Grey Lynn Tunnel

Watercare Services Limited

Settlement Assessment of Grey Lynn Tunnel and Tawariki Street Shafts

Draft
Revision No. 3

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Revision Log

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1	9 Nov 2018	Draft for internal review
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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.

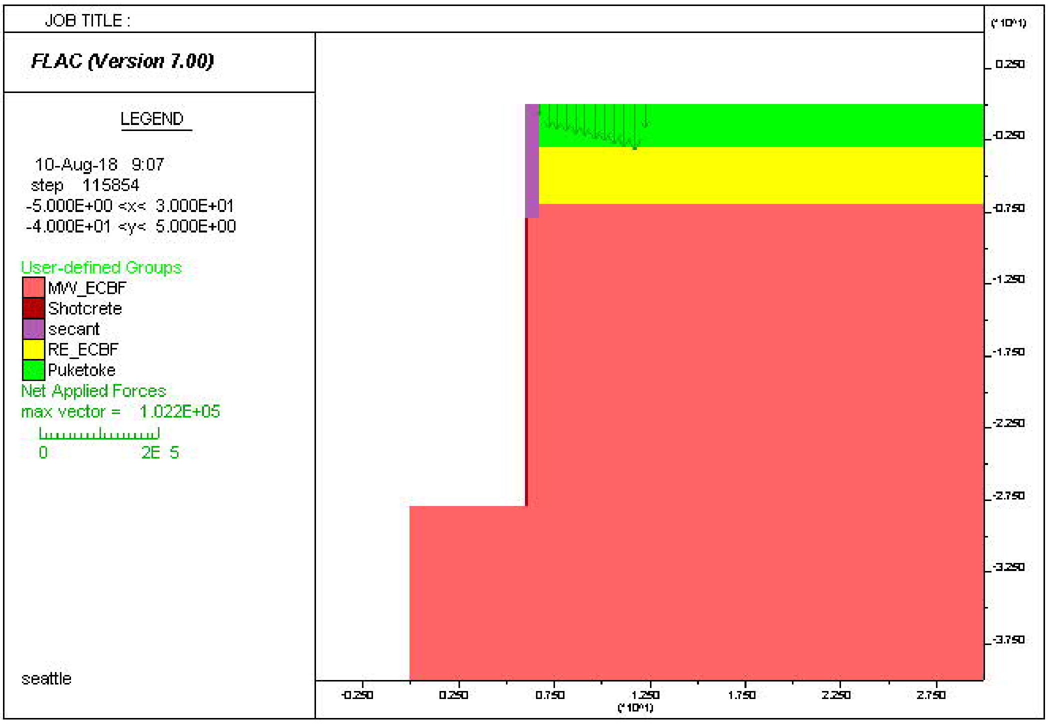

Figure 5-1: Model Cross Section

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)

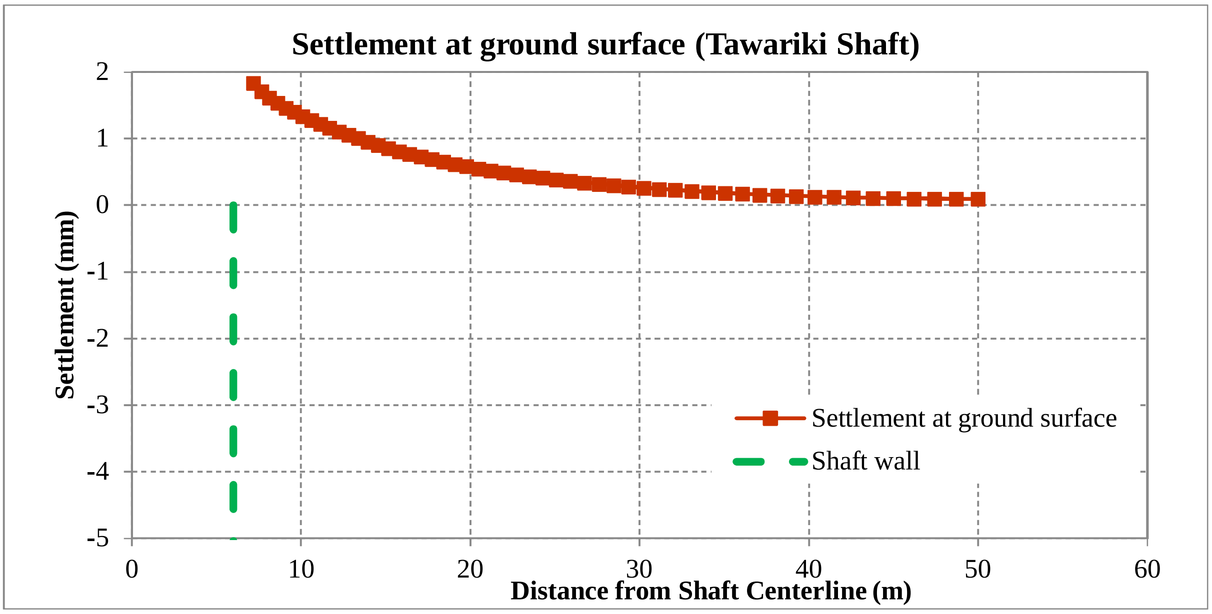

Figure 5-2: Ground Surface Settlement Model Output for Tawariki Shaft

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.

