REPORT

Herne Bay Connector Project – Groundwater and Settlement Assessment Report

Prepared for
Watercare Services Limited

Prepared by
Tonkin & Taylor Ltd

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1 Introduction

Watercare Services Limited (Watercare) has commissioned Tonkin & Taylor Ltd (T+T) to undertake a Groundwater and Settlement Assessment to support a resource consent application for the Herne Bay Connector Project (referred to as the Herne Bay Tunnel or Project in this assessment). The proposed alignment runs from Marine Parade in the south-west to the Point Erin drop shaft in the north-east where it connects into the proposed Central Interceptor (CI) tunnel.

This report provides a preliminary assessment of groundwater and settlement effects to:

1. Identify areas of the alignment where:
   - Building, structures, and/or services could be at risk of damage caused by ground settlement resulting from construction of the proposed tunnel.
   - The risk of damage to building, structures, and/or services is very low.
2. Understanding implications of the construction methodology, the effects on the receiving environment, and if there are any information gaps.
3. Undertake an assessment of potential effects based on the proposed construction methodology using published datasets and site-specific available information at the time of writing.

Our assessment is based on limited geotechnical information and concept level scheme plans provided to us by WSP[1]. Analysis has been undertaken adopting a conservative approach to estimate an upper bound potential effects envelope for the proposed works. The assumed ground conditions will be validated with further ground investigations and as designs are progressed. This further work is well underway and is expected to be complete by August / September 2023. It will be provided to Council as soon as it is completed.

[1] WSP (16 February 2023). *Herne Bay Trunk Sewer Upgrade, Marine Parade to Pt Erin.* DWG No. W-SL007.001 – W-SL007.008. Ref No. W-SL007.01.

2 Project summary

2.1 Overview

Watercare is working jointly with Auckland Council to deliver the WIWQIP, which will reduce wastewater overflows and improve water quality at local beaches. As part of the WIWQIP, this Project will upgrade the existing Branch 5 Sewer in the Herne Bay area by constructing a new wastewater trunk sewer. Upon completion, the new trunk sewer line will connect to the proposed CI wastewater conveyance and storage tunnel that is intended to be extended from Tawariki Street in Grey Lynn to a new drop shaft in Point Erin Park.

The Project's scope is for:

- Installation of approximately 1.5 km of 2.1 m internal diameter trunk sewer line, constructed via a tunnel-boring machine (TBM).
- Installation of approximately 150 m of 600 mm diameter trunk sewer within Marine Parade, constructed via open-cut trenching.
- Construction of 8 No. primary tunnel shafts, ranging in diameter from 3.5 m to 13.5 m, along with 4 No. 3.5 m diameter intercepting shafts.
- Installation of 4 interception pipes and 11 connections to existing engineered overflow points (EOPs).
- Relocation and reinstatement of utilities as required.

The primary purpose of the Project is to reduce engineered overflow spill frequencies resulting from the aging combined sewer network in the area and to comply with Watercare's Network Discharge Consent (NDC) conditions. This is expected to lead to improvements in bathing water quality conditions at the beaches, reduction of odour from stormwater catchpits and improved overall amenity.

This report does not consider effects from the Construction Support Areas (CSAs) as the proposed works in these areas are not anticipated to require significant earthworks (i.e. excavation below groundwater).

2.2 Project alignment

Figure 2.1 and Figure 2.2 below show the proposed alignment and key features of the Project.

Figure 2.1: Eastern portion of the Project

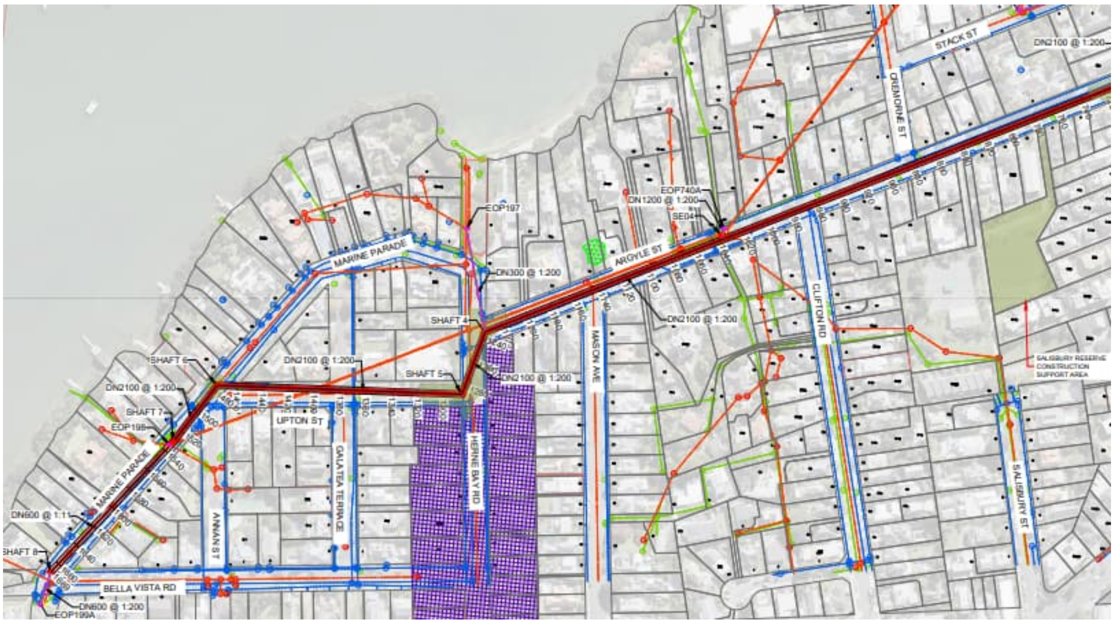

Figure 2.2: Western portion of the Project

The eastern terminus of the Project is at Point Erin Park, where it will connect to the proposed CI extension via a drop chamber, which is proposed as part of the CI extension project.

Works within Point Erin Park that are proposed as part of the CI extension resource consent application and therefore not forming part of the 'application area' for the Project include:

- The construction of the proposed CI tunnel terminal shaft, control chamber, and stub connection to facilitate a potential future connection to the proposed Herne Bay Trunk Sewer tunnel.
- The construction of a proposed plant room to house equipment to control the gates.
- Proposed connections between the terminal shaft, control chamber, vent, and plant room.
- Proposed tree works (pruning, works in the root zone, removal, relocation), and park reinstatement and landscaping following completion of construction works.

2.3 Assessment overview

Concept drawings of the scheme were provided by WSP on 16 February 2023[1] and are presented in Appendix A. The drawings include two design options being considered for the final tunnel depth along the main alignment, with the final tunnel elevation to be determined as part of detailed design. It understood Option 1 (deeper tunnel) is considered likely to be adopted due to the requirements for integration of the branch sewer to the proposed CI at Point Erin. However, for the purposes of this preliminary assessment both potential tunnel depths have been considered.

The Project has been developed to a concept design stage. As it moves through the detailed design process and as construction methodology is confirmed, it is anticipated that some details may need to change, but it is intended that geotechnical and groundwater effects should remain within the envelope of effects outlined in this assessment. All figures and dimensions provided are approximate and will be confirmed and refined during the detailed design stage.

A summary of the shaft depths shown in Table 2.1 for both of the current design options.

Table 2.1: Summary of shaft details based on WSP concept designs

Shaft ID	Shaft Location	External Shaft Diameter (m)	Approximate Shaft Depth (m bgl)	Pipe Invert Level (m RL)	Inner/Outer Pipe Diameter[1] (m)	Thrust/ Receiving of TBM
Shaft 1	Point Erin	13.5	11.46 to 16.89	1.43 to -4	2.1/2.5	Thrust
SE01	59 Hamilton Rd	3.5	17.96 to 23.39	2.66 to -2.77	2.1/2.5	Intercept
SE02	80 Sarsfield Rd	3.5	14.01 to 19.44	3.64 to -1.79	2.1/2.5	Intercept
SE03	91 Sarsfield St (Sarsfield WWPS)	3.5	9.03 to 14.46	4.10 to -1.35	2.1/2.5	Intercept
Shaft 2	58 Wallace St	9.0	13.97 to 19.41	4.56 to -0.87	2.1/2.5	Receiving
Shaft 3	50 Wallace St	13.5	17.25 to 22.68	5.05 to -0.38	2.1/2.5	Thrust
SE04	45 Argyle St	3.5	6.93 to 12.36	6.82 to 1.39	2.1/2.5	Intercept
Shaft 4	72 Argyle St	9.0	17.23 to 22.66	7.81 to 2.38	2.1/2.5	Receiving
Shaft 5	34 Herne Bay Rd	13.5	19.81 to 25.24	8.17 to 2.74	2.1/2.5	Thrust
Shaft 6	33 Marine Parade	9.0	8.17 to 13.60	9.21 to 3.79	2.1/2.5	Receiving
Shaft 7	22 Marine Parade	9.0	6.28 to 11.70	9.59/10.49 to 4.17/5.67	2.1/2.5 (0.6/0.72)[2]	TBC
Shaft 8	Marine Parade	3.5	2.42	21.96	0.6/0.72	TBC

Notes:

(1) Outer pipe diameter based on standard supplier pipe sizes.
(2) Bracketed item denotes the inflow pipe into the shaft where pipes into and out of the shaft are different dimensions.

3 Construction methodology

The following sections outline the current construction methodology proposed by WSP[2] for the various elements of the project upon which the assessment of effects has been undertaken.

Concept sequence programmes for the construction of scheme indicate construction is proposed to commence May 2024, with completion forecast for December 2026.

3.1 Tunnelling

An Earth Pressure Balance (EPB) TBM is to be used for the excavation of the main tunnel alignment between Shaft 1 (at Point Erin Park) and Shaft 7 (on Marine Parade). The TBM will be launched from the thrust shafts and retrieved from the reception shafts, with the pipe sections to be placed in behind the TBM as it progresses. The tunnelling methodology between Shafts 7 and Shafts 8 is proposed to comprise open trenching temporarily supported using a slide rail shoring system.

Tunnel depths below do not account for any over excavation due to the TBM construction methodology (but *volume loss* is allowed for in our assessment of effects, as discussed in section 6.2 of this report).

Key inputs for assessment include:

- Concept drawings[1] indicate the invert levels of the tunnel are approximately -4 m RL (16.89 m bgl) at the north-eastern end (Shaft 1), and 21.96 m RL (2.42 m bgl) at the south-western end (Shaft 8), with a maximum shaft depth of 25.24 m bgl at Shaft 5.
- Consistent with construction of the CI tunnel to date, the TBM is expected to predominantly operate in closed mode (with hydrostatic pressures balanced) for the bored portion of the alignment. In closed mode, no groundwater inflow into the tunnel or depressurisation of the hydrostratigraphic unit/ aquifer immediately outside the tunnel is expected.
- It may be necessary to operate the TBM in open mode over short sections of the tunnel alignment, where hydrostatic pressures will not be (or only partially) pressure balanced. For these short sections of the alignment, some groundwater inflow into the tunnel is expected resulting in some depressurisation of the hydrostratigraphic unit / aquifer immediately outside the tunnel. It is anticipated tunnelling in open mode would only be used where drilling within very weak ECBF rock.
- Operation of the TBM includes installation of concrete lining concurrent with progression with the advancing tunnelling. For this screening-level assessment, it has been assumed that the longitudinal distance from the cutter face to the permanent concrete lining is approximately 12.5 m, and tunnel progression will advance at a constant rate of approximately 10 m per day (or 20 m every two days).
  The implication for the assessment is that when the TBM operated in open mode, 12.5 m section(s) of the of the tunnel may be 'open' and therefore subject to groundwater inflow for up to two days. For the small sections of the alignment where open mode may be required, the rate of groundwater inflow into the tunnel will be a function of:
  - The hydraulic properties of the geological unit immediately outside the tunnel.
  - The dimensions of the tunnel.
  - The degree of any partial hydrostatic pressure balancing achieved by the TBM.

[2] WSP (3 March 2023). Memorandum to Technical Specialists. *Project Briefing and Request for Technical Assessments – Herne Bay Trunk Sewer, Watercare Services Limited.*

3.2 Thrust / receiving shafts

The thrust and receiving shafts are located where the direction of the alignment changes to follow the road alignment and allows for the deployment and retrieval of the TBM. The shafts generally have an external diameter of 9 m and 13.5 m for the receiving and launch shafts, respectively, with a finished shaft invert between 2.42 m deep (Shaft 8) and 25.24 m deep (Shaft 5).

Secant piles will form the shaft walls and will be constructed with a bored piling rig and support crane. The secant piles are proposed to extend to full depth of the shafts. The piled shaft will then be excavated, and a capping ring beam installed. The shaft is expected to be excavated by conventional mechanical equipment (e.g. 23 t excavator with 2 t excavator within the shaft) through overburden soils and East Coast Bay Formation (ECBF) soil and rock.

On completion of the shaft and tunnel excavations, a permanent manhole will be installed within the shaft, the space between the shaft walls and manhole will be backfilled with hardfill, and holes will be cut into the shaft walls to reduce groundwater impedance by the secant piles and allow for groundwater flow around the manhole in the long term.

Key input assumptions for this assessment include:

- Shaft design assumes the shaft will be constructed using secant piles in a circular arrangement assuming the piles extend to full depth of the shaft excavation.
- It is anticipated a maximum 0.5 m of over-dig beneath the base of the shaft to allow for reception of the TBM and construction of the base slab (200 mm thick). Shaft depths discussed herein refer to the shaft depths including overdig unless otherwise specified.
- Given that the proposed shaft excavations will be up to approximately 25.74 m deep, methods to control groundwater inflow will be required to maintain groundwater levels at or below the base of each shaft during the construction period, including:
  - Exclusion (secant piling extending to or below the base of the shaft will act as a hydraulic barrier impeding groundwater inflow).
  - Temporary dewatering/pumping to maintain groundwater levels below the base of each shaft during the construction period.

3.3 Interceptor shafts

Four intercept shafts (SE01 – SE04) will be constructed following the construction of the main tunnel to pick up wastewater drainage inflows from Engineered Overflow Points (EOPs). This will be accomplished by drilling vertical shafts beside the main tunnel.

From discussions with Brian Perry Civil (BPC)[3], it is understood that a trench box or casing will be installed for the stability of surficial soils prior to drilling of the shaft. This is to extend into competent ground that would maintain stability during excavation. If required, the casing can be deepened by welding additional segments as drilling progresses, to maintain stability of the surrounding soils.

The shaft would then be drilled to depth with a 75 t piling rig with a 3.5 m diameter auger following which a temporary steel casing would be installed to full depth. A window will then be cut into the base of the casing and a lateral passage excavated/mined to connect to the main tunnel. Once the lateral passage is lined, the permanent concrete manhole structure will be installed into the shaft and the temporary casing removed. Grout will then be poured or tremmied into the annulus between the permanent manhole and ground.

[3] Brian Perry Civil (10 February 2023 @ 3:50pm). Email from Tony Sage to Ric Wilkinson. *RE: Herne Bay geo discussion.*

Key inputs for assessment include:

- Concept drawings[1] indicate the interceptor shaft invert levels are a minimum of -3.27 m RL (23.89 m bgl) at SE01 and a maximum of 0.89 m RL (12.86 m bgl) at SE04.
- Shaft design assumes full length temporary casing will be installed following boring of the shaft (i.e. negligible soil relaxation / mechanical settlement) with minimal gap between the ground and the casing.
- Given that the proposed shaft excavations will be up to approximately 24 m deep, methods to control groundwater inflow will be required to maintain groundwater levels at or below the base of each shaft during the construction period. It is anticipated that groundwater drawdown would commence once the full-length temporary casing has been installed within the drilled shaft. A timeframe of 'continuous' dewatering of seven days has been allowed for the assessment of effects. Inflow rates are anticipated to be low, and pumping/dewatering would only occur when construction is taking place in the base of the shaft. In practice that would mean that groundwater could refill within the shaft overnight, and it is expected that pumping would mainly be required each morning to provide a reasonably dry work environment for constructability and safety purposes.

If required during construction, contingency options to minimise water inflows during construction may be considered such as concrete seal may be placed at the base of the bored shaft.

It is inferred that the construction methodology for the interception pipes will generally follow that of the intercept shafts. The pipe connection between the EOPs and the main tunnel shafts are expected to be either open trenched (for shallow connections) or horizontally drilled as shown in Table 3.1 below.

3.4 Interception pipes

It is understood that the construction methodology for the interception pipes, which will connect to the existing engineered overflow points, will generally be undertaken using temporary excavations shored with trench boxes (for shallower excavations where possible), or adopting a similar methodology to that of the intercept shafts at deeper locations. The pipe connections between the EOPs and the main tunnel shafts / intercept shafts are expected to be either trenched or directionally drilled as outlined in the WSP construction methodology[3]. The proposed methodology and maximum pipe invert level for the respective EOPs are summarised in Table 3.1.

Directional drilling will comprise excavation of drilling and receiving pits which will be shored with trench shields. A directional drill rig will then be mobilised to site and a bore drilled using a recirculating bentonite slurry. A polyethylene pipe (PE) will be pulled through and the grouted into place.

Open trenching will be undertaken for shallower connections using "traditional construction" with 23 t excavators and slide rail shoring systems to support the sides of the excavation. Where excavations intercept the groundwater table, additional consent conditions may be imposed in accordance with Chapter E7 of the Auckland Unitary Plan – Operative in Part (AUP). This would depend on dimensions, anticipated drawdown, proximity to property boundaries, and length of time the trench remains open.

Key inputs for assessment include:

- Concept drawings[1] indicate the invert levels of the EOPs are between 1.1 m bgl (9.39 m RL) at EOP202 and 14.56 m bgl (10.48 m RL) at EOP197.

- The slide rail shoring system has been assumed to be propped at multiple levels. For the purposes of modelling mechanical ground settlement effects, it is assumed to have sufficient stiffness to be considered 'high stiffness' in accordance with CIRIA C760[4].

Table 3.1: EOP pipe details

Asset Name	Location	Methodology	Pipe internal/external diameter (m)	Distance (m)	Depth to pipe invert (m bgl)	Pipe end depth (m bgl)
SE01 – 59 Hamilton Road						
EOP202	69 Hamilton Road	Horizontal Drill	0.30/0.37	85	1.1	11.7
EOP195	59 Hamilton Road	Open trench Excavation	0.45/0.54	6.4	2.0	2.3
SE02 – 80 Sarsfield Street						
EOP200	28 Sentinel Road	Open trench Excavation	0.45/0.54	184	4.0	4.7
SE03 – 91 Sarsfield Street						
EOP201	91 Sarsfield Street	Open trench Excavation	1.20/1.40	4.5	4.1	4.3
Shaft 2 – 58 Wallace St						
EOP1019 WWMH01	12 Stack Street	Horizontal Drill	0.45/0.54	85	7.3	8.7
EOP1019 WWMH02	1 Wairangi Street	Horizontal Drill	0.45/0.54	80	5	7.3
EOP1019	11 Cremorne Street	Open trench Excavation	0.45/0.54	55	2	5
SE04 – 45 Argyle Street						
EOP740A	45 Argyle Street	Open trench Excavation	1.20/1.40	5	4.1	4.2
Shaft 4 – 72 Argyle St						
EOP197	1 Marine Parade	Horizontal Drill	0.30/0.37	81	3.6	14.6
Shaft 7 – 22 Marine Parade						
EOP198	22 Marine Parade	Open trench Excavation	0.60/0.72	3.5	4.7	4.9
Shaft 8 – Marine Parade						
EOP199	Bella Vista Road x Marine Parade	Open trench Excavation	0.60/0.72	26.5	1.8	2.3

Note 1: Outer pipe diameter based on standard supplier pipe sizes.

[4] CIRIA (2019). CIRIA C760 – *Guidance on embedded retaining wall design.*

4 Hydrogeological conceptual model

4.1 Available data

A site investigation and drilling programme is running concurrently with this screening-level assessment. Drilling occurred during April to late June 2023 and the further reporting based on these site-specific investigations is currently scheduled to be completed in August – September 2023.

The conceptual model presented below is based on information available to the T+T project team at the time of writing this preliminary assessment report, including:

- Site investigation and testing data obtained from other Watercare projects in the same geologic formations with similar lithology descriptions.
- Bore log information sourced from New Zealand Geotechnical Database (NZGD).
- Published data including geological mapping by GNS.

It is noted that this conceptual model is conservative based on the historical information available, such that the results are anticipated to represent an upper bound of potential effects that may result from project works. The August/September reporting will be based on the site-specific investigations and provide a more accurate assessment of geotechnical and groundwater effects.

4.2 Geology

4.2.1 Regional geology

The underlying geological conditions are characterised by three major stratigraphic groups:

- Late Pliocene to Holocene alluvial and estuarine sediments of the Takaanini Formation, part of the Tauranga Group (Takaanini Formation was previously termed Puketoka Formation).
- Miocene Waitemata Group and Waitakere Group marine sedimentary and volcanic rocks.

Published geological maps[5] indicate that the tunnel and shafts will mainly be constructed within the East Coast Bays Formation (ECBF), part of the Waitemata Group. GNS describes the ECBF unit as: *alternating sandstone and mudstone with variable volcanic content and interbedded volcaniclastic grits*.

The upper surface of the ECBF has a variable weathering profile. This material is typically a firm to very stiff silt or clay with a variable sand content. Natural Pleistocene and Holocene deposits are expected in some of the low-lying parts of the alignment. Made ground (fill) may also be encountered at or near the ground surface in this urban environment.

4.2.2 Local geology

As outlined below, T+T developed a geological model based on the current available data, using 'Leapfrog' software.

4.2.3 Data sources

The alignment for the proposed Herne Bay Tunnel was provided to T+T (via WSP on 20 January 2023[6]). To compile relevant site investigation data from the T+T Geotechnical Database (TTGD) and

[5] Kermode, L.O. (1992). *Geological of the Auckland Area.* Institute of Geological and Nuclear Sciences.
[6] WSP (20 January 2023 @ 4:01pm). Email from Gabby Ip to Benjamin Westgate. *RE: Herne Bay Geological model and GI – wrap up notes (TT 1090120).*

the NZGD a ~300 m zone either side of the proposed pipe alignment has been modelled, and selected investigations were used for the development of the model.

Some of the available non-project specific geotechnical investigation data was reviewed and/or omitted during creation of the model due to overlaps with other data, or where the logged information did not appear to match the geological units identified in the area. The following information was extracted from each available geotechnical investigation location:

- Investigation location (Easting [NZTM2000], Northing [NZTM2000], Elevation [NZVD2016], and Depth [m]).
- Geological formation.
- Groundwater level (if recorded).
- In-situ testing:
  - Undrained shear strength (Su) in kPa.
  - Standard Penetrometer Tests (SPT by blows (N count).

No project-specific boreholes had been undertaken when the model was created and, on this basis, a number of conservative assumptions were adopted, as outlined in the subsequent sections.

The locations of boreholes used to develop the LeapFrog model are shown on Figure 4.1.

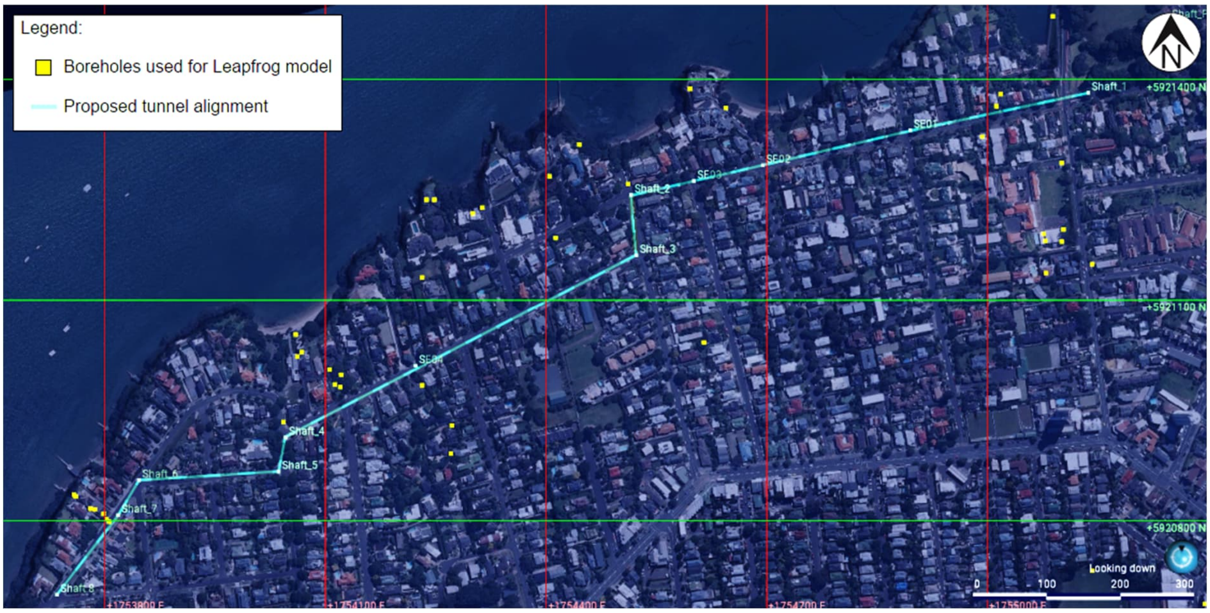

Figure 4.1: Locations of boreholes used to develop the LeapFrog model

The topographical surface used in the development of the model was created from public data available at the website https://opentopography.org. The point cloud used was LiDAR captured for Auckland Council by Aerial Surveys between August 2016 and August 2018. The point cloud file was downloaded for the area and imported into Global Mapper software for processing. An elevation grid was developed from this data on a 1 x 1 m grid. The elevation grid was then contoured in Global Mapper to create 1 m contours for the project area, representing the approximate ground surface levels measured between 2016 and 2018.

4.2.4 Modeller input

Descriptions of the geological units identified were taken directly from the bore logs available and LeapFrog software created the model based on the logged information 'as-written' in most cases.

Where there was a clear outlier in the logged information, it was typically reviewed so that geological consistency could be maintained.

To develop the geological model, it was necessary to include user generated points and lines to guide the geological surfaces. Where possible, the points and lines were used away from investigation data to keep the model as true to the raw data as possible. Where no investigation data was available, the modeller assumed the presence and extent of the unit based on our understanding of the site and previous experience of similar topography – specifically for the Fill material and Takaanini Group Alluvium material modelled to the south and west of Point Erin Park.

The location of all investigations has been taken as recorded on each log for Easting and Northing. Where no elevation was indicated on the log for that investigation, the top of the investigation was assumed to correspond with the modelled topographical surface elevation at that point.

4.2.5 Model assumptions

The following assumptions were made for the LeapFrog modelling:

- All historic investigation data for the area was available on NZGD and TTGD.
- The elevation of historic boreholes was assumed at the topographical surface used in the ground model unless elevation was specified on the logs.
- Ground conditions modelled away from investigation locations are indicative only, with actual ground conditions potentially differing to those indicated in the model.

4.2.6 Model results

Five geological units have been defined for the project site. A summary of the units is provided in Table 4.1. Supporting figures are shown in Appendix B, including line of section (plan view) and a series of geological long cross sections between shaft locations.

Table 4.1: Geological formations modelled

Code	Formation	Description
FILL	Fill	Clay, silt, gravel, sand, cobbles, boulders, and refuse.
TAKAANINI	Takaanini Formation (Alluvium)	Clay, silt, and sand mixtures, occasionally with minor organics
RES ECBF	East Coast Bays Formation	Residual soil (SPT N <20)
WECBF	East Coast Bays Formation	Weathered soil/rock (SPT N 20 – 50)
ECBF ROCK	East Coast Bays Formation	Very weak sandstone/siltstone rock

4.3 Hydrostratigraphic units and properties

Two hydrostratigraphic units (HSUs) have been defined for the proposed tunnel alignment, as follows:

- HSU1, which includes Takaanini Formation (Alluvium) and residually weathered and highly weathered ECBF (RES ECBF and WECBF).
- HSU2, which includes moderately weathered and fresh ECBF rock (ECBF ROCK).

Details of each HSU are tabulated in Table 4.2. The degree of hydraulic connection between the two HSUs cannot yet be quantified based on the existing available data; the HSU units could either be in direct hydraulic connection or disconnected such that HSU1 occurs as a perched system.

Table 4.2: Hydrostratigraphic units identified along the tunnel alignment

HSU	Geological Unit	Lithology	Thickness (m)	Hydraulic permeability range (m/s)[(a)]	Sy (-)[(b)]
1	Takaanini Formation (Alluvium) Residually weathered and weathered ECBF	Clay, silt, and sand mixtures, occasionally with minor organics Silt and clay (stiff), sand (dense), weathered mudstone and muddy sandstone	Generally < 5 m 5 to 19 m	10^{-7} to 10^{-6}	0.01 to 0.1
2	ECBF Rock	Weathered to unweathered mudstone and muddy sandstone	> 30	10^{-8} to 10^{-5}	0.01 to 0.1

Note:

a – CI Geotechnical Design Parameters Table

b – Anderson et al. (2015)7, p.228

4.4 Groundwater levels and flow regime

A review of groundwater levels obtained from available standpipe piezometers in proximity to the proposed tunnel alignment has been undertaken for this screening-level assessment. Additional and ongoing groundwater measurements will be taken over the coming months and incorporated into our further reporting on the assessment of effects.

A summary of the groundwater data available at the time of writing is presented in Table 4.3 below.

Table 4.3: Summary of standpipe piezometer monitoring records

BH ID	Location	Nearest Tunnel Feature	Investigation type	Standpipe tip depth (m bgl)	Groundwater level (m bgl)
Aurecon BH30	Sarsfield Street	Shaft 1	Machine borehole + standpipe	5.5	Dry [(1)]
Opus BH 15/2 (TTGD ID BH_110410)	Cnr Wallace St / Sarsfield St	Shaft 2	Machine borehole + standpipe	10	8.2
Opus BH 15/3 (TTGD ID BH_110409)	Cnr Herne Bay Rd / Argyle St	Shaft 4	Machine borehole + standpipe	16	10.6
Unknown [(2)]	24 Marine Parade	Shaft 7	Machine borehole + dual-standpipe	5.3	4.71
				15.9	2.88

Note 1: Piezometer dipped by a T+T engineer on 17 March 2023 to confirm reading presented on borehole log, which also recorded the piezometer as dry.

Note 2: Piezometer present within footpath outside 24 Marine Parade and dipped by T+T engineer on 23 March 2023. No borehole log records are available, with standpipe tip depths measured by the T+T engineer on site.

[7] Anderson, M. P., Woessner, W. W., & Hunt, R. J. (2015). Applied groundwater modelling: simulation of flow and advective transport. Academic press.

Hydrogeological conditions in the Auckland region, particularly within ECBF in coastal areas, generally comprise perched groundwater at shallower depths overlying a regional groundwater regime. The perched groundwater represents transient groundwater flow through the alternating permeable and less permeable sub-horizontally bedded shallow soil layers, with typical heads in the order of 1 to 2 m above the regional groundwater table[8]. The regional groundwater table in the area of Herne Bay is likely to be at an elevation of approximately RL 0 to +10 m, i.e. rising from the nearby coastline at a gradient of approximately 3 to 5 % (based on historical observations in the Auckland area).

