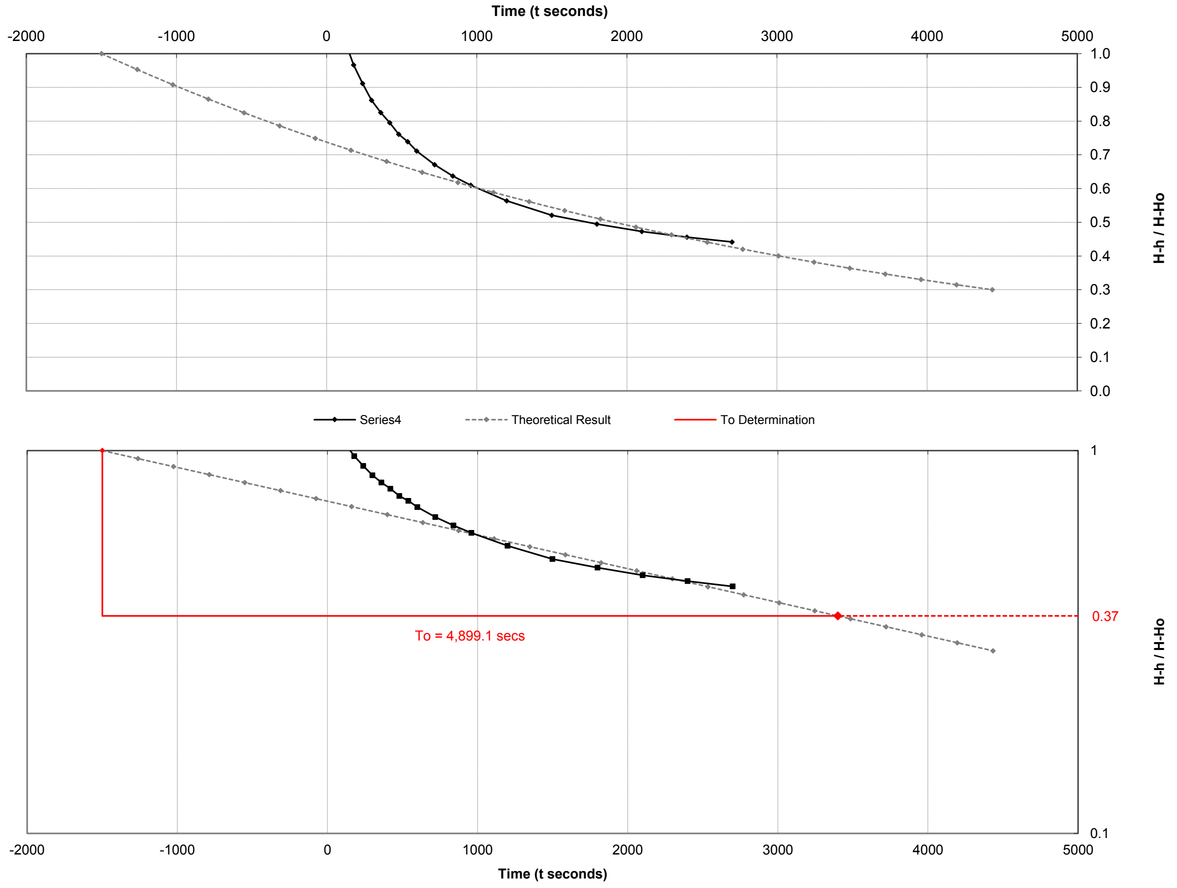
Calibrated Parameters

Intake factor, *F*	8.72
Time Factor, T_0	4,899.1

Measured Data

Time (Secs)	Depth (h) (m)	H - h (m)	H-h / H-H0
1	-2.248	0.53	1.27
2	-2.248	0.52	1.26
3	-2.252	0.52	1.25
4	-2.253	0.52	1.25
5	-2.254	0.52	1.25
6	-2.256	0.52	1.25
7	-2.257	0.52	1.24
8	-2.259	0.51	1.24
9	-2.261	0.51	1.23
10	-2.261	0.51	1.23
20	-2.272	0.50	1.21
30	-2.282	0.49	1.18
60	-2.305	0.47	1.13
120	-2.341	0.43	1.04
180	-2.372	0.40	0.97
240	-2.395	0.38	0.91
300	-2.415	0.36	0.86
360	-2.430	0.34	0.83
420	-2.443	0.33	0.79
480	-2.457	0.32	0.76
540	-2.466	0.31	0.74
600	-2.478	0.30	0.71
720	-2.495	0.28	0.67
840	-2.509	0.26	0.64
960	-2.520	0.25	0.61
1200	-2.539	0.23	0.56
1500	-2.557	0.22	0.52
1800	-2.568	0.21	0.49
2100	-2.577	0.20	0.47
2400	-2.584	0.19	0.46
2700	-2.590	0.18	0.44
3000			
3360			

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



Note: Hvorslev method is based on the slope of the best-fit line. This is calculated by taking the elapsed time, T_0, over 1 natural log interval of the graph, i.e. normalised head between 1 and 0.37.

Borehole Variable Head Permeability Test

Borehole ID

BH5_test 2

Project Details

Project Name	Grey Lynn Tunnel
Project Number	WWA047
Test Date	17/7/1018
Tested	JNS
Checked	

Test Parameters

Top of screen	25.000 m
Bottom of Screen	31.500 m
Screen Length, *L*	6.500 m
Static Water Level, *H*	-2.773 m
Initial Water Level, H_0	-2.358 m
Hole Radius, *R*	0.060 m
Casing Radius, *r*	0.050 m

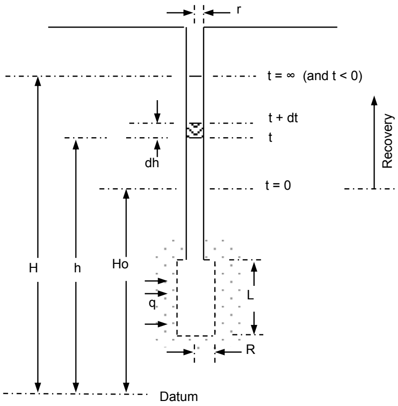
Note: If the datum is above the hole, the height/depth readings do not have to be negative numbers - as long as they are either all negative or all \

Result

Hydraulic Conductivity

***K* = 4.01E-07 m/s**

Test Schematic



Hvorslev (1951) method:

$$F = \frac{2\pi L}{\ln(L/R)}$$

$$K = \frac{\pi r^2}{FT_0}$$

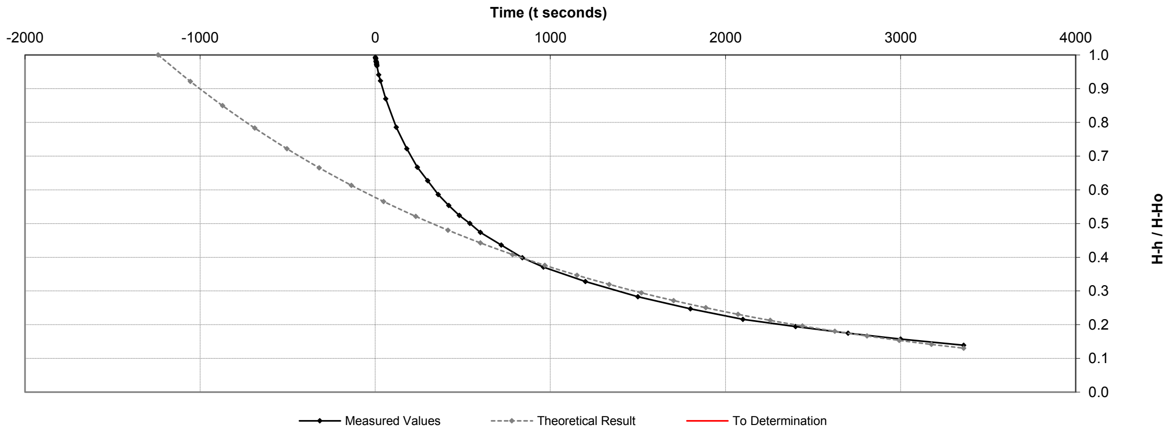
Calibrated Parameters

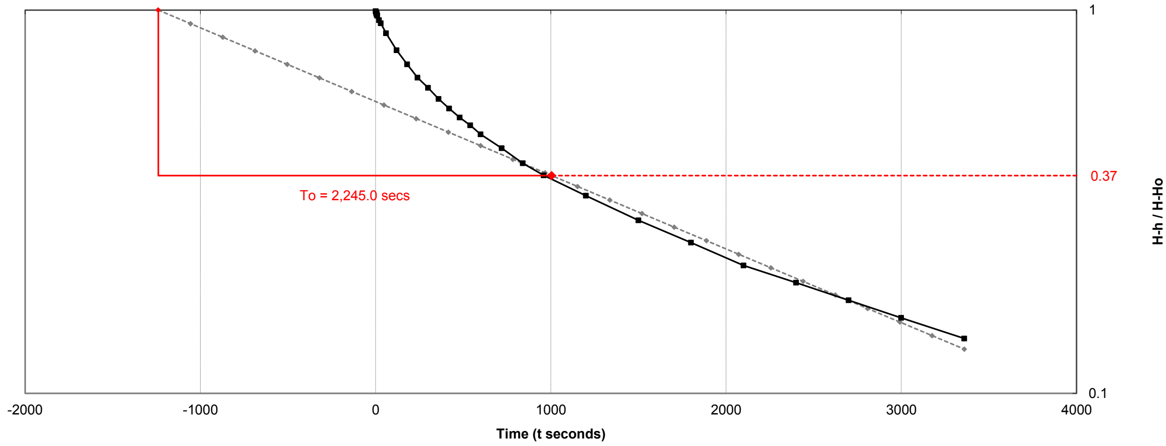
Intake factor, *F*	8.72
Time Factor, T_0	2,245.0

Measured Data

Time (Secs)	Depth (h) (m)	H - h (m)	H-h / H-H0
1	-2.361	0.41	0.99
2	-2.362	0.41	0.99
3	-2.363	0.41	0.99
4	-2.366	0.41	0.98
5	-2.366	0.41	0.98
6	-2.367	0.41	0.98
7	-2.368	0.41	0.98
8	-2.370	0.40	0.97
9	-2.370	0.40	0.97
10	-2.372	0.40	0.97
20	-2.383	0.39	0.94
30	-2.390	0.38	0.92
60	-2.412	0.36	0.87
120	-2.447	0.33	0.79
180	-2.473	0.30	0.72
240	-2.496	0.28	0.67
300	-2.513	0.26	0.63
360	-2.530	0.24	0.59
420	-2.543	0.23	0.55
480	-2.555	0.22	0.52
540	-2.565	0.21	0.50
600	-2.576	0.20	0.47
720	-2.592	0.18	0.44
840	-2.608	0.17	0.40
960	-2.619	0.15	0.37
1200	-2.637	0.14	0.33
1500	-2.656	0.12	0.28
1800	-2.670	0.10	0.25
2100	-2.684	0.09	0.22
2400	-2.692	0.08	0.19
2700	-2.700	0.07	0.17
3000	-2.708	0.07	0.16
3360	-2.715	0.06	0.14

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





Note: Hvorslev method is based on the slope of the best-fit line. This is calculated by taking the elapsed time, T_0, over 1 natural log interval of the graph, i.e. normalised head between 1 and 0.37.

Borehole Variable Head Permeability Test

Borehole ID

BH5_test 3

Project Details

Project Name	Grey Lynn Tunnel
Project Number	WWA047
Test Date	17/7/1018
Tested	JNS
Checked	

Test Parameters

Top of screen	25.000 m
Bottom of Screen	31.500 m
Screen Length, *L*	6.500 m
Static Water Level, *H*	-2.773 m
Initial Water Level, H_0	-2.358 m
Hole Radius, *R*	0.060 m
Casing Radius, *r*	0.050 m

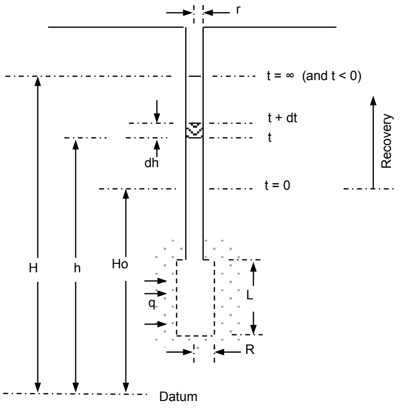
Note: If the datum is above the hole, the height/depth readings do not have to be negative numbers - as long as they are either all negative or all
\

Result

Hydraulic Conductivity

***K* = 3.92E-07 m/s**

Test Schematic



Hvorslev (1951) method:

$$F = \frac{2\pi L}{\ln(L/R)}$$

$$K = \frac{\pi r^2}{FT_0}$$

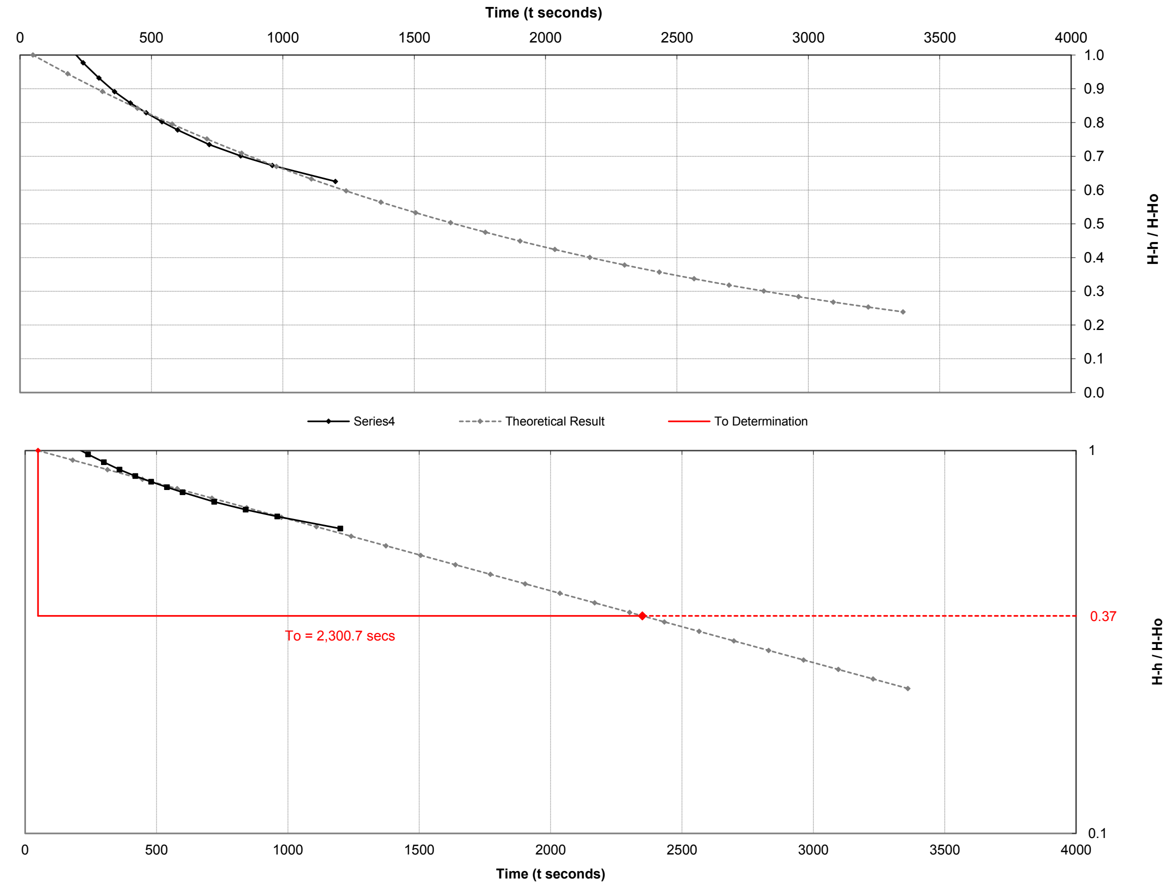
Calibrated Parameters

Intake factor, *F*	8.72
Time Factor, T_0	2,300.7

Measured Data

Time (Secs)	Depth (h) (m)	H - h (m)	H-h / H-H0
1	-2.221	0.55	1.33
2			
3	-2.228	0.55	1.31
4	-2.232	0.54	1.30
5	-2.234	0.54	1.30
6	-2.235	0.54	1.30
7	-2.236	0.54	1.29
8	-2.237	0.54	1.29
9	-2.239	0.53	1.29
10	-2.240	0.53	1.28
20	-2.250	0.52	1.26
30	-2.260	0.51	1.24
60	-2.283	0.49	1.18
120	-2.318	0.45	1.10
180	-2.346	0.43	1.03
240	-2.368	0.41	0.98
300	-2.386	0.39	0.93
360	-2.403	0.37	0.89
420	-2.417	0.36	0.86
480	-2.429	0.34	0.83
540	-2.440	0.33	0.80
600	-2.450	0.32	0.78
720	-2.468	0.30	0.73
840	-2.482	0.29	0.70
960	-2.494	0.28	0.67
1200	-2.513	0.26	0.63
1500			
1800			
2100			
2400			
2700			
3000			
3360			

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



Note: Hvorslev method is based on the slope of the best-fit line. This is calculated by taking the elapsed time, T_0, over 1 natural log interval of the graph, i.e. normalised head between 1 and 0.37.

Borehole Variable Head Permeability Test

Borehole ID

BH6_test 1

Project Details

Project Name	Grey Lynn Tunnel
Project Number	WWA047
Test Date	17/7/1018
Tested	JNS
Checked	

Test Parameters

Top of screen	47.200 m
Bottom of Screen	54.500 m
Screen Length, *L*	7.300 m
Static Water Level, *H*	3.300 m
Initial Water Level, H_0	4.750 m
Hole Radius, *R*	0.050 m
Casing Radius, *r*	0.025 m

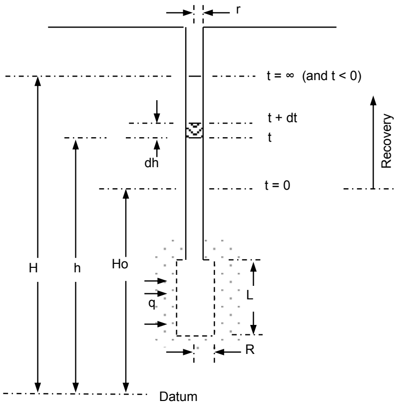
Note: If the datum is above the hole, the height/depth readings do not have to be negative numbers - as long as they are either all negative or all positive, the answer will be correct.

Result

Hydraulic Conductivity

K **= 1.05E-07 m/s**

Test Schematic



Hvorslev (1951) method:

$$F = \frac{2 \pi L}{\ln(L / R)}$$

$$K = \frac{\pi r^2}{F T_0}$$

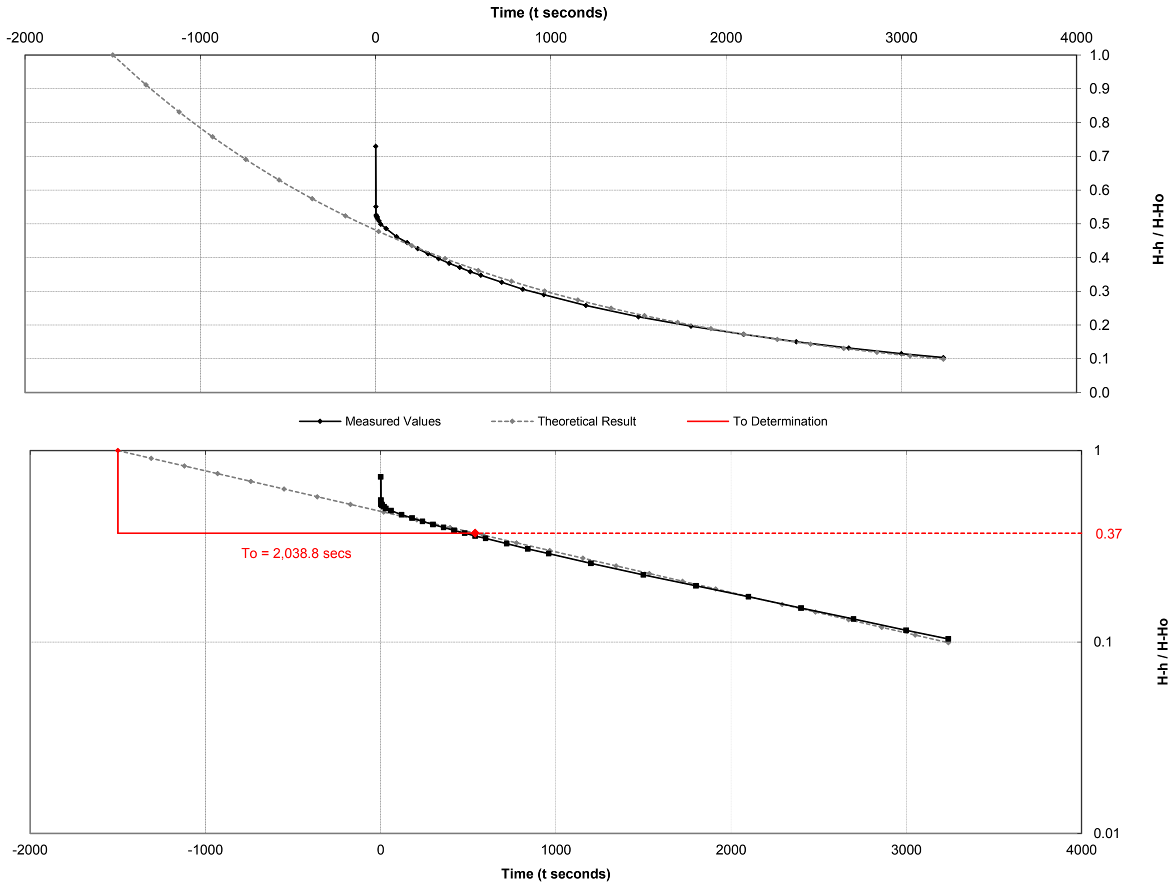
Calibrated Parameters

Intake factor, *F*	9.20
Time Factor, T_0	2,038.8

Measured Data

Time (Secs)	Depth (h) (m)	H - h (m)	H-h / H-H0
1	4.358	1.06	0.73
2	4.099	0.80	0.55
3	4.062	0.76	0.53
4	4.059	0.76	0.52
5	4.061	0.76	0.52
6	4.056	0.76	0.52
7	4.054	0.75	0.52
8	4.057	0.76	0.52
9	4.053	0.75	0.52
10	4.048	0.75	0.52
20	4.037	0.74	0.51
30	4.023	0.72	0.50
60	4.004	0.70	0.49
120	3.970	0.67	0.46
180	3.944	0.64	0.44
240	3.918	0.62	0.43
300	3.897	0.60	0.41
360	3.876	0.58	0.40
420	3.855	0.56	0.38
480	3.837	0.54	0.37
540	3.819	0.52	0.36
600	3.804	0.50	0.35
720	3.774	0.47	0.33
840	3.744	0.44	0.31
960	3.720	0.42	0.29
1200	3.674	0.37	0.26
1500	3.625	0.33	0.22
1800	3.585	0.29	0.20
2100	3.550	0.25	0.17
2400	3.518	0.22	0.15
2700	3.491	0.19	0.13
3000	3.467	0.17	0.12
3240	3.450	0.15	0.10

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



Note: Hvorslev method is based on the slope of the best-fit line. This is calculated by taking the elapsed time, T_0, over 1 natural log interval of the graph, i.e. normalised head between 1 and 0.37.