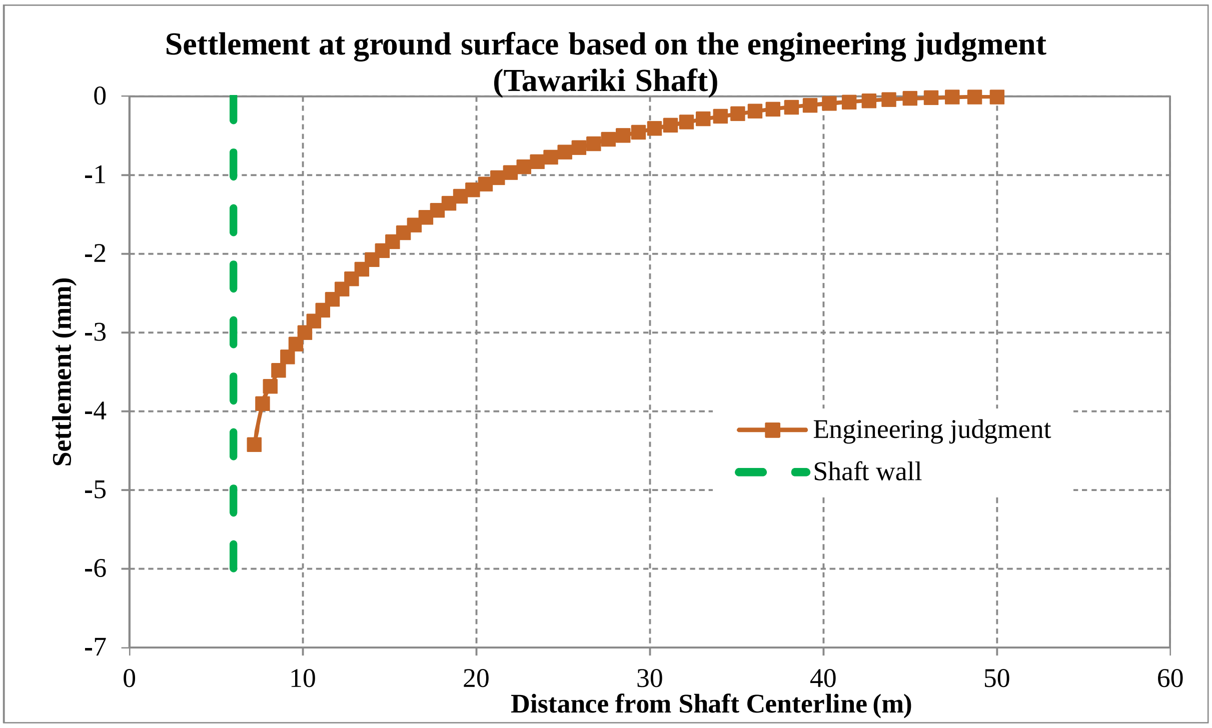

Figure 5-3: Ground Surface Settlement from Shaft Excavation

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

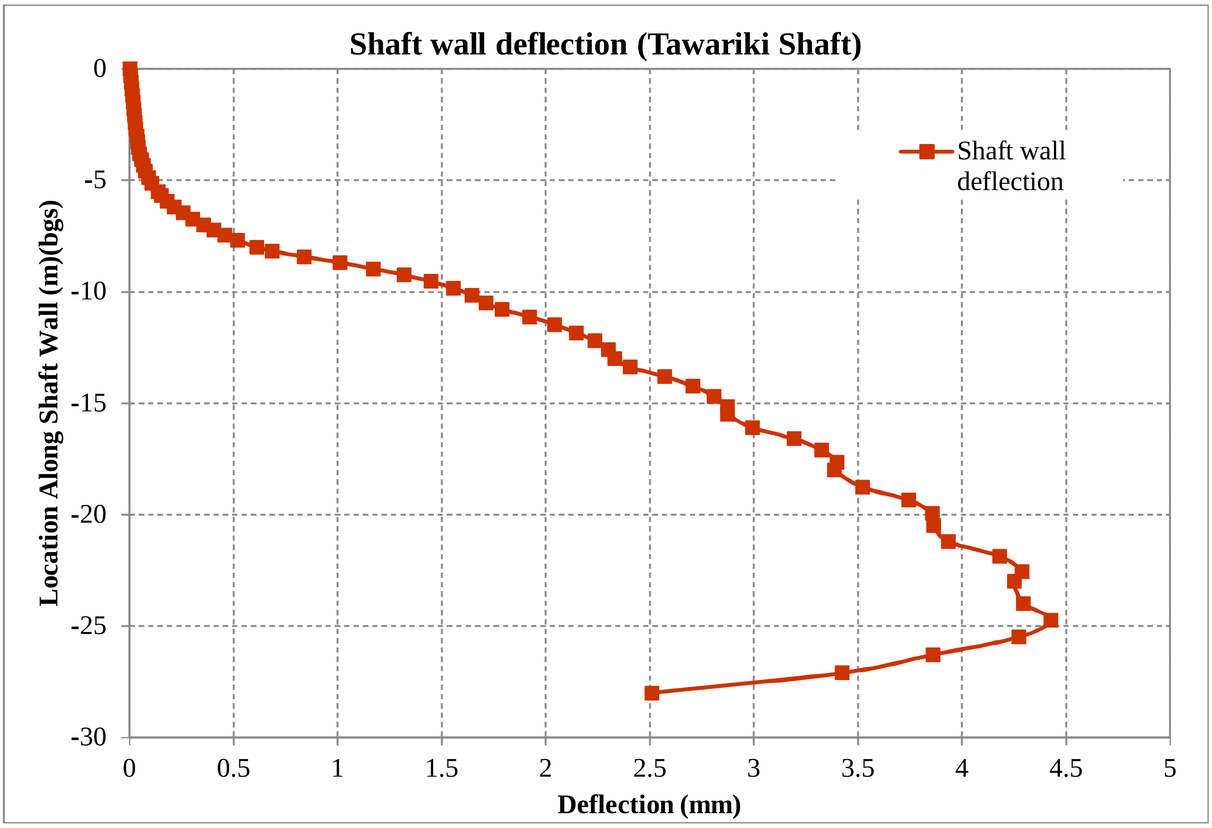

Figure 5-4: Shaft Wall Deflection for Tawariki Shaft (positive deflection is towards the excavation)

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.