For this preliminary assessment report, a general hydrostatic groundwater level has been adopted at 4 m bgl (based on the measurements presented above). This is considered to be appropriately conservative to reflect the lack of data available in some areas. An additional layer of conservatism has been applied to the groundwater model for this preliminary assessment by assuming that groundwater pressures are hydrostatic below 4 m bgl. This will be reviewed in our subsequent reporting based on site specific groundwater measurements.

[8] Aurecon (25 July 2014), *Auckland City Rail Link Enabling Works Contracts 1 and 2, Geotechnical Interpretative Report.* Document ref CRL-SYW-GEO-000-RPT-0003, revision 1.0.

5 Proposed shaft excavations

5.1 Modelling overview

For this preliminary assessment, three representative shaft configurations have been analysed to assess the upper bound ground settlement profile near surrounding buildings and infrastructure. The cases analysed are summarised below:

- Case 1 (Shaft 2) – model adopted to estimate the impact of significant thickness of compressible alluvial soils overlying ECBF. Shaft geometry adopted as per Shaft 2 geometry, due to the proximity of structures (including heritage structure at 58 Wallace St) and presence of alluvium at this location from available borehole data.
- Case 2 (Shaft 4/5) – model adopted to estimate the upper bound settlement profile for an anticipated typical ground profile across the scheme comprising residual ECBF soils overlying weathered ECBF and rock. The deepest shaft geometry has been modelled for this scenario (Shaft 5) while adopting the ground profile inferred at Shaft 4 which recorded a locally increased thickness of weathered ECBF.
- Case 3 (Intercept Shaft) – additional analysis case adopted to estimate the impact of the different construction methodology at the intercept shaft locations, adopting the temporary steel casing to full depth which will not provide outright groundwater cut-off. The ground profile and shaft geometry at SE01 (deepest intercept shaft) has been adopted, to represent the upper bound ground profile for EOP shafts where this shaft installation methodology is adopted.

The analysis cases undertaken are illustrated in Figure 5.1, Figure 5.2, and Figure 5.3 below.

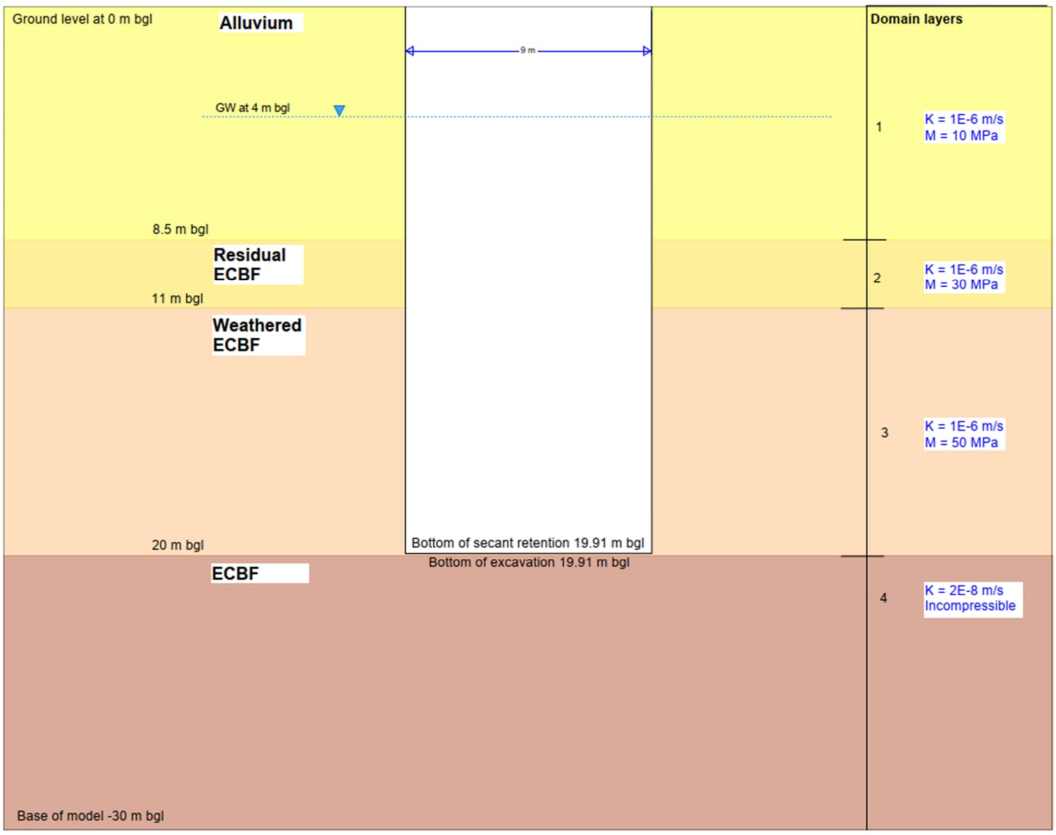

Figure 5.1: Graphical input summary for Case 1 (Shaft 2)

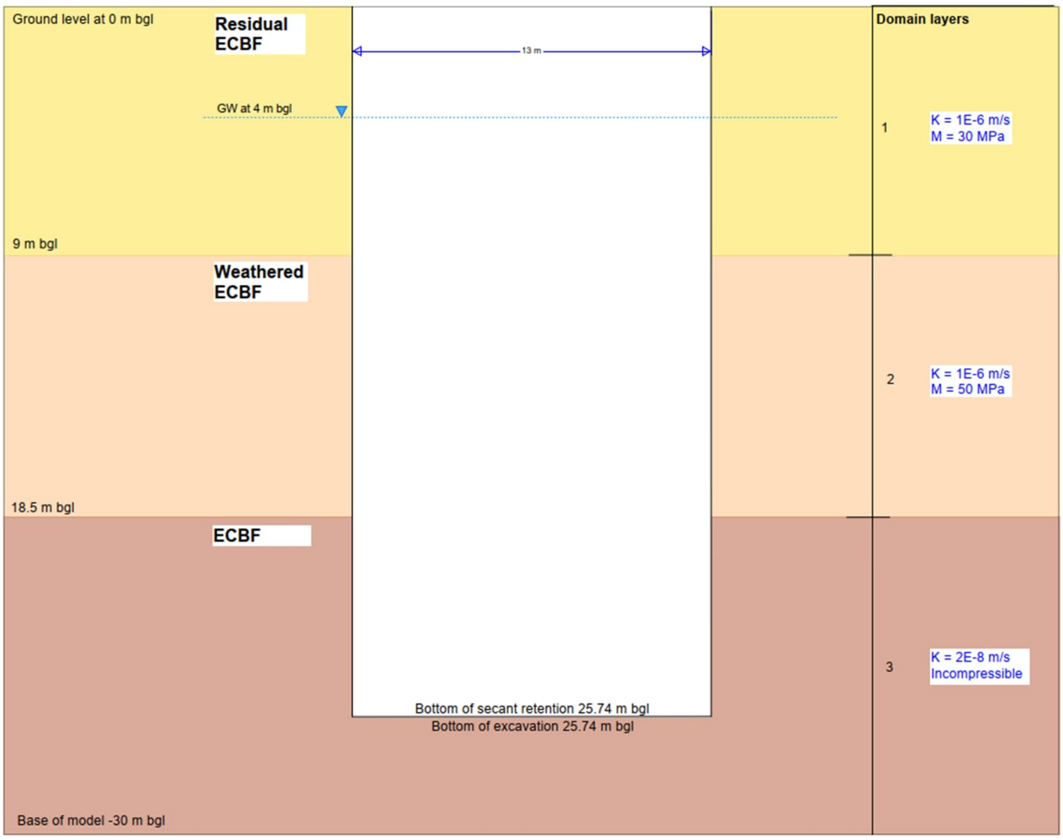

Figure 5.2: Graphical input summary for Case 2 (Shaft 4/5)

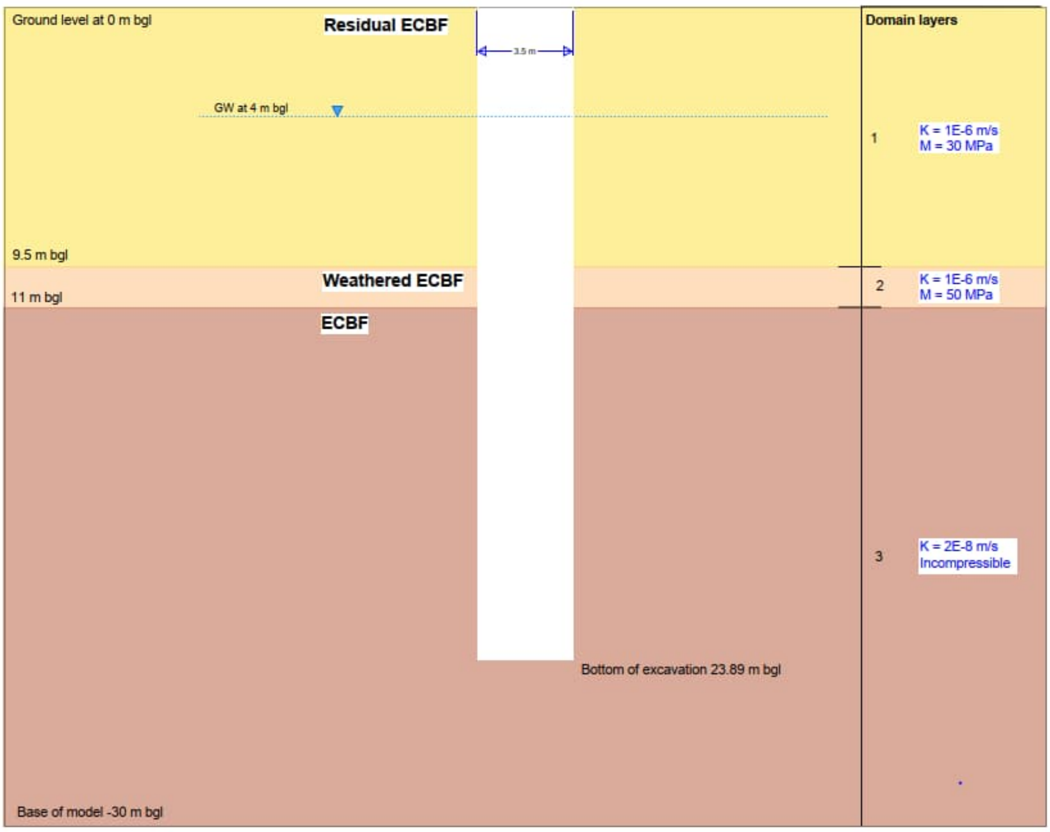

Figure 5.3: Graphical input summary for Case 3 (Intercept Shaft SE01)

Analysis of these three cases has been undertaken to estimate the following:

- Drawdown-induced ground settlements over the anticipated construction period.
- Shaft excavation retention deformation resulting in soil relaxation and displacement into the excavation as the retention walls displace inwards.

The results have been plotted onto a plan view at each shaft to present the potential extent of surface ground settlement at the locations of adjacent structures, infrastructure and residential dwelling. These plans are presented in Appendix C. It should be noted that the settlement effects presented on these plans are based on several simplifying conservative assumptions and therefore likely over-estimate settlement results, in particular at locations where a shallower shaft excavation is required and/or less adverse ground/ groundwater conditions are present. A detailed analysis at two to three representative sections for the shafts will be developed based on site-specific data to confirm the envelope of effects.

5.2 Groundwater drawdown and related settlement

5.2.1 Method

The ground models shown in Figure 5.1, Figure 5.2, and Figure 5.3 were adopted to undertake a groundwater drawdown-induced settlement analyses. Analytical Element Method (AEM) groundwater flow modelling software Analytical Aquifer Simulator (AnAqSim[9]) was used to estimate time dependent (transient) groundwater drawdown during the construction period. AnAqSim is capable of modelling groundwater flow in three dimensions.

Drawdown-induced settlement was calculated using an incremental layer summation method using python programming. This approach calculated the decrease in pore water pressure and corresponding increase in effective stress at the centre of each incremental layer caused by the groundwater drawdown in the unconfined units. This method assumed that the very weak ECBF rock was incompressible. Modulus of compressibility (m_V) values adopted for analysis are shown in Table 5.1 for each geological unit.

"Observation points" were added to the AnAqSim model for assessing groundwater drawdown due to dewatering. Drawdown-induced settlement was calculated for each drawdown observation point using the following approach:

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

[9] www.fittsgeosolutions.com
[10] www.python.org

- The following assumptions were made for the settlement assessment:
  - Initial static water levels were considered hydrostatic.
  - ECBF rock was considered incompressible.

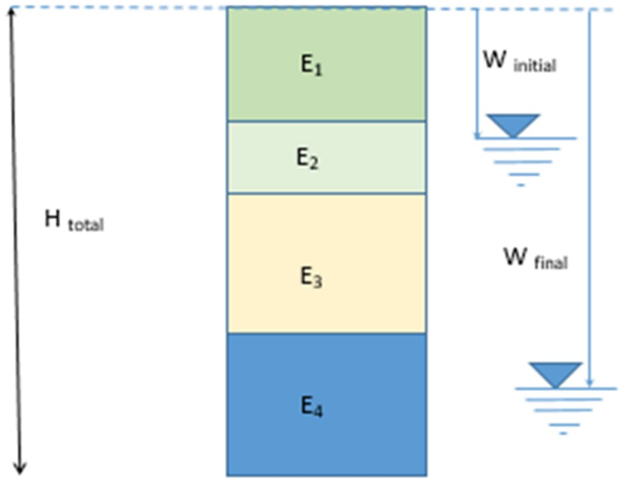

Figure 5.4: Example soil column and initial/final water level for calculating settlement using layer-wise summation method

Equation 1: Layer wise summation method:

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

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

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

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

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

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

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

Equation 2: Change in pore water pressure:

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

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

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

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

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

5.2.2 Model setup

The numerical model inputs and setup for the shaft analyses are presented in Table 5.1 below.

Table 5.1: Numerical modelling inputs

Geological Unit	Soil Permeability, k (m/s)	Modulus of compressibility, m_v (m^2/MN)
Alluvium	1×10^{-6}	0.1
Residual ECBF	1×10^{-6}	0.03
Weathered ECBF	1×10^{-6}	0.02
ECBF rock	2×10^{-8}	Incompressible

Note (1): For analysis case 3 (intercept shaft) the aquifer type was modelled as unconfined for the upper geological unit.

A construction period of 365 days was applied for assessment of the main shafts and seven days for the intercept shafts.

Two different secant pile conductance values were applied to represent the potential for variability in the performance and watertightness of the shaft. A conductance value of 1×10^{-2} day^{-1} has been adopted to represent a leaky retention system scenario which represents an upper bound (conservative) effect, and 1×10^{-3} day^{-1} for an expected "nominal" (reduced leakage) scenario. The analysis also included a sensitivity analysis to assess the difference in model results under elevated groundwater conditions. For the intercept shaft, we did not model an inflow barrier, as the proposed construction methodology is not anticipated to provide groundwater cut-off. If required, contingency measures may be considered to provide groundwater cut-off as outlined in Section 3.3.

5.2.3 Assumptions and limitations

Assumptions:

For modelling purposes we have conservatively assumed:

- Flat water table / negligible hydraulic gradient or direction.
- Rainfall recharge does not occur.
- Radius of AEM model set to 1 km from each shaft centre[11].
- Pseudo steady-state conditions are assumed after 365 days of continuous drainage. A transient 7-day timestep was adopted for intercept shafts based on the construction method provided for this excavation type.
- Hydrostatic conditions / groundwater levels apply to all geological units.
- For groundwater modelling, horizontal / uniform thickness geological units were assumed.
- Hydraulic permeability was assumed to be isotropic: Kv = Kh. In reality the vertical permeability will be much lower, and we anticipate that an anisotropic model with lower vertical permeability can be used in our updated modelling once site-specific permeability testing has been carried out.
- Very weak ECBF rock was assumed to be incompressible.

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

Implications:

The groundwater modelling has necessarily been based on a range of assumptions, as presented above. As site-specific investigations are carried out and as the design is developed, actual ground conditions and design variables may change. The groundwater modelling for the purposes of this preliminary assessment has therefore been carried out conservatively, with the intention that actual groundwater and geotechnical effects should be less than anticipated by this assessment. Implications of changes in ground conditions and design are discussed below:

- Design:
  - If the excavation level adopted for design is shallower the inflows and drawdowns estimates are expected to be less, and vice-versa. The analysis has adopted the maximum proposed excavation level based on the current scheme plans.
  - If the retention embedment (i.e. groundwater cut-off) depth adopted for design is shallower, the inflows and drawdowns estimates are expected to be greater, and vice-versa.
- Ground model:
  - Drawdown and inflow estimates are generally a function of the boundary conditions, the thickness of geological units, and the hydraulic permeability, Kv/Kh ratio, and specific yield values for each unit. Mapping and estimation of these are subject to inherent uncertainty; however, our selection of the hydraulic input parameter values and adopted thicknesses of geological units are considered to be suitably conservative.
  - If consolidation settlement has already occurred in the dewatered units (e.g. due to historical low groundwater levels), the observed settlement may be less than calculated. A sensitivity analysis has been undertaken adopting an upper and lower bound groundwater level based on existing data available.
  - If more compressible soils exist, the settlement estimates may be greater. However, our selection of the modulus of compressibility values adopted are considered to be suitably conservative.
- Excluding infiltration recharge suggests that the modelled drawdown and settlement estimates are over-estimated.

5.2.4 Analysis results

5.2.4.1 Case 1

Model results for Case 1 are presented for groundwater drawdown and settlement in Figure 5.5 and Figure 5.6, respectively. A groundwater level sensitivity analysis was also undertaken, adopting a design groundwater level at 4 m bgl (GW Scenario A) and 8 m bgl (GW Scenario B).

The findings of this preliminary assessment are summarised below:

- Maximum groundwater drawdown immediately adjacent to the shaft walls is conservatively estimated to be in the order of 1 to 1.2 m (assuming a leakage rate of 1×10^{-3} day^{-1} through the secant piled shaft walls).
- If the secant pile shaft walls are “leaky” (1×10^{-2} day^{-1}), maximum groundwater drawdown immediately adjacent to the shaft walls is conservatively estimated to be approximately 5 to 7 m. We note that these drawdowns are likely to be over-estimated due to the conservative assumptions made for the analysis (notably assuming isotropic hydraulic permeability throughout the hydrostratigraphic sequence).

- The analysis indicates that under typical construction techniques using secant piles allowing for a nominal amount of leakage (conductance = 1x10^{-3} day^{-1}), settlements of between approximately 8 mm and 3 mm are estimated at the shaft edge (not accounting for excavation-induced, or mechanical settlements related to pile installation and excavation).
- Results from the drawdown-induced settlement analysis for Scenario A, conductance = 1x10^{-2} day^{-1} which represents greater leakage due to poor construction of the secant pile wall (i.e. unplugged gaps between piles), are estimated to be less than 10 mm beyond approximately 30 m distance from the shaft edge. Settlements of less than 10 mm are generally considered to represent “less than minor effects” by Auckland Council.

Settlement profiles have been extrapolated onto a plan view (including addition of mechanical settlement) at each of the shaft locations as presented in Appendix C. These have been extrapolated adopting a secant pile conductance of 1x10^{-3} day^{-1} and a design groundwater level at 4 m bgl (GW Scenario A).

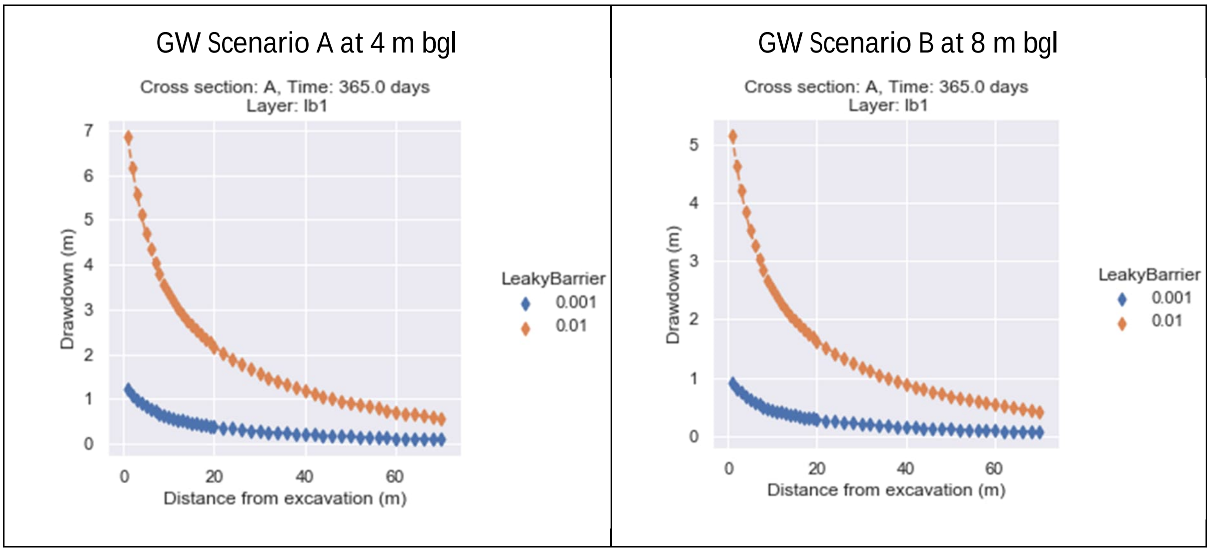

Figure 5.5: Case 1. Groundwater drawdown results.

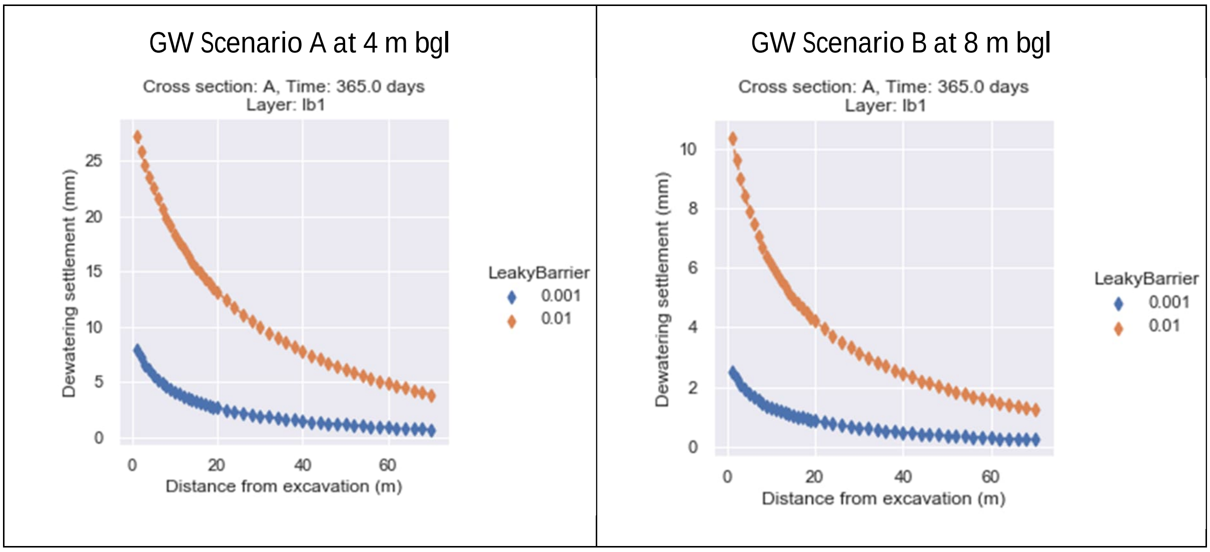

Figure 5.6: Case 1. Groundwater drawdown-induced settlement.

5.2.4.2 Case 2

Model results for Case 2 are presented for groundwater drawdown and settlement in Figure 5.7 and Figure 5.8, respectively. A groundwater level sensitivity analysis was also undertaken adopting a design groundwater level at 4 m bgl (GW Scenario A) and 8 m bgl (GW Scenario B).

The findings of this screening-level assessment are summarised below:

- Maximum groundwater drawdown immediately adjacent to the shaft walls is conservatively estimated to be in the order of 2 to 2.2 m (assuming a leakage rate of 1 x 10^{-3} day^{-1} through the secant piled shaft walls).
- If the secant pile shaft walls are "leaky" (1 x 10^{-2} day^{-1}), maximum groundwater drawdown immediately adjacent to the shaft walls is conservatively estimated to be approximately 9 to 11 m. As noted above, these drawdowns are likely to be over-estimated due to the conservative assumptions made for the analysis (notably assuming isotropic hydraulic permeability throughout the hydrostratigraphic sequence).
- The analysis indicates that under typical construction techniques using secant piles allowing for a nominal amount of leakage (conductance = $1x10^{-3}$ day^{-1}), settlements of between approximately 7 mm and 4 mm are estimated at the shaft edge (not accounting for excavation-induced, or mechanical settlements related to pile installation and excavation).
- Results from the drawdown-induced settlement analysis for Scenario A, conductance = $1x10^{-2}$ day^{-1} which represents greater leakage due to poor construction of the secant pile wall (i.e. unplugged gaps between piles), are estimated to be less than 10 mm beyond approximately 30 m distance from the shaft edge. Settlements of less than 10 mm are generally considered to represent "less than minor effects" by Auckland Council.

Settlement profiles have been extrapolated onto a plan view (including addition of mechanical settlement) at each of the shaft locations as presented in Appendix C. These have been extrapolated adopting a secant pile conductance of $1x10^{-3}$ day^{-1} and a design groundwater level at 4 m bgl (GW Scenario A).

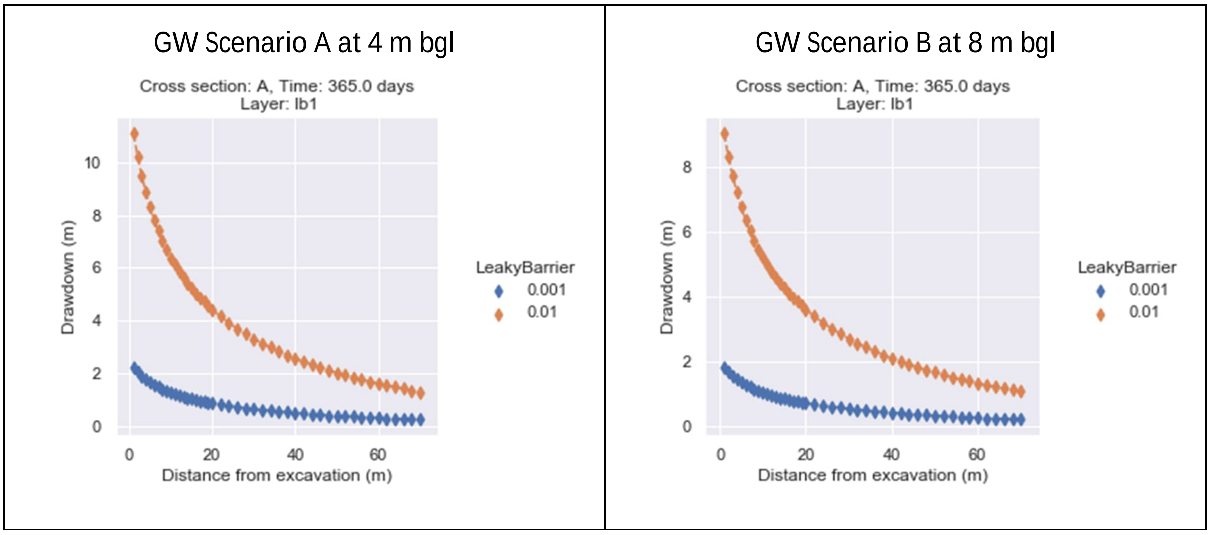

Figure 5.7: Case 2. Groundwater drawdown results.

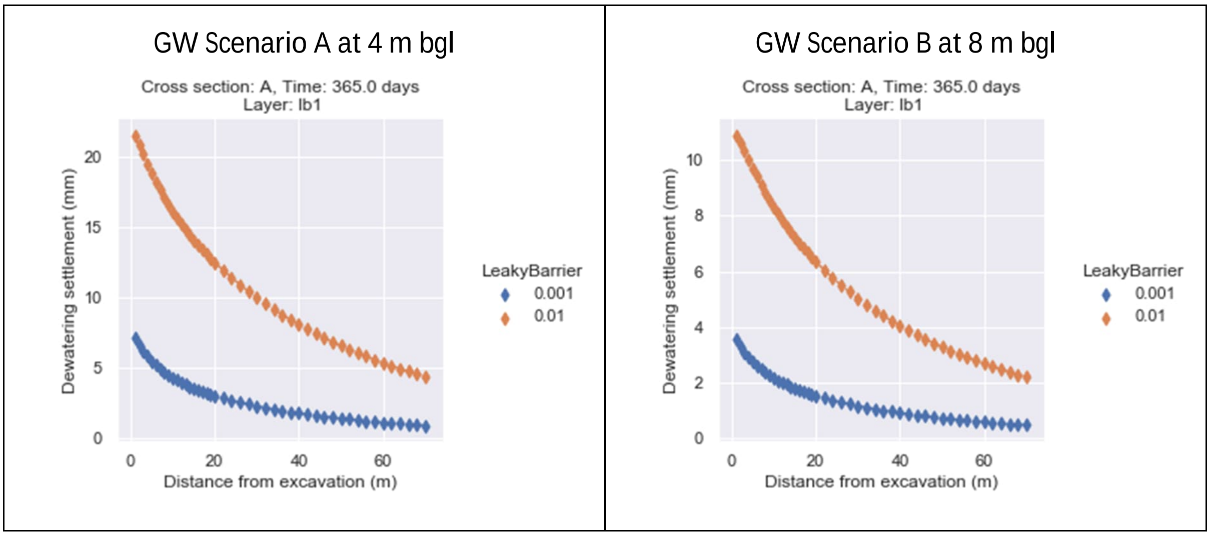

Figure 5.8: Case 2. Groundwater drawdown-induced settlement.

5.2.4.3 Case 3

Model results for Case 3 are presented for groundwater drawdown and settlement in Figure 5.5 and Figure 5.6, respectively. A groundwater level sensitivity analysis was also undertaken adopting a design groundwater level at 4 m bgl (GW Scenario A) and 8 m bgl (GW Scenario B).

The findings of this screening-level assessment are summarised below:

- Model results for the maximum groundwater drawdown immediately outside the intercept shaft for the range from approximately 3 m to 0.65 m.
- Settlements of between approximately 5 mm to 1 mm are estimated at the intercept shaft edge (not accounting for excavation-induced, or mechanical settlements). These are generally considered to represent “less than minor effects” by Auckland Council.

Settlement profiles have been extrapolated onto a plan view (including addition of mechanical settlement) at each of the intercept shaft locations as presented in Appendix C. These have been extrapolated adopting a design groundwater level at 4 m bgl (GW Scenario A).