WILLIAMSON WATER ADVISORY

Borehole Variable Head Permeability Test

Borehole ID

BH6_test 2

Project Details

Project Name	Grey Lynn Tunnel
Project Number	WWA047
Test Date	17/7/1018
Tested	JNS
Checked	

Test Parameters

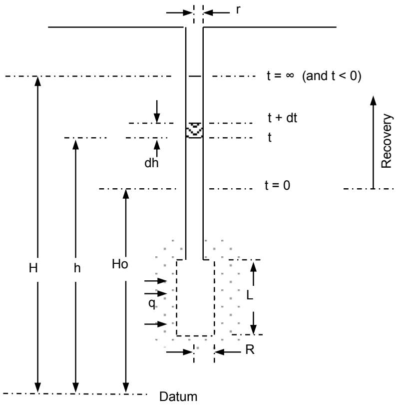
Top of screen	47.200 m
Bottom of Screen	54.500 m
Screen Length, *L*	7.300 m
Static Water Level, *H*	3.300 m
Initial Water Level, H_0	4.750 m
Hole Radius, *R*	0.050 m
Casing Radius, *r*	0.025 m

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

Result

Hydraulic Conductivity
***K* = 1.49E-07 m/s**

Test Schematic



Hvorslev (1951) method:

$$F = \frac{2\pi L}{\ln(L/R)} \qquad K = \frac{\pi r^2}{FT_0}$$

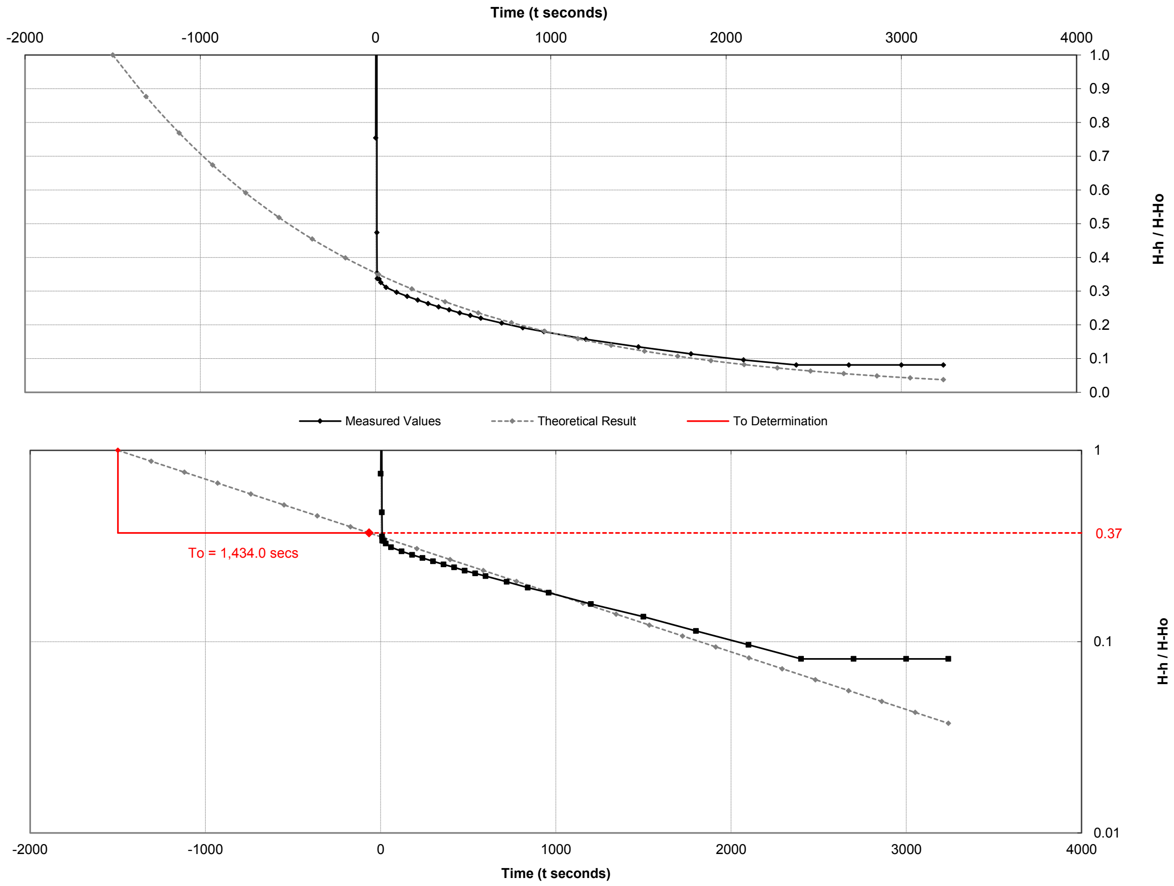
Calibrated Parameters

Intake factor, *F*	9.20
Time Factor, T_0	1,434.0

Measured Data

Time (Secs)	Depth (h) (m)	H - h (m)	H-h / H-H0
1	4.394	1.09	0.75
2	4.960	1.66	1.15
3			
4			
5			
6			
7			
8	3.987	0.69	0.47
9	3.814	0.51	0.35
10	3.789	0.49	0.34
20	3.787	0.49	0.34
30	3.772	0.47	0.33
60	3.752	0.45	0.31
120	3.730	0.43	0.30
180	3.713	0.41	0.28
240	3.697	0.40	0.27
300	3.682	0.38	0.26
360	3.668	0.37	0.25
420	3.655	0.35	0.24
480	3.641	0.34	0.24
540	3.630	0.33	0.23
600	3.619	0.32	0.22
720	3.598	0.30	0.21
840	3.578	0.28	0.19
960	3.561	0.26	0.18
1200	3.528	0.23	0.16
1500	3.496	0.20	0.13
1800	3.465	0.17	0.11
2100	3.440	0.14	0.10
2400	3.418	0.12	0.08
2700			
3000			
3240			

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



Note: Hvorslev method is based on the slope of the best-fit line. This is calculated by taking the elapsed time, T_0, over 1 natural log interval of the graph, i.e. normalised head between 1 and 0.37.

WILLIAMSON
WATER ADVISORY

Borehole Variable Head Permeability Test

Borehole ID

BH6_test 3

Project Details

Project Name	Grey Lynn Tunnel
Project Number	WWA047
Test Date	17/7/1018
Tested	JNS
Checked	

Test Parameters

Top of screen	47.200 m
Bottom of Screen	54.500 m
Screen Length, *L*	7.300 m
Static Water Level, *H*	3.300 m
Initial Water Level, H_0	4.750 m
Hole Radius, *R*	0.050 m
Casing Radius, *r*	0.025 m

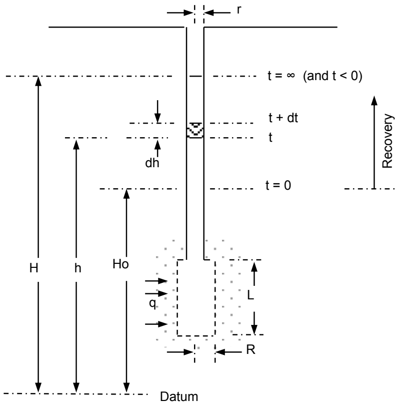
Note: If the datum is above the hole, the height/depth readings do not have to be negative numbers - as long as they are either all negative or all positive, the answer will be correct.

Result

Hydraulic Conductivity

***K* = 1.06E-07 m/s**

Test Schematic



Hvorslev (1951) method:

$$F = \frac{2\pi L}{\ln(L/R)}$$

$$K = \frac{\pi r^2}{FT_0}$$

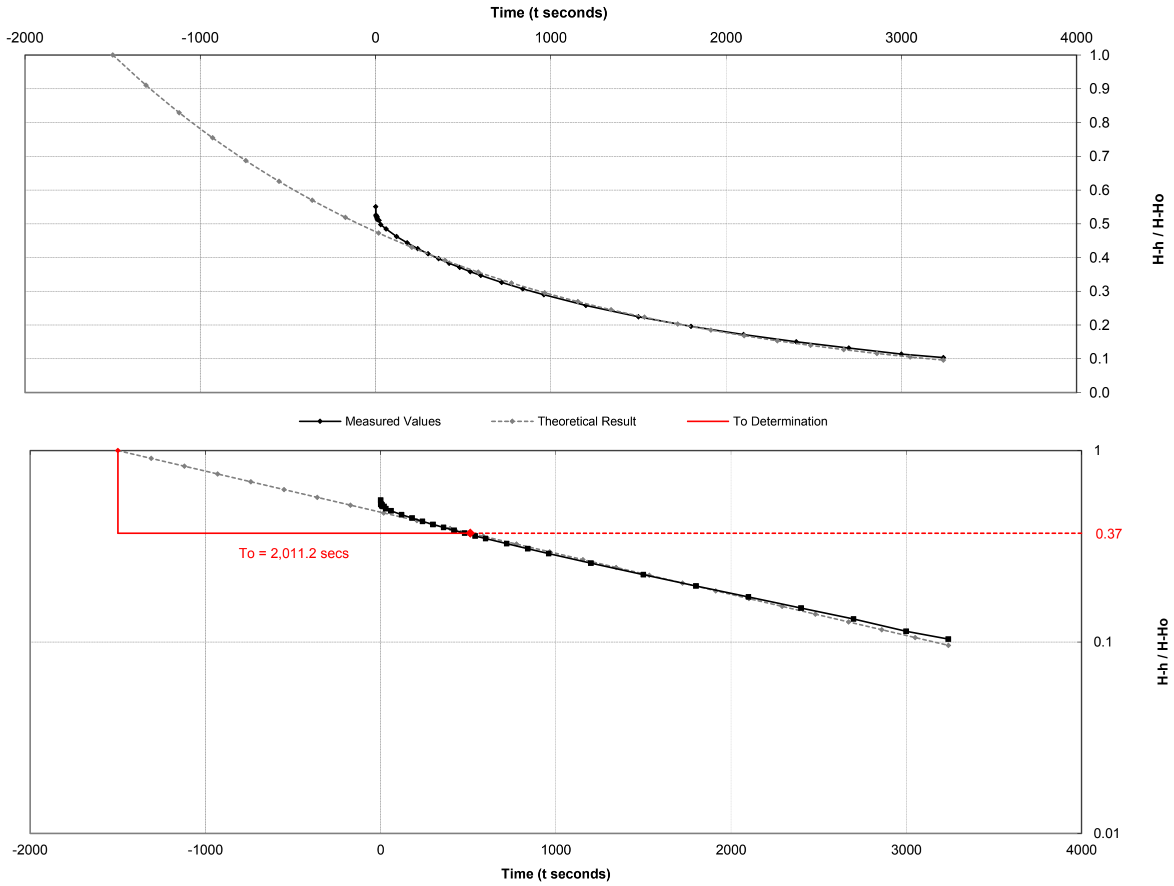
Calibrated Parameters

Intake factor, *F*	9.20
Time Factor, T_0	2,011.2

Measured Data

Time (Secs)	Depth (h) (m)	H - h (m)	H-h / H-H0
1	4.099	0.80	0.55
2	4.062	0.76	0.53
3	4.059	0.76	0.52
4	4.061	0.76	0.52
5	4.056	0.76	0.52
6	4.054	0.75	0.52
7	4.057	0.76	0.52
8	4.053	0.75	0.52
9	4.048	0.75	0.52
10	4.044	0.74	0.51
20	4.040	0.74	0.51
30	4.021	0.72	0.50
60	4.003	0.70	0.48
120	3.971	0.67	0.46
180	3.944	0.64	0.44
240	3.919	0.62	0.43
300	3.896	0.60	0.41
360	3.875	0.58	0.40
420	3.856	0.56	0.38
480	3.837	0.54	0.37
540	3.819	0.52	0.36
600	3.803	0.50	0.35
720	3.773	0.47	0.33
840	3.745	0.45	0.31
960	3.720	0.42	0.29
1200	3.674	0.37	0.26
1500	3.626	0.33	0.22
1800	3.584	0.28	0.20
2100	3.549	0.25	0.17
2400	3.518	0.22	0.15
2700	3.491	0.19	0.13
3000	3.465	0.17	0.11
3240	3.450	0.15	0.10

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



Note: Hvorslev method is based on the slope of the best-fit line. This is calculated by taking the elapsed time, T_0, over 1 natural log interval of the graph, i.e. normalised head between 1 and 0.37.

Appendix C. Packer Test Analyses

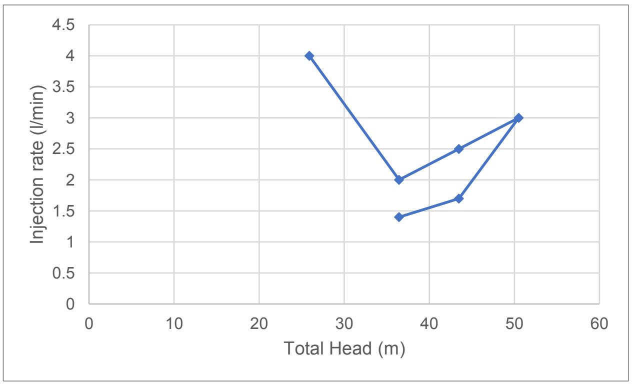
Packer Test Analysis

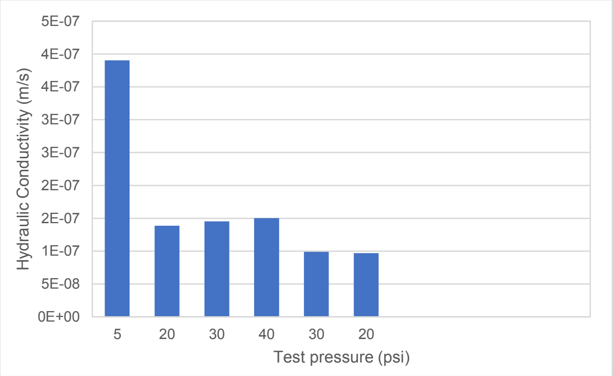
Project Name	Grey Lynn Tunnel	**Client**	Watercare
Borehole ID	CIE-BH01	**Contractor**	McMillans Drilling Group Ltd
Hole Diameter (m)	0.123	**Test Date**	16/03/2018
Top of test interval (mBGL)	17	**Collar Point (mAGL)**	1
Bottom of test interval (mBGL)	21.5	**Static WL (mBTC)**	2.6
Length of test interval (m)	4.5	**Gauge Height (m)**	0.9
		Static Pressure (m)	22.4

Test step	**Time elapsed**	**Water Take**			**Gauge Pressure**		**Net Test Pressure**	**Total Head**	**Injection Rate**	**Take**	**Lugeon Value**	**K^1**
	(min)	**Flow 1 (L)**	**Flow 2 (L)**	**Total flow**	**(psi)**	**(kPa)**	**(kPa)**	**(m)**	**(L/min)**	**(L/min/m)**		**(m/s)**
1	10	200	240	40	5	34	254	25.91	4	0.89	3.50	3.90E-07
2	20	240	260	20	20	138	358	36.46	2	0.44	1.24	1.39E-07
3	30	265	290	25	30	207	427	43.49	2.5	0.56	1.30	1.45E-07
4	40	290	320	30	40	276	496	50.51	3	0.67	1.35	1.50E-07
5	50	330	347	17	30	207	427	43.49	1.7	0.38	0.89	9.89E-08
6	60	350	364	14	20	138	358	36.46	1.4	0.31	0.87	9.71E-08
7												
8												
9												
10												
											Mean	**1.7E-07**

1. K is hydraulic conductivity. The relationship between K and Lugeon Units was defined by Richter and Lillich (1975).

Representative hydraulic conductivity (m/s):	9.71E-08
Test flow behaviour:	Void Filling
Comments:	Highest flow at low pressure indicates void filling. Final permeability used as representative value.





Lugeon Value	**Conductivity classification**	**Rock discontinuity condition**
<1	Very low	Very tight
1-5	Low	Tight
5-15	Moderate	Few partly open
15-50	Medium	Some open
50-100	High	Many open
>100	Very high	Open closely spaced or voids

Ref. http://www.geotechdata.info/geotest/Lugeon_test.html

Key
Background data to be entered
Test data
Spreadsheet calculation (do not change)
Hydraulic conductivity result

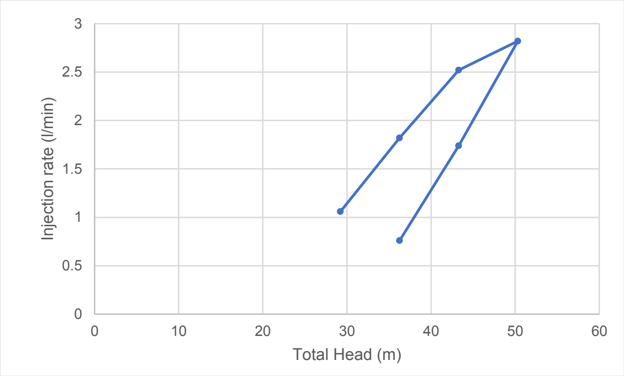
Packer Test Analysis

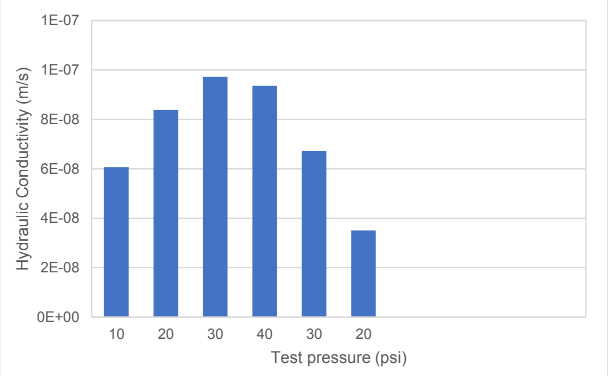
Project Name	Grey Lynn Tunnel	**Client**	Watercare
Borehole ID	CIE-BH02	**Contractor**	McMillans Drilling Group Ltd
Hole Diameter (m)	0.96	**Test Date**	21/03/2018
Top of test interval (mBGL)	18.7	**Collar Point (mAGL)**	0.7
Bottom of test interval (mBGL)	21.5	**Static WL (mBTC)**	6.2
Length of test interval (m)	2.8	**Gauge Height (m)**	0.7
		Static Pressure (m)	22.2

Test step	**Time elapsed**	**Water Take**			**Gauge Pressure**		**Net Test Pressure**	**Total Head**	**Injection Rate**	**Take**	**Lugeon Value**	**K^1**
	(min)	**Flow 1 (L)**	**Flow 2 (L)**	**Total flow**	**(psi)**	**(kPa)**	**(kPa)**	**(m)**	**(L/min)**	**(L/min/m)**		**(m/s)**
1	10	935	945.6	10.6	10	69	287	29.23	1.06	0.38	1.32	6.06E-08
2	10	948	966.2	18.2	20	138	356	36.26	1.82	0.65	1.83	8.38E-08
3	10	970	995.2	25.2	30	207	425	43.29	2.52	0.90	2.12	9.72E-08
4	10	998	1026.2	28.2	40	276	494	50.31	2.82	1.01	2.04	9.36E-08
5	10	1029	1046.4	17.4	30	207	425	43.29	1.74	0.62	1.46	6.71E-08
6	10	1047	1054.6	7.6	20	138	356	36.26	0.76	0.27	0.76	3.50E-08
7												
8												
9												
10												
											Mean	**7.3E-08**

1. K is hydraulic conductivity. The relationship between K and Lugeon Units was defined by Richter and Lillich (1975).

Representative hydraulic conductivity (m/s):	7.29E-08
Test flow behaviour:	Turbulent
Comments:	Laminar flow because of small range of permeability values. Average permeability was used.





Lugeon Value	**Conductivity classification**	**Rock discontinuity condition**
<1	Very low	Very tight
1-5	Low	Tight
5-15	Moderate	Few partly open
15-50	Medium	Some open
50-100	High	Many open
>100	Very high	Open closely spaced or voids

Ref. http://www.geotechdata.info/geotest/Lugeon_test.html

Key
Background data to be entered
Test data
Spreadsheet calculation (do not change)
Hydraulic conductivity result

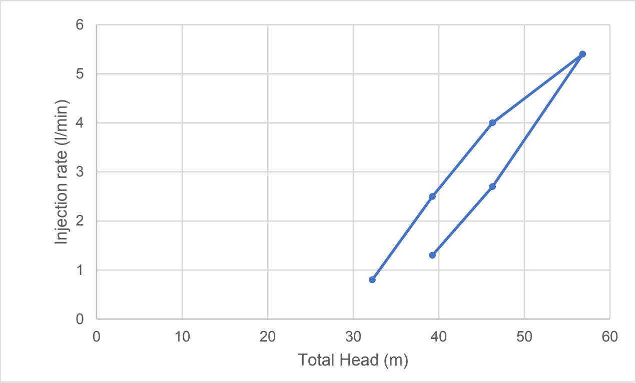
Packer Test Analysis

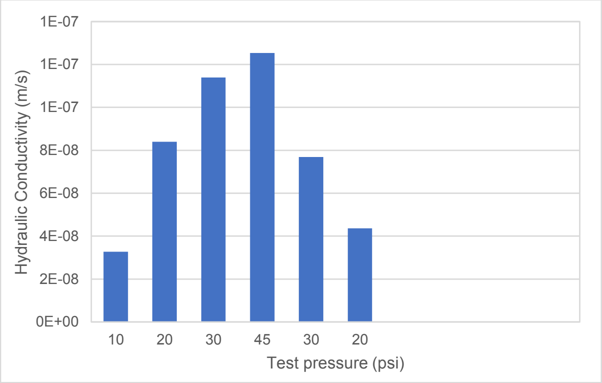
Project Name	Grey Lynn Tunnel	**Client**	Watercare
Borehole ID	CIE-BH03	**Contractor**	McMillans Drilling Group Ltd
Hole Diameter (m)	0.96	**Test Date**	27/03/2018
Top of test interval (mBGL)	20	**Collar Point (mAGL)**	0
Bottom of test interval (mBGL)	24.5	**Static WL (mBTC)**	0.9
Length of test interval (m)	4.5	**Gauge Height (m)**	0.7
		Static Pressure (m)	25.2

Test step	**Time elapsed**	**Water Take**			**Gauge Pressure**		**Net Test Pressure**	**Total Head**	**Injection Rate**	**Take**	**Lugeon Value**	**K[1]**
	(min)	**Flow 1 (L)**	**Flow 2 (L)**	**Total flow**	**(psi)**	**(kPa)**	**(kPa)**	**(m)**	**(L/min)**	**(L/min/m)**		**(m/s)**
1	10	65	73	8	10	69	316	32.23	0.8	0.18	0.56	3.27E-08
2	10	76	101	25	20	138	385	39.26	2.5	0.56	1.44	8.40E-08
3	10	104	144	40	30	207	454	46.29	4	0.89	1.96	1.14E-07
4	10	154	208	54	45	310	557	56.83	5.4	1.20	2.15	1.25E-07
5	10	212	239	27	30	207	454	46.29	2.7	0.60	1.32	7.69E-08
6	10	241	254	13	20	138	385	39.26	1.3	0.29	0.75	4.37E-08
7												
8												
9												
10												
											Mean	**7.9E-08**

1. K is hydraulic conductivity. The relationship between K and Lugeon Units was defined by Richter and Lillich (1975).

Representative hydraulic conductivity (m/s):	5.35E-08
Test flow behaviour:	Dilation
Comments:	Partial dilation occurred at 20 psi, increasing with pressure. Average of lower pressure values used as representative permeability.