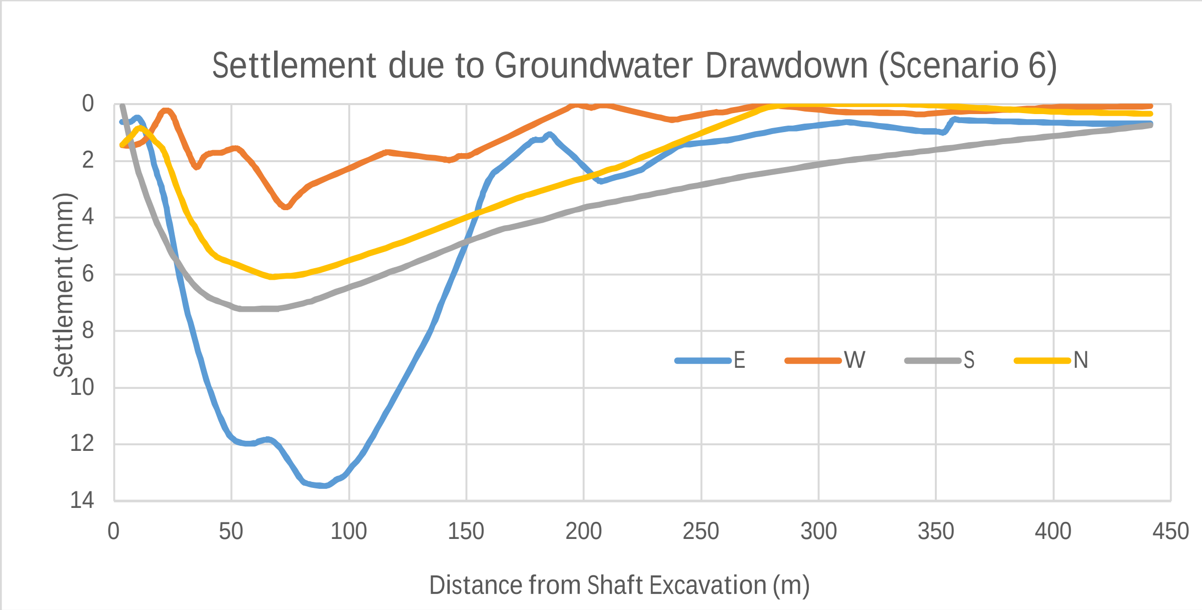

Figure 5-5: Settlement due to Groundwater Drawdown (the settlement profile for the east, west, south and north directions, where 0m is at the extent of shaft excavation and distance increases away from the shaft)

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.