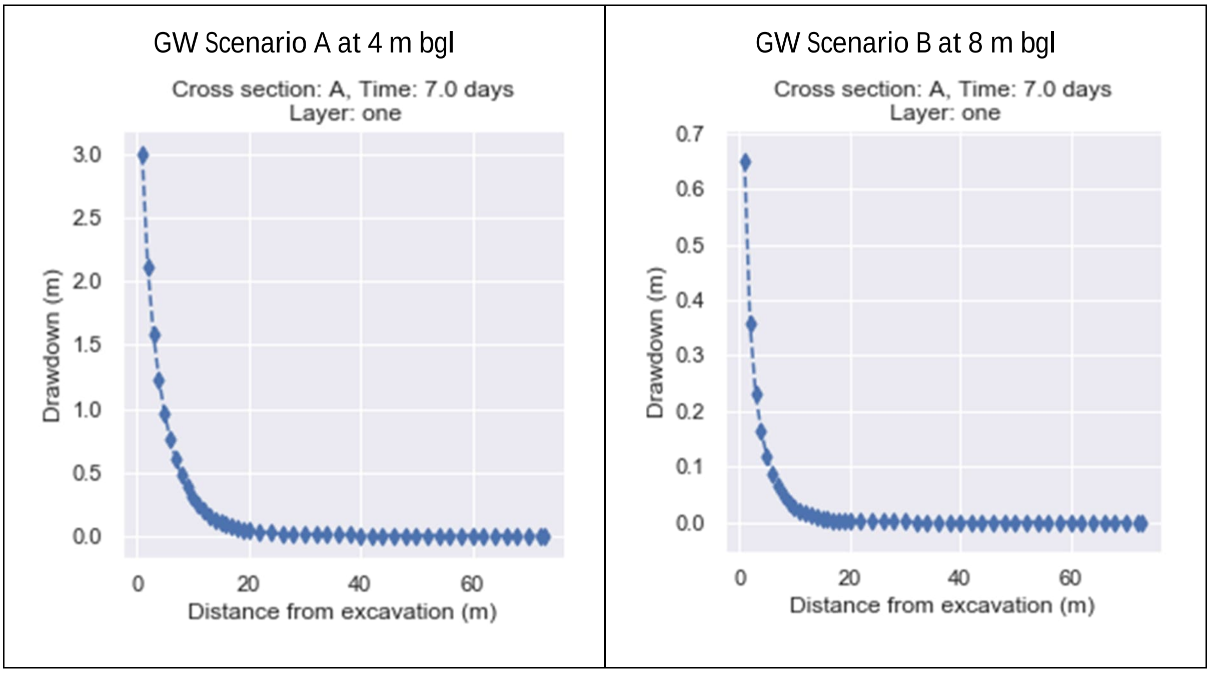

Figure 5.9: Groundwater drawdown results.

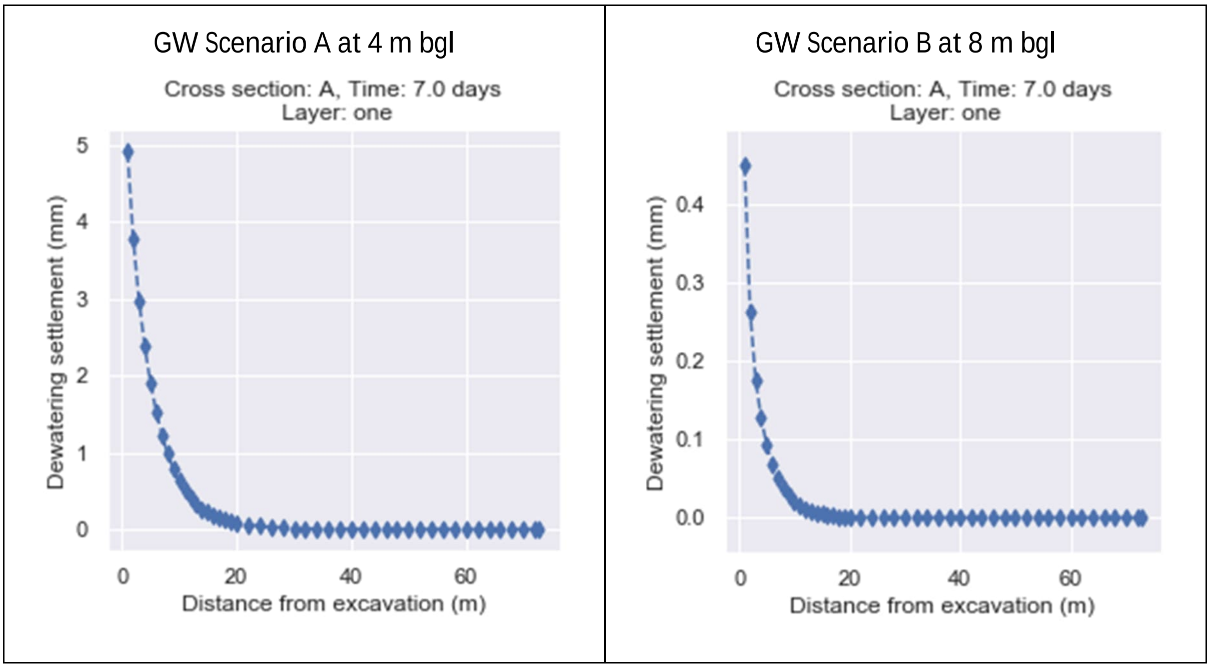

Figure 5.10: Groundwater drawdown-induced settlement.

5.3 Mechanical settlement due to excavation of shafts

The section assesses the surface ground settlement resulting from the excavation of the secant piled shafts (i.e. excluding any effects on groundwater). The proposed circular secant pile retention system is inherently rigid by design, as the structural system goes into compression to resist the lateral earth pressures, rather than deforming or relying on a 'toe embedment' as seen with conventional cantilevered or tied-back retention systems. This results in very small to negligible deformations as the shaft is excavated.

Accordingly, we have only assessed an upper bound scenario to demonstrate that mechanical settlement from shaft excavation is negligible. This case is derived from the Shaft 2 configuration (Figure 5.1), with the presence of lower-strength alluvium across the upper section of the shaft retention expected to result in the greater wall deformation and corresponding settlement. These deposits comprised layers of soft to firm clays which are expected to be highly compressible.

As an upper bound scenario has been assessed, the results should be conservative when adopted for the remaining shafts. Given the negligible surface ground settlement that arise from the deformation of the shaft retention, the effects of mechanical ground settlement from shaft excavation are not assessed in further detail beyond this section of the report.

5.3.1 Method

Two dimensional Fast Lagrangian Analysis of Continua (FLAC Version 7.0, Itasca Consulting Group) was used to model soil-structure interaction and estimate a settlement profile for Shaft 2. Figure 5.11 shows the model generated in FLAC.

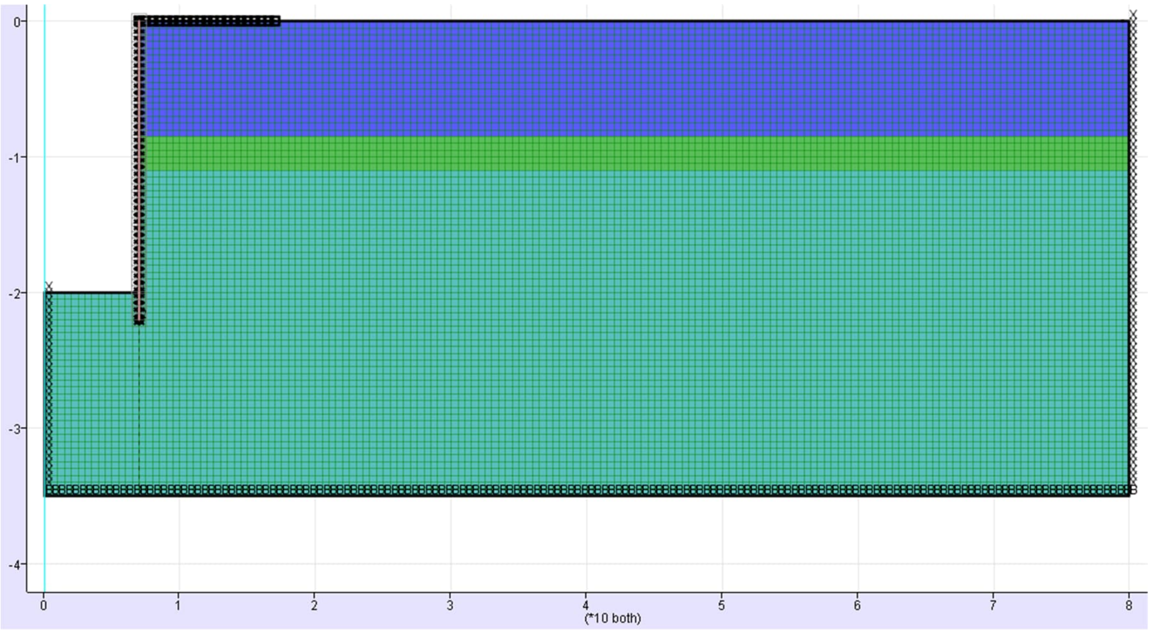

Figure 5.11: FLAC model (Shaft 2)

Table 5.2: Dimensions of shaft model

Shaft depth (m)	Pile embedment (m)	Outer Shaft Diameter (m)	Support
20	2	13.5	750 mm secant piles at 500 mm c/c spacing

As the support has yet to be designed, the configuration of the secant piles has been adopted based on similar shaft designs in similar geology. The modelled groundwater level has been conservatively assumed to be 4 m below the ground surface outside of the shaft throughout the construction. The shaft will be excavated “dry”, and therefore the groundwater level inside the shaft is assumed to be at the base of the excavation. This is considered appropriately conservative for the assessment of mechanical settlement as the secant piles will be subject to higher hydrostatic groundwater pressures than if groundwater drawdown was modelled.

A 35 kPa surcharge has been conservatively modelled over a 10 m wide area directly behind the shaft opening to account for construction machinery. This is based on a 120 tonne crane on a 7 m x 5 m wide pad.

5.3.2 Structural properties of ring beam installation

The properties in Table 5.3 have been adopted in our model.

Table 5.3: Structural properties used for modelling

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

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

**Based on the overlap thickness of 750 mm diameter piles at 500 mm centre-to-centre spacing.

5.3.3 Construction sequence

The following sequence has been assumed for the analysis:

1 Construct secant pile in an alternating hard and soft sequence.

2 Excavate down to final depth (~20 m below ground surface modelled).

5.3.4 Assumptions and analysis limitations

The modelling results are based on the following conservative assumptions:

1 An axisymmetric model has been used, which does not allow for the explicit modelling of unbalanced loading (variation in ground conditions, groundwater conditions or ground surcharges). Accordingly, the real 3D effects from the unbalanced loading are difficult to quantify with the adopted model. However, due to the conservative modelling approach and experience with similar designs and geology, we expect the effect on settlement is negligible.

2 The secant piles are interconnected and behave as a compression ring. Any adverse effect from loss of contact between secant piles has not been analysed; however, this is exceedingly unlikely as any complete loss of contact between piles is more likely to occur at depth due to out-of-verticality of piles, rather than within the top 5 metres of the walls where soils are more prone to movement.

3 Mechanical settlement associated with the construction of the tunnel connecting the shaft is considered to have a negligible contribution based on its size and depth. This should be considered as part of detailed design of the structure, however, is not considered necessary as part of the assessment of effects for the resource consent application.

5.3.5 Analysis results

Figure 5.12 below presents the settlement predicted at the ground surface due to the shaft excavation. The results indicate that mechanical induced settlement from the construction of Shaft 2 is less than 2 mm, i.e. effectively negligible and within tolerances of survey and the geotechnical model.

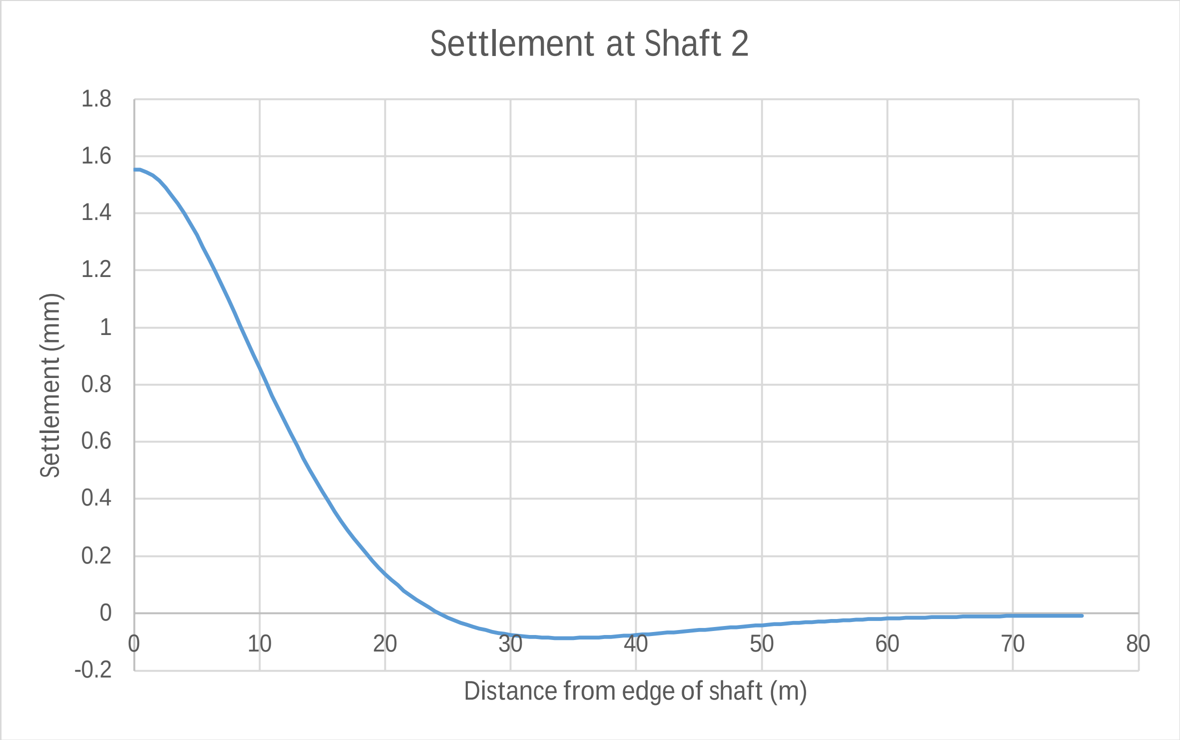

Figure 5.12: Ground surface settlement prediction

6 Proposed tunnel excavation

6.1 Assessment of groundwater inflows, drawdown, and related settlement

6.1.1 Design method and assumptions

A transient groundwater drawdown analysis has been undertaken using numerical modelling software SEEP/W. Two scenarios were considered for the assessment, to represent the “likely” ground conditions (Scenario 1) and a “conservative” assessment (Scenario 2).

Both scenarios assume, on a conservative basis, that at any one time, a 12.5 m longitudinal section of the of the tunnel may be ‘open’ when operating in open-mode and therefore subject to groundwater inflow for up to two days before the lining is constructed across the ‘open’ section, effectively sealing this section off from groundwater inflow.

Two-dimensional models were developed adopting the ground model and tunnel depths based on a representative ground profile for the outflow pipe from Shaft 4 as shown below in Figure 6.1. This ground model is inferred to be the maximum thickness of saturated soil above the deepest section of the tunnel alignment.

The tunnel is well above the regional groundwater table at its western extent (10.5 m RL), only dropping below the regional groundwater table as it approaches Point Erin (-4 mRL). The tunnel itself will be in ECBF rock which will experience negligible consolidation due to any reduction in pore water pressure; however, if pore water pressures drop below historical minimums in the overlying soils, they could experience consolidation settlement.

For this to occur, there needs to be vertical connectivity between the rock surrounding the tunnel and the overlying soils. Groundwater within ECBF is widely observed to be perched, with typical heads in the order of 1 to 2 m [(12)] above the regional groundwater table. Vertical connectivity between perched water tables is expected to be poor.

For this preliminary assessment, an assumption has been made that groundwater is hydrostatic from 4 m bgl down to the tunnel, whereas actual pore water pressures are expected to be significantly less. For that reason, this assessment of groundwater drawdown and resultant consolidation settlement is conservative, as actual groundwater pressures will not drop from such high initial values.

Perched groundwater will be investigated and measured, and recharge/drawdown effects analysed in detail in our further reporting to be provided to Council in due course.

[12] Aurecon (25 July 2014), *Auckland City Rail Link Enabling Works Contracts 1 and 2, Geotechnical Interpretative Report.* Document ref CRL-SYW-GEO-000-RPT-0003, revision 1.0.

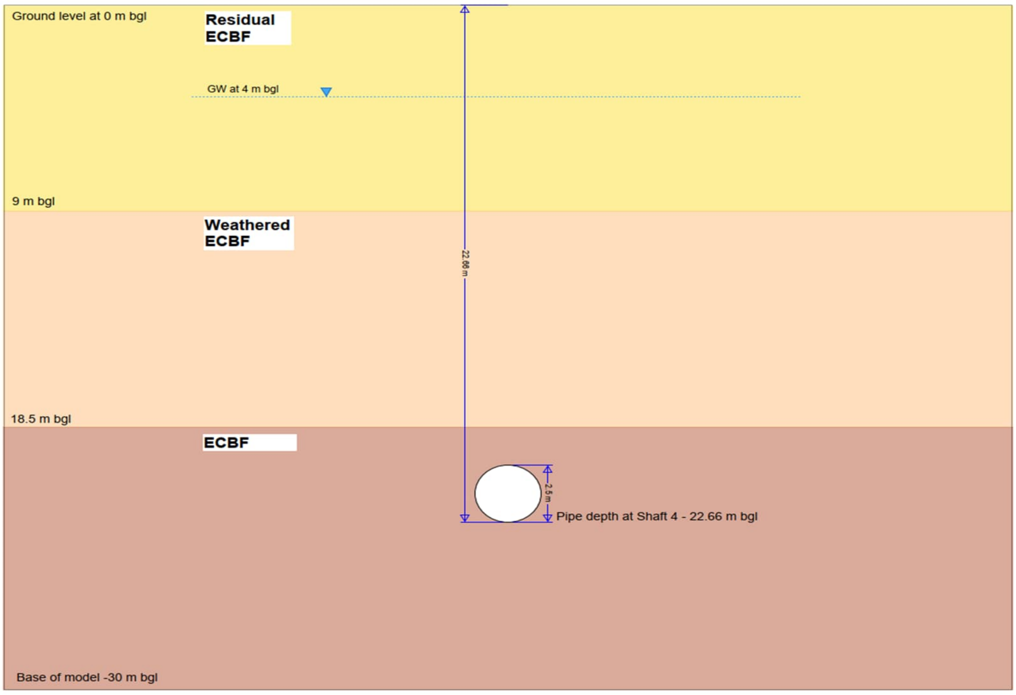

Figure 6.1: Ground model for tunnel groundwater drawdown assessment

6.1.2 Groundwater modelling

Initial groundwater conditions were defined in the SEEP/W model by applying the assumed groundwater pressures as fixed head boundary conditions at the model extents. The model was extended 100 m beyond the tunnel axis in both directions, beyond which it is assumed negligible groundwater drawdown due to tunnelling would occur.

Once initial groundwater pressures are established, a transient groundwater drawdown analysis was undertaken installing the tunnel excavation and applying a seepage face for a period of two days. An additional analysis was carried out to allow for potential shadowing effects of the tunnel excavation causing dewatering in front / behind the 'open' section, by extending the drawdown analysis to a 7 day period.

The following hydrogeological parameters were adopted in the analysis.

Table 6.1: Hydrogeological parameters adopted within SEEP/W

Geological Unit	Horizontal Permeability, k_x (m/s)	Vertical Permeability, k_v (m/s)	Modulus of compressibility, m_v (m^2/MN)
Residual ECBF	1×10^{-6}	1×10^{-7}	0.03
Weathered ECBF	1×10^{-6}	1×10^{-7}	0.02
ECBF rock	Scenario 1: 2×10^{-8} Scenario 2: 1×10^{-7}	Scenario 1: 2×10^{-9} Scenario 2: 1×10^{-8}	0.004

6.1.3 Analysis results

The transient SEEP/W analyses indicates that for both modelling scenarios, the open tunnel section results in a localised reduction in pore water pressure across the geological units at the elevation of the tunnel (typically the ECBF rock unit). Limited reduction of groundwater pressure is observed within the upper residual ECBF or weathered ECBF units, as shown in Figure 6.2 and Figure 6.3 below.

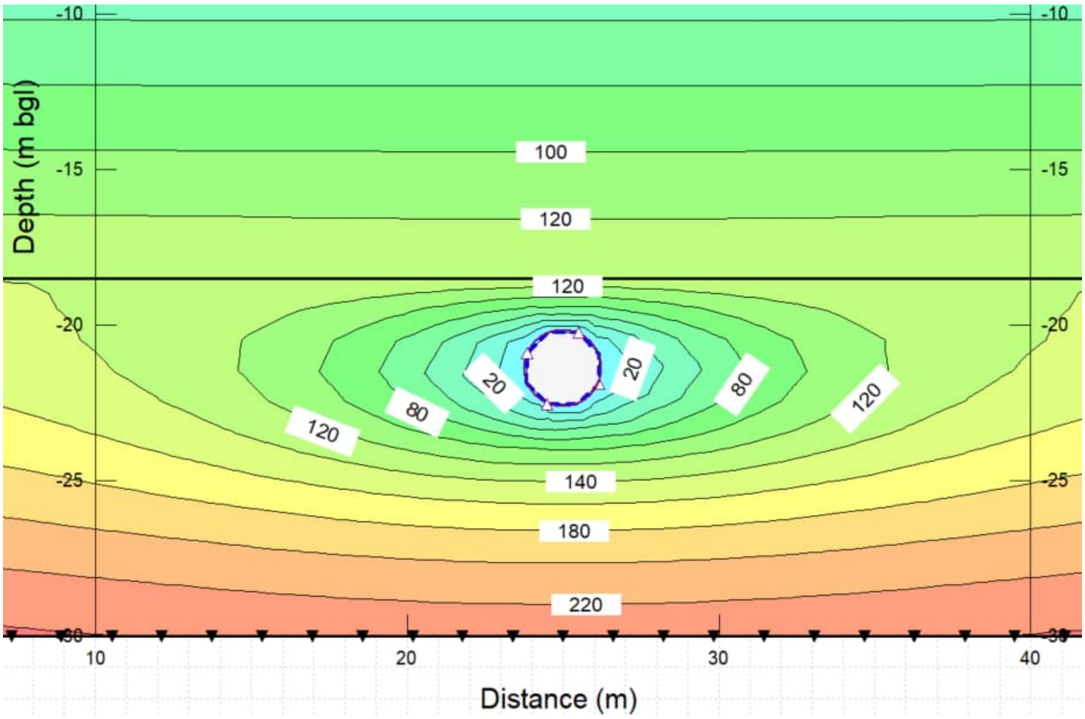

Figure 6.2: Groundwater pressure contours developed surrounding tunnel excavation (Scenario 1)

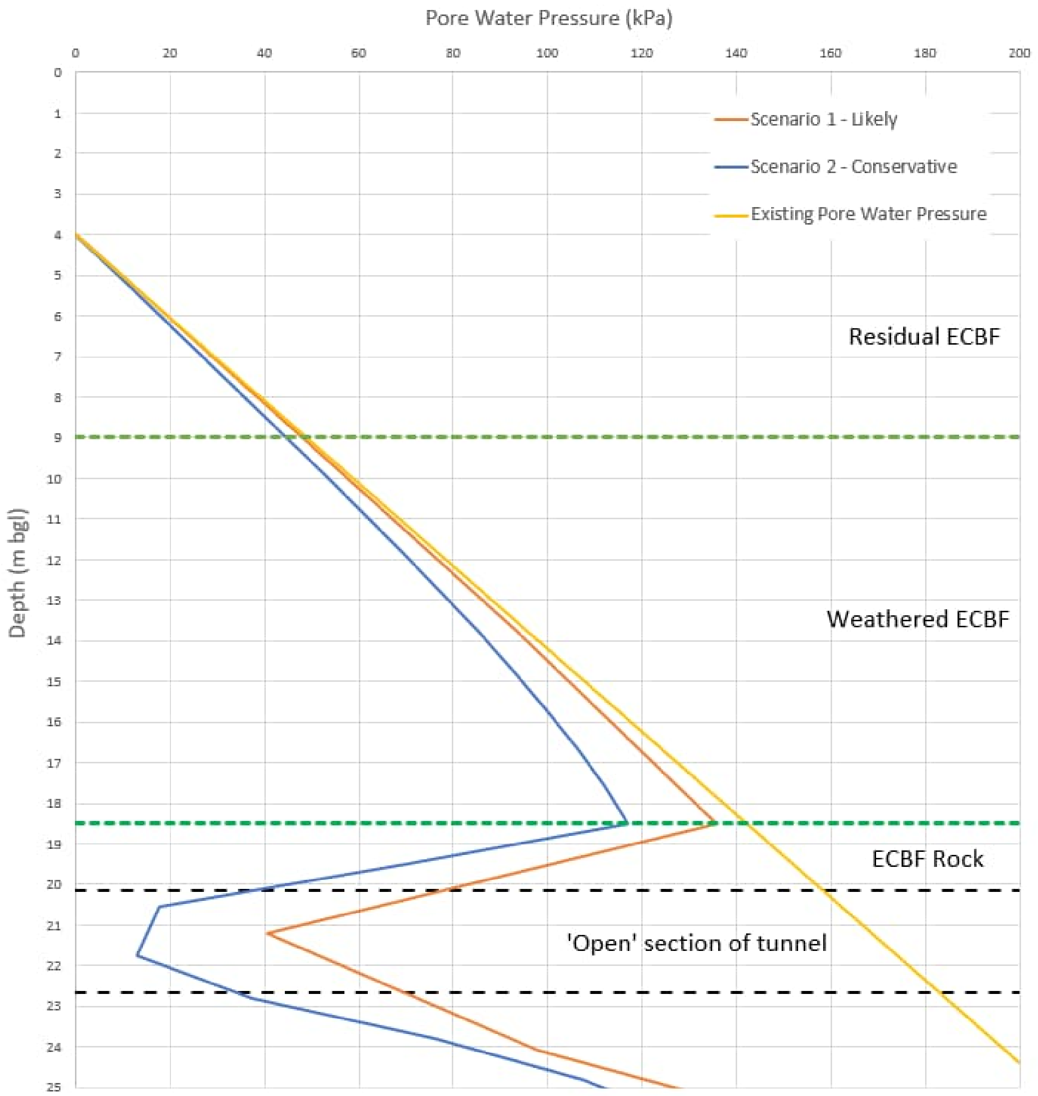

Figure 6.3: Pore water pressure profile with depth

The ECBF rock layer has been modelled with lower vertical permeability/connectivity than the overlying weathered ECBF and soil layers. As a sensitivity check, an additional analysis has been undertaken to consider the impact of a reduced thickness of ECBF rock above the crown of the tunnel excavation and the impact on pore pressures within the overlying more compressible units. This assessment has assumed a minimum thickness of very weak ECBF rock above the crown.

The pore water pressure profiles with depth for this assumption is shown in Figure 6.4 below, which indicates a greater reduction in pore pressure will occur when tunnelling near the weathered ECBF / ECBF rock interface. It is not anticipated that operation in open mode would be undertaken in such proximity to this interface. However, this pore water pressure profile has been adopted in the settlement analysis to assess an ‘upper bound’ settlement profile which accounts for unexpected ground conditions during construction.

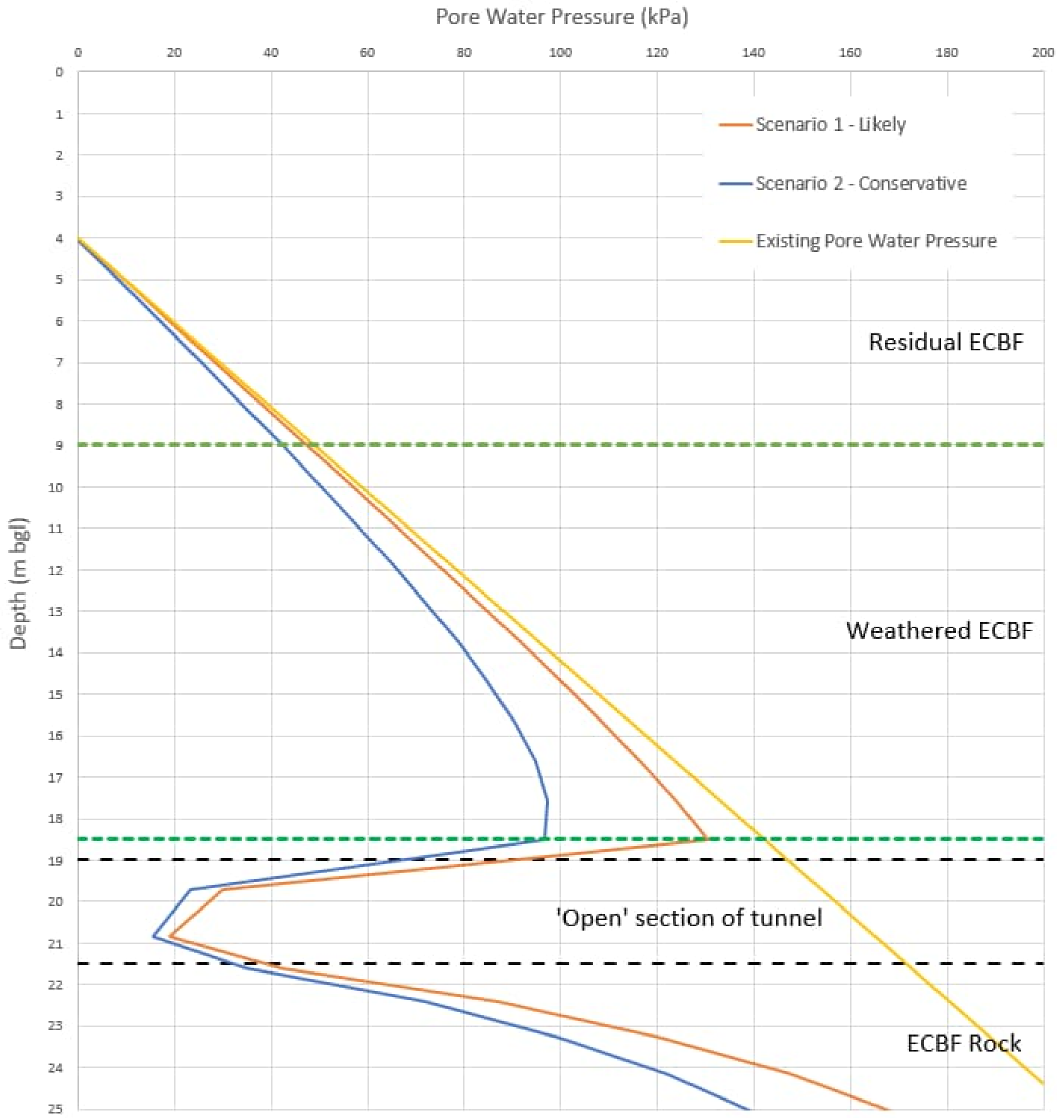

Figure 6.4: Pore water pressure profile with depth (0.5 m ECBF rock above tunnel crown)

6.1.4 Surface settlement

Ground surface settlement due to the above reduction in pore water pressures have been estimated based on:

- Change in stress (i.e. reduction in pore water pressure) estimated by SEEP/W analysis.
- Soil compressibility for the respective geological units as set out in Table 6.1.
- Equation below adopted the average change in stress across the respective geological units:

$$S = mv \,.\, \Delta\sigma v \,.\, H$$

S = settlement
mv = modulus of compressibility
$\Delta\sigma v$ = Change in stress
H = Saturated geological unit thickness

The ground surface settlement profile extending perpendicular to the tunnel axis are presented on Figure 6.5 below. The potential impact of these settlements is discussed in Section 8.1.2.