Lugeon Value	**Conductivity classification**	**Rock discontinuity condition**
<1	Very low	Very tight
1-5	Low	Tight
5-15	Moderate	Few partly open
15-50	Medium	Some open
50-100	High	Many open
>100	Very high	Open closely spaced or voids

Ref. http://www.geotechdata.info/geotest/Lugeon_test.html

Key
Background data to be entered
Test data
Spreadsheet calculation (do not change)
Hydraulic conductivity result

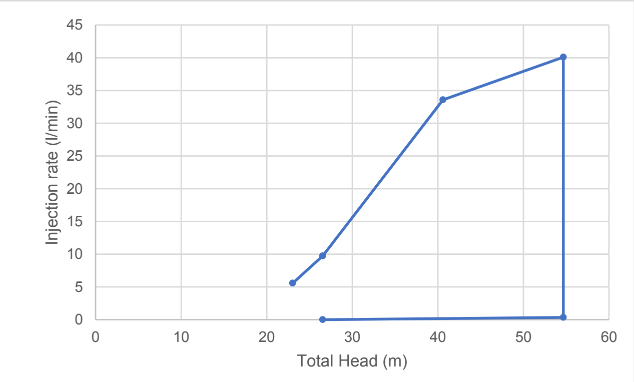
Packer Test Analysis

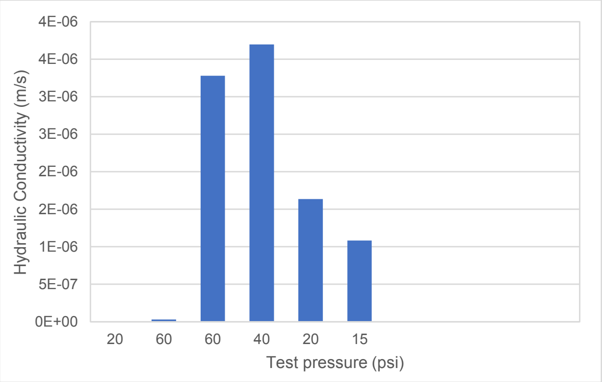
Project Name	Grey Lynn Tunnel	**Client**	Watercare
Borehole ID	CIE-BH04	**Contractor**	McMillans Drilling Group Ltd
Hole Diameter (m)	0.101	**Test Date**	6/07/2018
Top of test interval (mBGL)	9.75	**Collar Point (mAGL)**	1.2
Bottom of test interval (mBGL)	12	**Static WL (mBTC)**	1.13
Length of test interval (m)	2.25	**Gauge Height (m)**	0.5
		Static Pressure (m)	12.5

Test step	**Time elapsed**	**Water Take**			**Gauge Pressure**		**Net Test Pressure**	**Total Head**	**Injection Rate**	**Take**	**Lugeon Value**	**K^1**
	(min)	**Flow 1 (L)**	**Flow 2 (L)**	**Total flow**	**(psi)**	**(kPa)**	**(kPa)**	**(m)**	**(L/min)**	**(L/min/m)**		**(m/s)**
1	5	16727.3	16727.3	0	20	138	261	26.56	0	0.00	0.00	0.00E+00
2	5	16727.6	16729.4	1.8	60	414	536	54.67	0.36	0.16	0.30	2.95E-08
3	5	16730.1	16930.5	200.4	60	414	536	54.67	40.08	17.81	33.21	3.28E-06
4	5	16945	17112.8	167.8	40	276	398	40.61	33.56	14.92	37.44	3.70E-06
5	5	17123.6	17172.2	48.6	20	138	261	26.56	9.72	4.32	16.58	1.64E-06
6	5	17174.5	17202.4	27.9	15	103	226	23.04	5.58	2.48	10.97	1.08E-06
7												
8												
9												
10												
											Mean	**1.6E-06**

1. K is hydraulic conductivity. The relationship between K and Lugeon Units was defined by Richter and Lillich (1975).

Representative hydraulic conductivity (m/s):	2.95E-08
Test flow behaviour:	Dlation
Comments:	Dilation occurred after attempting to increase to 90 psi, then losing pressure and returning to 60. First 60 psi increment used.





Lugeon Value	**Conductivity classification**	**Rock discontinuity condition**
<1	Very low	Very tight
1-5	Low	Tight
5-15	Moderate	Few partly open
15-50	Medium	Some open
50-100	High	Many open
>100	Very high	Open closely spaced or voids

Ref. http://www.geotechdata.info/geotest/Lugeon_test.html

Key
Background data to be entered
Test data
Spreadsheet calculation (do not change)
Hydraulic conductivity result

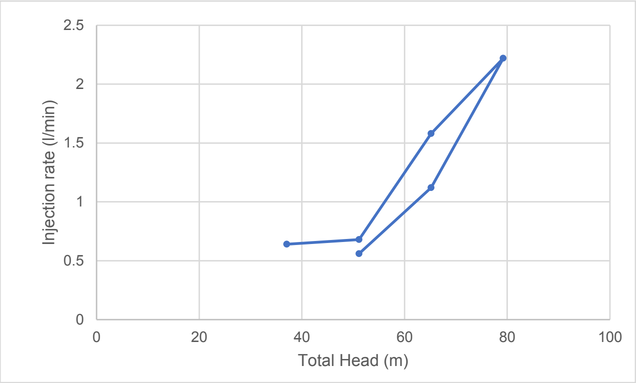
Packer Test Analysis

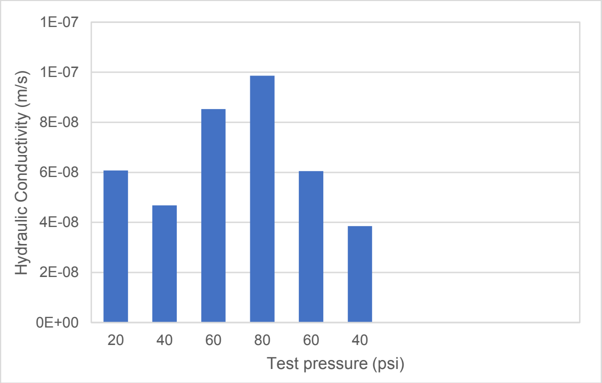
Project Name	Grey Lynn Tunnel	**Client**	Watercare
Borehole ID	CIE-BH04	**Contractor**	McMillans Drilling Group Ltd
Hole Diameter (m)	0.112	**Test Date**	6/07/2018
Top of test interval (mBGL)	19.5	**Collar Point (mAGL)**	1.85
Bottom of test interval (mBGL)	22.5	**Static WL (mBTC)**	0.97
Length of test interval (m)	3	**Gauge Height (m)**	0.5
		Static Pressure (m)	23.0

Test step	Time elapsed	Water Take			Gauge Pressure		Net Test Pressure	Total Head	Injection Rate	Take	Lugeon Value	K^1
	(min)	Flow 1 (L)	Flow 2 (L)	Total flow	(psi)	(kPa)	(kPa)	(m)	(L/min)	(L/min/m)		(m/s)
1	5	213.2	216.4	3.2	20	138	364	37.06	0.64	0.21	0.59	6.08E-08
2	5	219.8	223.2	3.4	40	276	501	51.11	0.68	0.23	0.45	4.68E-08
3	5	226.1	234	7.9	60	414	639	65.17	1.58	0.53	0.82	8.53E-08
4	5	236	247.1	11.1	80	552	777	79.23	2.22	0.74	0.95	9.86E-08
5	5	249.1	254.7	5.6	60	414	639	65.17	1.12	0.37	0.58	6.05E-08
6	5	255	257.8	2.8	40	276	501	51.11	0.56	0.19	0.37	3.86E-08
7												
8												
9												
10												
											Mean	**6.5E-08**

1. K is hydraulic conductivity. The relationship between K and Lugeon Units was defined by Richter and Lillich (1975).

Representative hydraulic conductivity (m/s):	6.51E-08
Test flow behaviour:	Laminar
Comments:	Laminar flow because of small range of permeability values. 40 psi appears to be an outlier. Average permeability was used.





Lugeon Value	Conductivity classification	Rock discontinuity condition
<1	Very low	Very tight
1-5	Low	Tight
5-15	Moderate	Few partly open
15-50	Medium	Some open
50-100	High	Many open
>100	Very high	Open closely spaced or voids

Ref. http://www.geotechdata.info/geotest/Lugeon_test.html

Key
Background data to be entered
Test data
Spreadsheet calculation (do not change)
Hydraulic conductivity result

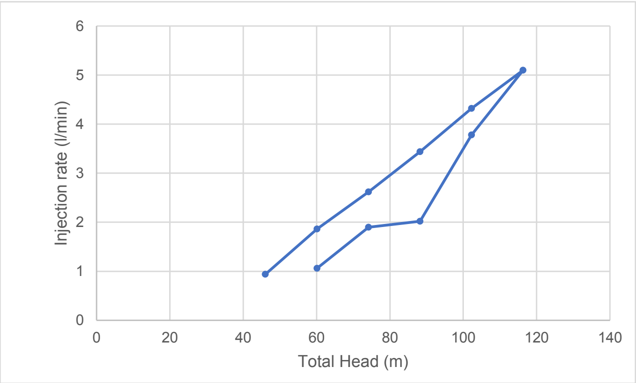
Packer Test Analysis

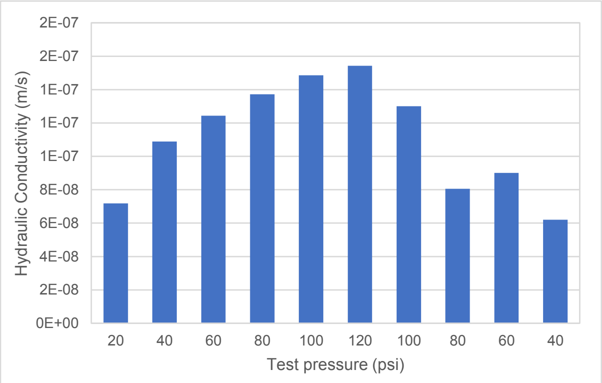
Project Name	Grey Lynn Tunnel	**Client**	Watercare
Borehole ID	CIE-BH04	**Contractor**	McMillans Drilling Group Ltd
Hole Diameter (m)	0.112	**Test Date**	10/07/2018
Top of test interval (mBGL)	28.5	**Collar Point (mAGL)**	1.85
Bottom of test interval (mBGL)	31.5	**Static WL (mBTC)**	0.9
Length of test interval (m)	3	**Gauge Height (m)**	0.5
		Static Pressure (m)	32.0

Test step	**Time elapsed**	**Water Take**			**Gauge Pressure**		**Net Test Pressure**	**Total Head**	**Injection Rate**	**Take**	**Lugeon Value**	**K^1**
	(min)	**Flow 1 (L)**	**Flow 2 (L)**	**Total flow**	**(psi)**	**(kPa)**	**(kPa)**	**(m)**	**(L/min)**	**(L/min/m)**		**(m/s)**
1	5	274.3	279	4.7	20	138	452	46.06	0.94	0.31	0.69	7.18E-08
2	5	280.5	289.8	9.3	40	276	590	60.11	1.86	0.62	1.05	1.09E-07
3	5	292.2	305.3	13.1	60	414	728	74.17	2.62	0.87	1.20	1.24E-07
4	5	308.4	325.6	17.2	80	552	866	88.23	3.44	1.15	1.32	1.37E-07
5	5	328.7	350.3	21.6	100	689	1003	102.28	4.32	1.44	1.44	1.49E-07
6	5	359.3	384.8	25.5	120	827	1141	116.34	5.1	1.70	1.49	1.54E-07
7	5	400	418.9	18.9	100	689	1003	102.28	3.78	1.26	1.26	1.30E-07
8	5	420.9	431	10.1	80	552	866	88.23	2.02	0.67	0.78	8.06E-08
9	5	435.1	444.6	9.5	60	414	728	74.17	1.9	0.63	0.87	9.01E-08
10	5	445	451	5.3	40	276	589.712	60.11	1.06	0.3533333	0.60	6.21E-08
											Mean	**1.1E-07**

1. K is hydraulic conductivity. The relationship between K and Lugeon Units was defined by Richter and Lillich (1975).

Representative hydraulic conductivity (m/s):	1.11E-07
Test flow behaviour:	Laminar
Comments:	Laminar flow because of small range of permeability values. Average was used.





Lugeon Value	**Conductivity classification**	**Rock discontinuity condition**
<1	Very low	Very tight
1-5	Low	Tight
5-15	Moderate	Few partly open
15-50	Medium	Some open
50-100	High	Many open
>100	Very high	Open closely spaced or voids

Ref. http://www.geotechdata.info/geotest/Lugeon_test.html

Key
Background data to be entered
Test data
Spreadsheet calculation (do not change)
Hydraulic conductivity result

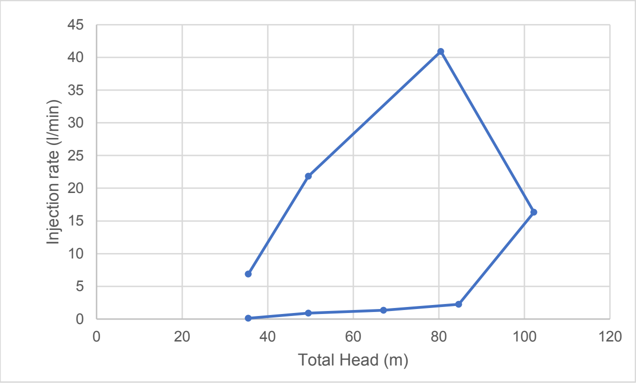
Packer Test Analysis

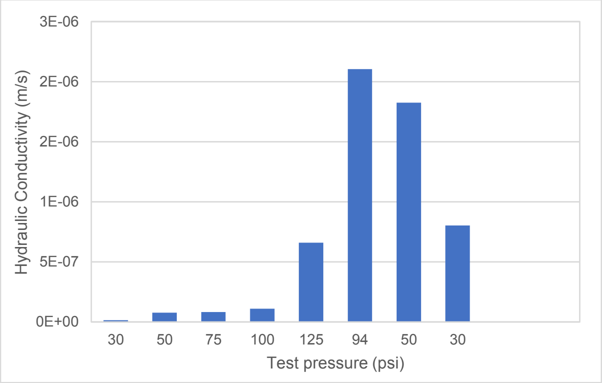
Project Name	Grey Lynn Tunnel	**Client**	Watercare
Borehole ID	CIE-BH05	**Contractor**	McMillans Drilling Group Ltd
Hole Diameter (m)	0.101	**Test Date**	12/07/2018
Top of test interval (mBGL)	11	**Collar Point (mAGL)**	1.1
Bottom of test interval (mBGL)	13.5	**Static WL (mBTC)**	0.2
Length of test interval (m)	2.5	**Gauge Height (m)**	0.9
		Static Pressure (m)	14.4

Test step	**Time elapsed**	**Water Take**			**Gauge Pressure**		**Net Test Pressure**	**Total Head**	**Injection Rate**	**Take**	**Lugeon Value**	**K[1]**
	(min)	**Flow 1 (L)**	**Flow 2 (L)**	**Total flow**	**(psi)**	**(kPa)**	**(kPa)**	**(m)**	**(L/min)**	**(L/min/m)**		**(m/s)**
1	5	3.84	4.46	0.62	30	207	348	35.49	0.124	0.05	0.14	1.45E-08
2	5	5.1	9.69	4.59	50	345	486	49.54	0.918	0.37	0.76	7.67E-08
3	5	11.1	17.75	6.65	75	517	658	67.11	1.33	0.53	0.81	8.20E-08
4	5	21.11	32.41	11.3	100	689	831	84.68	2.26	0.90	1.09	1.10E-07
5	5	37.5	119.1	81.6	125	862	1003	102.25	16.32	6.53	6.51	6.61E-07
6	5	135.3	339.9	204.6	94	648	789	80.47	40.92	16.37	20.74	2.10E-06
7	5	355.3	464.5	109.2	50	345	486	49.54	21.84	8.74	17.98	1.82E-06
8	5	474.8	509.2	34.4	30	207	348	35.49	6.88	2.75	7.91	8.02E-07
9												
10												
											Mean	**7.1E-07**

1. K is hydraulic conductivity. The relationship between K and Lugeon Units was defined by Richter and Lillich (1975).

Representative hydraulic conductivity (m/s):	8.97E-08
Test flow behaviour:	Dilation
Comments:	Dilation occurred at 125 psi. Average of 50-100 psi test used as representative permeability.





Lugeon Value	**Conductivity classification**	**Rock discontinuity condition**
<1	Very low	Very tight
1-5	Low	Tight
5-15	Moderate	Few partly open
15-50	Medium	Some open
50-100	High	Many open
>100	Very high	Open closely spaced or voids

Ref. http://www.geotechdata.info/geotest/Lugeon_test.html

Key
Background data to be entered
Test data
Spreadsheet calculation (do not change)
Hydraulic conductivity result

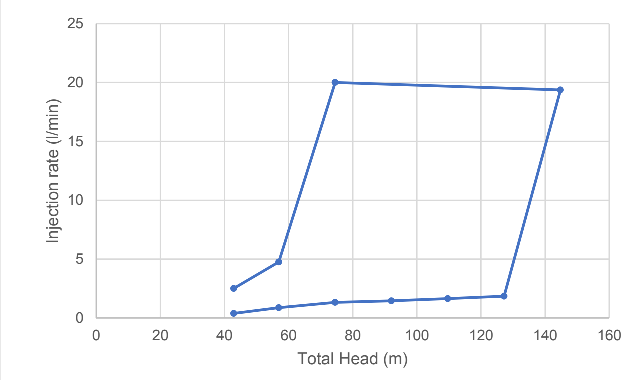
Packer Test Analysis

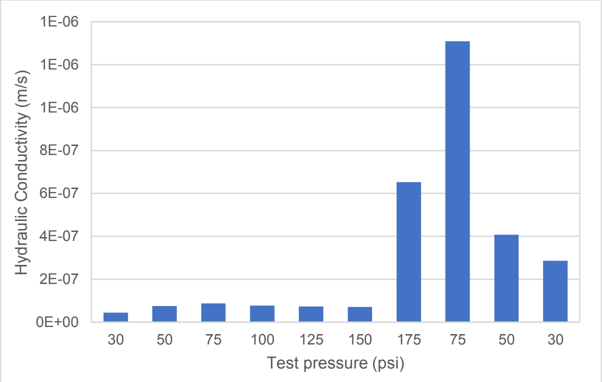
Project Name	Grey Lynn Tunnel	**Client**	Watercare
Borehole ID	CIE-BH05	**Contractor**	McMillans Drilling Group Ltd
Hole Diameter (m)	0.101	**Test Date**	13/07/2018
Top of test interval (mBGL)	19	**Collar Point (mAGL)**	1.1
Bottom of test interval (mBGL)	21	**Static WL (mBTC)**	-2.77
Length of test interval (m)	2	**Gauge Height (m)**	0.8
		Static Pressure (m)	21.8

Test step	**Time elapsed**	**Water Take**			**Gauge Pressure**		**Net Test Pressure**	**Total Head**	**Injection Rate**	**Take**	**Lugeon Value**	**K^1**
	(min)	**Flow 1 (L)**	**Flow 2 (L)**	**Total flow**	**(psi)**	**(kPa)**	**(kPa)**	**(m)**	**(L/min)**	**(L/min/m)**		**(m/s)**
1	5	0.85	2.79	1.94	30	207	421	42.89	0.388	0.19	0.46	4.41E-08
2	5	4.54	8.93	4.39	50	345	559	56.94	0.878	0.44	0.79	7.52E-08
3	5	9.86	16.51	6.65	75	517	731	74.51	1.33	0.67	0.91	8.71E-08
4	5	18.59	25.84	7.25	100	689	903	92.08	1.45	0.73	0.80	7.68E-08
5	5	27.29	35.54	8.25	125	862	1076	109.65	1.65	0.83	0.77	7.34E-08
6	5	38.27	47.52	9.25	150	1034	1248	127.23	1.85	0.93	0.74	7.09E-08
7	5	50.02	146.9	96.88	175	1207	1420	144.80	19.376	9.69	6.82	6.53E-07
8	5	161.6	261.61	100.01	75	517	731	74.51	20.002	10.00	13.68	1.31E-06
9	5	271.15	294.94	23.79	50	345	559	56.94	4.758	2.38	4.26	4.08E-07
10	5	298	311	12.56	30	207	420.702	42.89	2.512	1.256	2.99	2.86E-07
											Mean	**3.1E-07**

1. K is hydraulic conductivity. The relationship between K and Lugeon Units was defined by Richter and Lillich (1975).

Representative hydraulic conductivity (m/s):	7.09E-08
Test flow behaviour:	Wash out
Comments:	Wash out occurred at 175 psi, highest Lugeon value before wash out used as representative permeability.