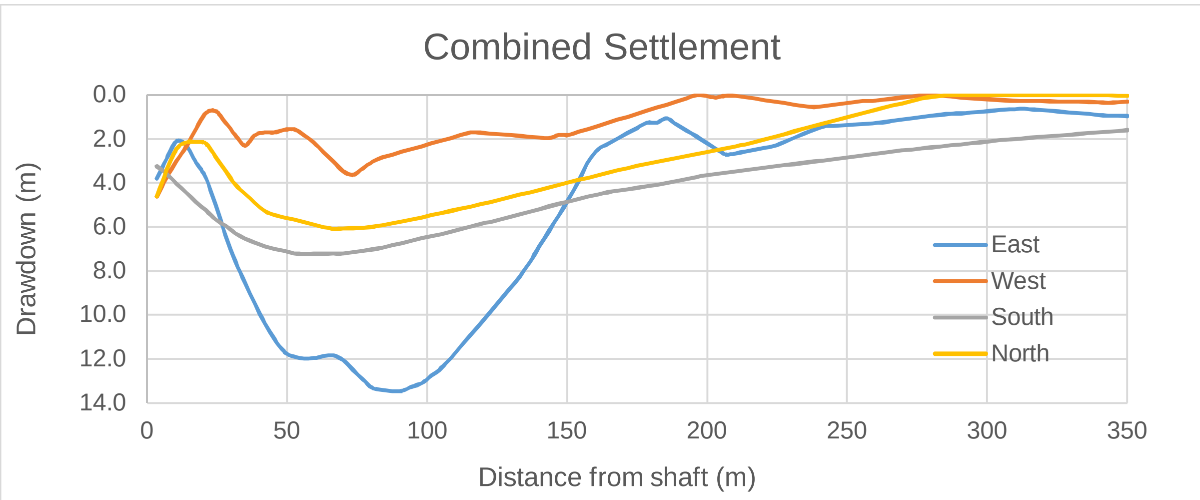

Figure 5-6: Combined Settlement (the settlement profile for the east, west, south and north directions, where 0m is at the extent of shaft excavation and distance increases from the shaft)

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

SETTLEMENT PLAN
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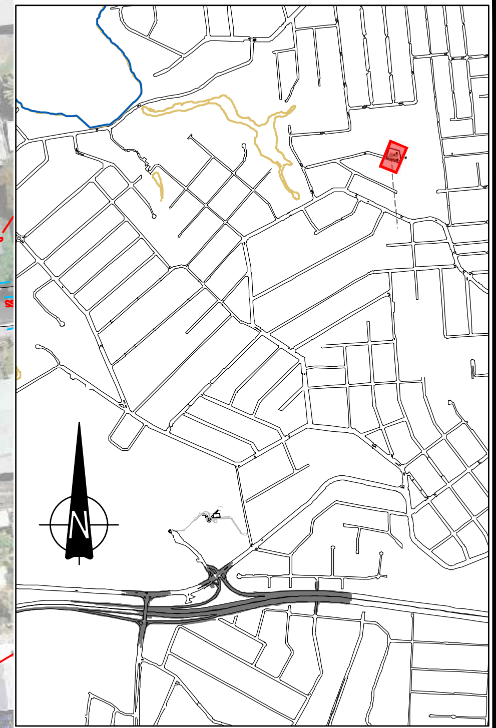

LEGEND

DIFFERENTIAL SETTLEMENT SLOPE

- 1:1000 - 1:500
- 1:500 - 1:200
- > 1:200
- SETTLEMENT CONTOUR LINE (mm)
- EX. STORMWATER
- EX. WASTEWATER
- EX. WATER SUPPLY

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Plot Date: Jan 30, 2019 - 4:33pm

JACOBS AECOM McMILLEN JACOBS ASSOCIATES

GREY LYNN TUNNEL

TAWARIKI STREET SHAFT SETTLEMENT CONTOUR PLOT

Watercare

Appendix B

Geological Profile

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

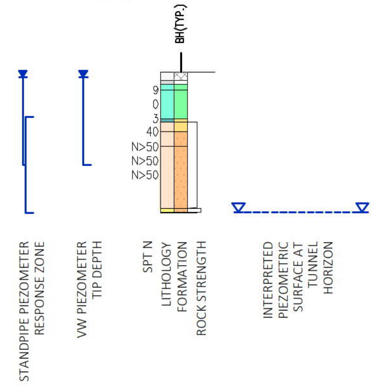
- No core
- Fill and topsoil
- Basalt
- Other volcanics
- Clay and clay dominated mixtures
- Organic clay and organic clay dominated mixtures
- Peat
- Silt and silt dominated mixtures
- Organic silt and organic silt dominated mixtures
- Sand and sand dominated mixtures (clean)
- Sand and sand dominated mixtures (with fines)
- Organic sand and organic sand dominated mixtures
- Sandstone
- Mudstone
- Interbedded mudstone and sandstone
- Conglomerate
- Coal