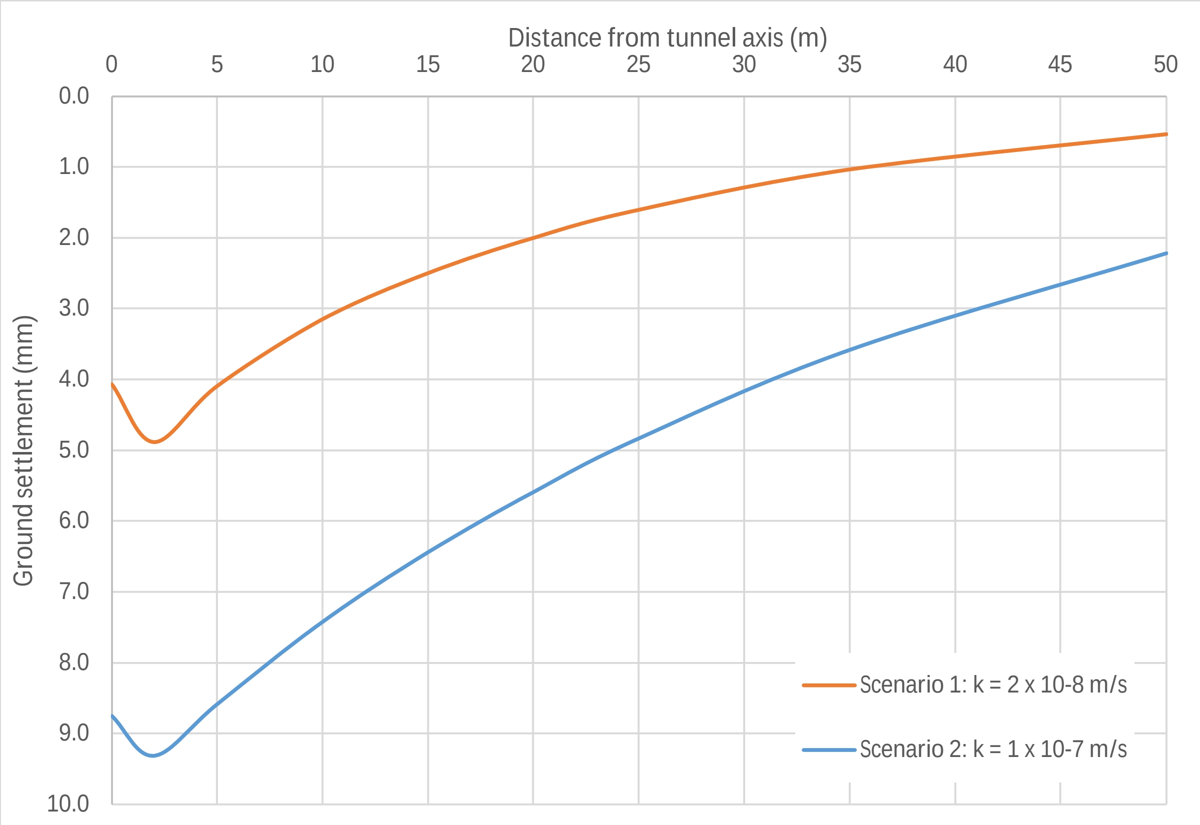

Figure 6.5: Ground surface settlement profile due to tunnel dewatering (tunnel approximately 20 m bgl)

6.2 Mechanical settlement due to tunnelling

The section below presents our assessment of surface ground settlement arising from the over excavation of face during tunnelling commonly referred to as "volume loss".

6.2.1 Method

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

Tunnelling publications comprising experience from thousands of projects suggest that volume loss can vary from 0% to 4% depending partly on the ground conditions, but primarily from the specific capabilities of the TBM being used and contractor performance. The parameters outlined in Table 6.2 below have been adopted for this assessment, which have been assessed based on our experience in these geological units.

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

Table 6.2: Parameters adopted for assessing settlement due to tunnelling volume loss

Parameter	Fill	Takaanini Formation	Residual ECBF	ECBF Rock
Volume loss (%)	2.5	2.5	1.0	0.5
Trough width factor, K	0.5	0.5	0.5	0.5

6.2.2 Analysis results

Table 6.3 presents a summary of the surface ground settlements for different tunnel depths based on the inferred geological model. The results indicate that surface ground settlements are expected to be less than 4 mm with differential settlement less than 1V:1000H along the shallowest sections of the alignment.

It should be noted that the section between Shaft 7 and 8 is currently proposed to be open trenched and therefore would not result in any settlement due to tunnelling. However, this has been assessed as the critical case to allow for potential variation in the construction methodology.

Table 6.3: Summary of surface ground settlement resulting from tunnelling volume loss

Volume loss (%)	Pipe Dia. (m)	Pipe Location	Depth to pipe centreline (m)	Surface ground settlement (mm)	Maximum differential settlement
0.5 (ECBF rock)	2.5	Shaft 7	5.03	<4	> 1V:1000H
1.0 (Residual ECBF)	0.72	Shaft 8	2.06	<2	> 1V:1300H
2.5 (Fill/ Takaanini Formation)	0.72	Shaft 7 – 8	3.72	<3	> 1V:1500H
		Shaft 7	5.02	<2	> 1V:2000H

A series of sewer lines are proposed associated with the EOP branch sewer, which is to connect to the main tunnel alignment. The lines are to comprise of horizontally drilled tunnel sections less than 1.2 m in diameter (refer to drawings in Appendix A) and are therefore exempt from the AUP Permitted Standards E27.6.1.10(2)-(6). These have not been assessed at this stage.

6.2.3 Sensitivity Analysis for unexpected ground conditions

The majority of the proposed tunnel alignment is expected to be within in ECBF rock, particularly within the deeper alignment (Option 1), however the inferred geological model indicates there is a possibility that the that tunnelling may extend into residual soils, Takaanini formation or fill.

The ground profile across the tunnel alignment is undulating, and the tunnel extends below three 'valley' features where there is potential for localised alluvial palaeo-channels to have formed within the shallow ground profile. These low points are located as follows:

1 Marine Parade – extending north of Shaft 6 to Shaft 8, with a local low point at Shaft 7.

2 Argyle St – extending between Shaft 3 and Shaft 5, with a local low point at interception shaft SE04.

3 Sarsfield St – extending between interception shaft SE01 and Shaft 2, with a local low point at interception shaft SE03.

A sensitivity analysis has been undertaken to account for the potential varied ground conditions across the tunnel alignment adopting potential worst-case ground conditions (Takaanini formation / Fill) modelling the shallowest TBM tunnel depths across the alignment. These are presented in Table

6.4. Tunnelling in open-mode would not be permitted within these materials, and therefore groundwater drawdown effects due to tunnelling are not anticipated at these locations.

Table 6.4: Summary of sensitivity analysis for tunnelling volume loss assuming Takaanini formation soils at local low-points within the alignment

Location	Depth to pipe centreline (m)	Surface ground settlement (mm)		Maximum differential settlement[1]	
		Within road corridor	Beyond road corridor	Within road corridor	Beyond road corridor
Marine Parade (Shaft 7)	5.03	19	<1	1(v):220(h)	>1(v):1,000(h)
Argyle Street (SE04)	5.68	17	<1	1(v):280(h)	>1(v):2,000(h)
Sarsfield St (SE03)	9.03	13	<2	1(v):520(h)	>1(v):2,000(h)

(1) Differential settlements presented perpendicular to the tunnel alignment. For services that run parallel or oblique to the tunnel direction, differential settlements are anticipated to be less.

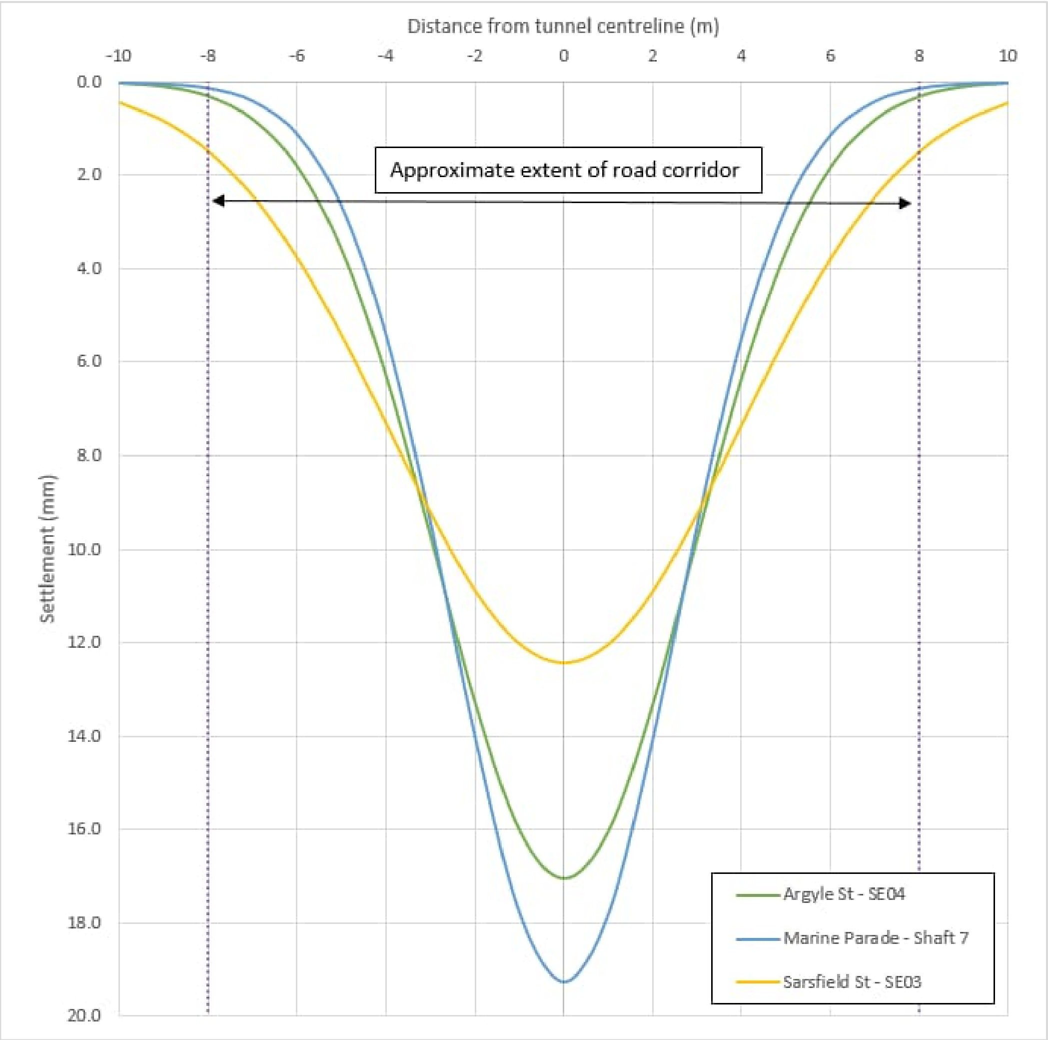

Figure 6.6: Ground surface settlement profiles for the sensitivity analysis at 3 No. low points in alignment

This analysis indicates that at the low points of the alignment, settlements of greater than 10 mm may occur if tunnelling extends into Takaanini Formation or Fill soils. However, the settlement rapidly decreases either side of tunnel centreline, with less than 10 mm predicted at 3 m offset and less than 2 mm predicted beyond the road corridor. Additionally, these potential effects would be isolated to these low points in the alignment, with settlement directly above the tunnel centreline reducing with increased depth of the tunnel as the ground level rises.

Accordingly, when considering the potential effects on residential properties in proximity to the tunnel alignment, the combined effects due to groundwater drawdown (operating in open-mode) with a deeper tunnel elevation remain critical (rather than tunnelling in closed mode through Takaanini Formation or Fill). Consideration of impacts to underground services which are located close to or intersecting the tunnel alignment at these low points may be required as part of our further assessment and reporting, which is discussed further in Section 8.3.

7 Proposed open trench excavation

Based on the currently proposed construction methodology[2], approximately 420 m of open trenched excavations are proposed. This includes approximately 140 m along the main alignment between Shaft 7 and 8 along Marine Parade, with remainder for installation overflow pipe connections as outlined in Table 3.1. The proposed open trenched sections are illustrated in Figure 7.1 below.

Figure 7.1: Alignment construction methodology; open trench excavations denoted in yellow

7.1 Assessment of groundwater inflows, drawdown, and related settlement

7.1.1 Method

Analytical Element Method (AEM) groundwater flow modelling software Analytical Aquifer Simulator (AnAqSim[14]) was used to estimate time dependent (transient) groundwater drawdown during the construction period. AnAqSim is capable of modelling groundwater flow in three dimensions.

Drawdown-induced settlement was calculated using an incremental layer summation method using Python programming. This approach calculated the decrease in pore water pressure and corresponding increase in effective stress at the centre of each incremental layer caused by the groundwater drawdown in the unconfined units.

The trench excavation geometry and ground model adopted in the analysis is presented in Figure 7.2 below. This model assesses the deepest section of proposed open trenching, currently proposed up to 5.9 m below ground level for the tunnel section between Shaft 7 and 8. The ground model was adopted in accordance with the current geological model for the tunnel alignment presented in Appendix B.

[14] www.fittsgeosolutions.com

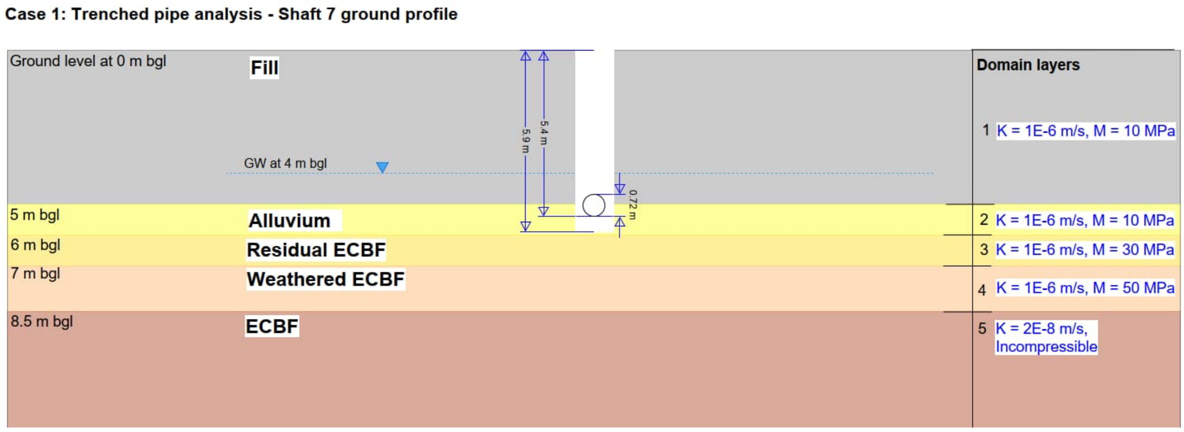

Figure 7.2: Graphical input summary for open trenching

The analysis method adopted is the same as described in Section 5.2.1 and design assumptions are the same as described in Section 5.2.3. The transient analysis was undertaken adopting a 30-day time period. No impedance was modelled for the proposed trench shield supports (i.e. no groundwater cut-off is provided). Groundwater was adopted at 4 m bgl for this analysis.

The following modelling input values were adopted within the analysis:

Table 7.1: Numerical modelling inputs

Geological Unit	Soil Permeability, k (m/s)	Modulus of compressibility, m_v (m^2/MN)
Fill / Alluvium	1×10^{-6}	0.1
Residual ECBF	1×10^{-6}	0.03
Weathered ECBF	1×10^{-6}	0.02
ECBF rock	2×10^{-8}	0.004

7.1.2 Analysis results

Analysis results are presented for groundwater drawdown and settlement in Figure 7.3.

The findings of this screening-level assessment are summarised below:

- Model results for the maximum groundwater drawdown of up to 1.7 m is estimated immediately behind the open trenched excavation.
- Ground surface settlements of up to 3 mm are estimated immediately behind the open trenched excavation (not accounting for excavation-induced, or mechanical settlements). These are generally considered to represent "less than minor effects" by Auckland Council.

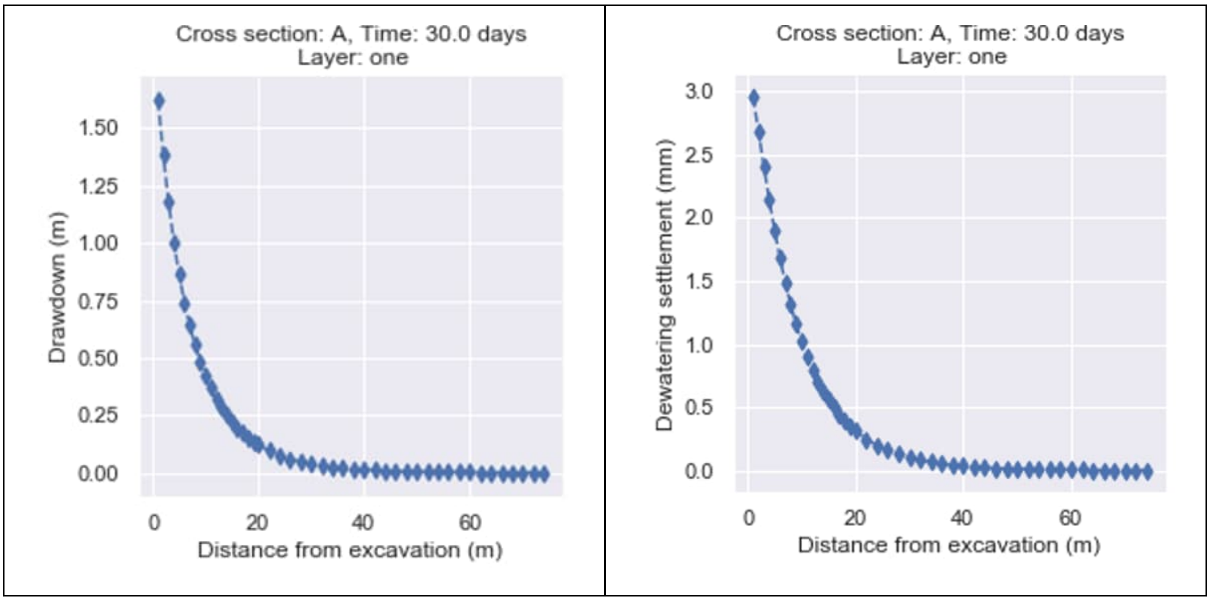

Figure 7.3: Summary of open trenching dewatering settlement results

7.2 Mechanical settlement

7.2.1 Design assumptions and method

A single representative model has conservatively been adopted across all proposed open trenched excavations to assess the zone of influence from mechanical settlement caused by the trench-shield supports, the following has been assumed:

- The maximum depth to invert for trenched sections is assumed to be 5.4 m bgl based on concept drawings. This equates to a maximum excavation depth of 5.9 m bgl, assuming 0.5 m of overdig for pipe bedding.
- The excavation will be retained using proprietary double slide-rail trench shields. It has been assumed the contractor will undertake temporary works design to confirm the trench shields to be used have sufficient capacity (i.e. prop buckling etc.) to support the excavation.
- The ends of each trench excavations will be stepped up to the ground surface with full trench shield support will be provided along the edge of the entire excavation (i.e. no lateral return wall will be formed at either extent).

Mechanical settlements due to the open trenched excavations have been assessed using the empirical method outlined in CIRIA C760[15]. This outlines an empirical method (Figure 6-15b, duplicated as Figure 7.4 below) to estimate surface mechanical settlements behind retaining walls based a number of case histories within stiff cohesive soils.

The controlling input parameters are the depth of excavation and the assumed stiffness of the retaining wall. For the purpose of this assessment, a high stiffness wall has been assumed which is applicable based on the support type (trench shields supported by multi-level props).

[15] CIRIA C760, 2018. Guidance on Embedded Retaining Wall Design. London, 455 pp Figure 6.15.

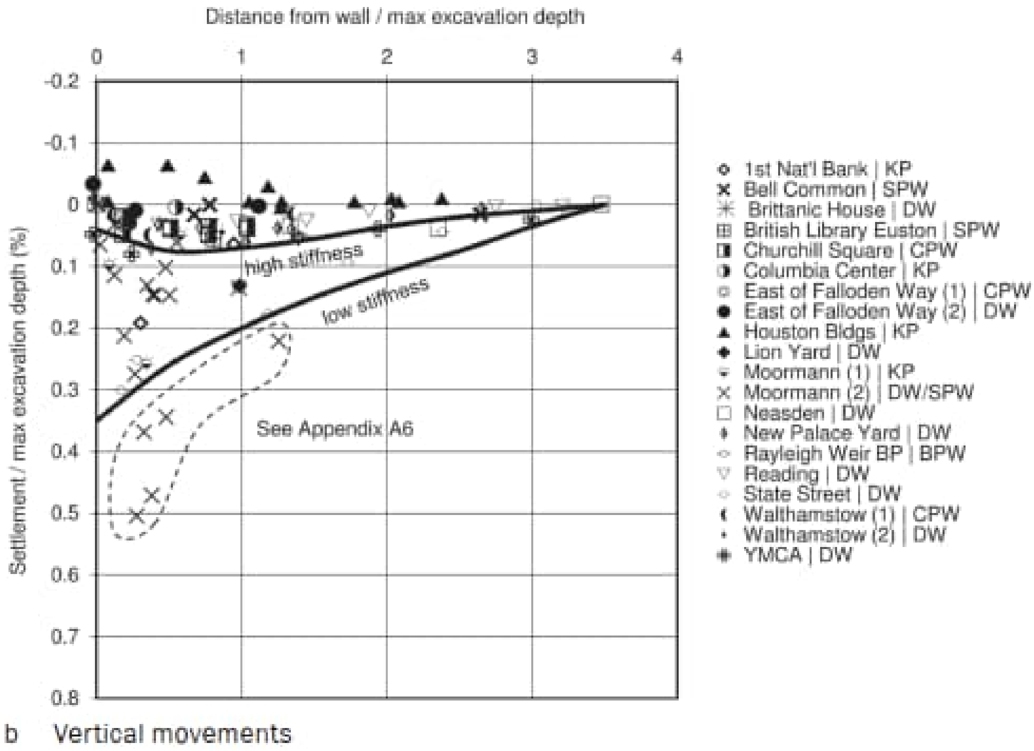

Figure 7.4: Extract of design chart (Figure 6.15b) from CIRIA C760.

7.2.2 Analysis results

The results of the mechanical induced settlement for the analysed trench excavation are presented in Figure 7.5. This presents the settlement for the maximum trenched excavation height of 5.9 m.

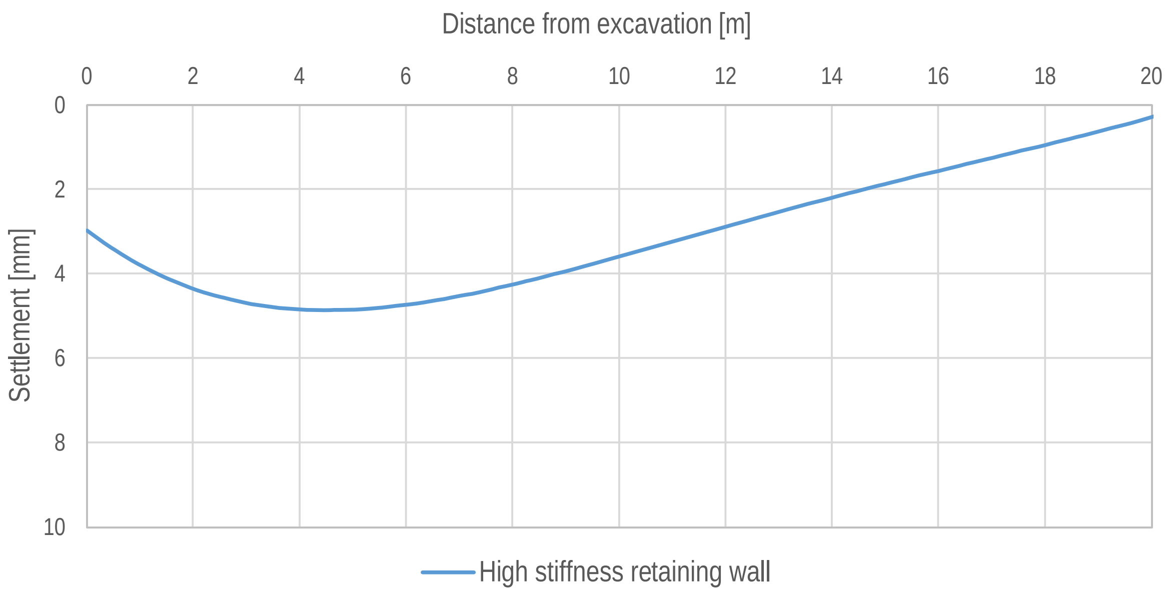

Figure 7.5: Mechanical settlement contour prediction due to the trench excavations

8 Risk of damage to existing buildings, structures and services

8.1 Combined ground settlements

8.1.1 Shaft construction

The combined settlement profiles for the proposed main shaft excavations have been plotted on Figure 8.1 below, adopting the following design cases:

- Estimated mechanical settlement profile due to shaft excavation.
- Groundwater drawdown settlement profile adopting:
  - GW Scenario A (4 m bgl) which is considered a conservative groundwater level assuming hydraulic continuity through all hydrostratigraphic units.
  - Secant wall conductance value of 1×10^{-3} day^{-1} which represents the expected "nominal" leakage through the secant wall, assuming good construction controls are in place.

This is profile represents the conservative 'upper bound' envelope of effects based on the information available at the time of writing.

The approximate location of the nearest neighbouring property boundary has been plotted based on the current scheme layout as detailed on WSP drawings presented in Appendix A.

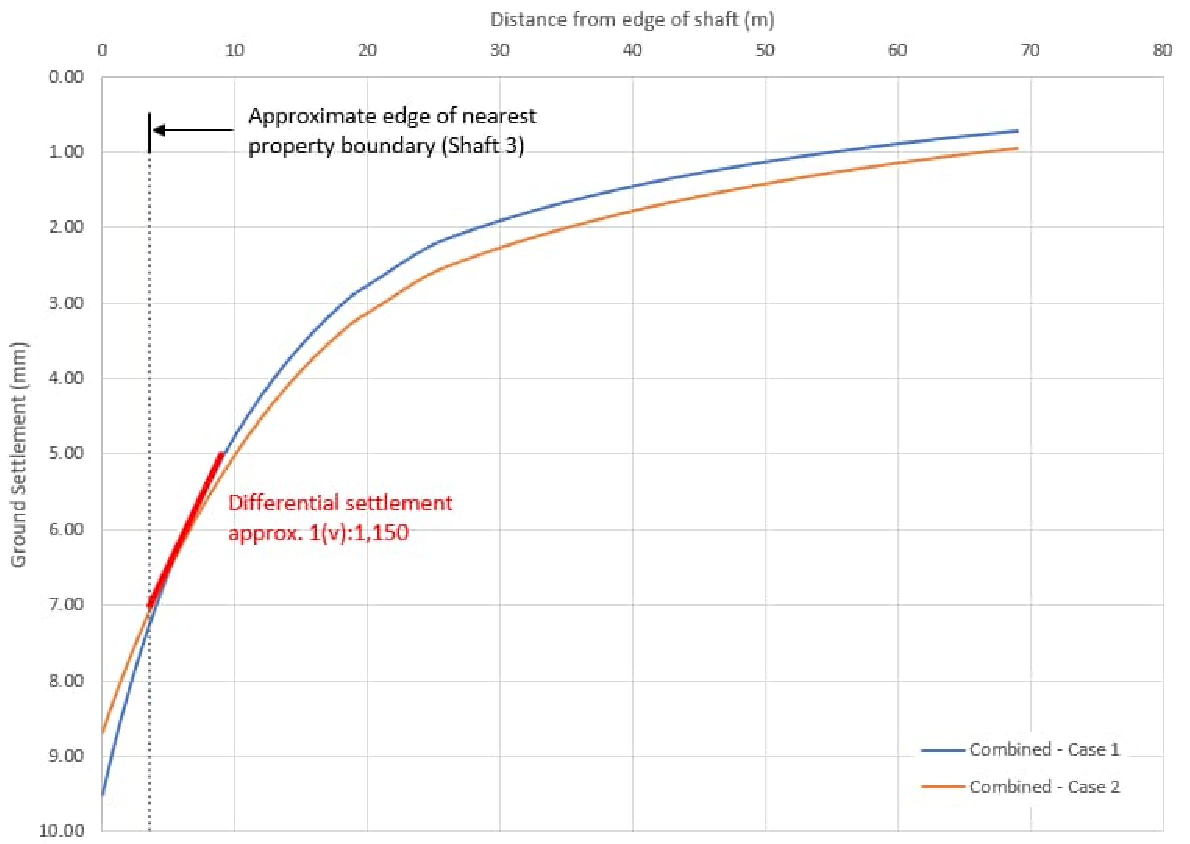

Figure 8.1: Estimated combined settlement profile due to shaft excavation

8.1.2 Tunnelling

The combined settlement profiles for the proposed tunnel construction have been plotted on Figure 8.2 below, adopting both analysis cases for tunnel dewatering outlined in Section 6.1 and mechanical settlements for the Shaft 4 scenario (Figure 6.1). This profile represents a conservative 'upper bound' envelope of effects, with respect to settlement beyond the road corridor, based on the information available at the time of writing.

The approximate location of the neighbouring property boundary has been plotted based on the current scheme layout as detailed on WSP drawings presented in Appendix A.