Lugeon Value	**Conductivity classification**	**Rock discontinuity condition**
<1	Very low	Very tight
1-5	Low	Tight
5-15	Moderate	Few partly open
15-50	Medium	Some open
50-100	High	Many open
>100	Very high	Open closely spaced or voids

Ref. http://www.geotechdata.info/geotest/Lugeon_test.html

Key
Background data to be entered
Test data
Spreadsheet calculation (do not change)
Hydraulic conductivity result

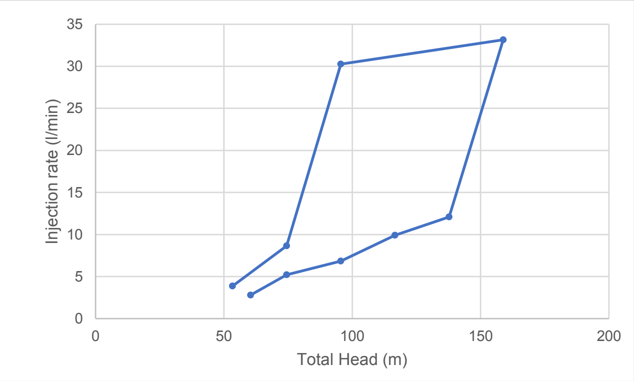
Packer Test Analysis

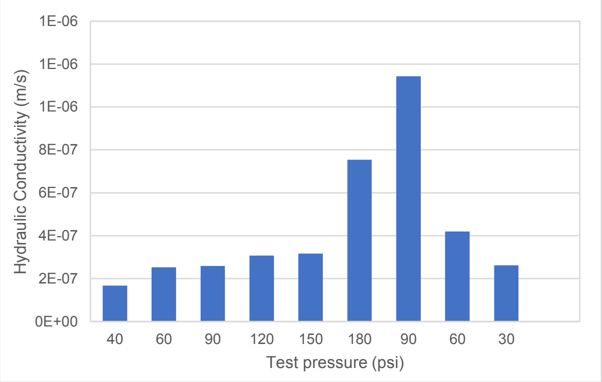
Project Name	Grey Lynn Tunnel	**Client**	Watercare
Borehole ID	CIE-BH05	**Contractor**	McMillans Drilling Group Ltd
Hole Diameter (m)	0.101	**Test Date**	13/07/2018
Top of test interval (mBGL)	28.5	**Collar Point (mAGL)**	1.1
Bottom of test interval (mBGL)	31.5	**Static WL (mBTC)**	-2.77
Length of test interval (m)	3	**Gauge Height (m)**	0.8
		Static Pressure (m)	32.3

Test step	Time elapsed	Water Take			Gauge Pressure		Net Test Pressure	Total Head	Injection Rate	Take	Lugeon Value	K[1]
	(min)	Flow 1 (L)	Flow 2 (L)	Total flow	(psi)	(kPa)	(kPa)	(m)	(L/min)	(L/min/m)		(m/s)
1	5	7.29	21.31	14.02	40	276	593	60.41	2.804	0.93	1.58	1.68E-07
2	5	30.59	56.67	26.08	60	414	731	74.47	5.216	1.74	2.38	2.53E-07
3	5	76.31	110.54	34.23	90	621	937	95.56	6.846	2.28	2.43	2.59E-07
4	5	118.57	168.17	49.6	120	827	1144	116.64	9.92	3.31	2.89	3.07E-07
5	5	191.84	252.29	60.45	150	1034	1351	137.73	12.09	4.03	2.98	3.17E-07
6	5	271.76	437.55	165.79	180	1241	1558	158.81	33.158	11.05	7.09	7.54E-07
7	5	479.7	631.02	151.32	90	621	937	95.56	30.264	10.09	10.76	1.14E-06
8	5	667.25	710.56	43.31	60	414	731	74.47	8.662	2.89	3.95	4.20E-07
9	5	716.89	736.27	19.38	30	207	524	53.39	3.876	1.29	2.47	2.62E-07
10												
											Mean	**4.3E-07**

1. K is hydraulic conductivity. The relationship between K and Lugeon Units was defined by Richter and Lillich (1975).

Representative hydraulic conductivity (m/s):	2.61E-07
Test flow behaviour:	Dilation
Comments:	Dilation occurred at 180 psi, average of values prior to dilation was used as representative permeability.





Lugeon Value	Conductivity classification	Rock discontinuity condition
<1	Very low	Very tight
1-5	Low	Tight
5-15	Moderate	Few partly open
15-50	Medium	Some open
50-100	High	Many open
>100	Very high	Open closely spaced or voids

Ref. http://www.geotechdata.info/geotest/Lugeon_test.html

Key
Background data to be entered
Test data
Spreadsheet calculation (do not change)
Hydraulic conductivity result

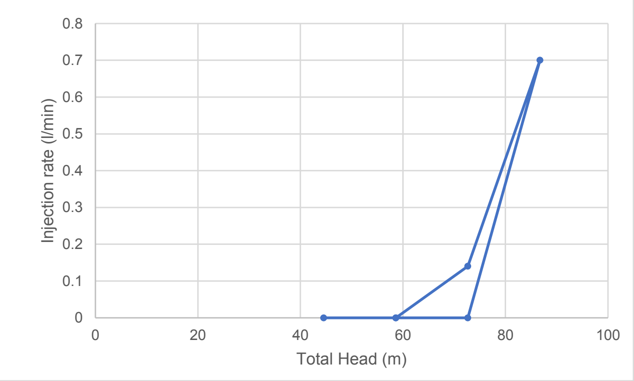
Packer Test Analysis

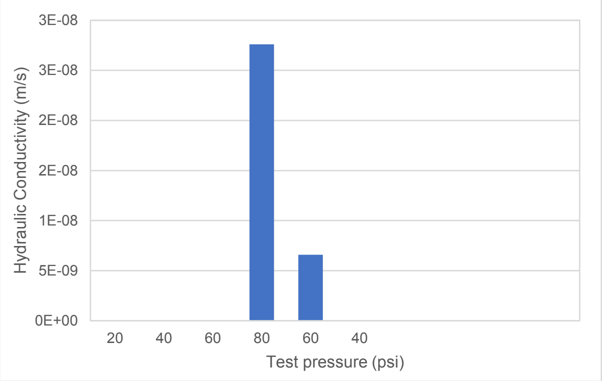
Project Name	Grey Lynn Tunnel	**Client**	Watercare
Borehole ID	CIE-BH06	**Contractor**	McMillans Drilling Group Ltd
Hole Diameter (m)	0.125	**Test Date**	29/06/2018
Top of test interval (mBGL)	27	**Collar Point (mAGL)**	1.2
Bottom of test interval (mBGL)	30	**Static WL (mBTC)**	0.6
Length of test interval (m)	3	**Gauge Height (m)**	0.5
		Static Pressure (m)	30.5

Test step	Time elapsed	Water Take			Gauge Pressure		Net Test Pressure	Total Head	Injection Rate	Take	Lugeon Value	K^1
	(min)	Flow 1 (L)	Flow 2 (L)	Total flow	(psi)	(kPa)	(kPa)	(m)	(L/min)	(L/min/m)		(m/s)
1	5	16644.2	16644.2	0	20	138	437	44.56	0	0.00	0.00	0.00E+00
2	5	16644.2	16644.2	0	40	276	575	58.61	0	0.00	0.00	0.00E+00
3	5	16644.2	16644.2	0	60	414	713	72.67	0	0.00	0.00	0.00E+00
4	5	16653.9	16657.4	3.5	80	552	851	86.73	0.7	0.23	0.27	2.76E-08
5	5	16657.6	16658.3	0.7	60	414	713	72.67	0.14	0.05	0.07	6.59E-09
6	5	16658.3	16658.3	0	40	276	575	58.61	0	0.00	0.00	0.00E+00
7												
8												
9												
10												
											Mean	**5.7E-09**

1. K is hydraulic conductivity. The relationship between K and Lugeon Units was defined by Richter and Lillich (1975).

Representative hydraulic conductivity (m/s):	2.76E-08
Test flow behaviour:	Dilation
Comments:	Flow did not occur until 80 psi. 80 psi used as representative permeability value.





Lugeon Value	Conductivity classification	Rock discontinuity condition
<1	Very low	Very tight
1-5	Low	Tight
5-15	Moderate	Few partly open
15-50	Medium	Some open
50-100	High	Many open
>100	Very high	Open closely spaced or voids

Ref. http://www.geotechdata.info/geotest/Lugeon_test.html

Key
Background data to be entered
Test data
Spreadsheet calculation (do not change)
Hydraulic conductivity result

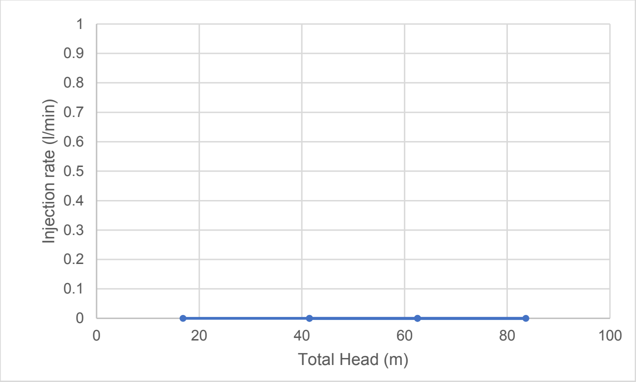
Packer Test Analysis

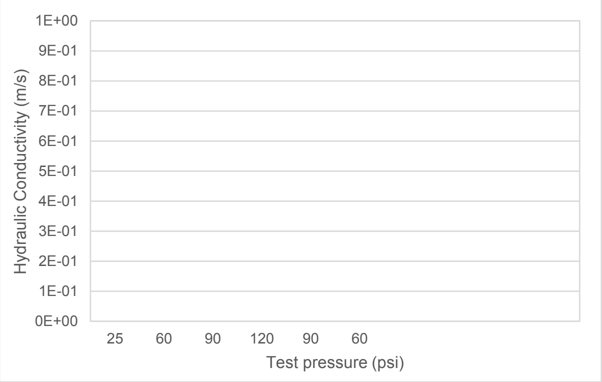
Project Name	Grey Lynn Tunnel	**Client**	Watercare
Borehole ID	CIE-BH06	**Contractor**	McMillans Drilling Group Ltd
Hole Diameter (m)	0.125	**Test Date**	2/07/2018
Top of test interval (mBGL)	50.25	**Collar Point (mAGL)**	1.2
Bottom of test interval (mBGL)	52.5	**Static WL (mBTC)**	0
Length of test interval (m)	2.25	**Gauge Height (m)**	0.5
		Static Pressure (m)	-0.7

Test step	**Time elapsed**	**Water Take**			**Gauge Pressure**		**Net Test Pressure**	**Total Head**	**Injection Rate**	**Take**	**Lugeon Value**	**K**^{1}
	(min)	**Flow 1 (L)**	**Flow 2 (L)**	**Total flow**	**(psi)**	**(kPa)**	**(kPa)**	**(m)**	**(L/min)**	**(L/min/m)**		**(m/s)**
1	5	16664.9	16664.9	0	25	172	166	16.87	0	0.00	0.00	0.00E+00
2	5	16664.9	16664.9	0	60	414	407	41.47	0	0.00	0.00	0.00E+00
3	5	16664.9	16664.9	0	90	621	614	62.56	0	0.00	0.00	0.00E+00
4	5	16664.9	16664.9	0	120	827	821	83.64	0	0.00	0.00	0.00E+00
5	5	16664.9	16664.9	0	90	621	614	62.56	0	0.00	0.00	0.00E+00
6	5	16664.9	16664.9	0	60	414	407	41.47	0	0.00	0.00	0.00E+00
7												
8												
9												
10												
											Mean	**0.0E+00**

1. K is hydraulic conductivity. The relationship between K and Lugeon Units was defined by Richter and Lillich (1975).

Representative hydraulic conductivity (m/s):	0.00E+00
Test flow behaviour:	No flow
Comments:	No Flow





Lugeon Value	**Conductivity classification**	**Rock discontinuity condition**
<1	Very low	Very tight
1-5	Low	Tight
5-15	Moderate	Few partly open
15-50	Medium	Some open
50-100	High	Many open
>100	Very high	Open closely spaced or voids

Ref. http://www.geotechdata.info/geotest/Lugeon_test.html

Key
Background data to be entered
Test data
Spreadsheet calculation (do not change)
Hydraulic conductivity result

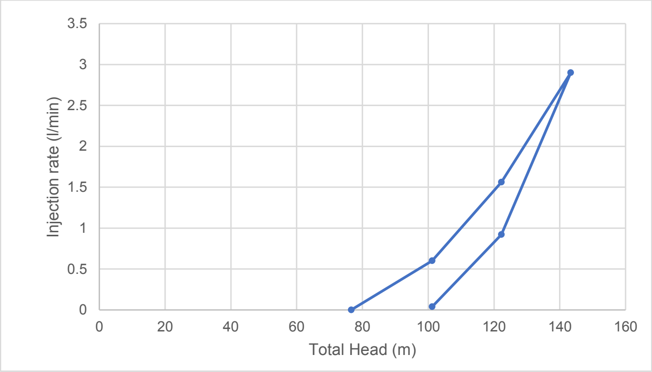
Packer Test Analysis

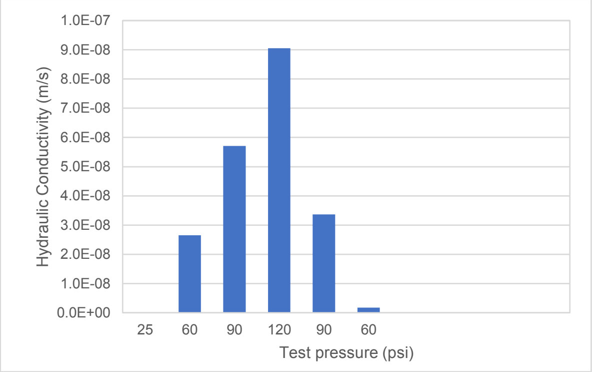
Project Name	Grey Lynn Tunnel	**Client**	Watercare
Borehole ID	CIE-BH06	**Contractor**	McMillans Drilling Group Ltd
Hole Diameter (m)	0.101	**Test Date**	3/07/2018
Top of test interval (mBGL)	56.25	**Collar Point (mAGL)**	1.2
Bottom of test interval (mBGL)	58.5	**Static WL (mBTC)**	0.25
Length of test interval (m)	2.25	**Gauge Height (m)**	0.5
		Static Pressure (m)	59.0

Test step	**Time elapsed**	**Water Take**			**Gauge Pressure**		**Net Test Pressure**	**Total Head**	**Injection Rate**	**Take**	**Lugeon Value**	**K[1]**
	(min)	**Flow 1 (L)**	**Flow 2 (L)**	**Total flow**	**(psi)**	**(kPa)**	**(kPa)**	**(m)**	**(L/min)**	**(L/min/m)**		**(m/s)**
1	5	16674.8	16674.8	0	25	172	751	76.57	0	0.00	0.00	0.00E+00
2	5	16675.7	16678.7	3	60	414	992	101.17	0.6	0.27	0.27	2.65E-08
3	5	16679.9	16687.7	7.8	90	621	1199	122.26	1.56	0.69	0.58	5.71E-08
4	5	16688.7	16703.2	14.5	120	827	1406	143.34	2.9	1.29	0.92	9.05E-08
5	5	16704.4	16709	4.6	90	621	1199	122.26	0.92	0.41	0.34	3.37E-08
6	5	16709.1	16709.3	0.2	60	414	992	101.17	0.04	0.02	0.02	1.77E-09
7												
8												
9												
10												
											Mean	**3.5E-08**

1. K is hydraulic conductivity. The relationship between K and Lugeon Units was defined by Richter and Lillich (1975).

Representative hydraulic conductivity (m/s):	4.18E-08
Test flow behaviour:	Dilation
Comments:	Dilation occurred at 120 psi. Average of 60 and 90 psi used as representative values.





Lugeon Value	**Conductivity classification**	**Rock discontinuity condition**
<1	Very low	Very tight
1-5	Low	Tight
5-15	Moderate	Few partly open
15-50	Medium	Some open
50-100	High	Many open
>100	Very high	Open closely spaced or voids

Ref. http://www.geotechdata.info/geotest/Lugeon_test.html

Key
Background data to be entered
Test data
Spreadsheet calculation (do not change)
Hydraulic conductivity result

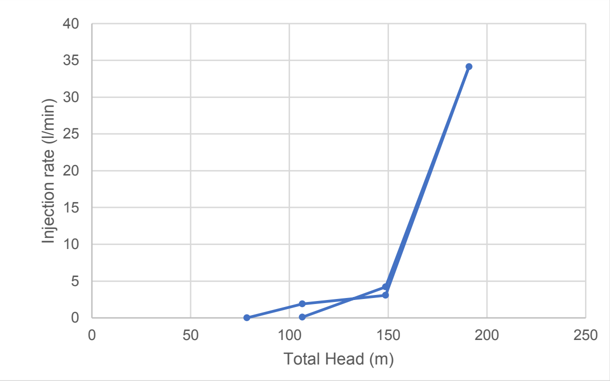
Packer Test Analysis

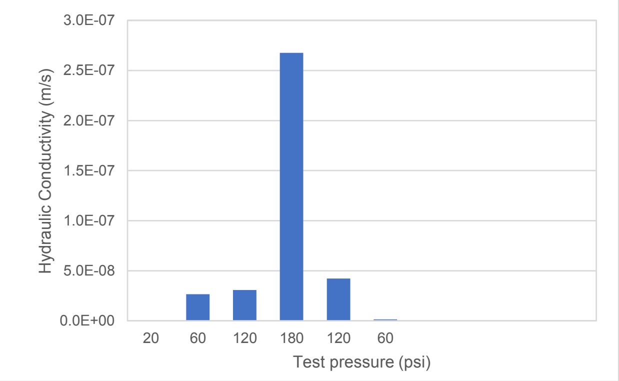
Project Name	Grey Lynn Tunnel	**Client**	Watercare
Borehole ID	CIE-BH06	**Contractor**	McMillans Drilling Group Ltd
Hole Diameter (m)	0.112	**Test Date**	10/07/2018
Top of test interval (mBGL)	54.5	**Collar Point (mAGL)**	1.3
Bottom of test interval (mBGL)	63.5	**Static WL (mBTC)**	1.8
Length of test interval (m)	9	**Gauge Height (m)**	0.9
		Static Pressure (m)	64.4

Test step	**Time elapsed**	**Water Take**			**Gauge Pressure**		**Net Test Pressure**	**Total Head**	**Injection Rate**	**Take**	**Lugeon Value**	**K[1]**
	(min)	**Flow 1 (L)**	**Flow 2 (L)**	**Total flow**	**(psi)**	**(kPa)**	**(kPa)**	**(m)**	**(L/min)**	**(L/min/m)**		**(m/s)**
1	5	30.17	30.17	0	20	138	770	78.46	0	0.00	0.00	0.00E+00
2	5	31.6	41.07	9.47	60	414	1045	106.57	1.894	0.21	0.20	2.66E-08
3	5	50.35	65.62	15.27	120	827	1459	148.74	3.054	0.34	0.23	3.07E-08
4	5	76.31	247	170.69	180	1241	1873	190.91	34.138	3.79	2.03	2.68E-07
5	5	275.9	296.9	21	120	827	1459	148.74	4.2	0.47	0.32	4.23E-08
6	5	298.6	299.07	0.47	60	414	1045	106.57	0.094	0.01	0.01	1.32E-09
7												
8												
9												
10												
											Mean	**6.1E-08**

1. K is hydraulic conductivity. The relationship between K and Lugeon Units was defined by Richter and Lillich (1975).

Representative hydraulic conductivity (m/s):	2.87E-08
Test flow behaviour:	Dilation
Comments:	Dilation occurred at 180 psi. Average of 60 and 120 psi used as representative value.





Lugeon Value	**Conductivity classification**	**Rock discontinuity condition**
<1	Very low	Very tight
1-5	Low	Tight
5-15	Moderate	Few partly open
15-50	Medium	Some open
50-100	High	Many open
>100	Very high	Open closely spaced or voids

Ref. http://www.geotechdata.info/geotest/Lugeon_test.html

Key
Background data to be entered
Test data
Spreadsheet calculation (do not change)
Hydraulic conductivity result

Map Title:

Project Area Overview

Project:

Grey Lynn Tunnel

Client:

Watercare Services Limited

★	**Tawariki shaft location**
▭	**Model boundary**
—	**Drainage Network**

Data Provenance

Topographic Data derived from Land Information New Zealand

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QGIS by:

WILLIAMSON WATER ADVISORY

Figure 1.

Map Title:

Study Area Geology

Project:

Grey Lynn Tunnel

Client:

Watercare Services Limited

★ **Tawariki shaft location**

Model boundary

— **Model area drains (surface & sub-surface)**

Geologic Units

▨ **East Coast Bays Formation**

▨ **Tauranga Group**

▨ **Auckland Basalts**

▨ **Puketoka Formation**

▨ **Fill**

Data Provenance

Topographic Data derived from Land Information New Zealand

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Figure 2.

Map Title:

Monitoring bore locations

Project:

Grey Lynn Tunnel

Client:

Watercare Services Limited

Data Provenance

Topographic Data derived from Land Information New Zealand
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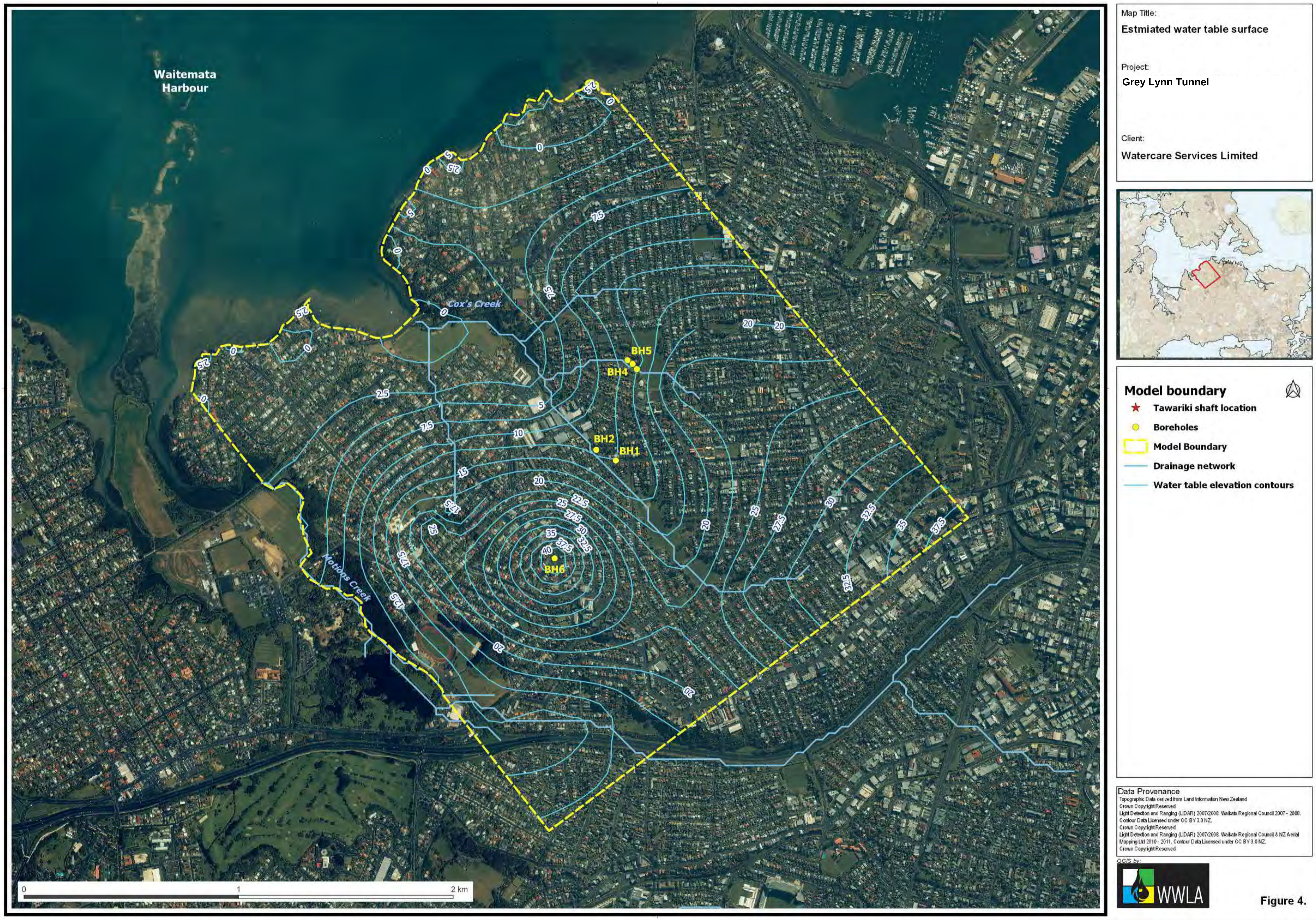
Figure 3.