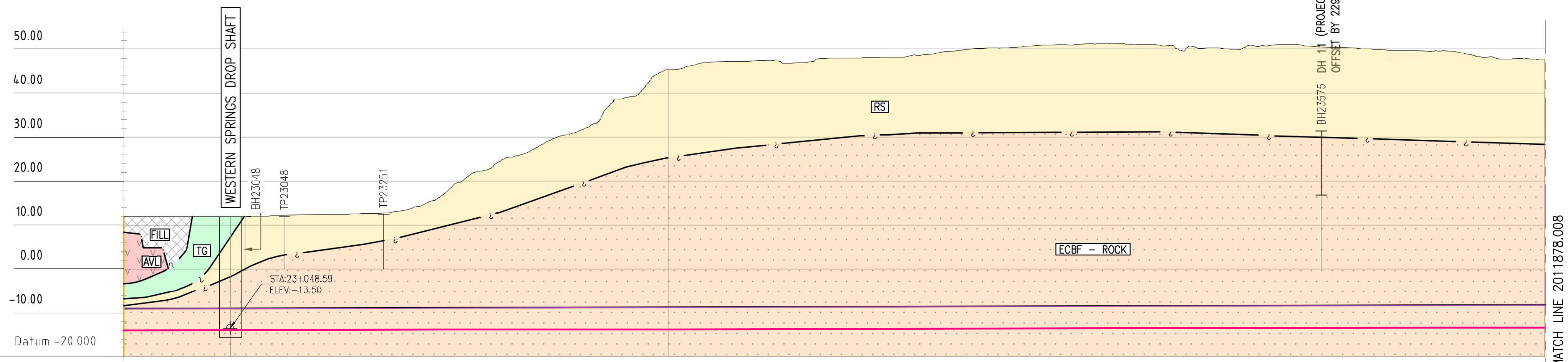
- Made Ground (FILL)
- Recent Alluvium (Q1a)

TAURANGA GROUP
- Undifferentiated Tauranga Group (TG)
- Kaawa Formation (KAAWA)

AUCKLAND VOLCANIC FIELD (PLEISTOCENE)
- Basalt (AVL)
- Tuff / Ash / Scoria (AVT)

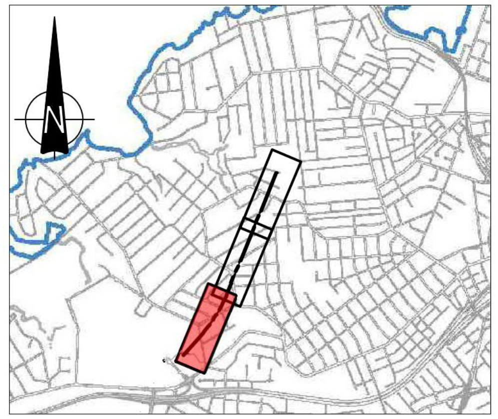
WAITEMATA GROUP (MIOCENE)
- Residually to Highly Weathered - Cohesive Soils (RS)
- Residually to Highly Weathered - Granular Soils (RS)
- Parnell Volcaniclastic Conglomerate (PVC)
- Moderately Weathered to Unweathered ECBF (ECBF-ROCK)