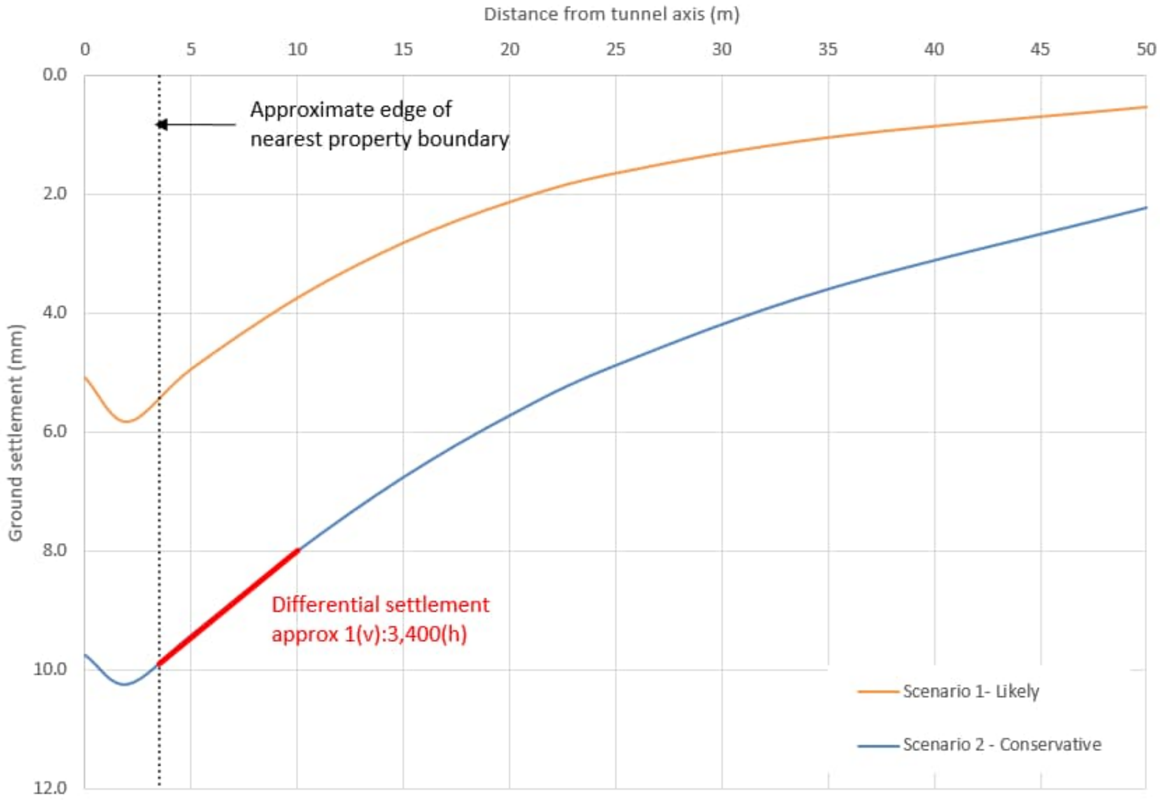

Figure 8.2: Estimated combined settlement profile due to tunnelling

8.1.3 Open trenching

The combined settlement profiles for the proposed tunnel construction have been plotted on Figure 8.3 below based on the analyses and design assumptions outlined in Section 5. This is profile represents a conservative 'upper bound' envelope of effects based on the information available at the time of writing.

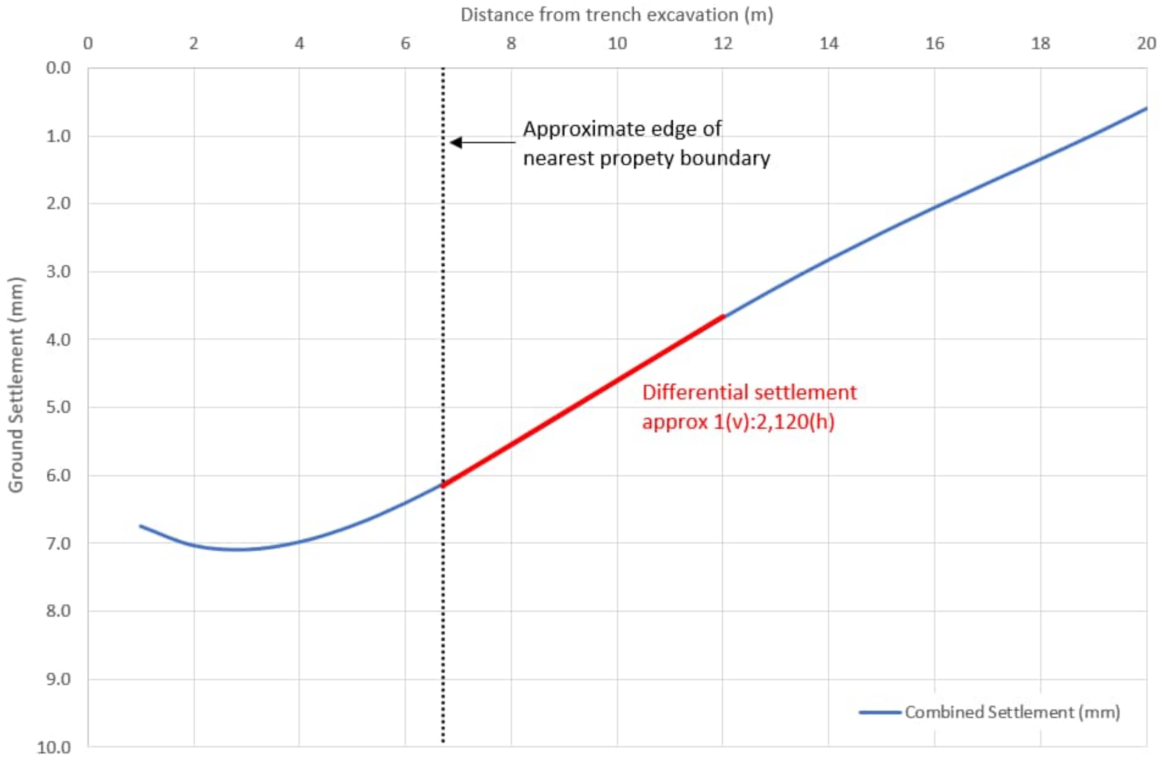

Figure 8.3: Estimated combined settlement profile due to open trenching

8.2 Assessment of effects on buildings and structures

In general, a building structure's tolerance to total and differential settlement depends upon the materials used in construction as well as the structures foundation system (e.g. shallow versus piled), the quality of the structure, and the existing condition of the structure.

The limiting values of total settlement and angular distortion along with damage classifications as presented in CIRIA PR30[16] have been used as guidance for assessment of potential effects on buildings (Table 8.1). In addition, the New Zealand Building Code – B1[17] states that designers should limit the probable maximum differential settlement of a building to 1V:240H (25 mm over a 6 m horizontal distance) under serviceability limit state loading, or total settlement to 50 mm unless the structure is specifically designed to resist damage under a greater settlement. While the NZ Building Code applies to these structures, the residential house dwellings are likely to have been designed using acceptable solutions outlined in NZS 3604[18] which is appropriate for ground with movements

[16] Lake, L.M., Rankin, W.J. and Hawley, J., 1996. Prediction and effects of ground movements caused by tunnelling in soft ground beneath urban areas. CIRIA PR30.
[17] Ministry of Business, Innovation & Employment (2004) Acceptable Solutions and Verification Methods – For New Zealand Building Code Clause B1 Structure.
[18] Standards New Zealand (2011). New Zealand Standard – Timber-framed buildings. NZS3604:2011

less than 25 mm. The ground deformation limits outlined in NZS 3604[18] are approximately equivalent to Risk Category 2 in CIRIA PR30.

Table 8.1: Settlement criteria for properties and buildings along the proposed project alignment

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

The values presented in the New Zealand Building Code are total amounts of ground movement over the life of a building. The buildings in the vicinity of the project are existing, with unknown histories; and therefore, may have already been subjected to some movement. To account for historical movement, ground movements of 10 mm vertical settlement and 1:500 distortion are considered suitable thresholds below which buildings are exposed to negligible risk of damage.

Table 8.2 below summarises the estimated 'upper bound' total and differential settlements within neighbouring properties. Note this has been conservatively assessed at the property boundary rather than actual building footprints. This indicates that no neighbouring properties are anticipated to experience ground settlement-related damage due to the proposed works.

Table 8.2: Summary of estimated upper bound settlement effects along proposed alignment

Construction Activity	Offset to property boundary (m)	Total Settlement (mm)	Differential Settlement	Risk Category
Open trenching	6.7	6.2	1 in 2,100	1
Tunnelling	3.5	10	1 in 3,400	1
Shaft construction	3.5	7	1 in 1,150	1

Some structures along the proposed alignment lie within the AUP historic heritage overlay. Some of these structures may be particularly sensitive to settlement and have been listed in Table 8.3 below, however it should also be appreciated that historical timber structures were often designed with greater tolerances than modern buildings.

Table 8.3: Historic heritage structures adjacent to alignment

Address	Details
85 Sarsfield Street	3-storey dwelling, timber and plaster clad, tile roof
58 Wallace Street	1-storey dwelling, timber clad (weatherboard), tile roof
72 Argyle Street	1-storey dwelling, timber clad (weatherboard)
31 Herne Bay Road	2-storey dwelling, timber clad (weatherboard)
29 Herne Bay Road	2-storey dwelling, timber clad (weatherboard)
27 Herne Bay Road	2-storey dwelling, timber clad (weatherboard), standalone garage (weatherboard), masonry fence
34 Herne Bay Road	1-storey dwelling, timber clad (weatherboard)

8.3 Assessment of effects on underground services and pavements

While many types of utilities can accommodate high levels of differential settlement, certain types can be susceptible to damage. In general, a utility's tolerance to settlement depends upon the construction type/material, existing condition, and whether the utility runs parallel or perpendicular to the excavation. Utilities running perpendicular to the excavation works are considered to be at the highest risk of damage. Utilities which run parallel and are near the excavation works may experience horizontal displacement associated with ground loss at the excavation face; however, they will experience a much gentler differential settlement.

The methodology to assess the effects on utilities is based on the method on O'Rourke and Trautman (1982)[19] – which provides guidance on allowable differential settlement for various utility construction types. A summary of the utility types is included in Table 8.4.

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

Table 8.4: Allowable differential settlement based on utility type

Utility type	Maximum allowable differential settlement (V:H)
Brick unlined	1:245
Welded steel pipe	1:122
Cast in-situ concrete	1:173
PVC & HDPE	1:67
Reinforced concrete pipe	1:229
Ductile iron pipe	1:229
Vitrified clay pipe	1:299
Cast iron pipe	1:150 – 1:500 (varies based on diameter)

Estimations of potential damage to a utility have been based off the calculated ground surface settlement profiles perpendicular to the excavation. This is likely to over-estimate the actual differential settlement experienced by the utility as:

- Where a utility crosses oblique to the alignment, the estimated differential settlement is expected to be lower than if the utility was crossing perpendicular to the alignment.
- It has been assumed that the differential settlement affecting each recorded utility is equal to the differential settlement at the ground surface. In reality the differential settlements at depth are likely to be less than at the ground surface, and consequently the settlement estimates are conservative.

Based on the settlement estimates presented earlier in this report, differential settlements are anticipated to be within the allowable tolerances of the services present within the carriageway. The estimated settlements are anticipated to have a negligible to less than minor impact on pavement surfaces and overland flow regimes.

The sensitivity analysis presented in Section 6.2.3 identified a potential risk to underground services should low-strength alluvial soils be encountered at points of shallow ground cover above the tunnel excavation. This estimated total settlements of up to 19 mm and differential settlements of up to 1(v):220(h) as a 'worst case'. These effects are expected to be highly localised, with effects reducing rapidly perpendicular to the tunnel corridor and as ground level rises along the alignment.

Detailed assessment of the effects on these services should be undertaken as part of any further assessment and reporting should these ground conditions be inferred across the tunnel alignment. This assessment will consider the depth, construction and orientation of these services which may affected.

8.4 High level conclusion of results

Our preliminary assessment indicates that the ground settlement and dewatering effects arising for the tunnelling and shaft excavation can be managed to within levels typically accepted by Auckland Council, provided that robust construction methods are adopted, and tunnel boring activities take into consideration locality of compressible materials where inverts are shallow. Based on our initial assessment we believe this can be achieved through:

- Having the contractor agree to achieving minimum seepage cut-off performance for shafts. We recommend that the contractor agree to maintaining a minimum secant pile seepage performance consistent with the effects modelled adopting a conductance value of $1x10^{-3}$ day-1 (no to minimal leakage) during construction of the shaft. Our assessment indicates if poor seepage cut-off (high leakage) is achieved this may affect neighbouring properties if not remediated immediately.

- Have a contingency and management plan on how to quickly address defects in the secant pile wall which may result in leakage. The contractor may consider a construction methodology that allows for sealing of defects progressively as excavations extend to depth rather than undertaking sealing of the defects upon completion of bulk excavation, in order to minimise groundwater effects and associated consolidation settlement.
- Open-mode tunnelling confined to within very weak ECBF rock, with careful consideration to the ground model and thickness of very weak ECBF rock present above the crown of the tunnel excavation. The contractor should carefully monitor groundwater inflows to the cutting face / spoil and revert to closed mode when high groundwater inflows are observed, to reduce potential for high groundwater drawdowns.
- A management plan that incorporates sufficient groundwater monitoring to assess groundwater drawdown impacts upon aquifers within the compressible materials where settlement may result in adverse effects.

9 Assessment of groundwater related effects

This section describes an assessment of potential groundwater related effects due to:

- Temporary dewatering of the shaft excavations.
- Temporary depressurisation of the hydrostratigraphic unit (i.e. ECBF) immediately outside the cutting face of the TBM.

9.1 Effects on regional groundwater availability

The tunnel alignment is through the Auckland Isthmus Waitematā aquifer i.e. primarily ECBF rock unit. Auckland Council confirmed that this aquifer has a groundwater availability of 1,302,001 m^3/year and that 966,664 m^3/year remained available for allocation in February 2023.

The dewatering rates presented in this report are several orders of magnitude less than the groundwater available for allocation, and significant volumes remain for allocation to other users.

9.2 Stream depletion effects

Streams naturally lose water to groundwater by outflow through the bed and banks when the water table elevation is lower than the water level in the stream and vice versa. Where the loss of water is increased by the effect of groundwater pumping the increase is referred to as stream depletion. Stream depletion is reported as the ratio of stream depletion caused by pumping to the total pumping volume.

The nearest stream mapped on the LINZ topographical map is located approximately 400 m south of the western end of the tunnel alignment (Marine Parade). There are no stream monitoring stations reported by Auckland Council[20] near any of the proposed excavations, further indicating that there are no major streams in proximity to the site. On this basis stream depletion effects are assessed as negligible.

9.3 Saltwater intrusion

Saltwater intrusion occurs when groundwater in an aquifer near the coast is replaced by seawater from the ocean. The Ghyben-Herzberg relation[21] predicts that the depth below sea level to the saline interface is approximately 40 times the height of the freshwater table above sea level. This height results from the assumption that the density of freshwater is 1,000 kg/m^3 and 1,025 kg/m^3 for seawater.

9.3.1 Saltwater intrusion along the tunnel alignment

Temporary dewatering of the tunnel alignment by the TBM will occur below mean sea level for short periods. The tunnel will be sealed as short sections of tunnelling are completed. We have considered the time that each tunnelled section is open, the static groundwater level above mean sea level along the alignment, the distance from the foreshore, and the temporary dewatering at low rates during tunnelling. Our assessment is that saltwater intrusion is unlikely to be observed during construction of the tunnel.

[20] https://environmentauckland.org.nz/Data/Map/Parameter/River%20Discharge/Statistic/LASTRECORD/Interval/Latest
[21] Fetter, C. W. (1994). Applied Hydrogeology. Third edition. Prentice Hall Inc., New Jersey, USA, p. 370.

9.3.2 Saltwater intrusion due to shaft excavations

Construction of the shafts are proposed between +21.96 m RL (2.42 m bgl) to -4 m RL (i.e. below mean sea level) and will require temporary dewatering (refer Table 2.1).

Saltwater intrusion is considered to be unlikely on the basis that dewatering at the shaft excavations is limited and temporary in nature. Furthermore, in the unlikely event that salt water intrusion occurs, our assessment is that effects will be inconsequential considering the low groundwater use in the aquifer. This assessment is supported by studies[22] on the impact of saline intrusion on coastal groundwater around New Zealand between 2001 and 2011. The studies have identified that most of the evidence for saline intrusion has been limited to shallow unconfined aquifers.

9.4 Effects on neighbouring groundwater users

Considering the location of the tunnel alignment across Herne Bay, it is unlikely that there are other groundwater takes for consumptive use along the alignment. On that basis and taking into account the hydraulic conductivity of the ECBF, our assessment is that predicted temporary drawdowns along the TBM alignment and at the proposed shaft and control chamber will not have a consequential impact on other groundwater users (if any).

[22] Pattle Delamore Partners Ltd, June 2011. New Zealand Guidelines for the monitoring and management of sea water intrusion risk on groundwater. Report ref. C02085500.

10 Slope instability and coastal regression

Auckland Council Geomaps presents an overlay of the predicted coastal regression lines due to potential erosion and slope instability, accounting for various sea-level rise predictions up to year 2130. The assessment[23] accounts for potential future toe erosion and cliff instability and presents the extents of 'Area susceptible to coast instability and/or erosion' (ASCIE).

The extents of the predicted ASCIE are shown in Figure 10.1 below. This indicates the proposed tunnel alignment is generally well outside the predicted area of potential instability for a period of greater than 100 years. The regression lines are drawn at the ground surface. The risk to the tunnel alignment is considered to be greatly reduced due to its depth below the ground surface, with the majority of the tunnel proposed a minimum of 10 m below ground level.

It is therefore considered no further quantitative assessment is required at this stage.

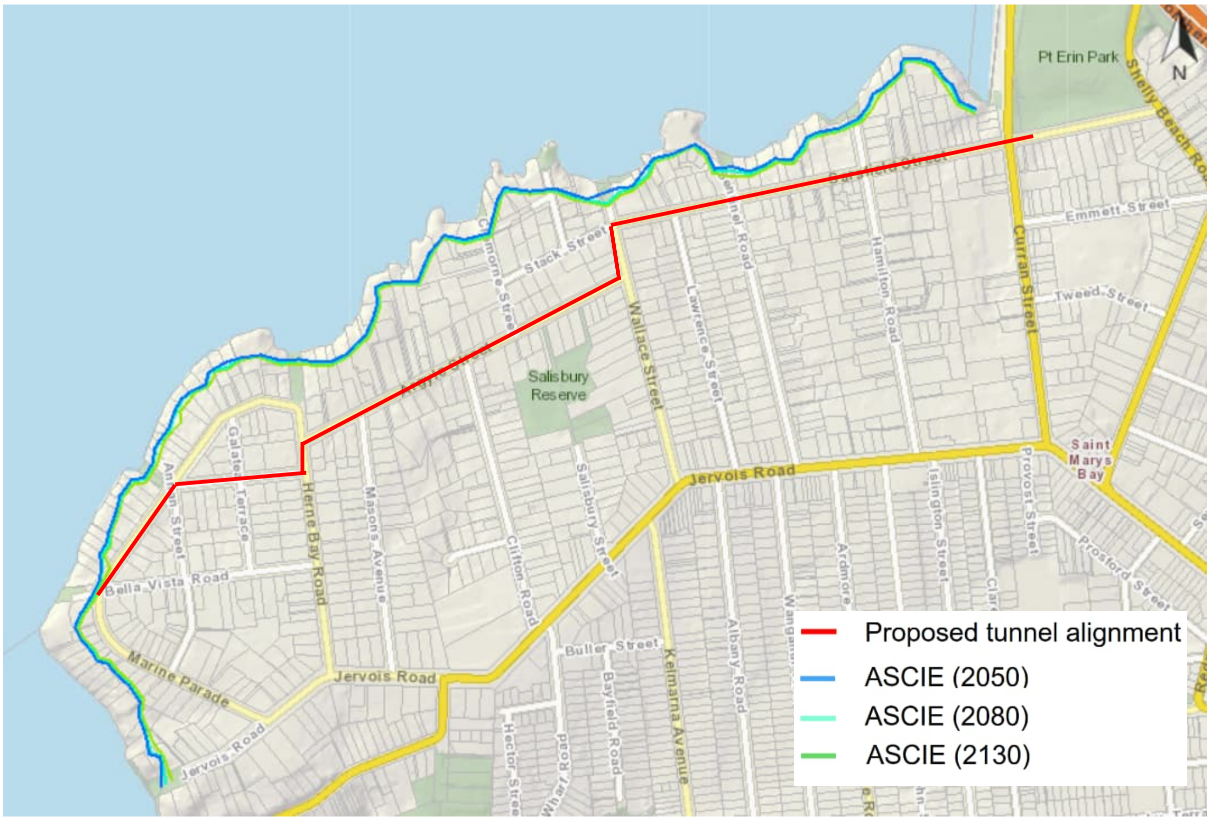

Figure 10.1: Predicted ASCIE profiles within Herne Bay area

[23] https://www.knowledgeauckland.org.nz/media/2432/tr2020-021-predicting-aucklands-exposure-to-coastal-instability-and-erosion.pdf

11 Resource consent considerations

11.1 Auckland Unitary Plan

A preliminary assessment of the geotechnical aspects of the project, with respect to the AUP requirements for "permitted activities", has been undertaken. Our interpretation of the geotechnical aspects of the development in relation to these conditions is attached in Appendix D.

Based on our preliminary assessment, the proposed works do not comply with Section E.7.6.10 of the AUP *"Diversion of groundwater caused by any excavation, (including trench) or tunnel".* It is therefore considered the development will require a groundwater take and divert consent as a restricted discretionary activity.

11.2 Construction monitoring and contingency

A draft Ground Settlement Monitoring and Contingency Plan (GSMCP) will be required to support the groundwater take and divert consent application. It is envisaged the monitoring regime for the development would comprise the following:

- Groundwater monitoring – in proximity to the shaft excavations to monitor groundwater drawdown behind the secant pile wall and near boundaries of neighbouring properties which may experience groundwater drawdown effects.
- Ground and building settlement monitoring – in proximity to the shaft excavations, along the tunnel alignment and at/near boundaries of neighbouring properties which may experience ground settlement effects.
- Building dilapidation survey – pre-construction and post-construction building condition inspections should be undertaken by an independent Chartered Professional Engineer (with approval from building owner). At a minimum, the following structures should be inspected:
  - Structures where estimated ground settlement due to the proposed works exceeds 10 mm total settlement and/or 1(v):1,000(h) differential settlement.
- The project manager may consider also undertaking condition survey of properties located within 1x the shaft depth of the proposed shaft locations (i.e. for a 25 m deep shaft monitoring of properties within 25 m plan distance the excavation shall be undertaken).

12 Further work and reporting

As noted earlier in this report, this assessment has been undertaken on a conservative basis to estimate the upper bound of potential effects that may result from project works. This report assumes pipe alignment, shaft locations and construction methodology as outlined in this report and Appendix A. This assessment relies upon the construction method being secant pile circular shafts. Should the alignment, shaft locations or construction method be altered then this assessment will need to be reviewed and if required, modified.

To refine the analysis done to date, further work is currently being undertaken and will be provided to Council as soon as it is completed (likely to be August / September 2023). The further work and reporting will:

1. Refine the hydrogeological conceptual model with site investigation data for the current ground investigation programme. This includes:
   - Groundwater level monitoring (VWPs and standpipe piezometers).
   - On site hydraulic permeability testing and modulus of compressibility testing.
2. Review and update the ground model with new information and update analyses as required based on project specific boreholes, CPTs and geophysical survey.
3. Confirm the proposed construction methods for the shaft excavations issued as part of the preliminary design. If construction methods other than secant pile circular shafts are proposed, undertake an assessment to determine settlements arising from mechanical deformation.
4. If the results from the further investigation works identify any substantial changes from the initial assumptions in this report, then undertake further detailed assessment to identify any areas of potential adverse settlement effects and undertake an assessment of the building types, infrastructure and likely tolerance to settlement based on published guidance. Specialised inputs from a civil and/or structural engineer may be required depending on the building type and proximity to the proposed works.

13 Applicability

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

We understand and agree that our client will submit this report as part of an application for resource consent and that Auckland Council as the consenting authority will use this report for the purpose of assessing that application.

Recommendations and opinions in this report are based on available published geological and borehole information undertaken by T+T and others at point locations. The nature and continuity of subsoil away from these locations is inferred but it must be appreciated that actual conditions may vary from the assumed model. This report has been provided in advance of site-specific investigations and must be updated after these investigations and subsequent analyses are carried out.

Tonkin & Taylor Ltd
Environmental and Engineering Consultants

Report prepared by:

....

Natalie Tan
Geotechnical Engineer

Report prepared by:

pp ....

Richard Bond
Engineering Geologist

Technical Review

....

Mark Thomas (CPEng)

Principal Geotechnical Engineer

Authorised for Tonkin & Taylor Ltd by:

....

Shannon Richardson

Project Director

Report has been reviewed by Ric Wilkinson, Geotechnical Engineer

29-Jun-23
\\ttgroup.local\files\aklprojects\1090120\issueddocuments\20230629 herne bay connector report final\herne bay connector - groundwater and settlement assessment report.docx

Appendix A Project Figures and Plans provided by WSP

PLAN

SCALE 1:2000 @ A1

NOTES:

1. REFER TO SHEET W-SL007.002 FOR PROPOSED TRUNK SEWER LONG SECTION
2. COORDINATES ARE IN TERMS OF NEW ZEALAND TRANSVERSE MERCATOR 2000 CIRCUIT.
3. LEVELS ARE IN TERMS OF METRES AUCKLAND 1946 LOCAL VERTICAL DATUM.
4. DIMENSIONS / DISTANCE ARE IN METRES UNLESS STATED OTHERWISE.
5. IT IS THE CONTRACTORS RESPONSIBILITY TO LOCATE ALL EXISTING SERVICES PRIOR TO CONSTRUCTION.
6. DESIGN AT CONCEPT STAGE AND SUBJECT TO CHANGE THROUGH DESIGN PROCESS.
7. NO SURVEY OF THE EXISTING UTILITIES AND FEATURES HAVE BEEN CARRIED OUT.
8. TUNNEL ALIGNMENT SHOWN WITH 2.5m WIDE CORRIDOR EITHER SIDE OF PROPOSED ALIGNMENT.

1:2000@ A1
1:4000@ A3
0 20 40 60 80 100 120 140 160 180 200 m

KEY:

- PARCEL BOUNDARY
- EXISTING WATER SUPPLY
- EXISTING LOCAL SEWER
- EXISTING TRANSMISSION SEWER
- EXISTING STORMWATER
- EXISTING GROUND PROFILE
- EXISTING EOP
- PROPOSED LOCAL SEWER
- PROPOSED TUNNEL SEWER
- PROPOSED SEWER MANHOLE
- PROPOSED SHAFT
- PROPOSED HERITAGE BUILDING
- PROPOSED NOTABLE TREE
- TUNNEL 2.5m WIDE CORRIDOR
- PROPOSED CONSTRUCTION AREA

ISSUE	DATE	AMENDMENT	BY	APPD.
1	16.02.23	DRAFT ISSUED FOR CONSENT APPLICATION	GI	MK

	BY	DATE
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DRAWN	G.IP	02-23
DWG. APPROVED	M.KUDOIC	02-23
WSL DESIGN MGMT.	B.DEVILLIERS	–
WSL PROJ. LEAD	–	–

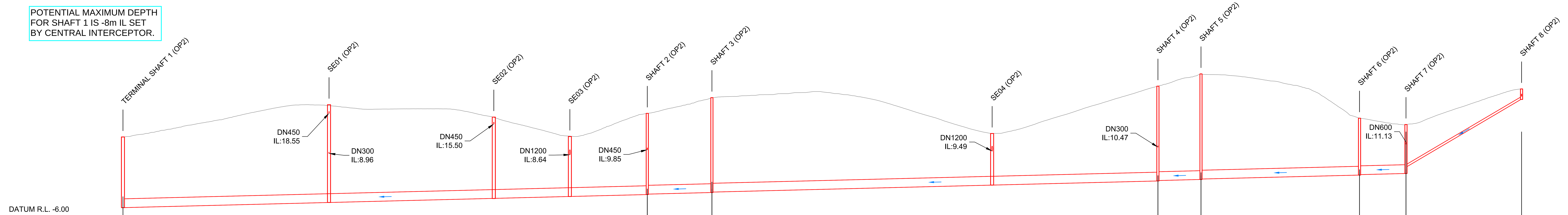
Watercare

HERNE BAY TRUNK SEWER UPGRADE
MARINE PARADE TO PT ERIN
PROJECT OVERVIEW – PLAN

DRAFT

CAD FILE W-SL007.001
DATE 16-02-2023
ORIGINAL SCALE A1 1:2000
CONTRACT No. 1
REF. No. W-SL007.01
DWG. No. W-SL007 001
ISSUE 1

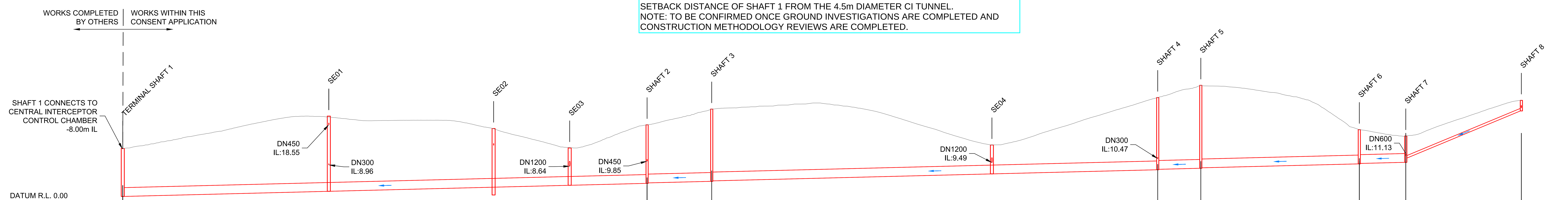
POTENTIAL MAXIMUM DEPTH
FOR SHAFT 1 IS -8m IL SET
BY CENTRAL INTERCEPTOR.



	Terminal Shaft 1	SE01	SE02	SE03	Shaft 2	Shaft 3	SE04	Shaft 4	Shaft 5	Shaft 6	Shaft 7	Shaft 8
GROUND LEVEL (m)	12.89	20.62	17.65	13.11	18.53 / 18.53	22.30 / 22.30	13.75	25.04 / 25.04	27.97 / 27.97	17.38 / 17.38	15.87 / 15.87	24.38
DEPTH TO INVERT (m)	16.89	23.39	19.44	9.03	19.41 / 19.30	22.68 / 22.58	12.36	22.66 / 22.56	25.24 / 25.13	13.60 / 13.49	11.70 / 10.20	2.42
PIPE INVERT LEVEL (m)	-4.00	-2.77	-1.79	4.08	-0.87 / -0.77	-0.38 / -0.28	1.39	2.38 / 2.48	2.74 / 2.84	3.79 / 3.89	4.17 / 5.67	21.96
DISTANCE (m)	0.0	245.9	442.4	533.3	625.6 / 625.6	702.7 / 702.7	1037.0	1234.8 / 1234.8	1286.2 / 1286.2	1475.3 / 1475.3	1530.8 / 1530.8	1668.7
GRADIENT	0.50% (1:200)				0.50% (1:200)	0.50% (1:200)		0.50% (1:200)	0.50% (1:200)	0.50% (1:200)	11.82% (1:8)	
PIPE SIZE AND MATERIAL	DN2100				DN2100	DN2100		DN2100	DN2100	DN2100	DN600	

HERNE BAY SEWER TRUNK MAIN LONGITUDINAL SECTION
OPTION 1

OPTION 1 IS ASSUMED TO BE THE PROBABLE DEPTH OF HERNE BAY TRUNK SEWER BASED ON SET LEVELS FROM CENTRAL INTERCEPTOR TUNNELS AND VERTICAL SETBACK DISTANCE OF SHAFT 1 FROM THE 4.5m DIAMETER CI TUNNEL.
NOTE: TO BE CONFIRMED ONCE GROUND INVESTIGATIONS ARE COMPLETED AND CONSTRUCTION METHODOLOGY REVIEWS ARE COMPLETED.