Map Title:

Estmiated water table surface

Project:

Grey Lynn Tunnel

Client:

Watercare Services Limited

Data Provenance

Topographic Data derived from Land Information New Zealand

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QGIS by:

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Figure 4.

Map Title:

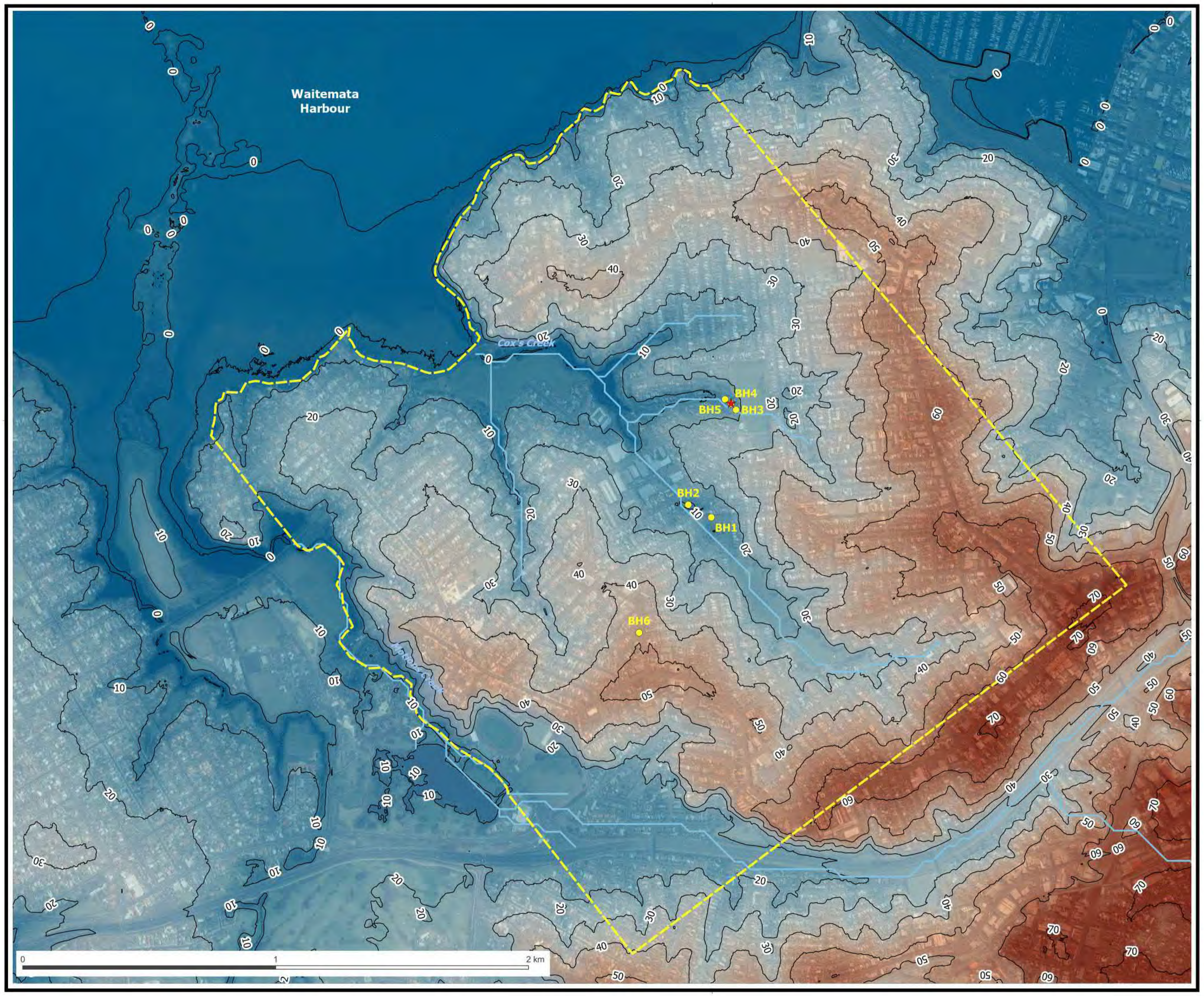
Study area surface elevation

Project:

Grey Lynn Tunnel

Client:

Watercare Services Limited



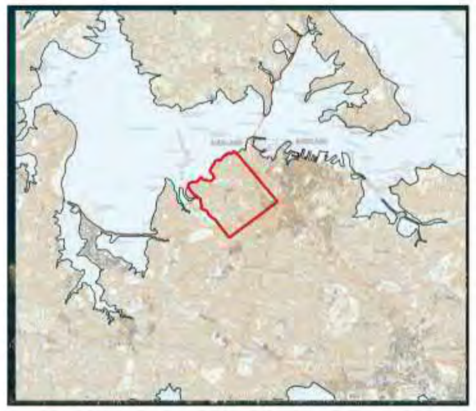


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Figure 6.

Map Title:

Predicted drawdown following shaft installation: Scenario 4-Layer 2

Project:

Grey Lynn Tunnel

Client:

Watercare Services Limited

★ **Tawariki shaft location**

— **Drawdown contour (m)**

Data Provenance

Topographic Data derived from Land Information New Zealand

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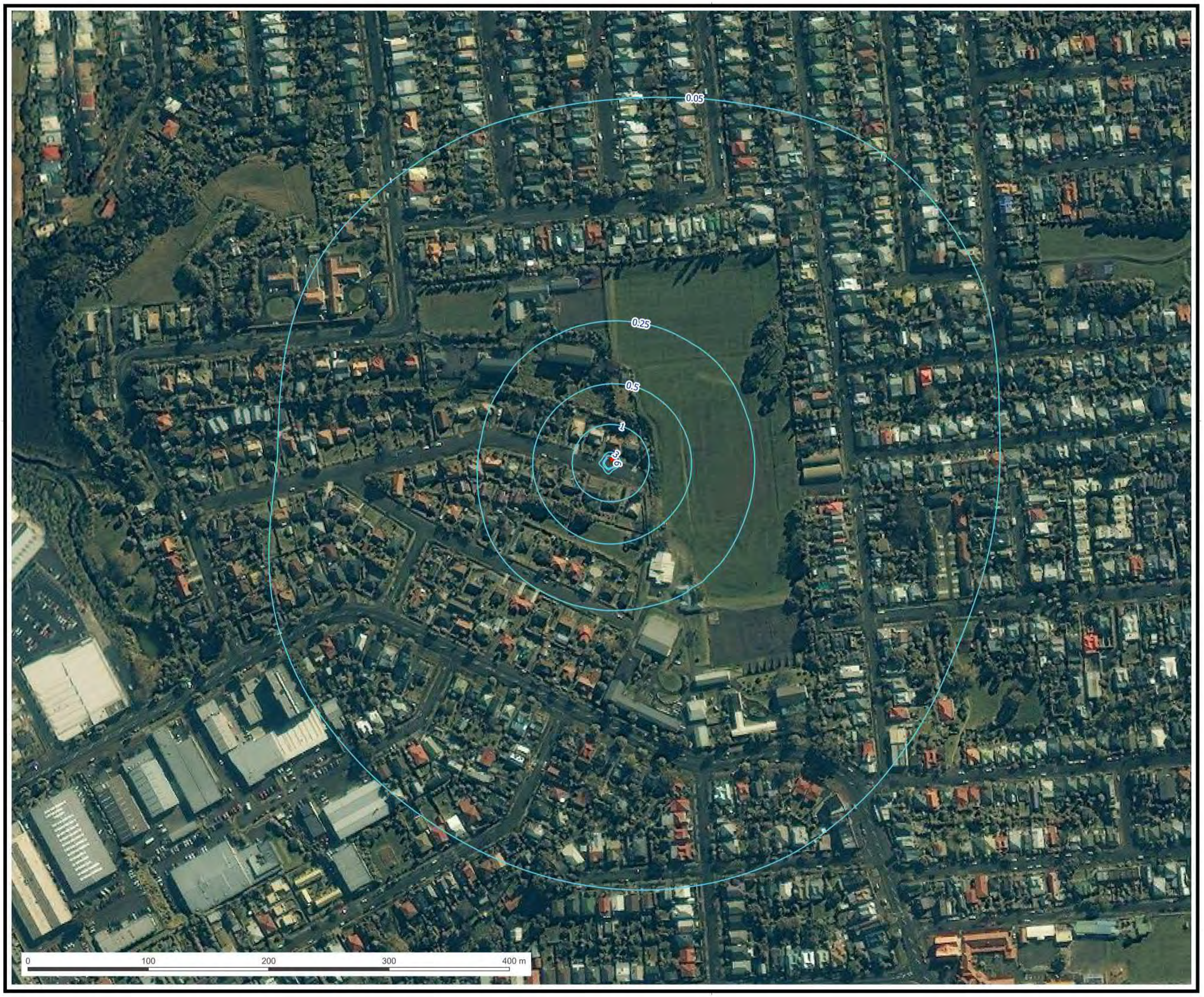
Figure 11.



Map Title:

Predicted drawdown following shaft installation: Scenario 4-Layer 4

Project:

Grey Lynn Tunnel

Client:

Watercare Services Limited

★ **Tawariki shaft location**

— **Drawdown contour (m)**

Data Provenance

Topographic Data derived from Land Information New Zealand

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Figure 12.

Map Title:

Predicted drawdown following shaft installation: Scenario 6-Layer 2

Project:

Grey Lynn Tunnel

Client:

Watercare Services Limited

★ **Tawariki shaft location**

— **Drawdown contour (m)**

Data Provenance

Topographic Data derived from Land Information New Zealand

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Figure 13.

Map Title:

Predicted drawdown following shaft installation: Scenario 6-Layer 4

Project:

Grey Lynn Tunnel

Client:

Watercare Services Limited

★ **Tawariki shaft location**
— **Drawdown contour (m)**

Data Provenance
Topographic Data derived from Land Information New Zealand
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Figure 14.

Appendix O

Grey Lynn Tunnel

Watercare Services Limited

Settlement Assessment of Grey Lynn Tunnel and Tawariki Street Shafts

Draft
Revision No. 3

31 January 2018

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Table of Contents

List of Tables

List of Figures

Distribution

To:	Karyn Sinclair Jacobs New Zealand Limited
From:	Victor Romero McMillen Jacobs Associates
Prepared By:	Sam Burgess McMillen Jacobs Associates
Reviewed By:	Victor Romero McMillen Jacobs Associates

Revision Log

Revision No.	Date	Revision Description
1	9 Nov 2018	Draft for internal review
2	20 Nov 2018	Issued to planning team
3	31 January	Issued for Consent

Executive Summary

This report summarises the assessment undertaken to identify existing structures and utilities at risk of damage because of settlement caused by shaft construction or tunnelling activities from construction of the Grey Lynn Tunnel.

This settlement assessment considers both the mechanical (i.e. excavation) settlement associated with excavation of tunnels and shafts, and the consolidation settlement that could occur because of dewatering during construction. Mechanical and consolidation settlements have been combined on a settlement contour plot in Appendix A. Consistent with the approach taken to and conditions imposed on the Central Interceptor (Consent Ref 40836),these drawings also show areas where 50mm total settlement or 1:1000 differential settlement may be exceeded. These settlement values are conservative. Total settlement does not have any significant effect on buildings, it is the differential settlement which can affect buildings, if a building settles uniformly, then damage is not likely. However, Lake et al (1996) proposed 10-50mm settlement as ‘possible superficial damage which is unlikely to have structural significance’. In reality, a building is unlikely to show any signs of cracking at this level. The safe limit of differential settlement for no cracking of buildings is 1:500 (danger of structural damage occurs at 1:150) and the limit for machinery sensitive to settlement is 1:750 Wahls (1981). Polshin and Tokar (1957) describe 0.7:1000 to 1:1000 as the limit where cracking could occur in walls and partitions for end bays, however subsequent research has suggested this limit is too conservative. Adopting a differential settlement of 1:1000 and total settlement of 50mm for this project is a conservative and reasonable limit.

Mechanical settlements were estimated by numerical modelling. Groundwater drawdown was assessed by 3D numerical modelling, followed by calculation of consolidation settlements.

The tunnel alignment is in bedrock and settlement is anticipated to be negligible. Potential settlement effects outside of the Tawariki Street Shaft Site will be primarily from consolidation settlements related to dewatering and are considered to be less than minor.

No buildings or utilities are predicted to be adversely impacted by the construction of the tunnel or shaft components of the Grey Lynn Tunnel.

Recommendations

1. The settlement assessment was conducted based on available project geotechnical data from Addendum No. 2 to the Geotechnical Factual Report PWCIN-DEL-REP-GT-J-100452 and property, aerial photography, and contour data information available via Auckland Council’s geographical information system. More detailed information about the existing building and utility conditions should be collected during pre-construction surveys where appropriate.
2. The settlement assessment herein assumes shafts will be excavated with stiff wall support systems. Should the contractor elect to excavate through soils in the shaft using a flexible support system, mechanical settlements will likely be more than predicted herein. With the use of a flexible support system, the 50mm total settlement or 1:1000 differential settlement limits can be met with appropriate mitigation. If chosen by the contractor, a settlement assessment should be undertaken to confirm this.

3. The conditions from the Central Interceptor project (Consent Ref: 40836) should be adopted. These limits are:
   a) Differential Settlement Limit: 1:1,000 between any two adjacent settlement monitoring points required under the consent; or
   b) Total Settlement Limit: 50mm at any settlement monitoring point required under the consent.

The contractor should develop and implement a Monitoring and Contingency Plan as detailed in section 7.2.1

1.0 Introduction

Watercare Services Limited ("**Watercare**") is the water and wastewater service provider for Auckland. Watercare is proposing to construct a wastewater interceptor from Tawariki Street, Grey Lynn to Western Springs ("**Grey Lynn Tunnel**"). The Grey Lynn Tunnel will connect to the Central Interceptor at Western Springs.

1.1 Project Overview

The Grey Lynn Tunnel involves the elements shown in the drawings and outlined in more detail in the reports which form part of the application. These elements are summarised as follows.

1.1.1 Grey Lynn Tunnel

The Grey Lynn Tunnel involves construction, operation and maintenance of a 1.6km gravity tunnel from Western Springs to Tawariki Street, Grey Lynn with a 4.5m internal diameter, at an approximate depth of between 15 to 62m below ground surface, depending on local topography. The tunnel will be constructed northwards from Western Springs using a Tunnel Boring Machine ("TBM"). The Grey Lynn Tunnel will connect to the Central Interceptor at Western Springs via the Western Springs shaft site.

1.1.2 Tawariki Street Shaft Site

The Grey Lynn Tunnel also involves construction, operation and maintenance of two shafts and associated structures at Tawariki Street, Grey Lynn ("Tawariki Street Shaft Site").

The Tawariki Street Shaft Site will be located at 44-48 Tawariki Street where the majority of the construction works will take place. Construction works will also take place within the road reserve at the eastern end of Tawariki Street and a small area of school land (St Paul's College) bordering the end of Tawariki Street (approximately 150m2).

The Tawariki Street Shaft Site will involve the following components:

1.1.2.1 Main Shaft

- A 25m deep shaft, with an internal diameter of approximately 10.8m, to drop flow from the existing sewers into the Grey Lynn Tunnel;
- Diversion of the Tawariki Local Sewer to a chamber to the north of the shaft. This chamber will be approximately 12m long, 5m wide and 5m deep below ground, and will connect to the shaft via a trenched sewer;
- Diversion of the Orakei Main Sewer to a chamber to the south of the shaft. This chamber will be approximately 10m long, 5m wide and 11m deep below ground;
- Construction of a stub pipe on the western edge of the shaft to enable future connections (that are not part of this proposal) from the CSO network;
- Construction of a grit trap within the property at 48 Tawariki St to replace the existing grit trap located within the Tawariki Street road reserve. The replacement grit trap will be approximately 16m long, 5m wide and 13m deep below ground;

- Permanent retaining of the bank at the end of Tawariki Street to enable the construction of the chamber for the Orakei Main Sewer. The area of the bank requiring retaining will be approximately 44m long, 3m wide and 2m high; and
- An above ground plant and ventilation building that is approximately 14m long, 6m wide and 4m high. An air vent in a form of a stack will be incorporated into the plant and ventilation building and discharge air vertically via a roof vent. The vent stack will be designed with a flange to allow future extension of up to 8m in total height and approximately 1m in diameter in the unexpected event of odour issues.

1.1.2.2 Tawariki Connection Sewer Shaft – Secondary Shaft

A secondary shaft will be constructed at the Tawariki Street Shaft Site to enable the connection of future sewers (that are not part of this proposal) from the Combined Sewers Overflows ("CSO") network. This will involve the following components:

- A 25m deep drop shaft with an internal diameter of approximately 10.2m; and

- A sewer pipe constructed by pipe-jacking to connect the secondary shaft to the main shaft.

1.2 Construction Timeframe

The construction works for the main shaft, chambers and tunnel will occur at the same time as works for the Central Interceptor. Construction will be up to 2 ½ years total duration. The construction of the main shaft and chambers is estimated to take approximately 12 months initially, followed by a hiatus of several months waiting for the TBM to arrive at Tawariki Street Shaft Site. This will be followed by approximately 9 months of activity to remove the TBM and complete the internal structure of the main shaft.

The secondary shaft will be constructed in conjunction with the future sewers at a later date but (subject to need) within a 10-year period following construction of the main shaft and tunnel. The construction period for the secondary shaft and future sewer connections is estimated to be up to 2 years total duration.

1.3 Assessment

This report summarises the assessment undertaken to identify existing buildings and structures at risk of damage due to estimated settlement caused by shaft construction at Tawariki Street or tunnelling activities along the Grey Lynn Tunnel alignment.

This settlement assessment considers both the mechanical settlement associated with excavation of the Grey Lynn Tunnel and Tawariki Street shafts, and the potential consolidation settlement that could occur because of dewatering during construction.

2.0 Report Scope

This report describes the assessment of ground settlement that could result from the construction of the shafts at Tawariki Street and the tunnel between those shafts and the Central Interceptor at Western Springs, and the effects of these settlements on the existing buildings, services and infrastructure.

This report considers both the mechanical settlement associated with excavation and construction, and the consolidation settlement that could occur as a result of dewatering. Settlement will result from different aspects of the construction. Each of the sources is described in the report, along with the methodologies for analysing and combining the settlements.

This report does not assess any potential settlement at the Western Springs shaft. That analysis was included in Main Tunnel and Shafts – Settlement Assessment (Reference: DSCIN-DEL-REP-T-J-100252, 05 September 2017). This assessment follows a similar methodology as report ref. DSCIN-DEL-REP-T-J-100252.

2.1 Abbreviations and Facility Codes

Abbreviations used in this report are as shown in Table 2-1.

Table 2-1: Abbreviations used in this Report

Abbrev.	Description
CI	Central Interceptor
ECBF	East Coast Bays Formation
EPB	Earth Pressure Balance
GFR	Geotechnical Factual Report
GW	Groundwater
HDPE	High-density Polyethylene
ID	Internal Diameter
PVC	Polyvinyl Chloride
RCP	Reinforced Concrete Pipe
TBM	Tunnel Boring Machine
WSP	Welded Steel Pipe

Watercare facility codes for the shafts are as shown below in Table 2-2.

Table 2-2: Watercare Central Interceptor Facility Codes

Code	Facility Name
DSCIN	Central Interceptor Tunnel (including Grey Lynn Tunnel)
DSCIN009	Western Springs
DSCIN010	Tawariki Street
PWCIN	Project Wide

2.2 Related Reports

This report refers to the following project reports:

- Addendum No. 2 to Geotechnical Factual Report –Reference: PWCIN-DEL-REP-GT-J-100452, 15 June 2018.
- Groundwater Effects Assessment – Reference: WWA0047.
- Main Tunnel and Shafts – Settlement Assessment – Reference: DSCIN-DEL-REP-T-J-100252.
- Central Interceptor Main Works, Resource Consent Conditions – Reference: STD00538.01953, 19 December 2013.

3.0 Existing Environment

3.1 Overview

The Grey Lynn tunnel commences at the end of Tawariki Street, Grey Lynn, where it curves generally towards the south, terminating at Western Springs shaft where it ties in to the Central Interceptor.

3.2 Geology

The subsurface geology along the Grey Lynn Tunnel alignment is dominated by the weak sandstones and mudstones/siltstones of the Waitemata Group rocks, in particular the ECBF, with Tauranga Group alluvium deposits within the present day and paleo-drainage channels cut into the Waitemata Group rocks.

Geologic units that will be encountered along the Grey Lynn Tunnel alignment include the Tauranga Group, and the ECBF of the Waitemata Group, including isolated lenses of the Parnell Volcaniclastic Conglomerate (PVC) of the ECBF. Shaft excavations will encounter surficial deposits of Made Ground (undifferentiated fill), Undifferentiated Tauranga Group alluvium, residual ECBF soils and weathered ECBF rock.

A detailed geologic profile is provided in Appendix B.

3.3 Buildings and Land Use

The Grey Lynn Tunnel alignment is generally situated under residential areas that are characterised by 1–2 storey stand-alone buildings. There are some larger buildings, from north to south, which are:

- Grey Lynn Community Centre (2 storeys).
- 490 Richmond Rd: Child, Youth and Family, Ministry of Social Development (3 storeys, with basement).
- 172 Surrey Crescent: The Church of Jesus Christ of Latter-day Saints (2 storeys).