PIPE DIAMETER AND GRADE	4500 MM ID PRECAST SEGMENTAL LINING (1:1000)												
EXISTING GROUND LEVELS (mRL)	12.00	12.52	18.92	30.33	45.40	47.06	48.20	50.32	51.34	50.34	50.52	49.67	47.77
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
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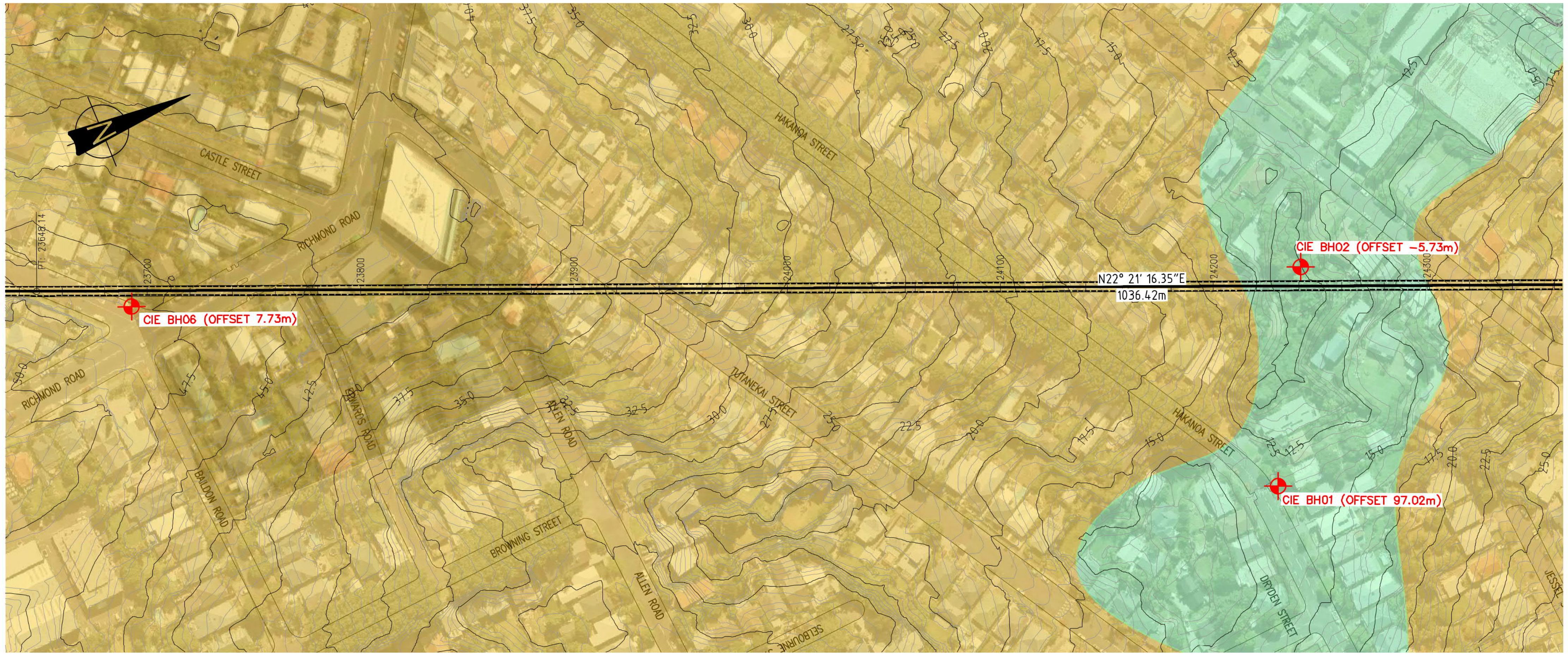
Watercare

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
DWG. No. 2011878 .007 | 1



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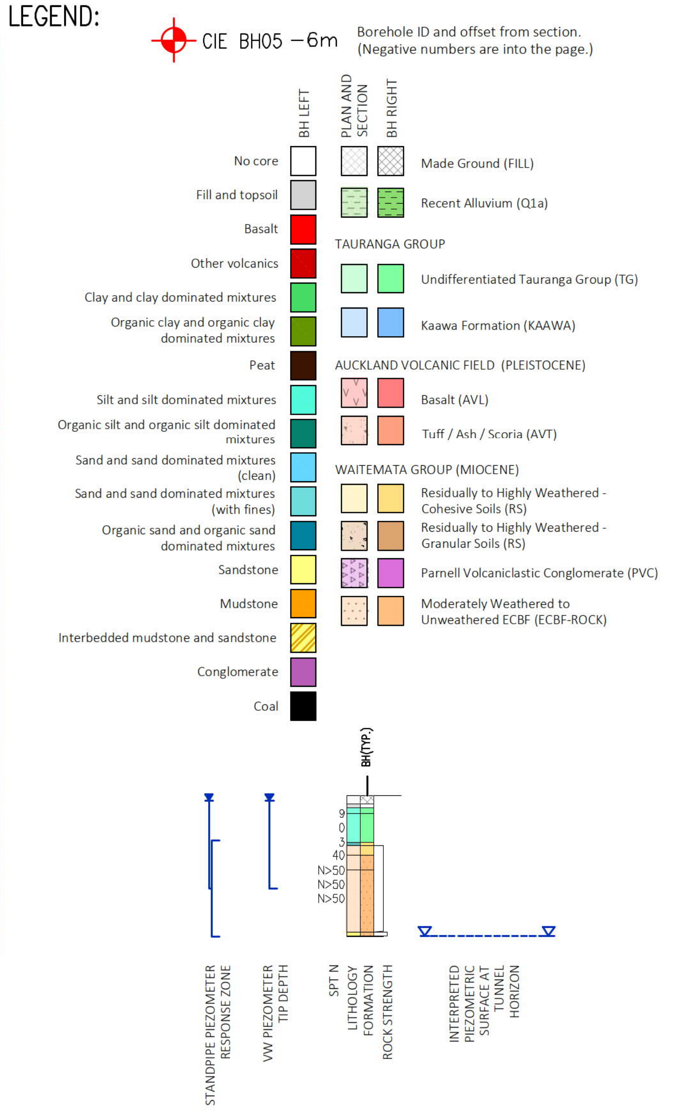