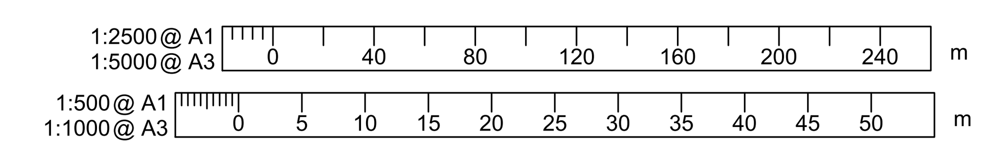


	Terminal Shaft 1	SE01	SE02	SE03	Shaft 2	Shaft 3	SE04	Shaft 4	Shaft 5	Shaft 6	Shaft 7	Shaft 8
GROUND LEVEL (m)	12.89	20.62	17.65	13.11	18.53 / 18.53	22.30 / 22.30	13.75	25.04 / 25.04	27.97 / 27.97	17.38 / 17.38	15.87 / 15.87	24.38
DEPTH TO INVERT (m)	11.46	17.96	14.01	9.03	13.97 / 13.87	17.25 / 17.15	6.93	17.23 / 17.13	19.81 / 19.71	8.17 / 8.07	6.28 / 5.38	2.42
PIPE INVERT LEVEL (m)	1.43	2.66	3.64	4.10	4.56 / 4.66	5.05 / 5.15	6.82	7.81 / 7.91	8.17 / 8.27	9.21 / 9.31	9.59 / 10.49	21.96
DISTANCE (m)	0.0	245.9	442.4	533.3	625.6 / 625.6	702.7 / 702.7	1037.0	1234.8 / 1234.8	1286.2 / 1286.2	1475.3 / 1475.3	1530.8 / 1530.8	1668.7
GRADIENT	0.50% (1:200)				0.50% (1:200)	0.50% (1:200)		0.50% (1:200)	0.50% (1:200)	0.50% (1:200)	8.32% (1:12)	
PIPE SIZE AND MATERIAL	DN2100				DN2100	DN2100		DN2100	DN2100	DN2100	DN600	

HERNE BAY SEWER TRUNK MAIN LONGITUDINAL SECTION
OPTION 2

SCALE 1:2500H 1:500V @ A1

OPTION 2 IS ASSUMED TO BE THE MINIMUM DEPTH OF HERNE BAY TRUNK SEWER BASED ON HYDRAULIC REQUIREMENTS TO CAPTURE EOP FLOWS.
NOTE: TO BE CONFIRMED ONCE GROUND INVESTIGATIONS AND SURVEY ARE COMPLETED AND AND CONSTRUCTION METHODOLOGY REVIEWS ARE COMPLETED.



NOTES:

1. REFER TO SHEET W-SL007.002 FOR PROPOSED TRUNK SEWER LONG SECTION
2. COORDINATES ARE IN TERMS OF NEW ZEALAND TRANSVERSE MERCATOR 2000 CIRCUIT.
3. LEVELS ARE IN TERMS OF METRES AUCKLAND 1946 LOCAL VERTICAL DATUM.
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8. TUNNEL ALIGNMENT SHOWN WITH 2.5m WIDE CORRIDOR EITHER SIDE OF PROPOSED ALIGNMENT

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WSL DESIGN MGMT.	B.DEVILLIERS	–
WSL PROJ. LEAD	–	–

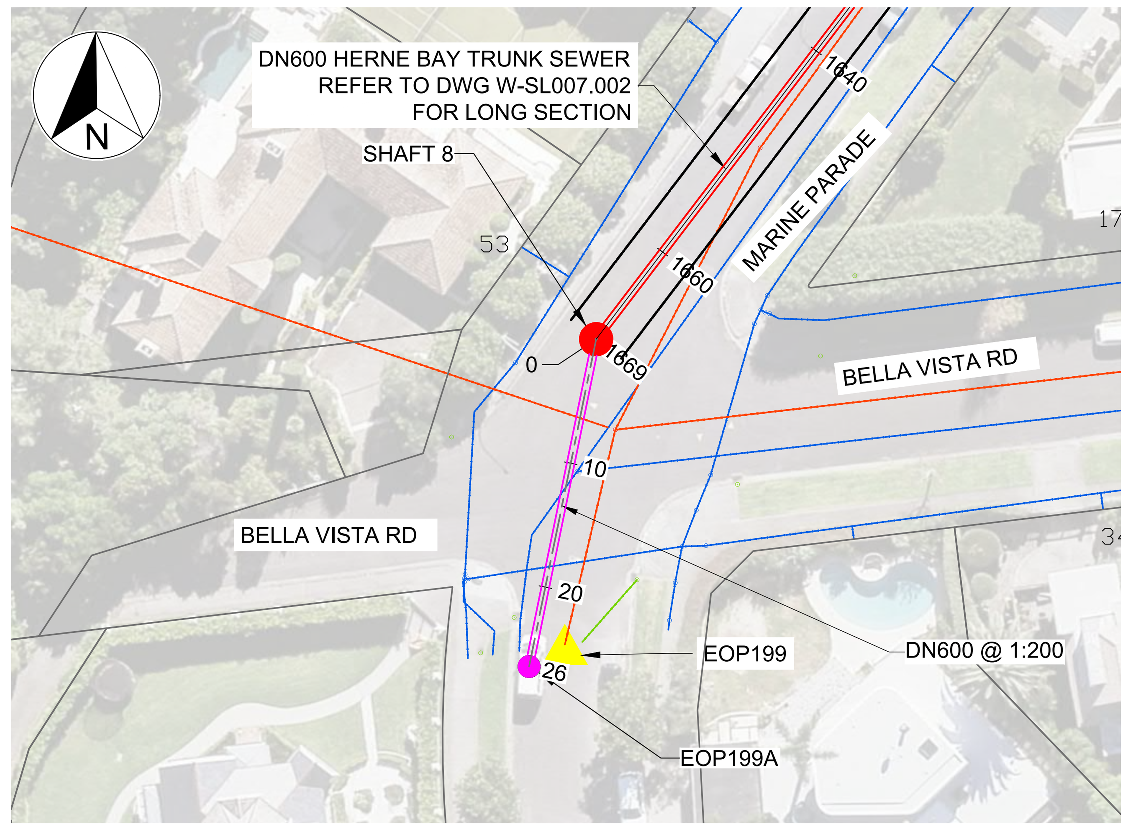
Watercare

HERNE BAY TRUNK SEWER UPGRADE
MARINE PARADE TO PT ERIN
LONGITUDINAL SECTION – TRUNK SEWER

DRAFT

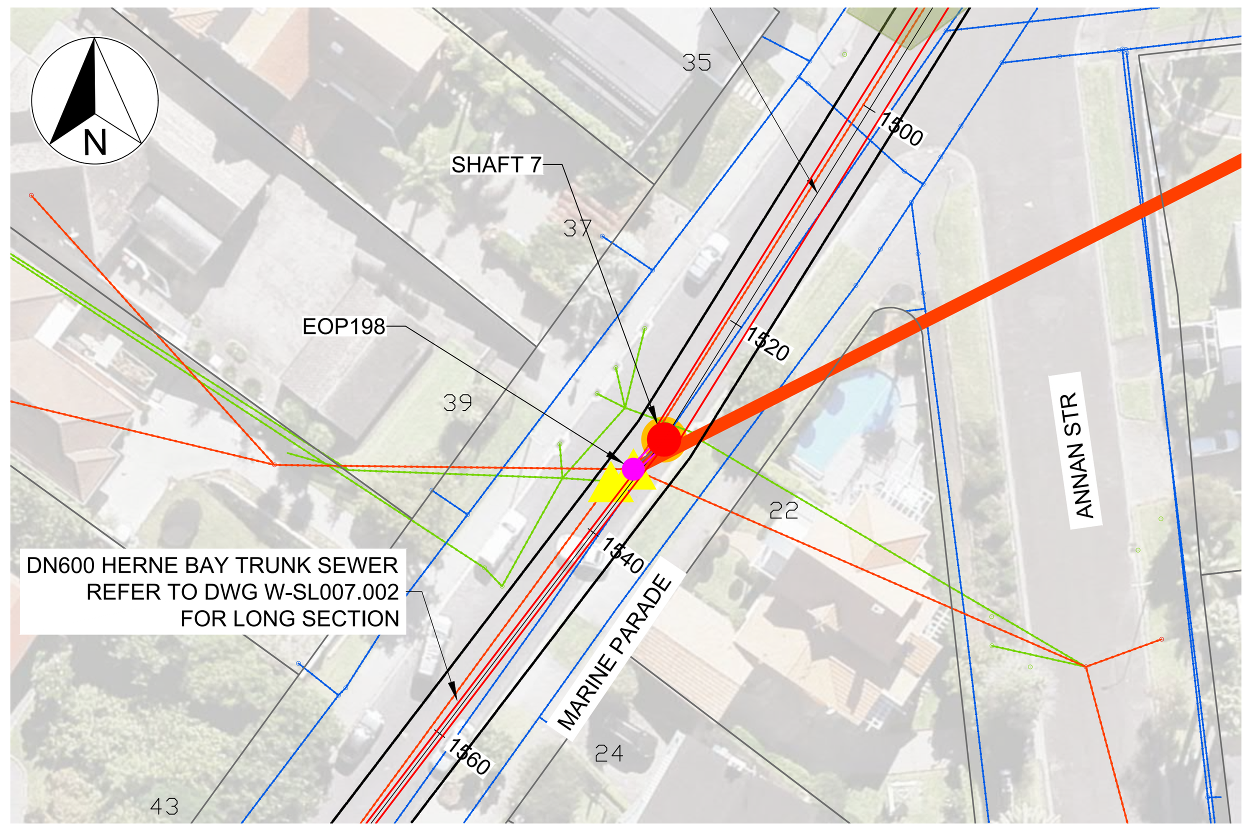
CAD FILE W-SL007.002
DATE 16-02-023
ORIGINAL SCALE A1 1:2000
CONTRACT No. 1
REF. No. W-SL007.01
DWG. No. W-SL007 .002
ISSUE 1

PIPE SIZE OF EOP CONNECTIONS ARE TBC.
EOP LONG SECTIONS ARE IN RELATION TO OPTION 2 (MINIMUM DEPTH)



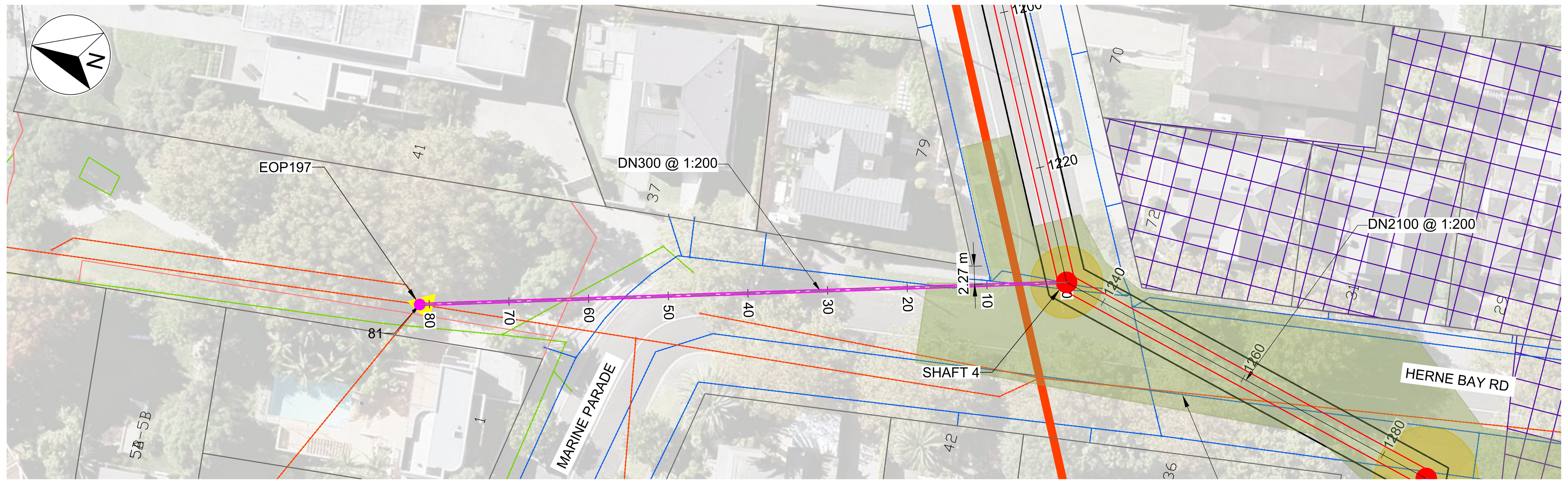
PLAN - EOP 199 - SHAFT 8

SCALE 1:500 @ A1



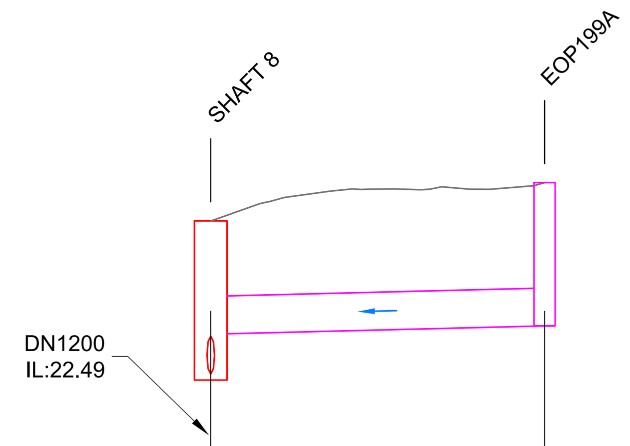
PLAN - EOP 198 - SHAFT 7

SCALE 1:500 @ A1



PLAN - EOP 197 - SHAFT 4

SCALE 1:500 @ A1



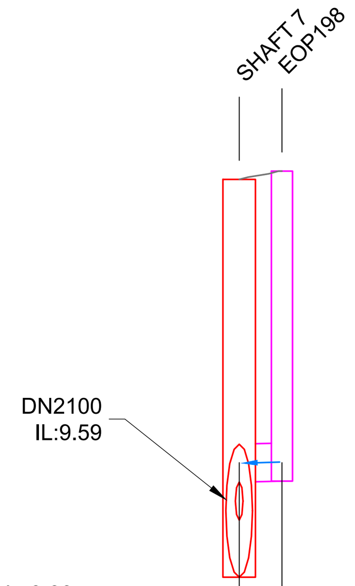
DATUM R.L. 20.00

GROUND LEVEL (m)	24.38	24.99
DEPTH TO INVERT (m)	1.79	2.27
PIPE INVERT LEVEL (m)	22.59	22.72
DISTANCE (m)	0.0	26.4
GRADIENT	0.50% (1:200)	
PIPE SIZE AND MATERIAL	DN600	

LONGITUDINAL SECTION
EOP 199 - SHAFT 8

SCALE 1:500H 1:100V @ A1

LONGSECTIONS SHOWN HERE ARE FOR DESIGN OPTION 1 IN W-SL007.002

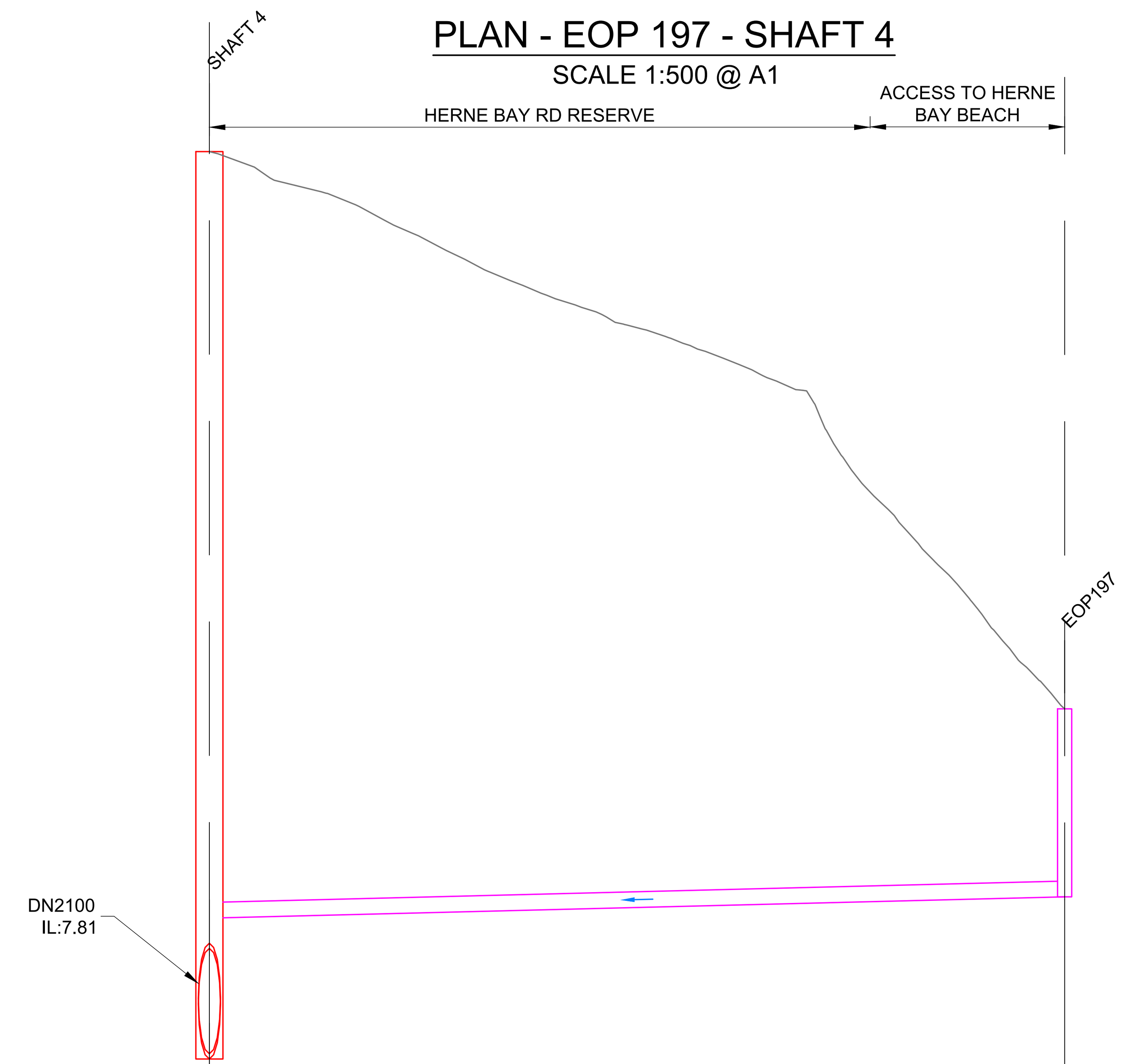


DATUM R.L. 9.00

GROUND LEVEL (m)	15.87	16.00
DEPTH TO INVERT (m)	4.78	4.90
PIPE INVERT LEVEL (m)	11.09	11.11
DISTANCE (m)	0.0	3.4
GRADIENT	0.50% (1:200)	
PIPE SIZE AND MATERIAL	DN600	

LONGITUDINAL SECTION
EOP 198 - SHAFT 7

SCALE 1:500H 1:100V @ A1



DATUM R.L. 6.00

GROUND LEVEL (m)	25.04	14.46
DEPTH TO INVERT (m)	14.56	3.57
PIPE INVERT LEVEL (m)	10.48	10.89
DISTANCE (m)	0.0	81.2
GRADIENT	0.50% (1:200)	
PIPE SIZE AND MATERIAL	DN300	

LONGITUDINAL SECTION
EOP 197 - SHAFT 4

SCALE 1:500H 1:100V @ A1

1:100@ A1 / 1:200@ A3: 0 1 2 3 4 5 6 7 8 9 10 m
1:500@ A1 / 1:1000@ A3: 0 5 10 15 20 25 30 35 40 45 50 m

KEY:

- PARCEL BOUNDARY
- EXISTING WATER SUPPLY — W
- EXISTING LOCAL SEWER — WSL
- EXISTING TRANSMISSION SEWER
- EXISTING STORMWATER — SW
- EXISTING GROUND PROFILE
- EXISTING EOP
- PROPOSED LOCAL SEWER
- PROPOSED TUNNEL SEWER
- PROPOSED SEWER MANHOLE
- PROPOSED SHAFT
- PROPOSED HERITAGE BUILDING
- PROPOSED NOTABLE TREE
- TUNNEL 2.5m WIDE CORRIDOR
- PROPOSED CONSTRUCTION AREA

NOTES:

1. REFER TO SHEET W-SL007.002 FOR PROPOSED TRUNK SEWER LONG SECTION
2. COORDINATES ARE IN TERMS OF NEW ZEALAND TRANSVERSE MERCATOR 2000 CIRCUIT.
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	BY	DATE
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DWG. APPROVED	M.KUDOIC	02-23
WSL DESIGN MGMT.	B.DEVILLIERS	–
WSL PROJ. LEAD	–	–

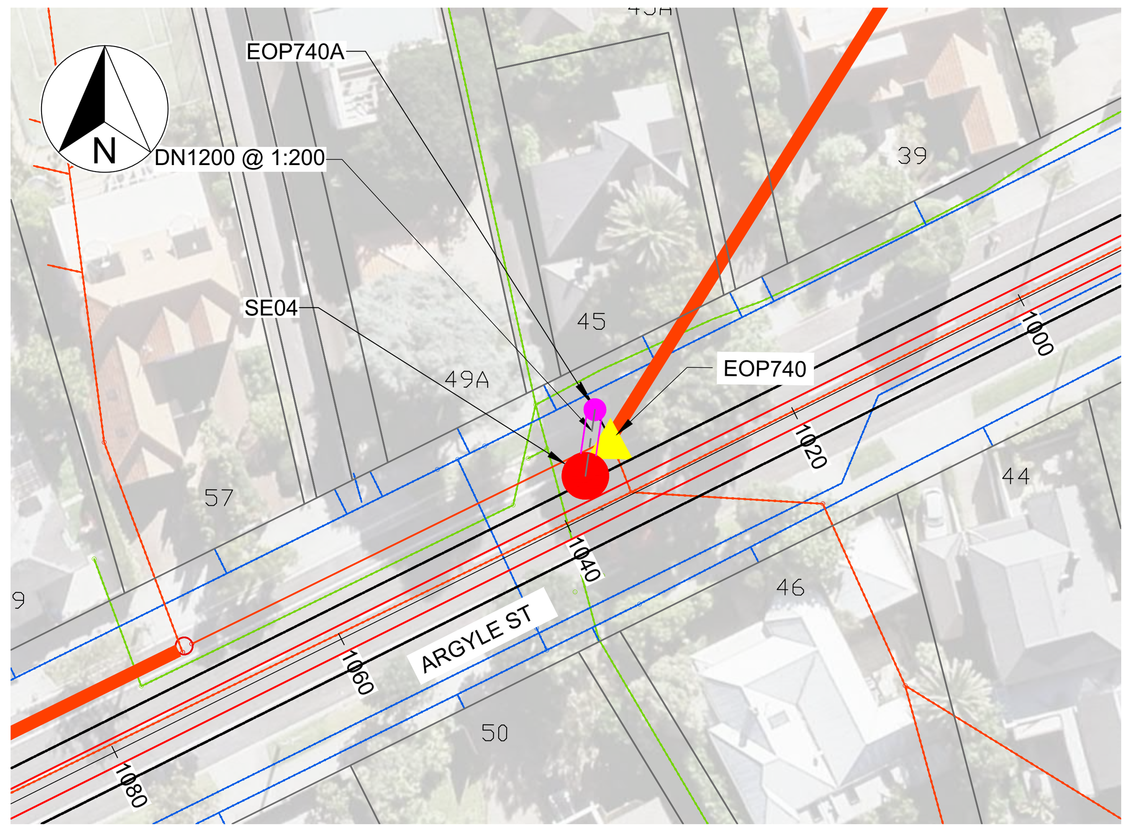
Watercare

HERNE BAY TRUNK SEWER UPGRADE
MARINE PARADE TO PT ERIN
LONGITUDINAL SECTIONS – LOCAL NETWORK SHEET 1

DRAFT

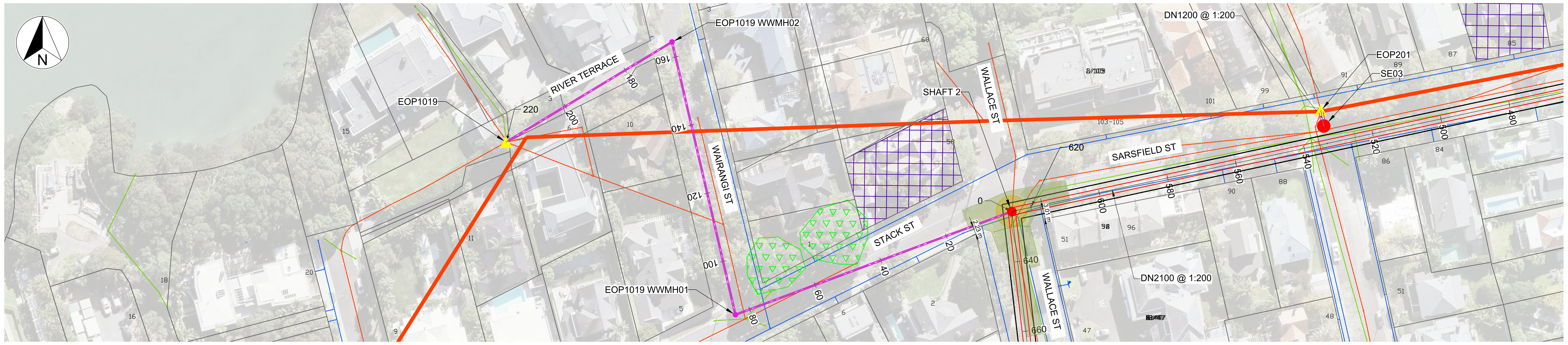
CAD FILE W-SL007.003	DATE 16-02-2023
ORIGINAL SCALE A1 1:500	CONTRACT No. 1
REF. No. W-SL007.01	
DWG. No. W-SL007 .003	ISSUE 1

PIPE SIZE OF EOP CONNECTIONS ARE TBC.
EOP LONG SECTIONS ARE IN RELATION TO OPTION 2 (MINIMUM DEPTH)



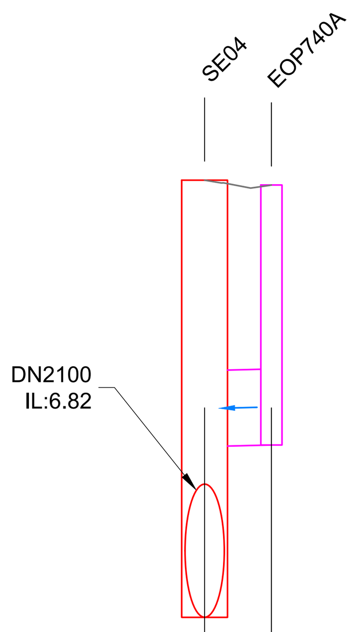
PLAN - EOP 740 - SE04

SCALE 1:500 @ A1



PLAN - EOP 1019 - SHAFT 2

SCALE 1:750 @ A1



DATUM R.L. 6.00

GROUND LEVEL (m)	13.72	13.65
DEPTH TO INVERT (m)	4.21	4.11
PIPE INVERT LEVEL (m)	9.51	9.54
DISTANCE (m)	0.0	5.3
GRADIENT	0.50% (1:200)	
PIPE SIZE AND MATERIAL	DN1200	

LONGITUDINAL SECTION
EOP 740 - SE04

SCALE 1:500 1:100V @ A1

DATUM R.L. 4.00

GROUND LEVEL (m)	18.53		17.54 17.54		15.74 15.74		13.03
DEPTH TO INVERT (m)	8.69		7.27 7.22		5.02 4.97		1.98
PIPE INVERT LEVEL (m)	9.85		10.27 10.32		10.72 10.77		11.05
DISTANCE (m)	0.0		84.4 84.4		164.4 164.4		219.8
GRADIENT		0.50% (1:200)		0.50% (1:200)		0.50% (1:200)	
PIPE SIZE AND MATERIAL		DN450		DN450		DN450	

LONGSECTIONS SHOWN HERE ARE
FOR DESIGN OPTION 1 IN W-SL007.002

LONGITUDINAL SECTION
EOP 1019 - SHAFT 2

SCALE 1:500H 1:100V @ A1

1:500 @ A1 / 1:1000 @ A3: 0 5 10 15 20 25 30 35 40 45 50 m
1:100 @ A1 / 1:200 @ A3: 0 1 2 3 4 5 6 7 8 9 10 m
1:750 @ A1 / 1:1500 @ A3: 0 10 20 30 40 50 60 70 m
1:150 @ A1 / 1:300 @ A3: 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 m

NOTES:

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KEY:

- PARCEL BOUNDARY
- EXISTING WATER SUPPLY (W)
- EXISTING LOCAL SEWER (WSL)
- EXISTING TRANSMISSION SEWER
- EXISTING STORMWATER (SW)
- EXISTING GROUND PROFILE
- EXISTING EOP
- PROPOSED LOCAL SEWER
- PROPOSED TUNNEL SEWER
- PROPOSED SEWER MANHOLE
- PROPOSED SHAFT
- PROPOSED HERITAGE BUILDING
- PROPOSED NOTABLE TREE
- TUNNEL 2.5m WIDE CORRIDOR
- PROPOSED CONSTRUCTION AREA