Pre-construction building structure and dilapidation surveys have not yet been conducted, but in general small residential buildings are anticipated to be wood frame and masonry structures, while larger buildings are anticipated to be mixed structural systems of wood, steel frames, masonry or concrete frame structures.

No historic buildings are shown on Heritage New Zealand’s map of historic buildings, New Zealand Heritage List/Rārangi Kōrero, within 50 metres of the alignment.

3.4 Utilities

Around the shaft site, utilities consist of pipes and conduits. Water retail pipes are most commonly 100mm ID pressurised pipe, made of asbestos cement or concrete-lined cast iron. These pipes are buried approximately 1 metre belowground. Wastewater retail pipes are commonly polyethylene, earthenware or concrete, with a wide variety of sizes, but typically 150 to 450mm ID. Stormwater-only pipes are most commonly concrete pipes 225mm ID and greater. Both wastewater and stormwater networks are gravity fed and are typically buried 1–4 metres belowground.

Retail services connect into the network via larger wholesale pipes. Water wholesale mains are typically concrete-lined steel pipes that are pressurised and buried approximately 1 metre deep. Wastewater wholesale pipes are commonly reinforced concrete and vary in depth. These pipes can be quite deep underground as they rely on gravity flow with the occasional pumping station. Some of these are larger utilities that were installed by tunnelling methods. The Orakei Main Sewer is one of these larger utilities and is situated to the north of the main shaft. This is a 1500mm diameter unlined brick sewer.

4.0 Anticipated Settlement Limits

4.1 Anticipated Consent Settlement Limits

.To compare expected movements to relevant limit criteria, the previous CI limits have been adopted for the purposes of identifying potential impacts. These limits are considered to be conservative; where damage is unlikely, and reasonable; where the limits can be met during construction. These limits are defined per CI Consent Condition 4.33 (Consent Ref: 40836) as:

- Differential Settlement Limit: 1:1,000 between any two adjacent settlement monitoring points required under the consent; or
- Total Settlement Limit: 50mm at any settlement monitoring point required under the consent.

4.2 Damage Trigger Levels

4.2.1 Buildings

Each structure within the zone of predicted settlement was evaluated for potential distortion due to settlement. The intent was not to precisely quantify the effect of settlement, but to determine which buildings are potentially at risk to damage and thus require further evaluation.

Criteria for allowable settlement of structures were originally a topic related to foundation engineering. The initial motivation for studies of building settlement and the degree of damage was to establish a basis for design of building foundations. The classic works and most comprehensive studies that set the early engineering precedents were by Skempton and MacDonald (1956) and Polshin and Tokar (1957). Additions to the experience base and summaries of world-wide practices developed over a number of years, such as by Bjerrum, 1963, and later in the United States, in particular Wahls (1981). These studies concluded that differential settlement was a key factor influencing observed building damage. Since most of the observed building damage appeared to be related to distortional deformations, ‘angular distortion’ (β) was used as a critical index of damage. Angular distortion is a measure of differential settlement. Limiting angular distortions and potential types of damage are given in below:

Table 4-1: Limiting Angular Distortion

Category of Potential Damage (after Wahls,1981)	β=δ/L (note 1)
Danger to machinery sensitive to settlement	1/750 (0.0013)
Danger to frames with diagonals	1/600 (0.0017)
Safe limit for no cracking of buildings (note 2)	1/500 (0.002)
First cracking of panel walls Difficulties with overhead cranes	1/300 (0.0033)
Tilting of high rigid buildings becomes visible	1/250 (0.004)
Considerable cracking of panel and brick walls Danger of structural damage to general buildings Safe limit for flexible brick walls, L/H >4b	1/150 (0.0067)
(1) β = angular distortion, δ = differential settlement, H = building height, and L = span length of beam or building. (2) Safe limits include a factor of safety.	

On recent urban tunnelling projects, angular distortion criteria on the order of 1/500 to 1/600 have been used as threshold values for decisions regarding settlement mitigation measures.

Prior to the work of Bjerrum (1963) and Wahls (1981), tunnels and deep excavations for tunnel construction promoted substantial research regarding the effects on existing structures of excavation-induced ground movements. The work of Mair et al. (1996), also referred to as the ‘Burland Method’, added the additional effects of horizontal ground movement to the effects of angular distortion as a further refinement to building damage prediction. Their work, supported by world-wide settlement data derived from actual field measurements of low-rise buildings, has gained worldwide acceptance in engineering practice.

4.2.2 Utilities

Each pipeline within the zone of predicted settlement was evaluated for potential distortion due to settlement. This distortion predominantly depends on pipe material and diameter, and the settlement profile. The trigger values shown in Table 4-2 are 80% of the maximum slope calculated (see Table 6-4).

Table 4-2: Utility Deformation Trigger Values

Utility Type (note 1)	Utility Dia. (mm)	Trigger Level
WSP	-	1:55
Cast-in-situ Concrete	-	1:75
PVC & HDPE	-	1:30
RCP	-	1:290
Ductile Iron Pipe	-	1:290

Utility Type (note 1)	Utility Dia. (mm)	Trigger Level
Vitrified Clay Pipe		1:290
Cast Iron Pipe	150	1:65
	200	1:80
	300	1:110
	400	1:150
	500	1:200
	600	1:270
	750	1:330
(1) HDPE = High-density polyethylene. PVC = Polyvinyl chloride. RCP = Reinforced concrete pipe. WSP= Welded steel pipe.		

5.0 Settlement Assessment Methodology and Results

5.1 Sources of Settlement Effect

The sources of settlement associated with the construction of the Grey Lynn Tunnel are the following:

- Mechanical settlement of the ground due to excavation of the tunnel. The relaxation of the rock and soil above the tunnel can result in settlement that occurs within a short period after the excavation is done, and is concentrated over the tunnel alignment.
- Mechanical settlement of the ground due to excavation of soil around the shaft. Lateral deflection of the temporary shaft walls used during excavation can result in settlement that occurs within a short period and is concentrated in the area immediately behind the wall.
- Consolidation of the ground due to extraction of groundwater. Depending on the compressibility properties of the soils, draining of the groundwater into the excavation can result in consolidation of the ground around shafts, with the resulting settlement occuring over a longer period. The watertight final linings proposed for the shaft will not allow for permanent draining of groundwater, and only of ground consolidation occurring during construction (short-term draining) has been considered.

5.2 Expected Areas of Effect

Settlement assessment is required on areas where tunnel excavation may result in excavation-related settlements exceeding measurable levels. These areas include lengths of tunnel where:

- The tunnel crown is in alluvium or residual soil and where there is no basalt or thin basalt cap (i.e. less than 1.5m thick).
- The tunnel crown is within 3m of the top of the ECBF rock (unweathered to highly weathered), which is overlain by alluvium and/or residual soils without a basalt cap of less than 1.5m.

Sections of the tunnel that do not meet the above criteria are excluded from the detailed settlement assessment because of favourable geological conditions that will result in negligible settlement.

The Tawariki Street shafts are analysed in Section 5.4 below. No exclusion criteria are applied for shafts. The major component of settlement is expected to be consolidation settlement due to groundwater drawdown at the shafts, where, settlement contours are expected to extend beyond site boundaries at the shaft site.

Alluvial deposits and uncontrolled fill deposits at the Tawariki Street shaft site are likely to exhibit some degree of settlement. The shaft site is surrounded by mostly residential properties. St Paul's College field is located to the east and non-residential properties nearby include Marist Catholic School and Our Lady of Perpetual Help, a church.

5.3 Tunnel Mechanical Settlement Assessment

5.3.1 Assumed Construction Methods

The Grey Lynn Tunnel will be constructed using an earth pressure balance (EPB) TBM and a single-pass segmental lining. The EPB TBM must be able to apply a positive pressure to the tunnel face, balancing the earth and groundwater pressures at all times to effectively control the ground and prevent groundwater inflows into the tunnel. The EPB TBM will operate in closed mode where soil or mixed face conditions are expected.

The one-pass gasketed precast concrete segmental lining system will be erected in the tail of the TBM concurrent with TBM advance. The annulus between the erected segmental lining and the excavation perimeter will be completely filled with grout. Annulus grouting provides continuous and intimate contact between the excavated ground and the precast concrete segmental lining and must be performed in a timely manner to reduce the risk of settlement resulting from closure of the annular tail shield void. Annulus grouting is also required to control the flow of water along the annulus, which may result in consolidation-related settlements.

5.3.2 Results

The Grey Lynn Tunnel is situated in competent ECBF rock and meets the exclusion criteria above. This analysis was conducted with a -2m/+2m vertical alignment tolerance. There are no areas of effect for tunnel related settlement.

5.4 Shaft Mechanical Settlement Assessment

5.4.1 Assumed Construction Methods

The shaft excavations for the Grey Lynn Tunnel include two shafts at the Tawariki Street Shaft Site. The preliminary layout of these shafts is shown relative to Tawariki Street in Appendix A. During construction, the site will be levelled, the three houses (44, 46, and 48 Tawariki St) located within the site boundary will be demolished/removed and the stormwater main crossing the site will be rerouted. The water services located in the footpath will be excavated where required.

The contractor shall select shaft excavation methods to be compatible with the ground conditions and ground behaviour anticipated. Conventional shaft excavation methods are anticipated. The contractor is responsible for the design of temporary excavation support systems, subject to the requirements in the specifications, compatible with the expected ground conditions and behaviours. The soil support system anticipated in the two shafts is secant piles. The rock support system is anticipated to include rock bolts, shotcrete and/or rock mesh.

The shafts will be built separately in two stages, with the main shaft DSCIN010B (drop shaft/TBM receiving shaft) being completed and put into service before construction of the secondary shaft DSCIN010A commences. Construction of the secondary shaft is anticipated to commence within a 10-year period following the construction of the main shaft.

Settlement at the shafts is primarily dependent on excavation support rigidity in overburden soils; settlement at shafts is dictated by the degree the shaft wall can flex inwards and allow for soil movement. Steel pipe casings, caissons and secant piles limit this movement and are considered rigid, whereas shafts constructed with sheet piles will result in more settlement. The anticipated rigidity of each shaft is given in Table 5-1.

Table 5-1: Assume Shaft Rigidity

Shaft No.	Shaft Name	Anticipated Soil Support Type	Rigidity
DSCIN010A	Grey Lynn (drop shaft)	Secant piles	Rigid
DSCIN010B	Grey Lynn (work shaft)	Secant piles	Rigid

Soil-structure interaction sensitivity modelling indicates larger settlements unless measures are taken to stabilise the ground and minimise the ground loss during the shaft excavation in soils. Upward displacements from invert heave are predicted for flexible support systems unless mitigation measures such as excavation of soil ‘in the wet’ (i.e. shaft flooded with underwater grab) or other methods are utilised to provide support pressure in soils, or relief of groundwater pressures, prior to reaching the top ECBF bedrock. Once excavation reaches ECBF bedrock, the construction method switches to dry excavation.

5.4.2 Methodology

Two dimensional Fast Lagrangian Analysis of Continua (FLAC 2D Version 7.0, Itasca Consulting Group) was used to model soil-structure interaction and yield a settlement profile for both Tawariki Street shafts. Figure 5-1 shows the model generated in FLAC.

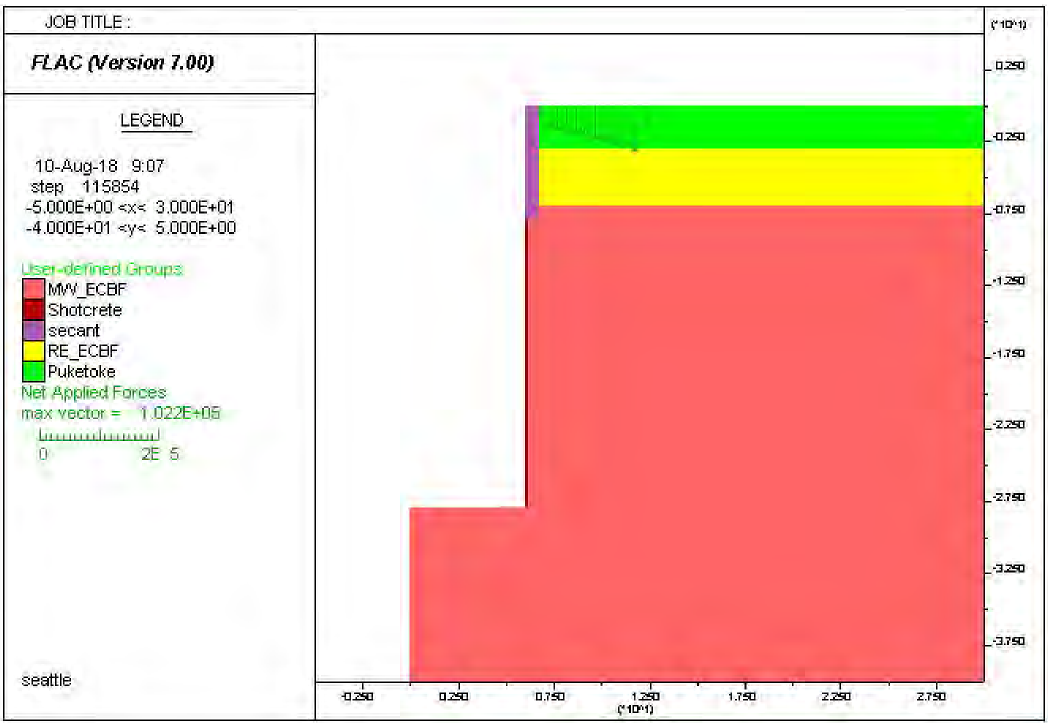

Figure 5-1: Model Cross Section

Modelling procedures, assumptions and parameters can be found in Appendix C.

5.4.3 Summary of Results

Table 5-2 summarises the maximum ground surface vertical displacement due to the excavation of the Tawariki Street Shafts, as also shown in Figure 5-2. Based on these results, a heave of up to approximately 2 mm is predicted near the shaft wall.

Table 5-2: Summary of Maximum Displacements at ground surface

Case	Maximum Vertical Displacement at Ground Surface (mm)
Settlement at ground surface	+1.84 (heave)

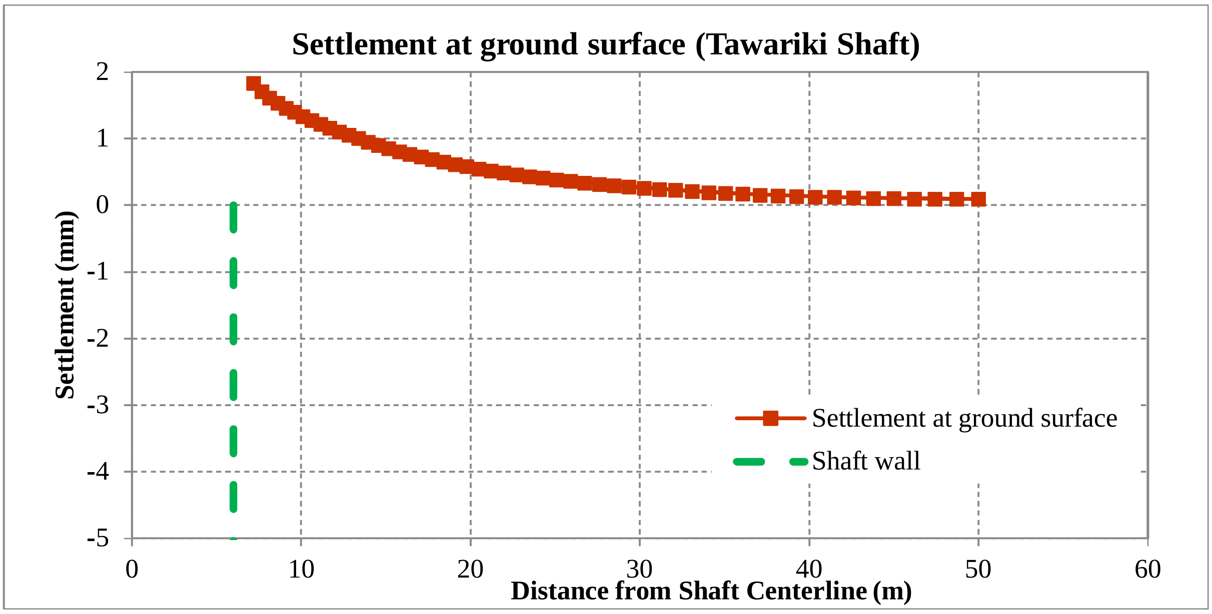

Figure 5-2: Ground Surface Settlement Model Output for Tawariki Shaft

It is unlikely that the small upward movement on the surface predicted by the numerical model would happen or be detected. The model predicted some small wall deflection (about 4mm). Past experiences from similar projects indicated that the wall deflection would result in some surface settlement. Therefore, based on engineering judgement, previous practical experience with similar shafts in Auckland and considering the predicted wall deflection, it is reasonable to assume a maximum surface settlement of about 4mm near the shaft wall. An approximation of the settlement distribution on the surface adjacent to the shaft is also assumed and depicted in Figure 5-3, where surface settlement approaches zero value as the distance to the shaft wall increases.

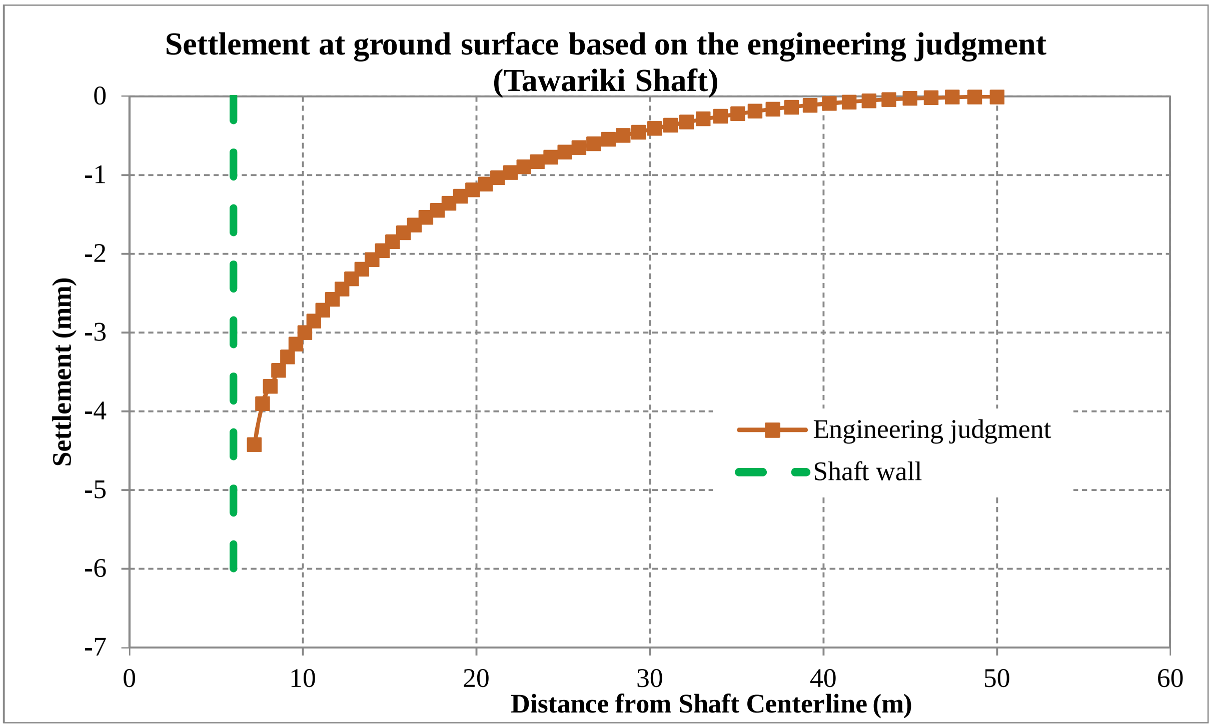

Figure 5-3: Ground Surface Settlement from Shaft Excavation

Table 5-3 summarises the maximum horizontal deflection due to the excavation of the Tawariki Street Shafts. See Figure 5-4 for the predicted shaft wall deflection (ground surface to shaft invert (at 28m below ground surface)). Based on these results, maximum predicted horizontal deflection is in the range of 4.5mm. The maximum deflection is observed at about 3 m above the shaft bottom.

Table 5-3: Summary of Maximum Horizontal Displacement at Shaft Wall

Case	Maximum Horizontal Displacement at Shaft Wall (mm)
Shaft wall deflection	4.5

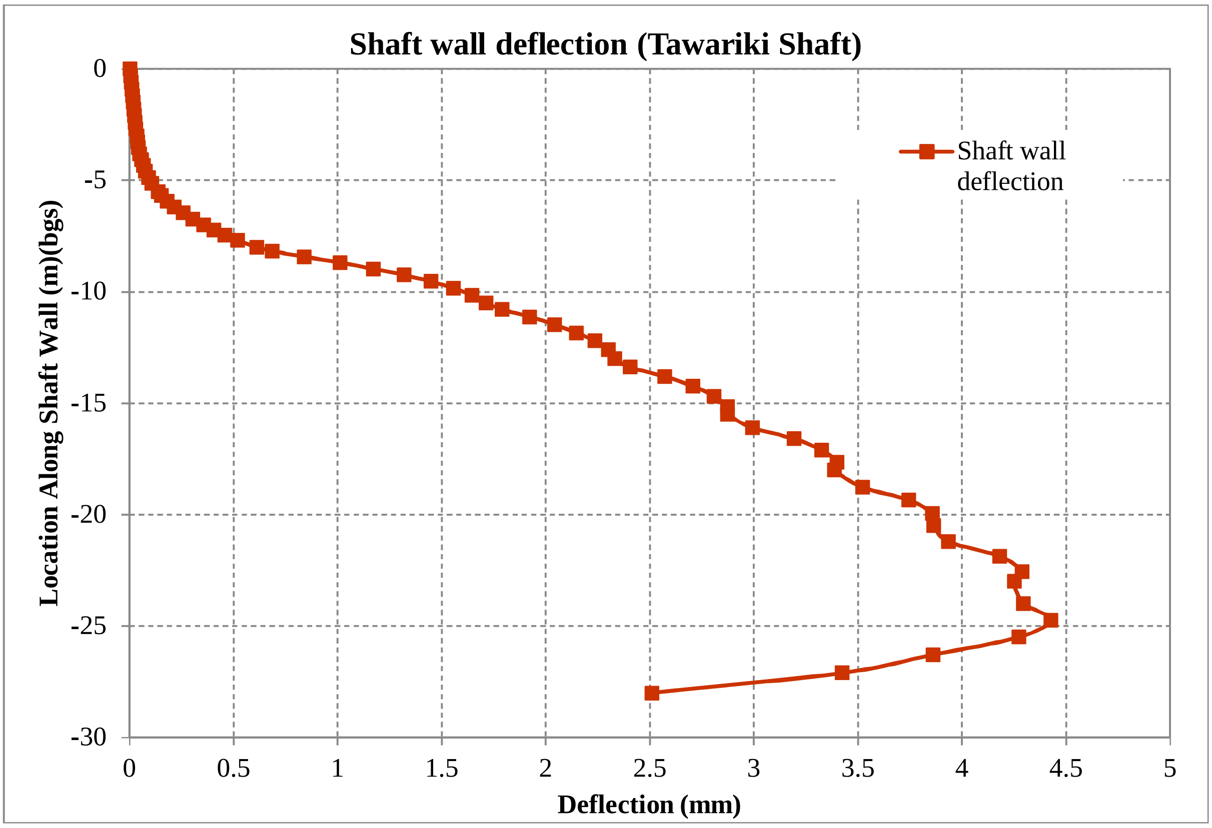

Figure 5-4: Shaft Wall Deflection for Tawariki Shaft (positive deflection is towards the excavation)

5.5 Consolidation Settlement Assessment

The consolidation settlement assessment consists of two main steps:

- Predicting groundwater drawdown; and
- Predicting the ground settlement response to groundwater drawdown.