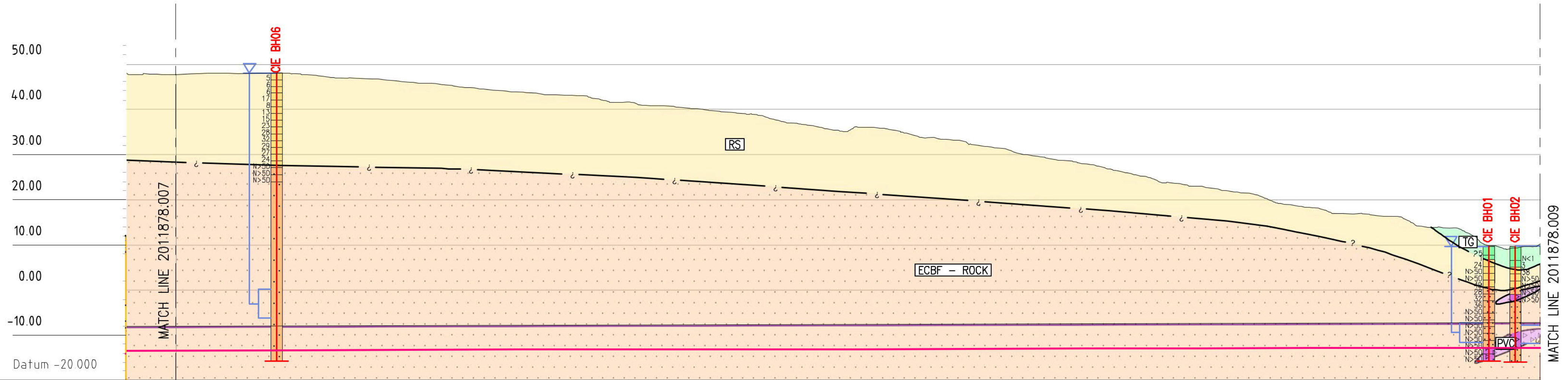

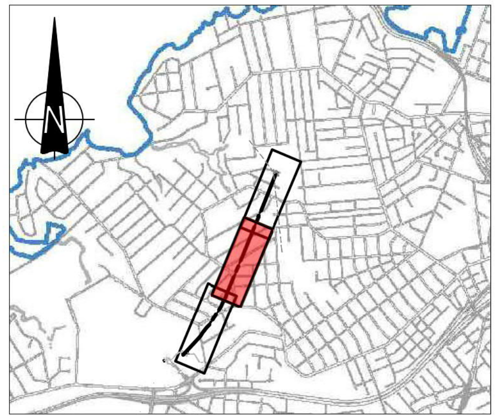

PIPE DIAMETER AND GRADE	4500 MM ID PRECAST SEGMENTAL LINING (1:1000)												
EXISTING GROUND LEVELS (mRL)	47.77	47.88	46.10	43.81	41.55	39.00	36.08	32.25	27.95	23.00	18.32	14.03	9.79
INVERT LEVEL (mRL)	-13.40	-13.35	-13.30	-13.25	-13.20	-13.15	-13.10	-13.05	-13.00	-12.95	-12.90	-12.85	-12.80
DEPTH 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:

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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DWG. CHECKED	V. LEE	01.19
PROJECT LEADER	N. KAY	01.19
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GREY LYNN TUNNEL
SITE GENERAL
GREY LYNN TUNNEL LONG SECTION – SHEET 2 of 3