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					WSL PROJ. LEAD	-	-

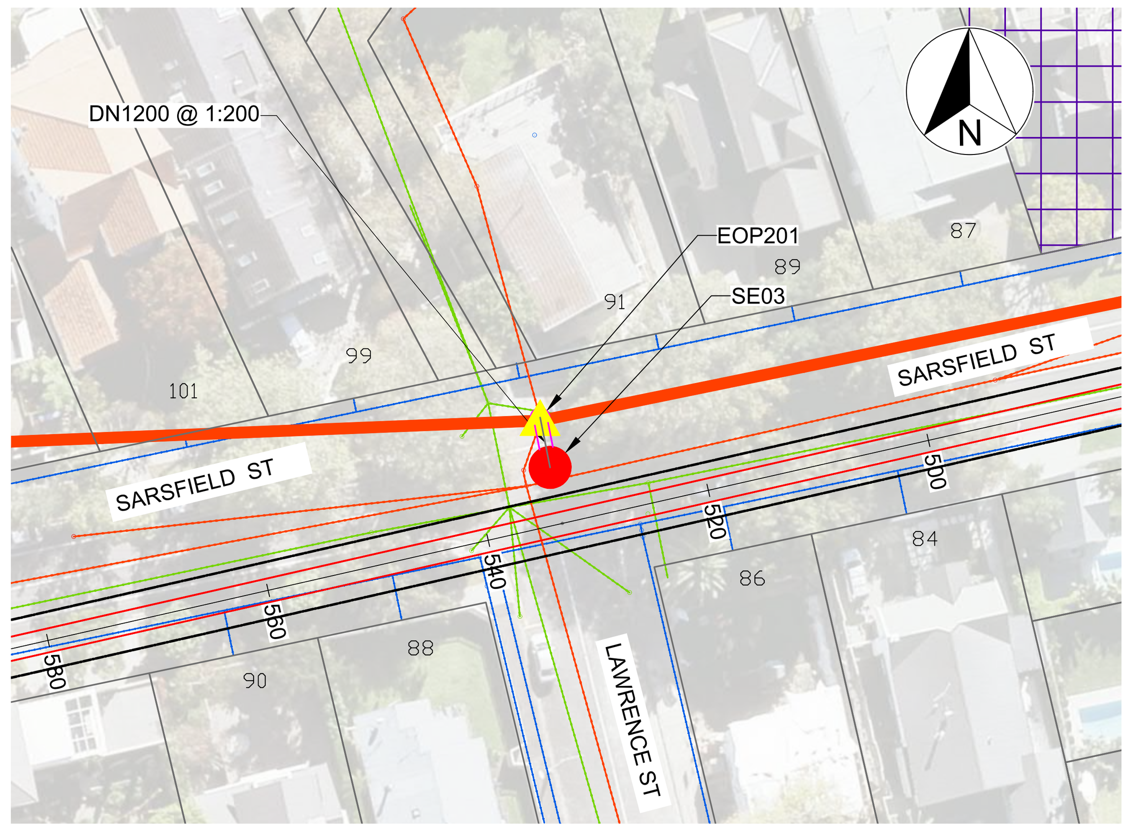
Watercare

HERNE BAY TRUNK SEWER UPGRADE
MARINE PARADE TO PT ERIN
LONGITUDINAL SECTIONS – LOCAL NETWORK SHEET 2

DRAFT

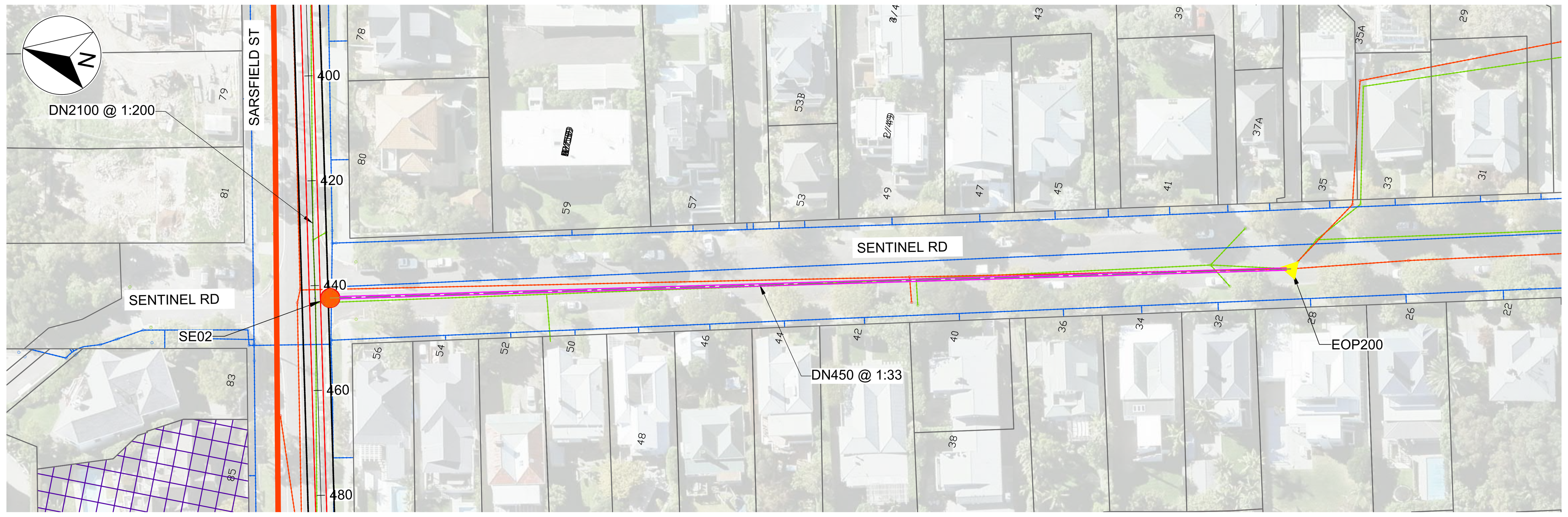
CAD FILE W-SL007.004 | DATE 16-02-2023
ORIGINAL SCALE A1 1:2000 | CONTRACT No. 1
REF. No. W-SL007.01
DWG. No. W-SL007 .004 | ISSUE 1

PIPE SIZE OF EOP CONNECTIONS ARE TBC.
EOP LONG SECTIONS ARE IN RELATION TO OPTION 2 (MINIMUM DEPTH)



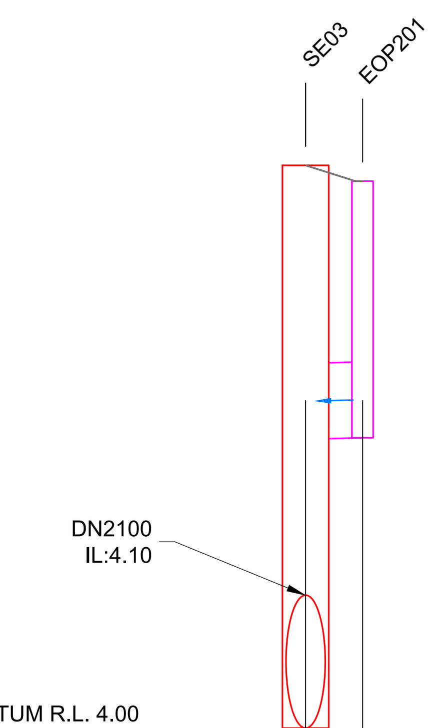
PLAN - EOP 201 - SE03

SCALE 1:500 @ A1



PLAN - EOP 200 - SE02

SCALE 1:750 @ A1

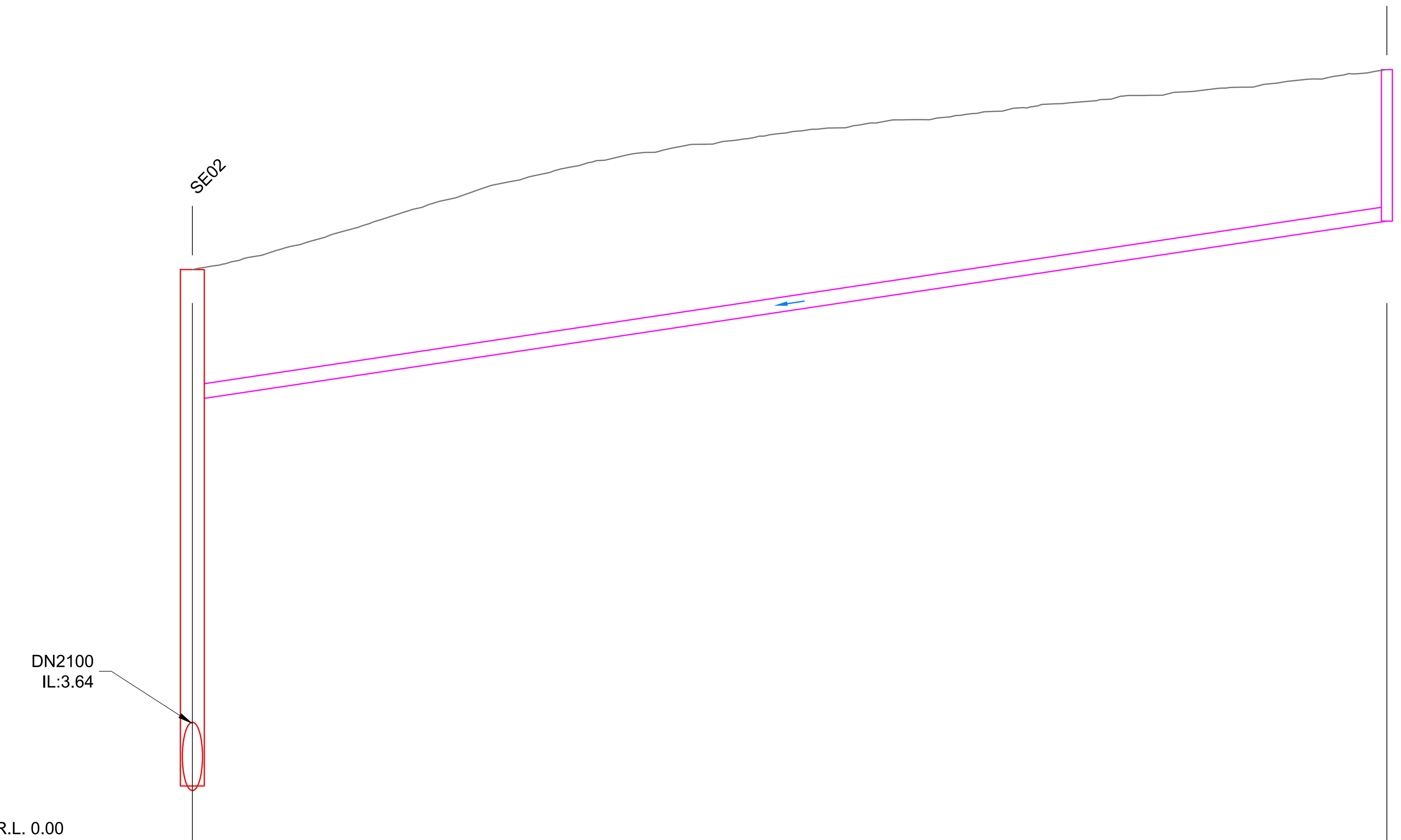


DATUM R.L. 4.00

GROUND LEVEL (m)	13.00	12.75
DEPTH TO INVERT (m)	4.33	4.06
PIPE INVERT LEVEL (m)	8.67	8.69
DISTANCE (m)	0.0	4.5
GRADIENT	0.50% (1:200)	
PIPE SIZE AND MATERIAL	DN1200	

LONGITUDINAL SECTION
EOP201 - SE03

SCALE 1:500H 1:100V @ A1



DATUM R.L. 0.00

GROUND LEVEL (m)	17.65	23.80
DEPTH TO INVERT (m)	4.01	4.66
PIPE INVERT LEVEL (m)	13.64	19.14
DISTANCE (m)	0.0	183.3
GRADIENT	3.00% (1:33)	
PIPE SIZE AND MATERIAL	DN450	

LONGITUDINAL SECTION
EOP200 - SE02

SCALE 1:750H 1:150V @ A1

LONGSECTIONS SHOWN HERE ARE FOR DESIGN OPTION 1 IN W-SL007.002

NOTES:

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1:500@ A1 1:1000@ A3 0 5 10 15 20 25 30 35 40 45 50 m
1:100@ A1 1:200@ A3 0 1 2 3 4 5 6 7 8 9 10 m
1:750@ A1 1:1500@ A3 0 10 20 30 40 50 60 70 m
1:150@ A1 1:300@ A3 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 m

KEY:

PARCEL BOUNDARY
EXISTING WATER SUPPLY
EXISTING LOCAL SEWER
EXISTING TRANSMISSION SEWER
EXISTING STORMWATER
EXISTING GROUND PROFILE
EXISTING EOP
PROPOSED LOCAL SEWER
PROPOSED TUNNEL SEWER
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PROPOSED NOTABLE TREE
TUNNEL 2.5m WIDE CORRIDOR
PROPOSED CONSTRUCTION AREA

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	BY	DATE
DESIGNED	G.IP	02-23
DES. APPROVED	C.STOKES	02-23
DRAWN	G.IP	02-23
DWG. APPROVED	M.KUDOIC	02-23
WSL DESIGN MGMT.	B.DEVILLIERS	–
WSL PROJ. LEAD	–	–

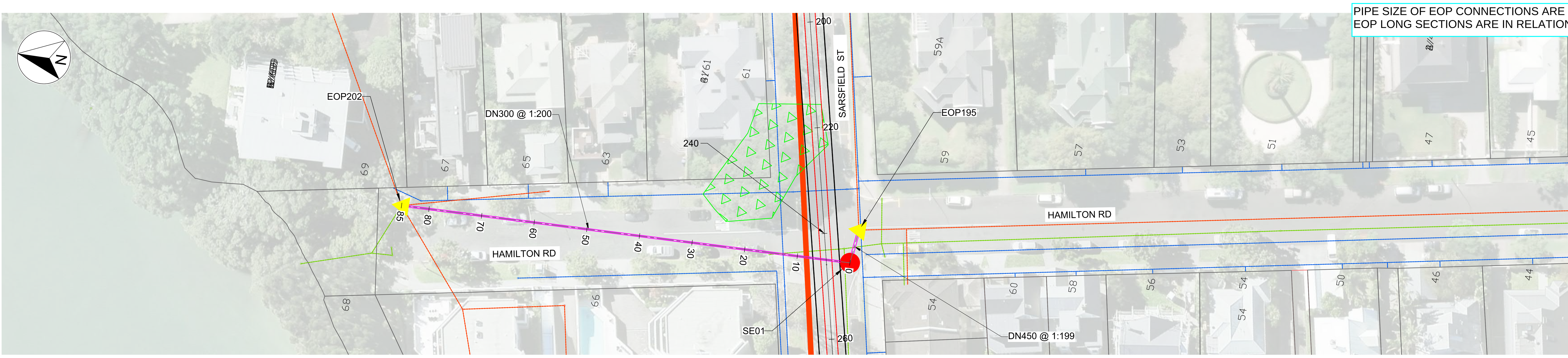
Watercare

HERNE BAY TRUNK SEWER UPGRADE
MARINE PARADE TO PT ERIN
LONGITUDINAL SECTIONS – LOCAL NETWORK SHEET 3

DRAFT

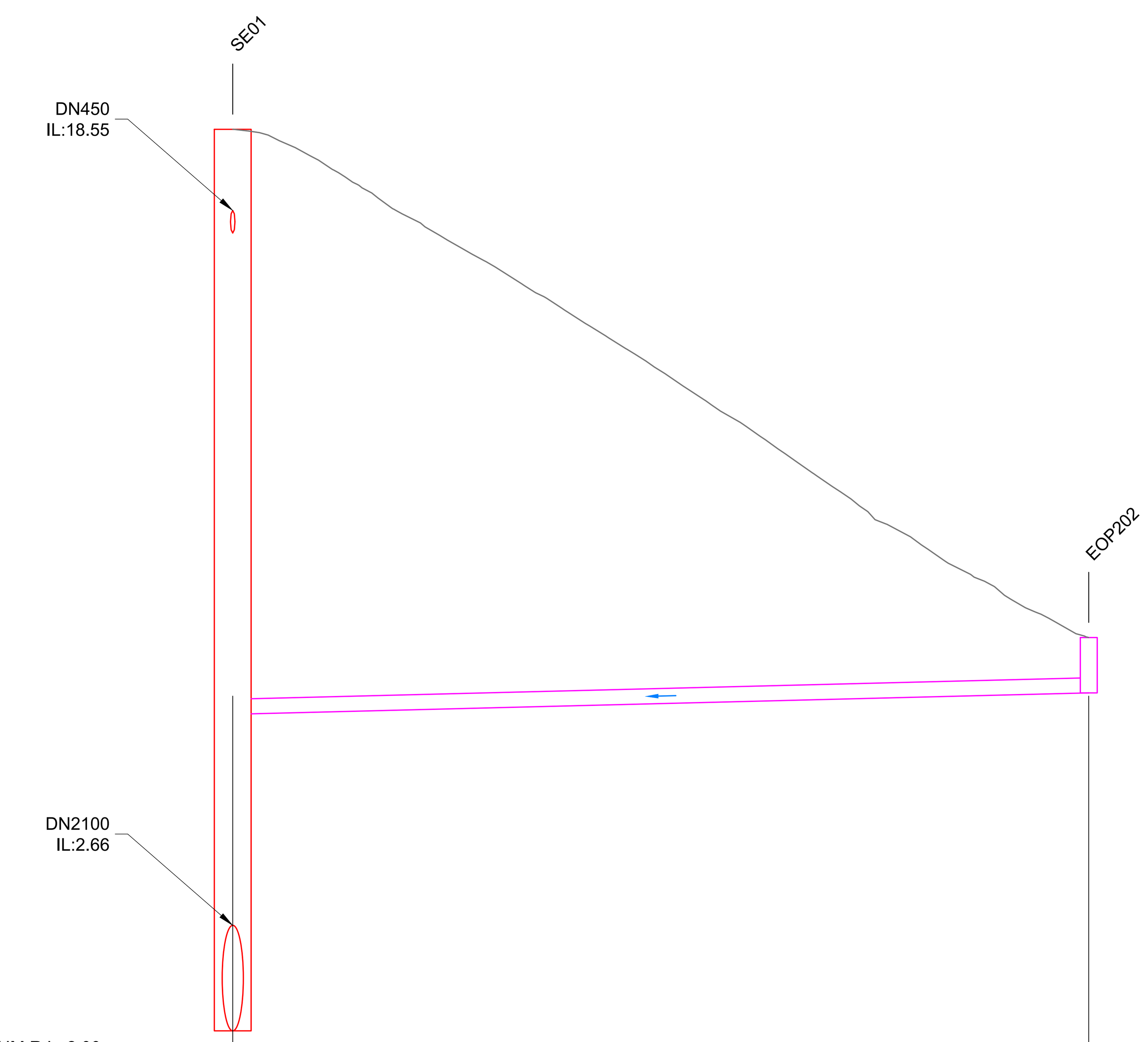
CAD FILE W-SL007.005
DATE 16-02-2023
ORIGINAL SCALE A1 1:2000
CONTRACT No. 1
REF. No. W-SL007.01
DWG. No. W-SL007 .005
ISSUE 1

PIPE SIZE OF EOP CONNECTIONS ARE TBC.
EOP LONG SECTIONS ARE IN RELATION TO OPTION 2 (MINIMUM DEPTH)



PLAN - EOP202 AND EOP 195 - SE01

SCALE 1:500 @ A1

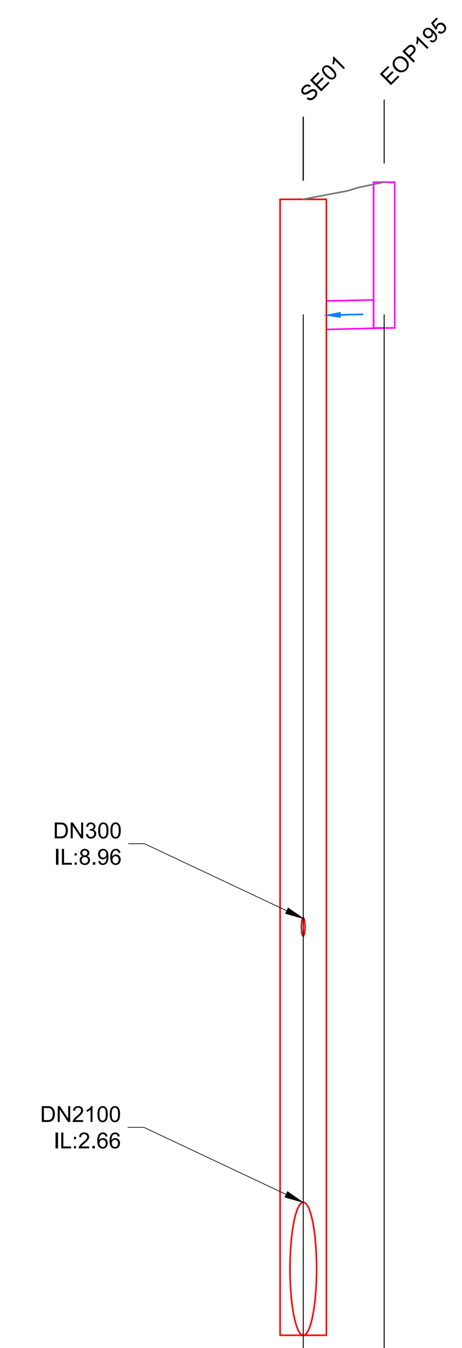


DATUM R.L. 2.00

GROUND LEVEL (m)	20.62	10.49
DEPTH TO INVERT (m)	11.65	1.10
PIPE INVERT LEVEL (m)	8.96	9.39
DISTANCE (m)	0.0	85.2
GRADIENT	0.50% (1:200)	
PIPE SIZE AND MATERIAL	DN300	

LONGITUDINAL SECTION
EOP202 - SE01

SCALE 1:500H 1:100V @ A1



DATUM R.L. 2.00

GROUND LEVEL (m)	20.62	20.89
DEPTH TO INVERT (m)	2.07	2.31
PIPE INVERT LEVEL (m)	18.55	18.58
DISTANCE (m)	0.0	6.4
GRADIENT	0.50% (1:199)	
PIPE SIZE AND MATERIAL	DN450	

LONGITUDINAL SECTION
EOP 195 - SE01

SCALE 1:500H 1:100V @ A1

1:2500@ A1
1:5000@ A3 0 40 80 120 160 200 240 m

1:500@ A1
1:1000@ A3 0 5 10 15 20 25 30 35 40 45 50 m

LONGSECTIONS SHOWN HERE ARE FOR DESIGN OPTION 1 IN W-SL007.002

NOTES:

1. REFER TO SHEET W-SL007.002 FOR PROPOSED TRUNK SEWER LONG SECTION
2. COORDINATES ARE IN TERMS OF NEW ZEALAND TRANSVERSE MERCATOR 2000 CIRCUIT.
3. LEVELS ARE IN TERMS OF METRES AUCKLAND 1946 LOCAL VERTICAL DATUM.
4. DIMENSIONS / DISTANCE ARE IN METRES UNLESS STATED OTHERWISE.
5. IT IS THE CONTRACTORS RESPONSIBILITY TO LOCATE ALL EXISTING SERVICES PRIOR TO CONSTRUCTION.
6. DESIGN AT CONCEPT STAGE AND SUBJECT TO CHANGE THROUGH DESIGN PROCESS.
7. NO SURVEY OF THE EXISTING UTILITIES AND FEATURES HAVE BEEN CARRIED OUT.
8. TUNNEL ALIGNMENT SHOWN WITH 2.5m WIDE CORRIDOR EITHER SIDE OF PROPOSED ALIGNMENT.

KEY:

- PARCEL BOUNDARY
- EXISTING WATER SUPPLY
- EXISTING LOCAL SEWER
- EXISTING TRANSMISSION SEWER
- EXISTING STORMWATER
- EXISTING GROUND PROFILE
- EXISTING EOP
- PROPOSED LOCAL SEWER
- PROPOSED TUNNEL SEWER
- PROPOSED SEWER MANHOLE
- PROPOSED SHAFT
- PROPOSED HERITAGE BUILDING
- PROPOSED NOTABLE TREE
- TUNNEL 2.5m WIDE CORRIDOR
- PROPOSED CONSTRUCTION AREA

ISSUE	DATE	AMENDMENT	BY	APPD.
1	16.02.23	DRAFT ISSUED FOR CONSENT APPLICATION	GI	MK

	BY	DATE
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DWG. APPROVED	M.KUDOIC	02-23
WSL DESIGN MGMT.	B.DEVILLIERS	-
WSL PROJ. LEAD	-	-

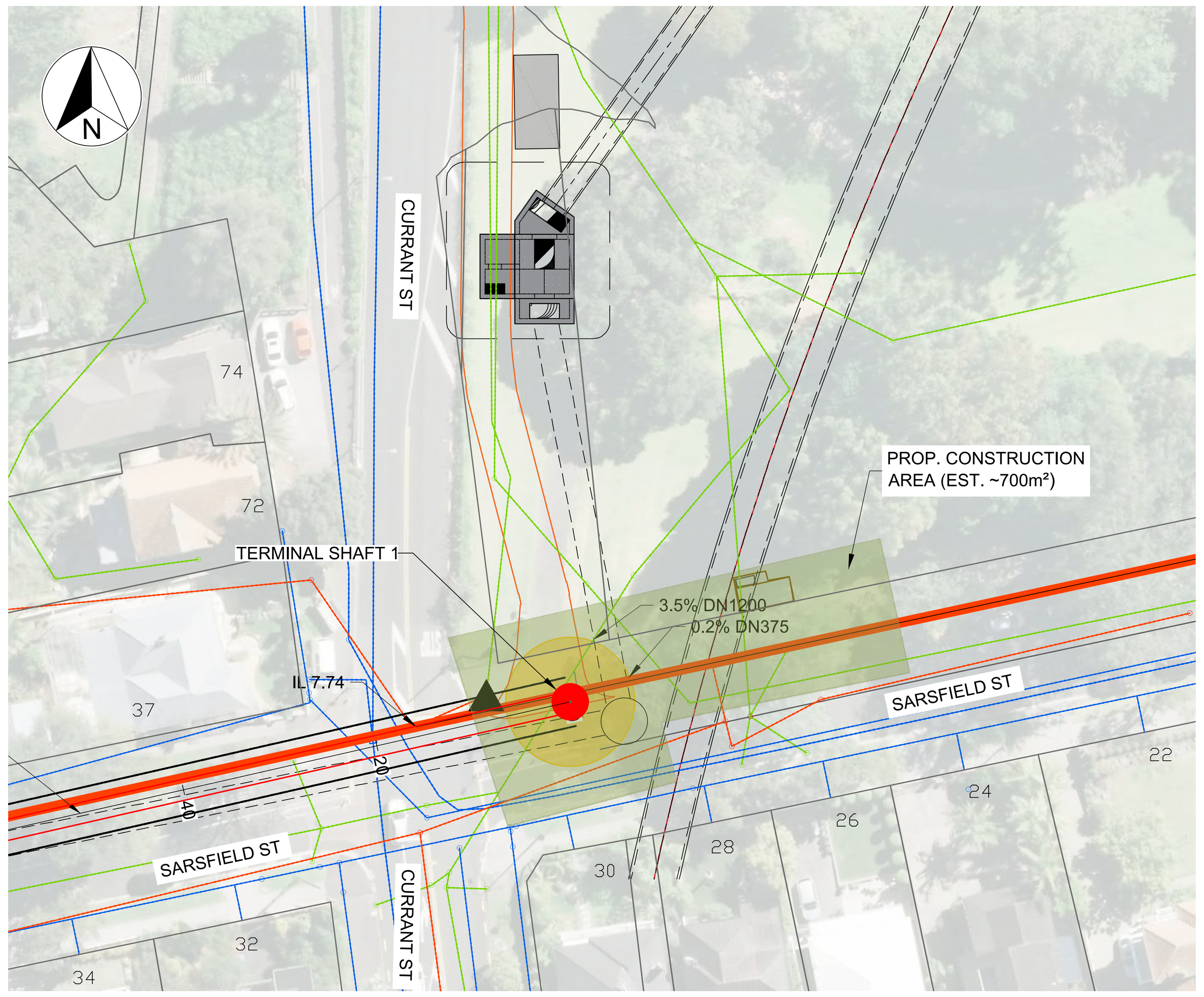
Watercare

HERNE BAY TRUNK SEWER UPGRADE
HAMILTON ROAD, HERNE BAY

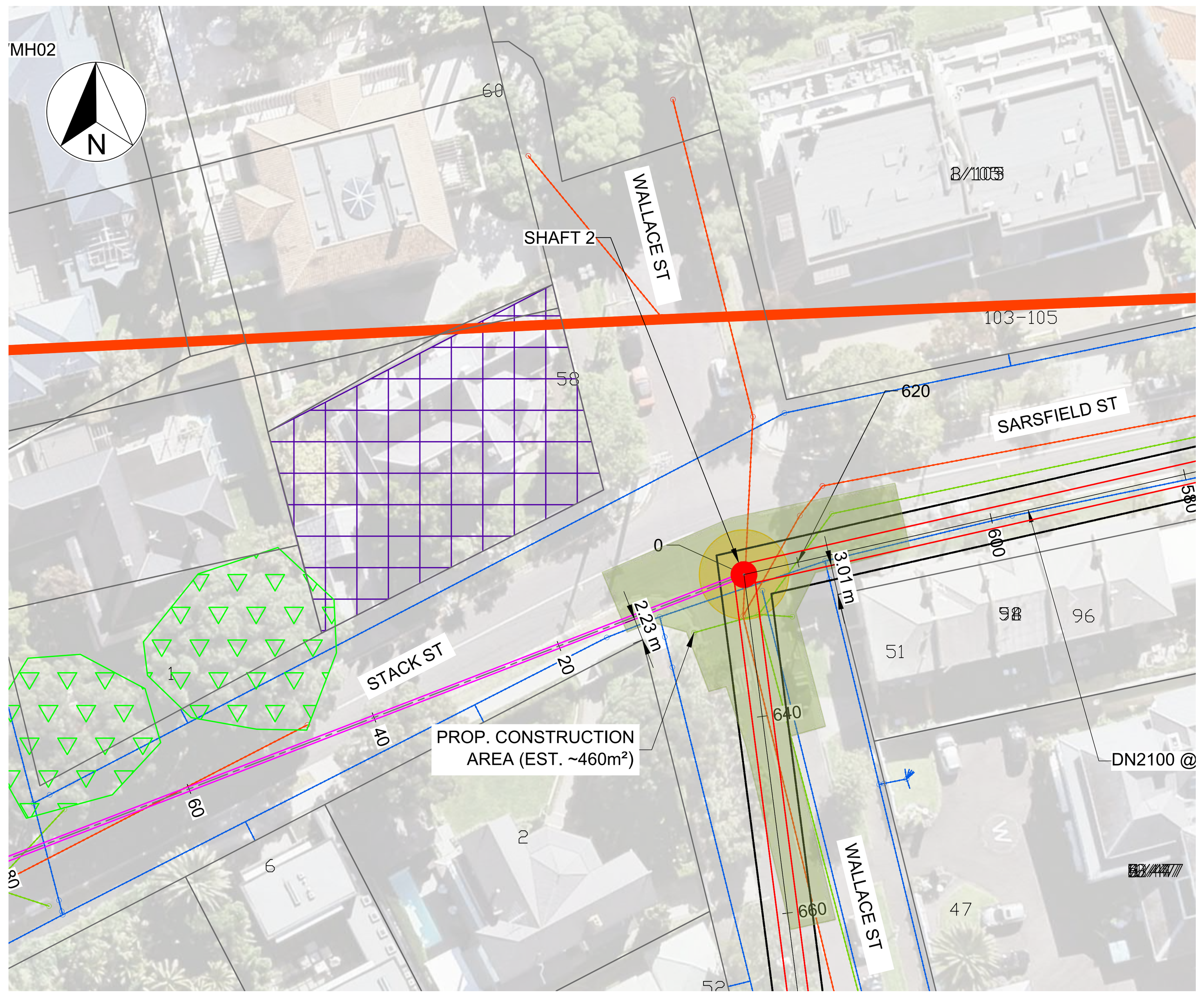
LONGITUDINAL SECTIONS – LOCAL NETWORK SHEET 4