5.5.1 Methodology for Groundwater Drawdown Prediction

The groundwater drawdown assessment is described in Grey Lynn Tunnel – Tawariki Street Shaft – Groundwater Effects Assessment (Reference: WWA0047).

Several drawdown models were created to model various indicative shaft hydrogeologic conditions and configurations. These models were analysed for transient conditions, and a conservative approach was taken for construction methodology. Recharge from rainfall and watercourses was considered where appropriate. Excavation support systems in soils were modelled to have a low conductivity to impede groundwater flow directly into the shafts. The excavated shafts were modelled as ‘open’ for a construction period of 2 years, before the permanent impermeable linings are installed and groundwater conditions return to pre-existing levels.

Of these models, scenarios 4 and 6 best represent the Tawariki shaft site construction. Both scenarios have a shaft lining with a permeability of 10-9 ms-1, but in scenario 6 the lining extends to 7 m BGL.

The drawdown model selected to calculate the consolidation settlement is scenario 6 (see Figure 5-5) because this scenario best approximates the temporary condition during construction and is the most conservative for settlement prediction out of scenarios 4 and 6.

5.5.2 Methodology for Groundwater Drawdown Settlement Assessment

Dewatering settlement of the soils surrounding each shaft was analysed using the following method:

$$\delta = m_v \, \Delta\sigma' \, H$$

Where δ is settlement, m_v is the one-dimensional volume of compressibility (m^2/kN), and $\Delta\sigma'$ is change in vertical effective stress at mid-height of the compressible layer depth H. The value of m_v was derived from one-dimensional compressibility results from laboratory tests on representative soil samples at the Tawariki Street Shaft site.

Settlement is calculated for different depths and different geological units over the profile and then added together to give a total settlement for that point. Geological information is based on the Addendum No. 2 to Geotechnical Factual Report (PWCIN-DEL-REP-GT-J-100452) .

5.5.3 Summary of Results

The predicted consolidation settlements were computed using Microsoft Excel, as presented in Figure 5-5.

5.5.3.1 St Paul's College Field

No geotechnical investigations, and hence no laboratory consolidation testing, was performed on St. Paul's College property. However, there is evidence in report 'Grey Lynn Tunnel: Archaeological and Historic Heritage Assessment, October 2018' by Clough & Associates that the playing field is constructed on top of man-made fill. The settlement contours shown in Appendix A for the St. Paul's College playing field are based on a conservative assumption for thickness and compressibility of fill materials under the playing field. This should be considered a "worst case" settlement for this playing field, as the dewatering levels and compressibility assumptions are both from conservative analyses.

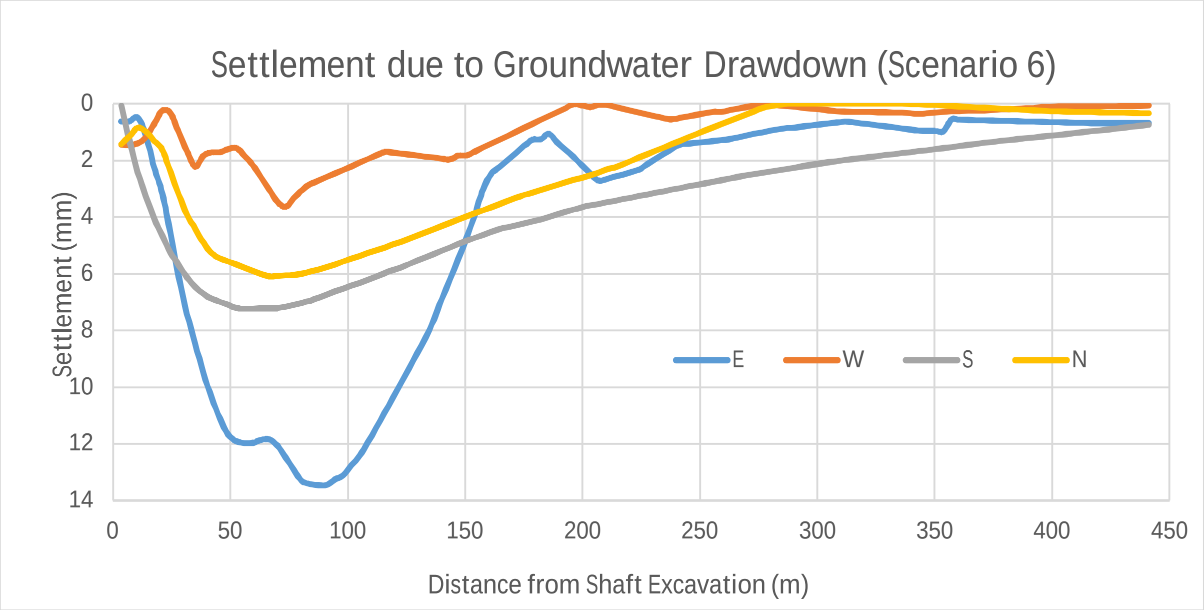

Figure 5-5: Settlement due to Groundwater Drawdown (the settlement profile for the east, west, south and north directions, where 0m is at the extent of shaft excavation and distance increases away from the shaft)

5.6 Combined Settlement

Tunnel mechanical, shaft mechanical and consolidation settlements are theoretically cumulative and can be combined arithmetically. There were no predicted tunnel mechanical settlements so only shaft mechanical and consolidation settlements have been plotted.

The combined settlement is shown in Figure 5-6 and the combined settlement contour drawings are provided in Appendix A. The maximum settlement is 14mm. This occurs over the playing fields within St Paul’s College to the east of the shafts. The areas that exceed differential slopes of 1:1000, 1:500 and 1:200 are shown where applicable. The 1:1000 slope limit is only exceeded in a limited area around each shaft within the Tawariki Street Shaft Site.

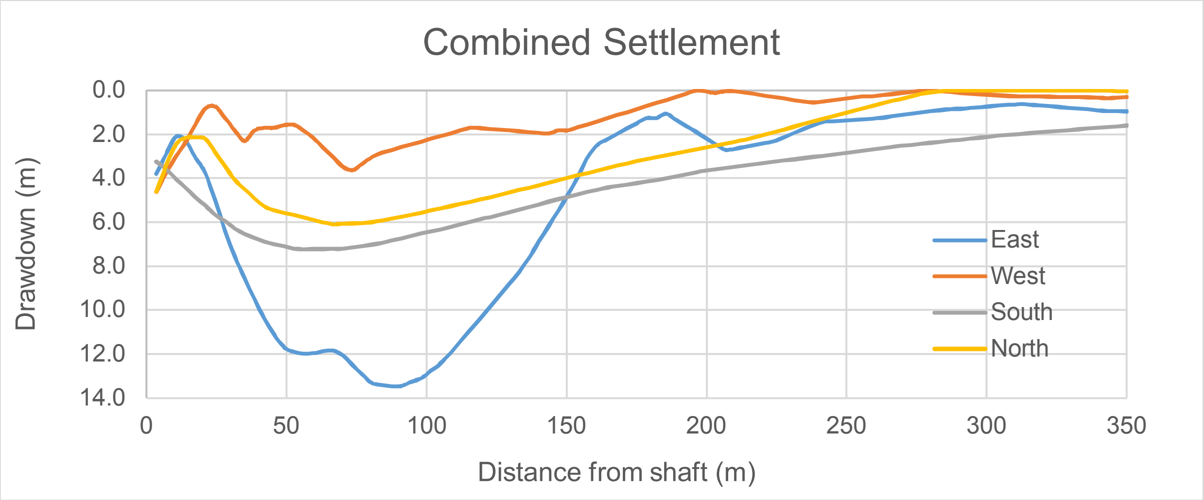

Figure 5-6: Combined Settlement (the settlement profile for the east, west, south and north directions, where 0m is at the extent of shaft excavation and distance increases from the shaft)

6.0 Effects Assessment

Each structure and identified utility within the zone of predicted settlement was evaluated for potential damage. Differential settlement is a key factor influencing predicted damage.

6.1 Potential for Damage to Buildings

The procedure to assess building damage is a two-step process:

1. Identify all buildings in settlement zones exceeding 1:1000 differential or 50mm total settlement.
2. Perform a potential damage assessment on these buildings per the Burland Method described in Section 4.2.1(if required).

As shown in Table 6-1 the only buildings that exceed these criteria are those on 44, 46, and 48 Tawariki Street. These buildings are to be demolished or removed to enable the project to proceed. All other buildings experience less than 1:1000 differential and 50mm total settlement.

Table 6-1: Building Settlement Screening

Bldg. Number	Address	50mm+	1:1000 – 1:500	1:500 – 1:200	1:200 +	Note
1	44 Tawariki St	-	✓	-	-	To be demolished or removed
2	46 Tawariki St	-	✓	-	-	To be demolished or removed
3	48 Tawariki St	-	✓	-	-	To be demolished or removed

No further analysis is required for buildings.

6.2 Potential for Damage to Utilities

Three types of settlement impacts typically affect buried pipeline utilities, as summarised in O’Rourke and Trautmann (1982):

- Tensile pull-apart at joints, caused by relative tensile axial movements along the pipeline.
- Opening of joints between pipe segments, due to relative rotation between two pipe segments.
- Straining of pipe caused by flexural deformations, and lateral deformations that lead to rupture or intolerable deformation.

The first two impacts focus on failures occurring at well-defined joints and would be more likely to occur in fairly rigid, jointed pipe such as reinforced concrete pipe. The third type of impact is caused by differential settlements and lateral ground movements and is most likely to occur in flexible pipelines with well-designed rigid joints that can take significant rotation, such as welded steel pipelines.

The maximum allowable value for each of the above deformation modes is given in Table 6-2 for each pipe material, based on the recommendations of O’Rourke and Trautmann (1982).

Table 6-2: Building Settlement Screening

Utility Type (note 1)	Utility Dia. (mm)	Allowable Joint Displacement (mm)	Allowable Joint Rotation (º)	Allowable Tensile Strain (µ mm/mm)
WSP	-	NA	NA	600
Cast-in-situ Concrete	-	NA	NA	300
PVC & HDPE	-	NA	NA	2000
RCP	-	10.2	0.250	300
Ductile Iron Pipe	-	10.2	0.250	600
Cast Iron Pipe	150	2.1	1.140	400
	200	2.1	0.930	400
	300	2.1	0.670	400
	400	2.0	0.490	400
	500	1.8	0.370	400
	600	1.6	0.270	400
	750	1.6	0.220	400

(1) HDPE = High-density polyethylene. PVC = Polyvinyl chloride. RCP = Reinforced concrete pipe. WSP= Welded steel pipe.

The Grey Lynn Tunnel alignment was screened for any sensitive services, such as the Marsden to Wiri gas line, fibre optic lines and water wholesale mains. Damage to these utilities has a much higher consequence, so they are screened separately. No gas lines, fibre optic lines or water wholesale mains were identified.

All services that intersected a zone of settlement exceeding 1:1000 differential or 50mm total settlement were then tabulated in Table 6-3. The only section which met these criteria were the areas directly around each shaft within the Tawariki Shaft Site. As can be expected, large-diameter wastewater and combined stormwater and wastewater mains were identified within settlement zones because the Tawariki Street shafts will be intercepting these wastewater flows. These pipes are not under pressure and have been analysed in the same manner as all the other services.

Table 6-3: Infrastructure within Settlement Zone

Pipe Code (assigned)	Utility Dia. (mm)	Material	Analysis Material	Predicted Slope
SS01	375	Ceramic/Earthenware	Vitrified Clay	1:1000
SW01	150	PE	PVC & HDPE	1:1000
W01	100	PVC	PVC & HDPE	1:1000
W02	20	PE	PVC & HDPE	1:1000

Using the utility deformation criteria in Table 6-4, maximum slopes for utilities at risk around shafts were back-calculated based on typical utility lengths and are presented in Table 6-4. Based on this analysis, a 1:500 maximum slope was identified irrespective of utility type and diameter to screen these utilities for analysis. No utilities were found to exceed this value, aside from utilities that will be connected into the system.

The CI construction specification requires the contractor to support existing utilities where they connect into the drop shaft or related control chambers. The same approach should be adopted for the Grey Lynn Tunnel project.

Table 6-4: Utility Deformation Maximum Slopes

Utility Type (note 1)	Utility Dia. (mm)	Maximum Slope
WSP	-	1:41
Cast-in-situ Concrete	-	1:58
PVC & HDPE	-	1:22
RCP	-	1:229
Ductile Iron Pipe	-	1:229
Vitrified Clay Pipe		1:229
Cast Iron Pipe	150	1:50
	200	1:62
	300	1:86
	400	1:117
	500	1:155
	600	1:212
	750	1:260
(1) HDPE = High-density polyethylene. PVC = Polyvinyl chloride. RCP = Reinforced concrete pipe. WSP= Welded steel pipe.		

7.0 Monitoring and Mitigation

The following monitoring and mitigation strategies are considered good practice in tunnelling and shaft construction and aide in managing effects.

7.1 Preconstruction Monitoring

Pre-construction monitoring is recommended to establish baseline ground surface movements associated with seasonal variations in soil moisture content and associated shrink/swell behaviour unrelated to construction of the Grey Lynn Tunnel and both Tawariki Street Shafts. The monitoring should be undertaken over a minimum period of 12 months.

7.2 Construction Monitoring

7.2.1 Surface Settlement Monitoring

Complementary to the preconstruction monitoring, and before any shaft sinking or tunnelling activities commence, the contractor should develop and implement a surface settlement monitoring programme This programme should be described in a Monitoring and Contingency Plan and should include:

- a location Plan of settlement and building (if required) deformation marks;
- details of the shaft wall monitoring described in Section 7.2.2;
- deformation and settlement Alert and Alarm Levels (Trigger Levels) to be utilised for early warning of settlement with the potential to cause damage to buildings and services and details of the processes used to establish, and if necessary, to review these triggers;
- details on the procedures for notification of the Manager in the event that Trigger Levels are exceeded;

7.2.2 Shaft Monitoring

Shaft instrumentation in both shafts is anticipated to consist of shaft convergence and/or ground movement measurements in soils, e.g. inclinometer monitoring is recommended in the deeper soil profiles around shafts.

7.2.3 Utilities Monitoring

No utilities are at risk for settlement and utility damage, aside from those utilities directly linked with construction. Utility monitoring is therefore not required for utilities.

7.2.4 Tunnel Convergence Monitoring

In-tunnel instrumentation will consist of instruments installed to monitor convergence of the precast segmental lining, as required to verify the design and stability of the lining. A typical instrumented section of tunnel will consist of an array of convergence survey reference points, which will be shown on the Drawings. Monitoring requirements in the tunnel will be provided in the geotechnical instrumentation and monitoring specification.

7.3 Proactive Mitigations

7.3.1 Pressurisation of the TBM

The requirement for an EPB TBM with annular grouting of the segmental lining through the TBM tail shield will minimise mechanical settlements related to tunnelling. Operation of the TBM in closed-mode or partial-mode will prevent dewatering around the tunnel, thus minimising or eliminating risk of consolidation settlements due to dewatering.

7.3.2 Watertight Shafts

Watertight or very low permeability shaft support systems will be specified, and dewatering of soil materials will be minimised. Therefore, consolidation settlements resulting from dewatering will be reduced significantly.

7.3.3 Building Protection Measures

Building and utility protection methods, if required, will be the responsibility of the contractor based on the selected construction means and methods.

8.0 Conclusions

The construction of the Grey Lynn Tunnel does not produce any measurable settlement due to the favourable alignment in ECBF sandstone. The construction of the Tawariki Street shafts produces mechanical and groundwater settlement, that has been modelled and combined to produce a settlement contour plot. The maximum settlement from this is 14mm occurring over the playing fields within St Paul's College to the east of the shafts. Settlements of this magnitude are insignificant in a greenfield environment and the potential settlement effects are considered to be less than minor.

No buildings or utility services are predicted to be impacted by the construction of both the tunnel and shaft components of the Grey Lynn Tunnel

9.0 References

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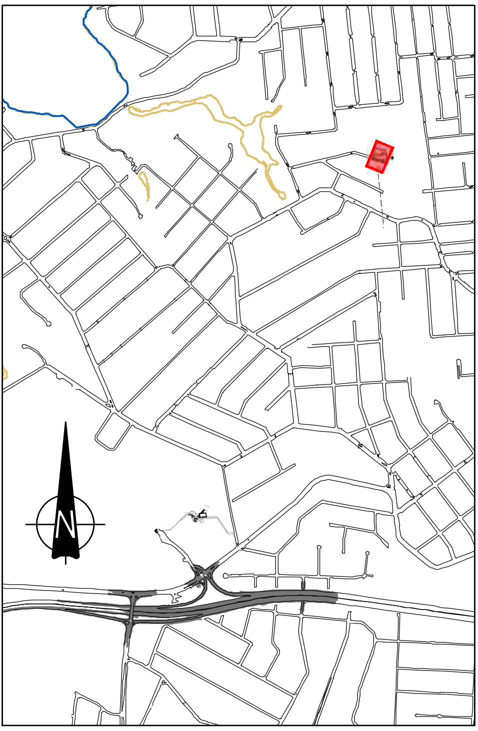
Appendix A

Settlement Contours

SCALE 1:600

12 6 0 12 24 36 48 60m

SETTLEMENT PLAN
SCALE. 1:600



KEY PLAN
SCALE 1:15,000

LEGEND

DIFFERENTIAL SETTLEMENT SLOPE

- ☐ 1:1000 - 1:500
- ☐ 1:500 - 1:200
- ☐ > 1:200

——— 2 ———	SETTLEMENT CONTOUR LINE (mm)
– SW – SW –	EX. STORMWATER
– SS – SS –	EX. WASTEWATER
– ▼ – ▼ –	EX. WATER SUPPLY

JACOBS

AECOM

McMILLEN JACOBS ASSOCIATES

GREY LYNN TUNNEL
TAWARIKI STREET SHAFT SETTLEMENT CONTOUR PLOT

Watercare

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Plot Date: Jan 30, 2019 - 4:33pm

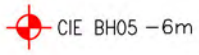
Appendix B

Geological Profile

PLAN

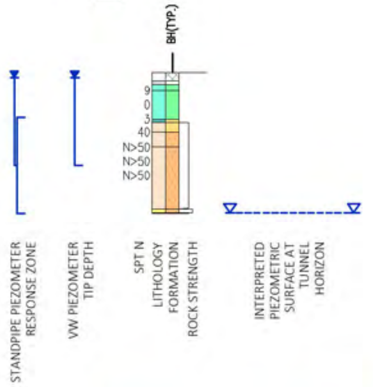
SCALE. 1:1250

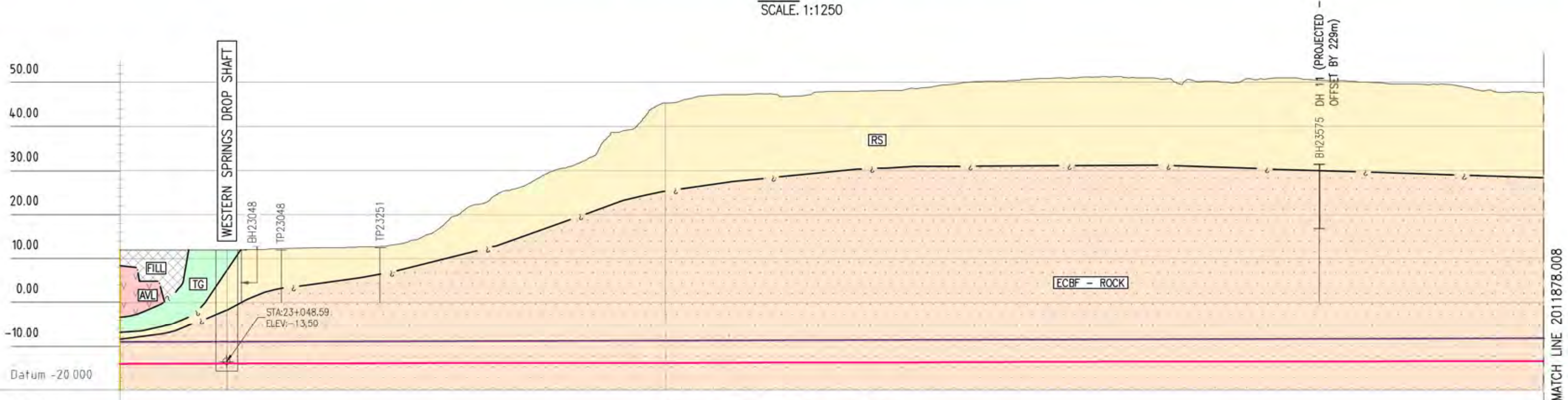
LEGEND:



Borehole ID and offset from section.
(Negative numbers are into the page.)