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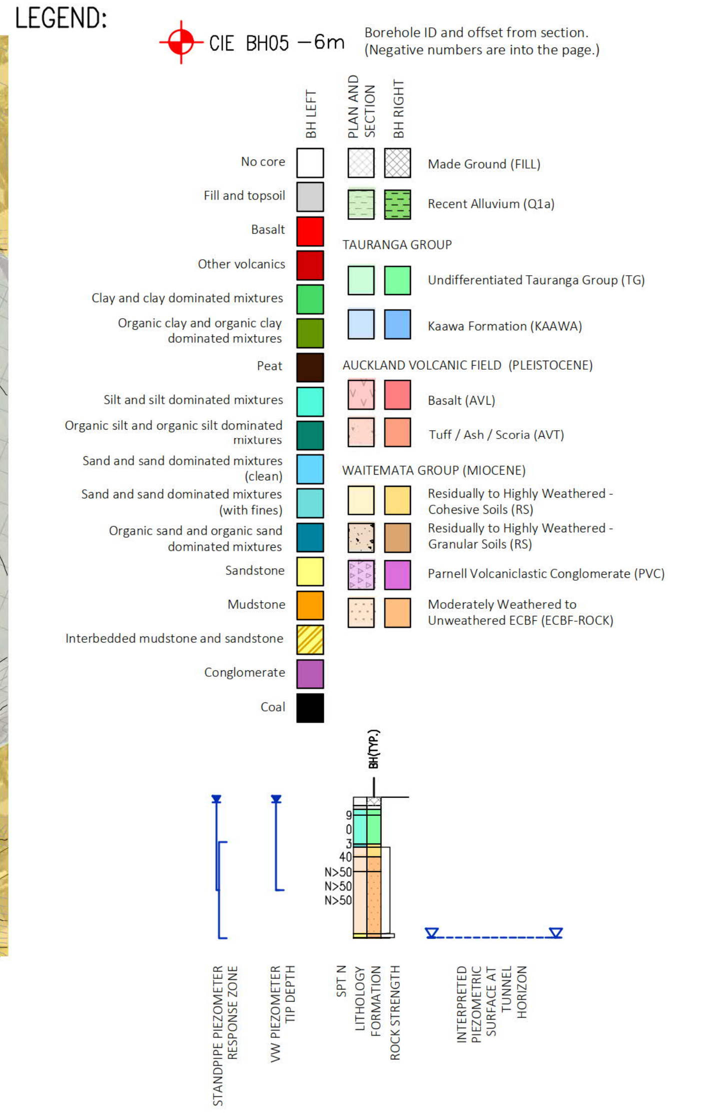
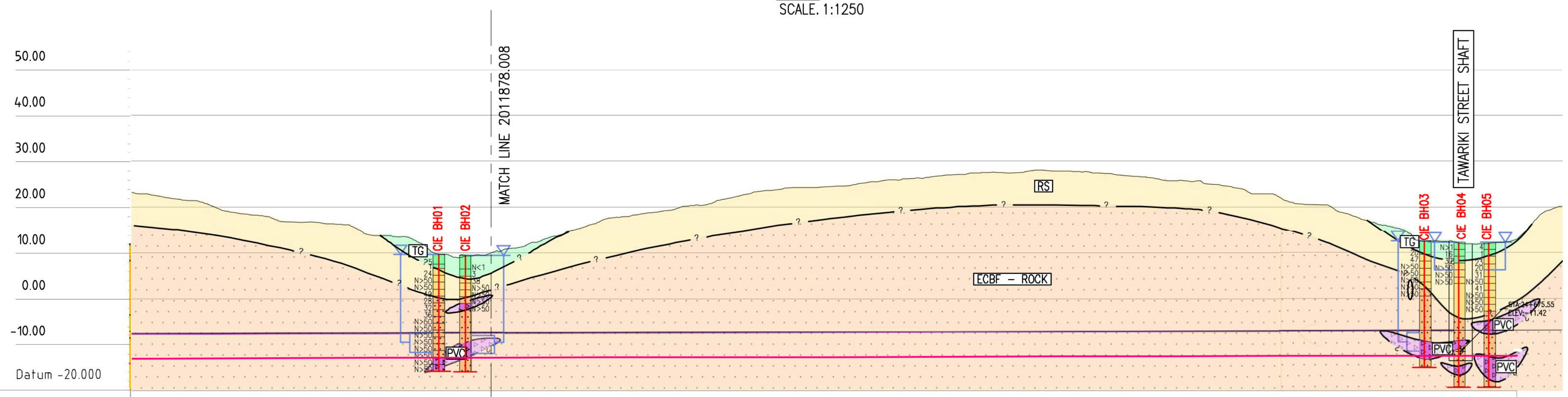
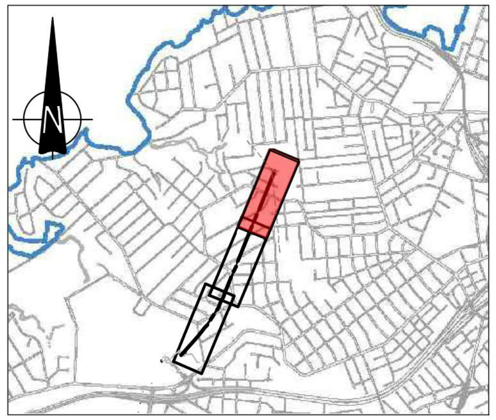




PIPE DIAMETER AND GRADE	4500 MM ID PRECAST SEGMENTAL LINING (1:1000)													
EXISTING GROUND LEVELS (mRL)	23.00	18.32	14.03	9.79	17.22	21.44	24.63	27.13	28.01	25.86	21.40	15.23		
INVERT LEVEL (mRL)	-12.95	-12.90	-12.85	-12.80	-12.75	-12.70	-12.65	-12.60	-12.55	-12.50	-12.45	-12.40	-12.36	
DEPTH 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:

1. ALL SETTING OUT COORDINATES ARE TO NEW ZEALAND TRANSVERSE MERCATOR PROJECTION AND ALL LEVELS ARE TO AUCKLAND 1946 DATUM.
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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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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