	BH LEFT	PLAN AND SECTION	BH RIGHT	
No core	☐	▨	▨	Made Ground (FILL)
Fill and topsoil	■	🟩	🟩	Recent Alluvium (Q1a)
Basalt	🟥			TAURANGA GROUP
Other volcanics	🟥	🟩	🟩	Undifferentiated Tauranga Group (TG)
Clay and clay dominated mixtures	🟩	🟦	🟦	Kaawa Formation (KAAWA)
Organic clay and organic clay dominated mixtures	🟩			AUCKLAND VOLCANIC FIELD (PLEISTOCENE)
Peat	🟫	🟥	🟥	Basalt (AVL)
Silt and silt dominated mixtures	🟦	🟧	🟧	Tuff / Ash / Scoria (AVT)
Organic silt and organic silt dominated mixtures	🟦			WAITEMATA GROUP (MIOCENE)
Sand and sand dominated mixtures (clean)	🟦	🟨	🟨	Residually to Highly Weathered - Cohesive Soils (RS)
Sand and sand dominated mixtures (with fines)	🟦	🟫	🟫	Residually to Highly Weathered - Granular Soils (RS)
Organic sand and organic sand dominated mixtures	🟦	🟪	🟪	Parnell Volcaniclastic Conglomerate (PVC)
Sandstone	🟨	🟧	🟧	Moderately Weathered to Unweathered ECBF (ECBF-ROCK)
Mudstone	🟧			
Interbedded mudstone and sandstone	▨			
Conglomerate	🟪			
Coal	■			





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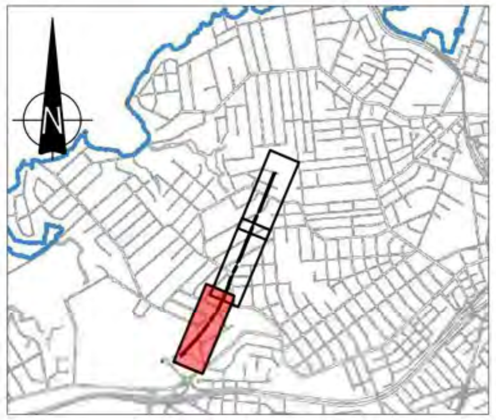
INVERT LEVEL (mRL)	-14.00	-13.95	-13.90	-13.85	-13.80	-13.75	-13.70	-13.65	-13.60	-13.55	-13.50	-13.45	-13.40
DEPTH TO INVERT (m)	26.00	26.47	32.82	44.18	59.20	60.81	61.90	63.97	63.89	63.89	64.02	63.12	45.30
CHAINAGE (m)	23050	23100	23150	23200	23250	23300	23350	23400	23500	23500	23550	23600	23650

LONG SECTION

SCALE. 1:1250 H 1:2500 V



NOTES:

1. ALL SETTING OUT COORDINATES ARE TO NEW ZEALAND TRANSVERSE MERCATOR PROJECTION AND ALL LEVELS ARE TO AUCKLAND 1946 DATUM.
2. GROUND ELEVATION MODEL DEVELOPED FROM AUCKLAND COUNCIL GIS CONTOUR DATA (2013).
3. GEOLOGY CONTACTS INTERPOLATED FROM AVAILABLE BOREHOLES, GNS MAP 1:250 000 (AUCKLAND) AND AUCKLAND COUNCIL GIS CONTOUR (LIDAR) DATA (2017).

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ISSUE	DATE	AMENDMENT	BY	APPD.
1	31.01.19	ISSUED TO WATERCARE	VL	VSR
0	12.12.18	ISSUED TO WATERCARE	LD	VSR

	BY	DATE
DESIGNED	V. LEE/S. BURGESS	01.19
DES. CHECKED	V. ROMERO	01.19
DRAWN	C. VALENCIA	01.19
DWG. CHECKED	V. LEE	01.19
PROJECT LEADER	N. KAY	01.19
INFRAST'R APP'D		

OPERATIONS

INFRASTRUCTURE

Watercare

GREY LYNN TUNNEL
SITE GENERAL
GREY LYNN TUNNEL LONG SECTION – SHEET 1 of 3

CAD FILE 2011878.007
DATE 31.01.2019
ORIGINAL SCALE A3 AS SHOWN
CONTRACT No. 5747
REF. No. CI–GEOTECH
ISSUE 1
DWG. No. 2011878 .007

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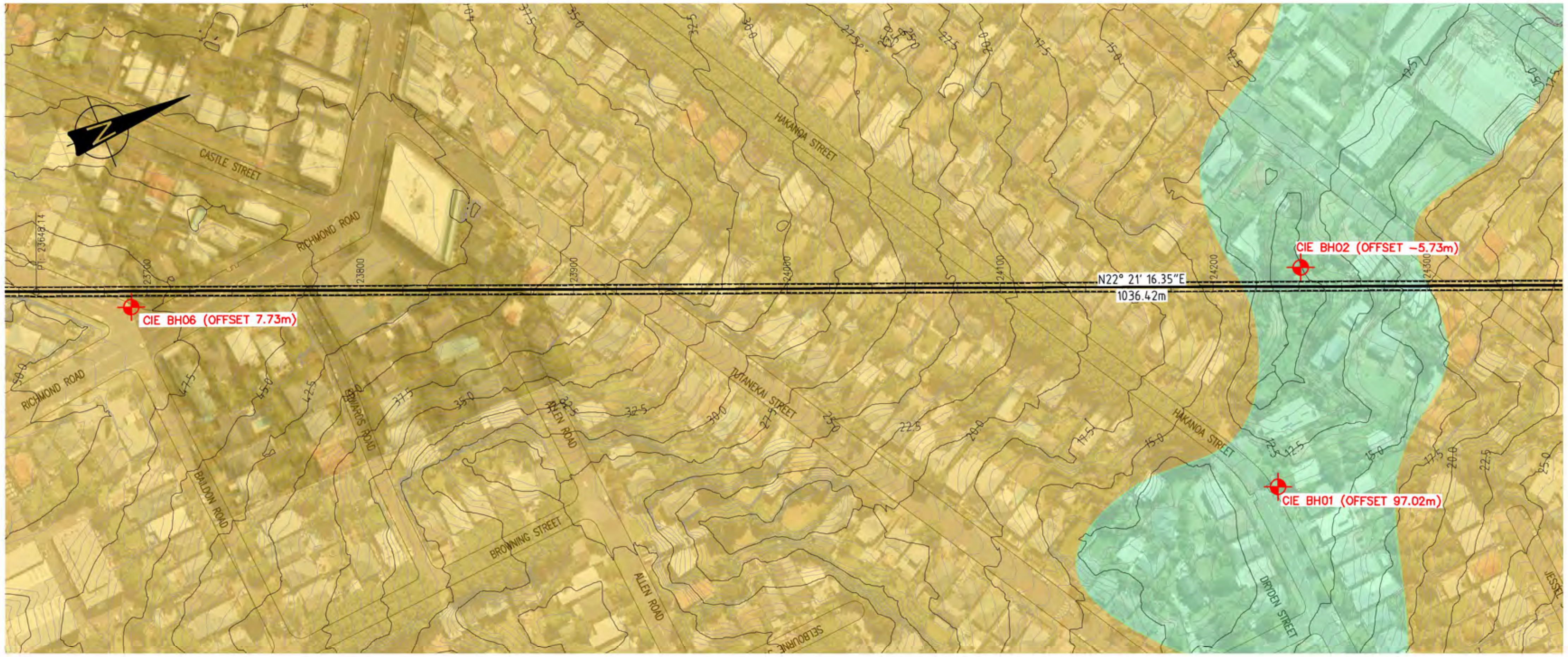


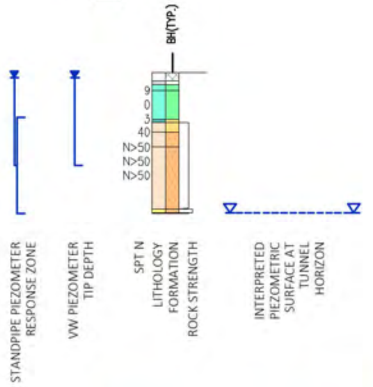
PLAN
SCALE. 1:1250

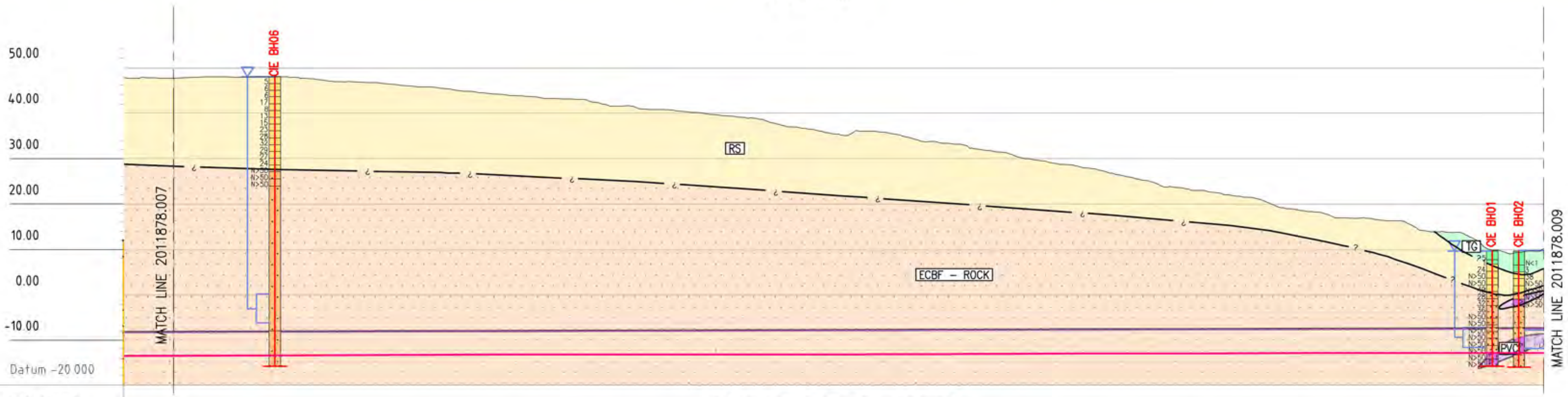
LEGEND:

CIE BH05 −6m Borehole ID and offset from section. (Negative numbers are into the page.)

	BH LEFT
No core	☐
Fill and topsoil	▨
Basalt	🟥
Other volcanics	🟥
Clay and clay dominated mixtures	🟩
Organic clay and organic clay dominated mixtures	🟩
Peat	🟫
Silt and silt dominated mixtures	🟦
Organic silt and organic silt dominated mixtures	🟦
Sand and sand dominated mixtures (clean)	🟦
Sand and sand dominated mixtures (with fines)	🟦
Organic sand and organic sand dominated mixtures	🟦
Sandstone	🟨
Mudstone	🟧
Interbedded mudstone and sandstone	▨
Conglomerate	🟪
Coal	■

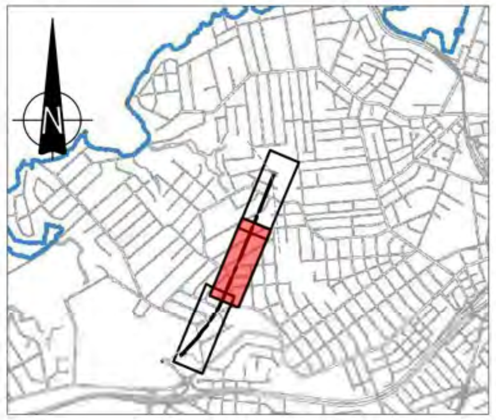
PLAN AND SECTION	BH RIGHT	
▨	▨	Made Ground (FILL)
🟩	🟩	Recent Alluvium (Q1a)
TAURANGA GROUP		
🟩	🟩	Undifferentiated Tauranga Group (TG)
🟦	🟦	Kaawa Formation (KAAWA)
AUCKLAND VOLCANIC FIELD (PLEISTOCENE)		
🟥	🟥	Basalt (AVL)
🟧	🟧	Tuff / Ash / Scoria (AVT)
WAITEMATA GROUP (MIOCENE)		
🟨	🟨	Residually to Highly Weathered - Cohesive Soils (RS)
🟫	🟫	Residually to Highly Weathered - Granular Soils (RS)
🟪	🟪	Parnell Volcaniclastic Conglomerate (PVC)
🟧	🟧	Moderately Weathered to Unweathered ECBF (ECBF-ROCK)





PIPE DIAMETER AND GRADE	4500 MM ID PRECAST SEGMENTAL LINING (1:1000)												
EXISTING GROUND LEVELS (mRL)	47.77	47.88	46.10	43.81	41.55	39.00	36.08	32.25	27.95	23.00	18.32	14.03	9.79
INVERT LEVEL (mRL)	-13.40	-13.35	-13.30	-13.25	-13.20	-13.15	-13.10	-13.05	-13.00	-12.95	-12.90	-12.85	-12.80
DEPTH TO INVERT (m)	45.30	61.23	59.40	57.06	54.75	52.15	49.18	45.30	40.95	35.95	31.22	26.88	22.58
CHAINAGE (m)	23650	23700	23750	23800	23850	23900	23950	24000	24050	24100	24150	24200	24250

LONG SECTION
SCALE. 1:1250 H 1:2500 V



NOTES:

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3. GEOLOGY CONTACTS INTERPOLATED FROM AVAILABLE BOREHOLES, GNS MAP 1:250 000 (AUCKLAND) AND AUCKLAND COUNCIL GIS CONTOUR (LIDAR) DATA (2017).

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DWG. CHECKED	V. LEE	01.19
PROJECT LEADER	N. KAY	01.19
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Watercare

GREY LYNN TUNNEL
SITE GENERAL
GREY LYNN TUNNEL LONG SECTION – SHEET 2 of 3

CAD FILE 2011878.008
DATE 31.01.2019
ORIGINAL SCALE A3 AS SHOWN
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REF. No. CI–GEOTECH
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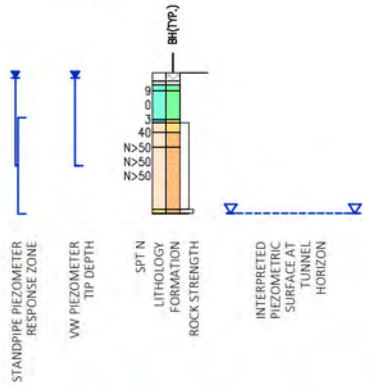
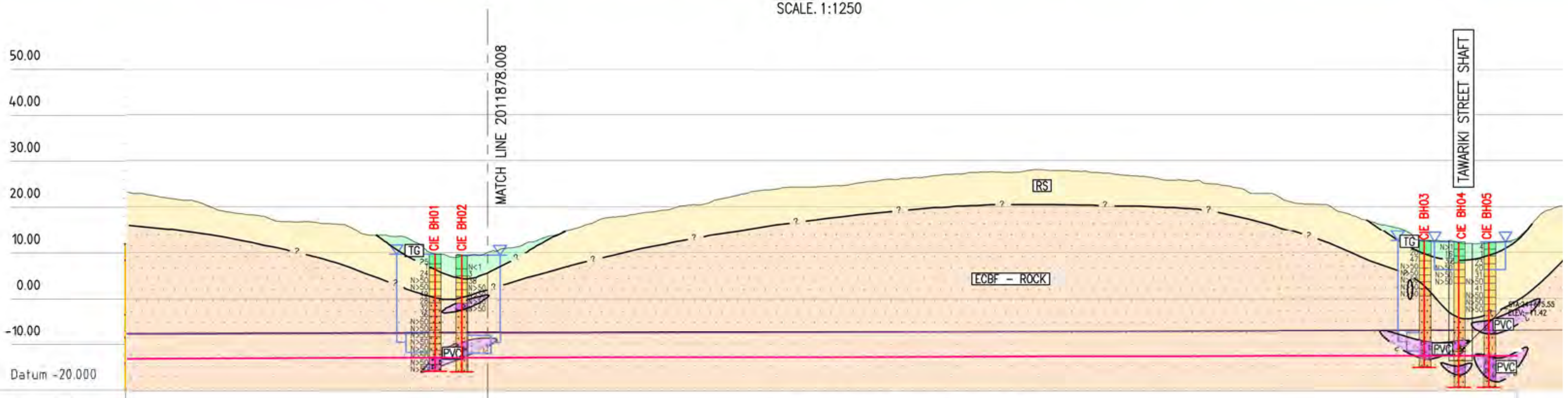
PLAN
SCALE. 1:1250

LEGEND:

CIE BH05 −6m Borehole ID and offset from section. (Negative numbers are into the page.)

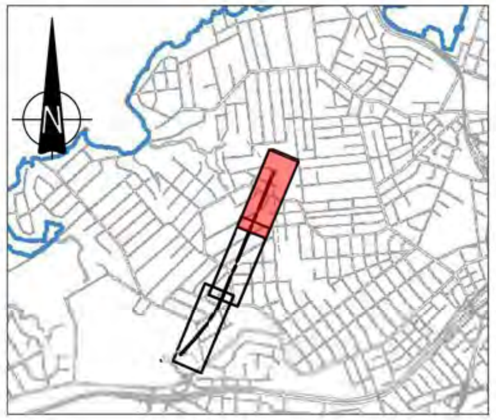
BH LEFT			PLAN AND SECTION	BH RIGHT	
No core	☐		▨	▨	Made Ground (FILL)
Fill and topsoil	■		▨	▨	Recent Alluvium (Q1a)
Basalt	🟥	TAURANGA GROUP			
Other volcanics	🟥		🟩	🟩	Undifferentiated Tauranga Group (TG)
Clay and clay dominated mixtures	🟩				
Organic clay and organic clay dominated mixtures	🟩		🟦	🟦	Kaawa Formation (KAAWA)
Peat	🟫	AUCKLAND VOLCANIC FIELD (PLEISTOCENE)			
Silt and silt dominated mixtures	🟦		▨	🟥	Basalt (AVL)
Organic silt and organic silt dominated mixtures	🟩		▨	🟧	Tuff / Ash / Scoria (AVT)
Sand and sand dominated mixtures (clean)	🟦	WAITEMATA GROUP (MIOCENE)			
Sand and sand dominated mixtures (with fines)	🟦		🟨	🟨	Residually to Highly Weathered - Cohesive Soils (RS)
Organic sand and organic sand dominated mixtures	🟦		▨	🟫	Residually to Highly Weathered - Granular Soils (RS)
Sandstone	🟨		▨	🟪	Parnell Volcaniclastic Conglomerate (PVC)
Mudstone	🟧		▨	🟧	Moderately Weathered to Unweathered ECBF (ECBF-ROCK)
Interbedded mudstone and sandstone	▨				
Conglomerate	🟪				
Coal	■				





PIPE DIAMETER AND GRADE	4500 MM ID PRECAST SEGMENTAL LINING (1:1000)												
EXISTING GROUND LEVELS (mRL)	23.00	18.32	14.03	9.79	17.22	21.44	24.63	27.13	28.01	25.86	21.40	15.23		
INVERT LEVEL (mRL)	-12.95	-12.90	-12.85	-12.80	-12.75	-12.70	-12.65	-12.60	-12.55	-12.50	-12.45	-12.40	-12.36	
DEPTH TO INVERT (m)	35.95	31.22	26.88	22.58	29.97	34.14	37.28	39.73	40.15	38.36	33.85	27.63		
CHAINAGE (m)	24100	24150	24200	24250	24300	24350	24400	24450	24500	24550	24600	24650	24675.55	24700

LONG SECTION
SCALE. 1:1250 H 1:2500 V



NOTES:

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2. GROUND ELEVATION MODEL DEVELOPED FROM AUCKLAND COUNCIL GIS CONTOUR DATA (2013).
3. GEOLOGY CONTACTS INTERPOLATED FROM AVAILABLE BOREHOLES, GNS MAP 1:250 000 (AUCKLAND) AND AUCKLAND COUNCIL GIS CONTOUR (LIDAR) DATA (2017)

JACOBS AECOM McMILLEN JACOBS ASSOCIATES

ISSUE	DATE	AMENDMENT	BY	APPD.
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0	12.12.18	ISSUED TO WATERCARE	LD	VSR

	BY	DATE
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DES. CHECKED	V. ROMERO	01.19
DRAWN	C. VALENCIA	01.19
DWG. CHECKED	V. LEE	01.19
PROJECT LEADER	N. KAY	01.19
INFRAST'R APP'D		

OPERATIONS

INFRASTRUCTURE

Watercare

GREY LYNN TUNNEL
SITE GENERAL
GREY LYNN TUNNEL LONG SECTION – SHEET 3 of 3

CAD FILE 2011878.009
DATE 31.01.2019
ORIGINAL SCALE A3 AS SHOWN
CONTRACT No. 5747
REF. No. CI–GEOTECH
ISSUE 1
DWG. No. 2011878 .009

Plot Date: Feb 12, 2019 - 4:16pm C:\Users\Valencia\Box\5222.0 Central Interceptor Detailed Design\10 CAD\04. drawings\ 2011878.007.dwg

Appendix C

Appendix C. Shaft Mechanical FLAC Modelling Results

Modelling Procedure

The modelling steps are as following:

1. Set up the initial geometry and apply initial stress and boundary conditions. Add groundwater (GW level is assumed to be at ground surface). Solve to equilibrium.
2. Apply a uniform surcharge of 16kPa over an approximately 6m wide annulus adjacent to the shaft wall.
3. Install secant pile and solve to equilibrium.
4. Reset ground displacements to zero to establish the baseline condition.
5. Excavate Lift 1. For a span between the shaft centerline and the back of scant pile, lower the GW level down to the bottom of Lift 1, and solve to equilibrium.
6. Repeat step 5 until the excavation reaches the secant pile toe.
7. Excavate the subsequent lift; lower the GW level inside the shaft down to the bottom of the excavation.
8. Install the shotcrete layer with full strength on the excavation wall for the current lift. Solve to equilibrium.
9. Repeat steps 7 and 8 until shaft excavation is completed.

Modelling Assumptions

Key modelling assumptions are as follows:

1. Axisymmetric configuration was used for this modelling.
2. The response of the soil and rock mass to static loading is modelled to be elasto-plastic. The plastic response for the rock mass and soil is governed by Mohr Coulomb yield criteria.
3. Groundwater (GW) level is assumed to be at ground surface. During the shaft excavation, GW level is lowered inside the shaft area and is assumed to be at the bottom of the excavation at each stage.
4. Shaft is considered to have a diameter of 12m and a depth of 28m.
5. Shaft is assumed to be excavated in 2.5m lifts, except for the first lift, which is assumed to be 3m.
6. The soil and rock mass parameters and in situ stress condition (K_0) are as shown in Table C-1.
7. Secant piles and shotcrete layer are modelled as continuum elements. The support properties are as shown in Table C-2.
8. Secant piles are assumed to be 8m long and extend 1m into the rock.
9. Shotcrete layers are installed immediately after each lift is excavated (no relaxation; with full strength).
10. Rock bolts are ignored in this modelling.
11. A surcharge of 16kPa is assumed to exist adjacent to the shaft wall over a 6m wide annulus.

Material Parameters

Subsurface Conditions

The subsurface ground conditions are interpreted from the data presented in the Geotechnical Factual Report (PWCIN-DEL-REP-GT-J-100452). Table C-1 presents a summary of the parameters used in the analyses.

Table C-1: Summary of Soil and Rock Mass Parameters

Medium	Total Unit Weight (kN/m3)	Deformation Modulus (MPa)	Poisson's Ratio	Friction angle	c′ (kPa)	K0
Puketoka Formation	16	3	0.40	28	7	0.50
Residual ECBF	19	30	0.40	32	6	0.47
MW-UW ECBF	20	400	0.25	34	100	1.2

Structure Material Properties

Table C-2 summarizes the properties that are used for each support element.

Table C-2: Summary of Maximum displacements at ground surface

Support Element	Unit Weight (kN/m3)	Thickness/ diameter (mm)	Elastic Modulus (GPa)	Poisson's Ratio
Secant Pile	24	750	30	0.2
Shotcrete Lining	24	~200	15	0.2