DRAFT

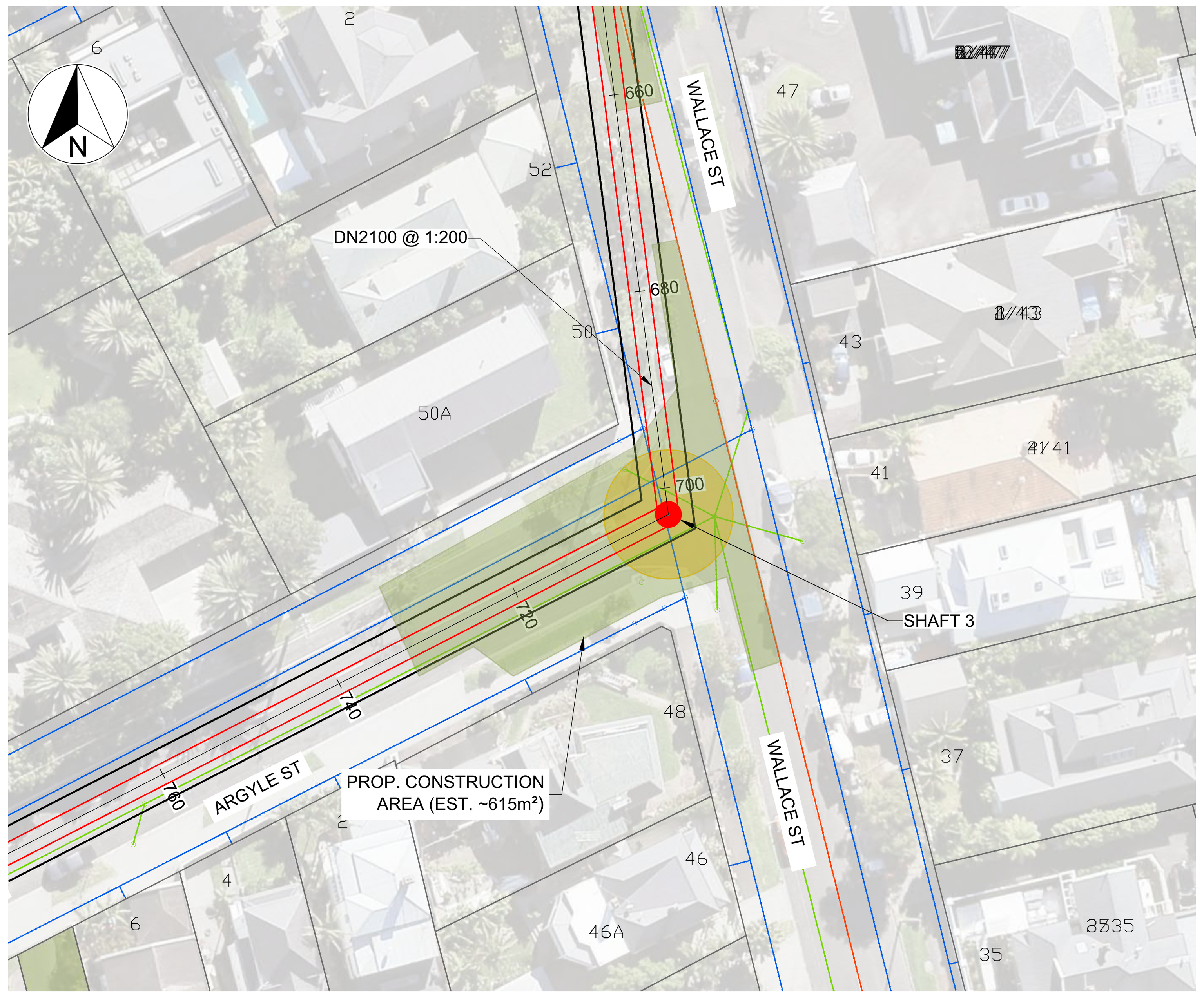
CAD FILE W-SL007.006
DATE 16-02-2023
ORIGINAL SCALE A1 1:2000
CONTRACT No. 1
REF. No. W-SL007.01
DWG. No. W-SL007 .006
ISSUE 1



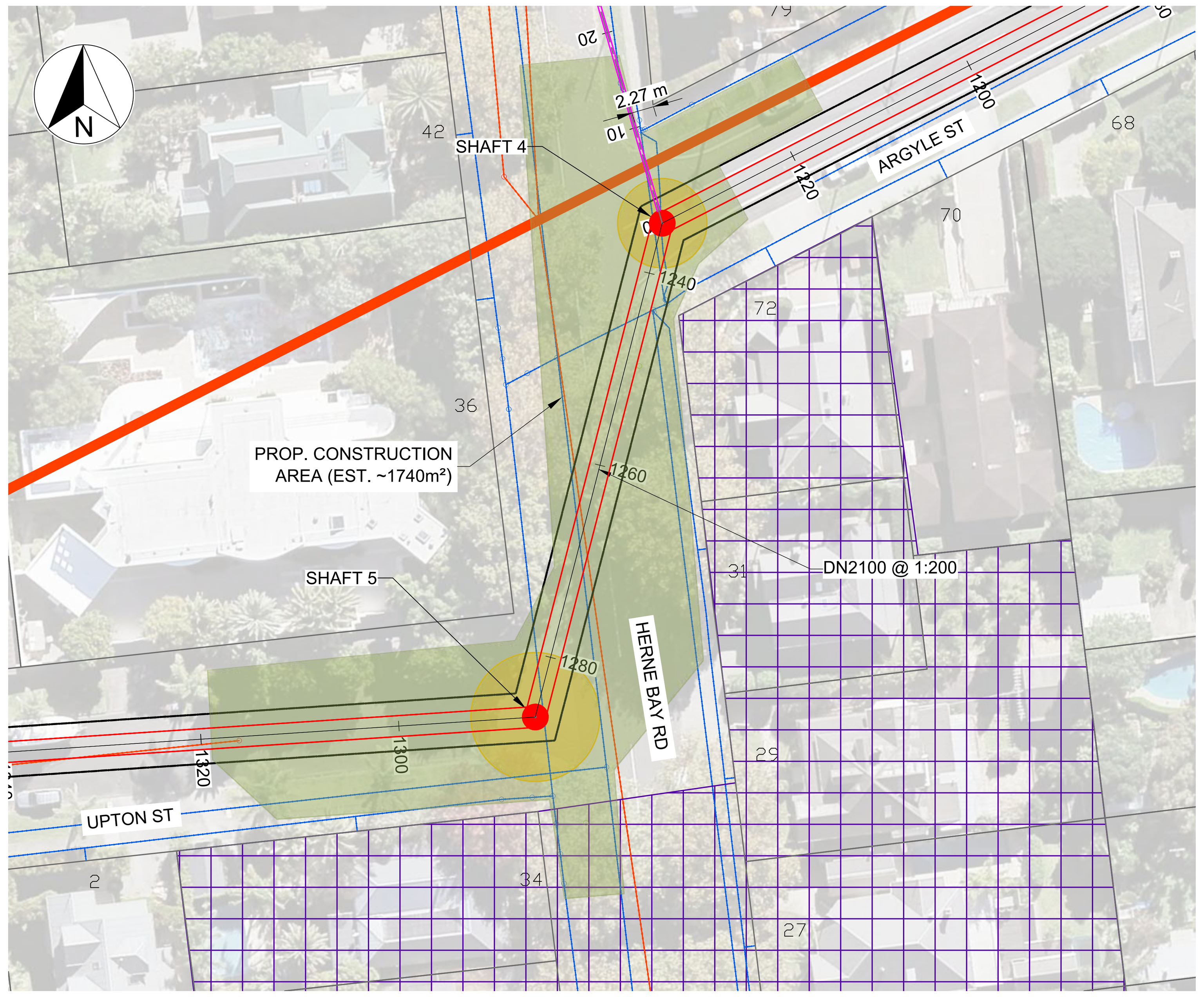
SHAFT 1 LOCATION
SCALE 1:500 @ A1



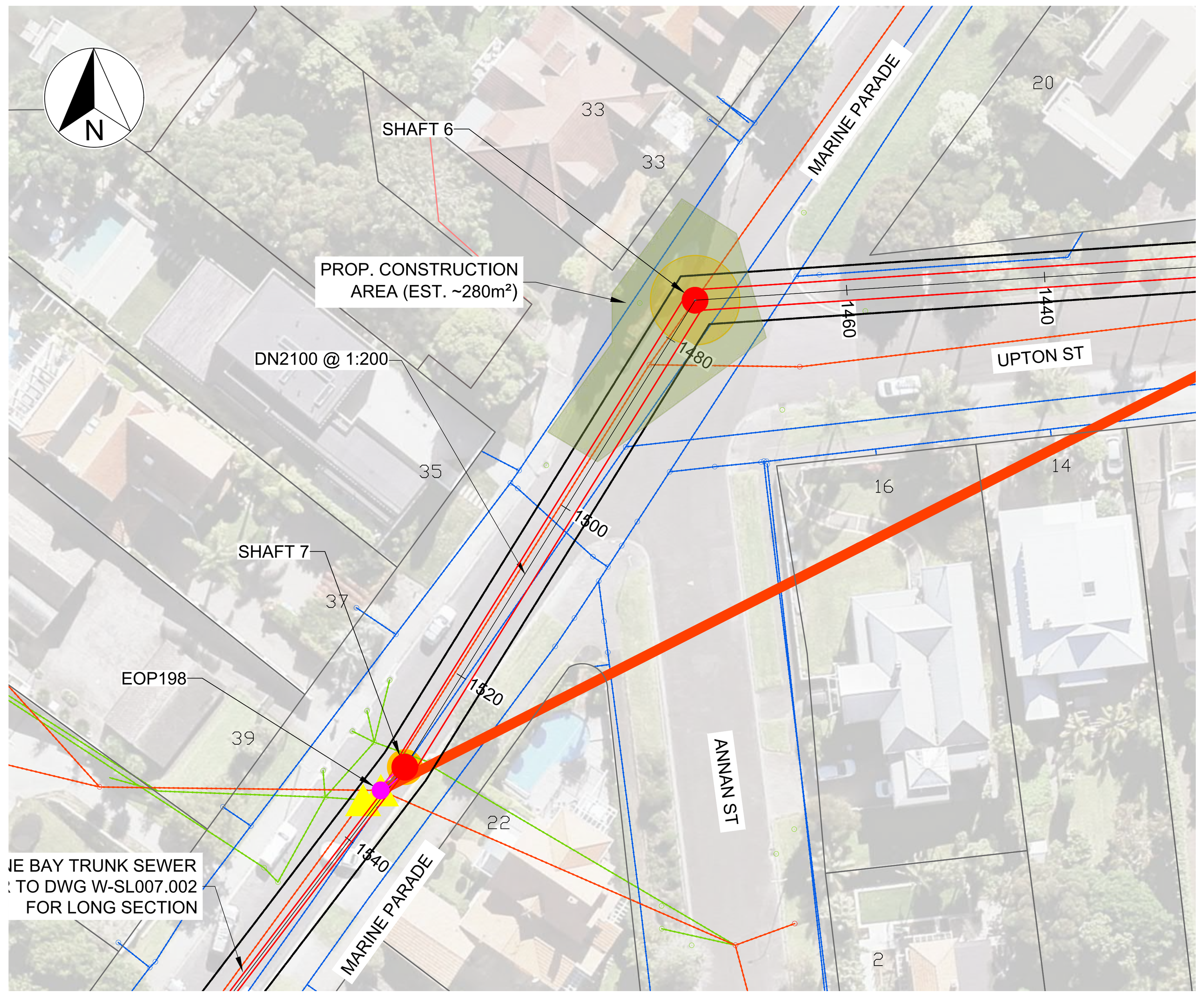
SHAFT 2 LOCATION
SCALE 1:500 @ A1



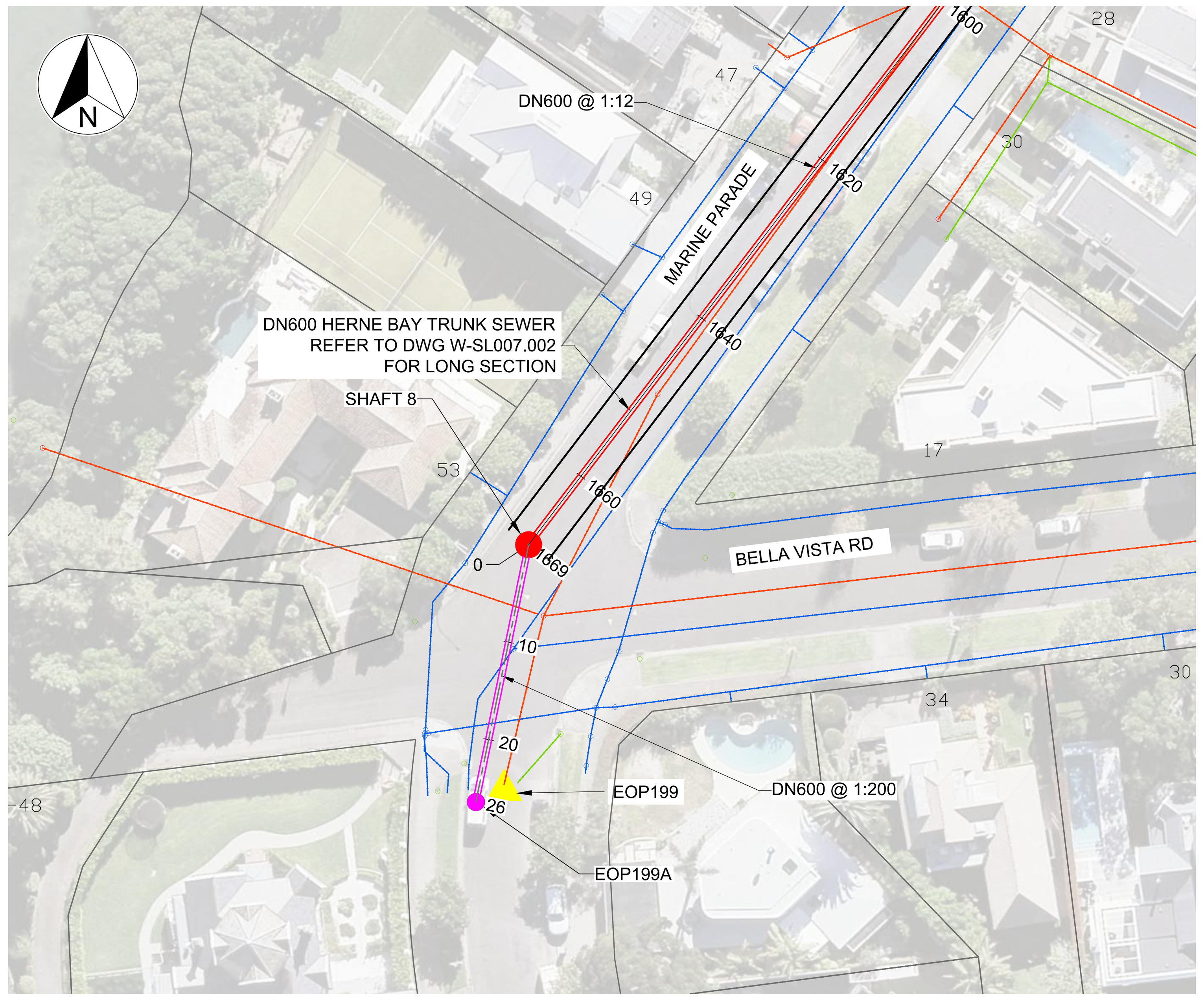
SHAFT 3 LOCATION
SCALE 1:500 @ A1



SHAFT 4 & 5 LOCATION
SCALE 1:500 @ A1



SHAFT 6 & 7 LOCATION
SCALE 1:500 @ A1



SHAFT 8 LOCATION
SCALE 1:500 @ A1

KEY:

- PARCEL BOUNDARY
- EXISTING WATER SUPPLY — W —
- EXISTING LOCAL SEWER — WSL —
- EXISTING TRANSMISSION SEWER
- EXISTING STORMWATER — SW —
- EXISTING GROUND PROFILE
- EXISTING EOP
- PROPOSED LOCAL SEWER
- PROPOSED TUNNEL SEWER
- PROPOSED SEWER MANHOLE
- PROPOSED SHAFT
- PROPOSED HERITAGE BUILDING
- PROPOSED NOTABLE TREE
- TUNNEL 2.5m WIDE CORRIDOR
- PROPOSED CONSTRUCTION AREA

NOTES:

1. REFER TO SHEET W-SL007.002 FOR PROPOSED TRUNK SEWER LONG SECTION
2. COORDINATES ARE IN TERMS OF NEW ZEALAND TRANSVERSE MERCATOR 2000 CIRCUIT.
3. LEVELS ARE IN TERMS OF METRES AUCKLAND 1946 LOCAL VERTICAL DATUM.
4. DIMENSIONS / DISTANCE ARE IN METRES UNLESS STATED OTHERWISE.
5. IT IS THE CONTRACTORS RESPONSIBILITY TO LOCATE ALL EXISTING SERVICES PRIOR TO CONSTRUCTION.
6. DESIGN AT CONCEPT STAGE AND SUBJECT TO CHANGE THROUGH DESIGN PROCESS.
7. NO SURVEY OF THE EXISTING UTILITIES AND FEATURES HAVE BEEN CARRIED OUT.
8. TUNNEL ALIGNMENT SHOWN WITH 2.5m WIDE CORRIDOR EITHER SIDE OF PROPOSED ALIGNMENT.

1:500 @ A1 / 1:1000 @ A3 — 0 5 10 15 20 25 30 35 40 45 50 m

ISSUE	DATE	AMENDMENT	BY	APPD.
1	16.02.23	DRAFT ISSUED FOR CONSENT APPLICATION	GI	MK

	BY	DATE
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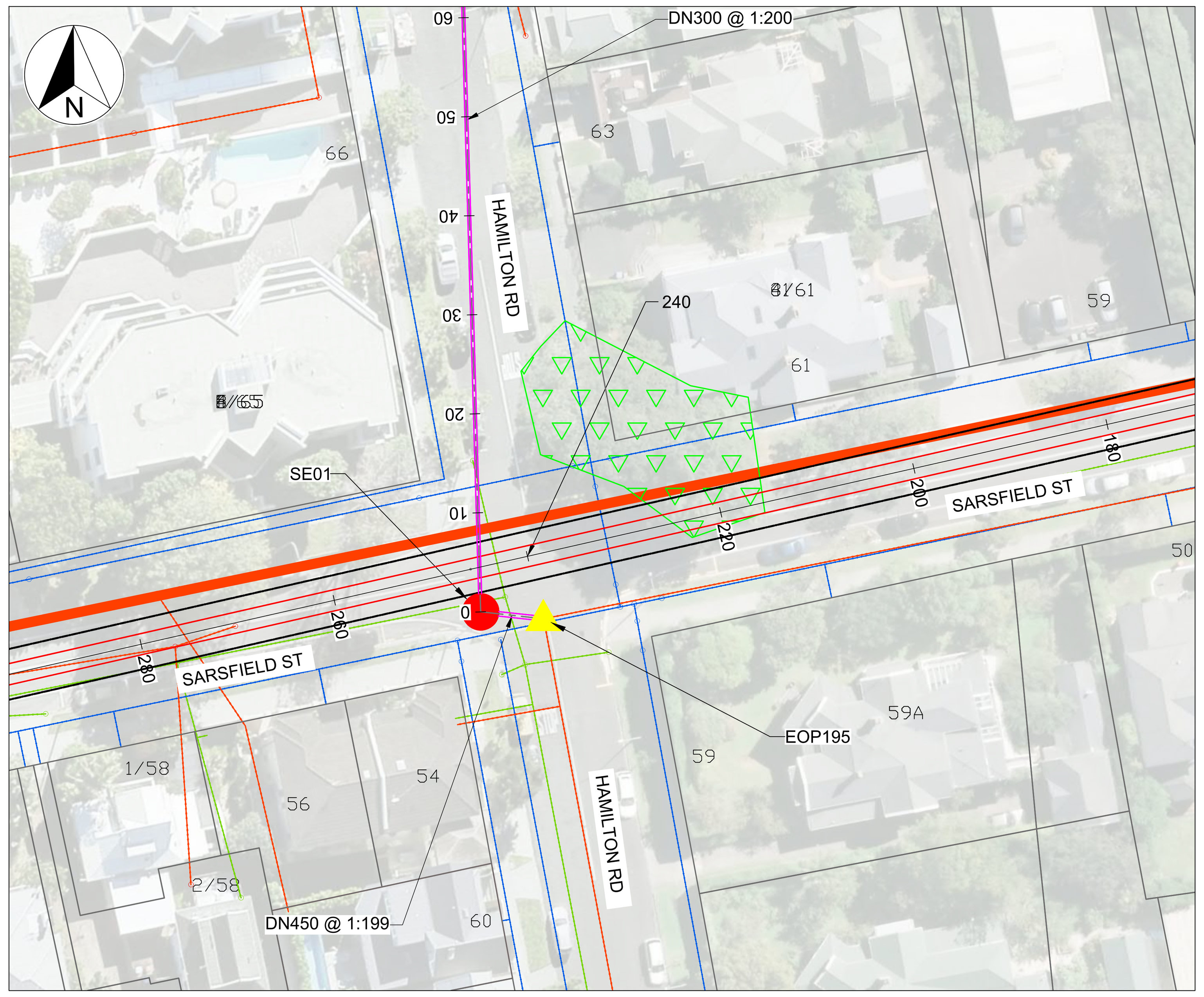
DES. APPROVED	C.STOKES	02-23
DRAWN	G.IP	02-23
DWG. APPROVED	M.KUDOIC	02-23
WSL DESIGN MGMT.	B.DEVILLIERS	-
WSL PROJ. LEAD	-	-

Watercare

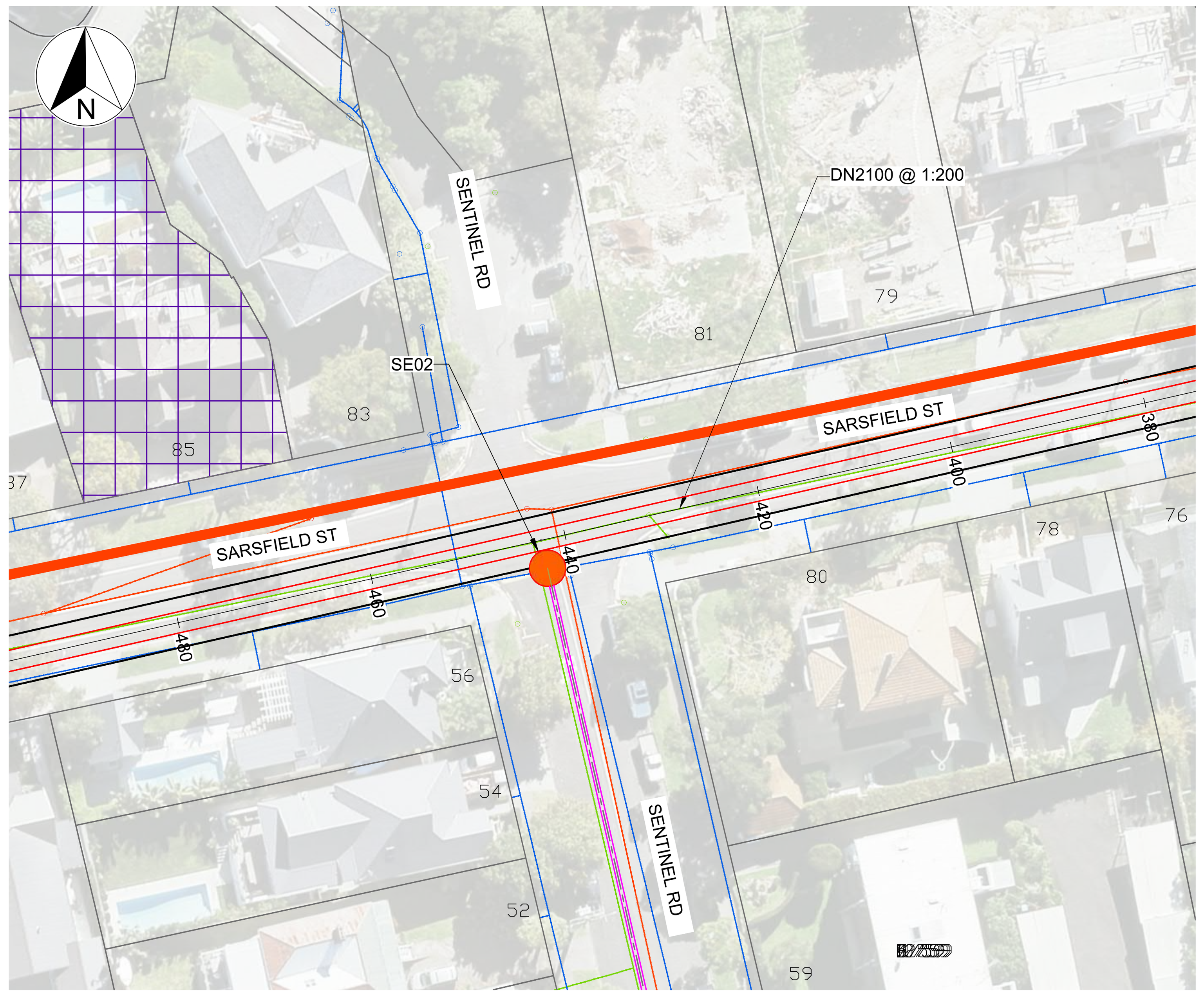
HERNE BAY TRUNK SEWER UPGRADE
HAMILTON ROAD, HERNE BAY
CONSTRUCTION PLAN – TUNNEL SHAFT LOCATIONS

DRAFT

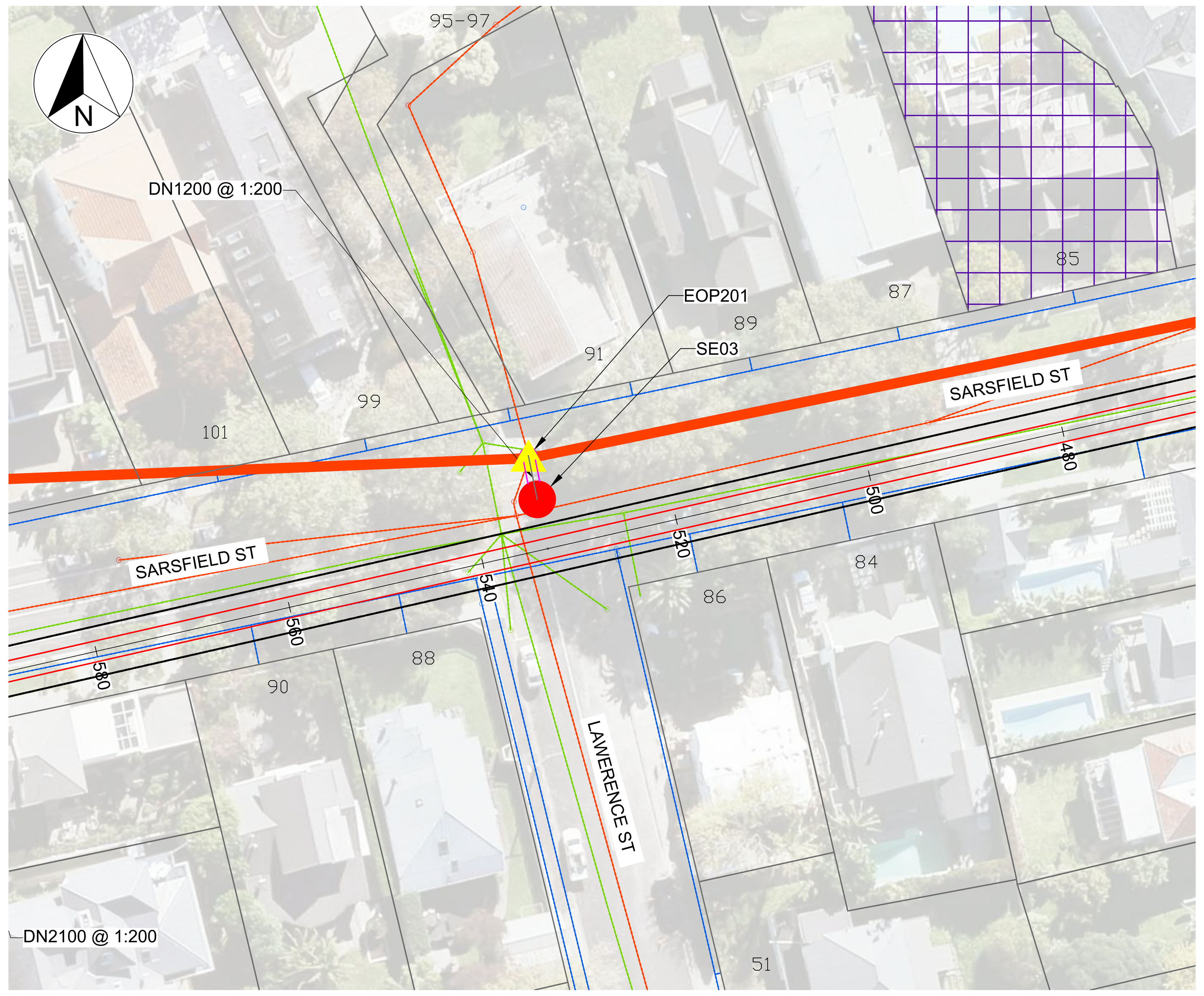
CAD FILE W-SL007.007
DATE 16-02-2023
ORIGINAL SCALE A1 1:2000
CONTRACT No. 1
REF. No. W-SL007.01
DWG. No. W-SL007 .007
ISSUE 1



SHAFT SE01
SCALE 1:500 @ A1



SHAFT SE02
SCALE 1:500 @ A1



SHAFT SE03
SCALE 1:500 @ A1

SHAFT SE04
SCALE 1:500 @ A1

KEY:

PARCEL BOUNDARY	
EXISTING WATER SUPPLY	W
EXISTING LOCAL SEWER	WSL
EXISTING TRANSMISSION SEWER	
EXISTING STORMWATER	SW
EXISTING GROUND PROFILE	
EXISTING EOP	
PROPOSED LOCAL SEWER	
PROPOSED TUNNEL SEWER	
PROPOSED SEWER MANHOLE	
PROPOSED SHAFT	
PROPOSED HERITAGE BUILDING	
PROPOSED NOTABLE TREE	
TUNNEL 2.5m WIDE CORRIDOR	
PROPOSED CONSTRUCTION AREA	

NOTES:

1. REFER TO SHEET W-SL007.002 FOR PROPOSED TRUNK SEWER LONG SECTION
2. COORDINATES ARE IN TERMS OF NEW ZEALAND TRANSVERSE MERCATOR 2000 CIRCUIT.
3. LEVELS ARE IN TERMS OF METRES AUCKLAND 1946 LOCAL VERTICAL DATUM.
4. DIMENSIONS / DISTANCE ARE IN METRES UNLESS STATED OTHERWISE.
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7. NO SURVEY OF THE EXISTING UTILITIES AND FEATURES HAVE BEEN CARRIED OUT.
8. TUNNEL ALIGNMENT SHOWN WITH 2.5m WIDE CORRIDOR EITHER SIDE OF PROPOSED ALIGNMENT.

1:500@ A1
1:1000@ A3
0 5 10 15 20 25 30 35 40 45 50 m

ISSUE	DATE	AMENDMENT	BY	APPD.
1	16.02.23	DRAFT ISSUED FOR CONSENT APPLICATION	GI	MK

	BY	DATE
DESIGNED	G.IP	02-23
DES. APPROVED	C.STOKES	02-23
DRAWN	G.IP	02-23
DWG. APPROVED	M.KUDOIC	02-23
WSL DESIGN MGMT.	B.DEVILLIERS	–
WSL PROJ. LEAD	–	–

Watercare

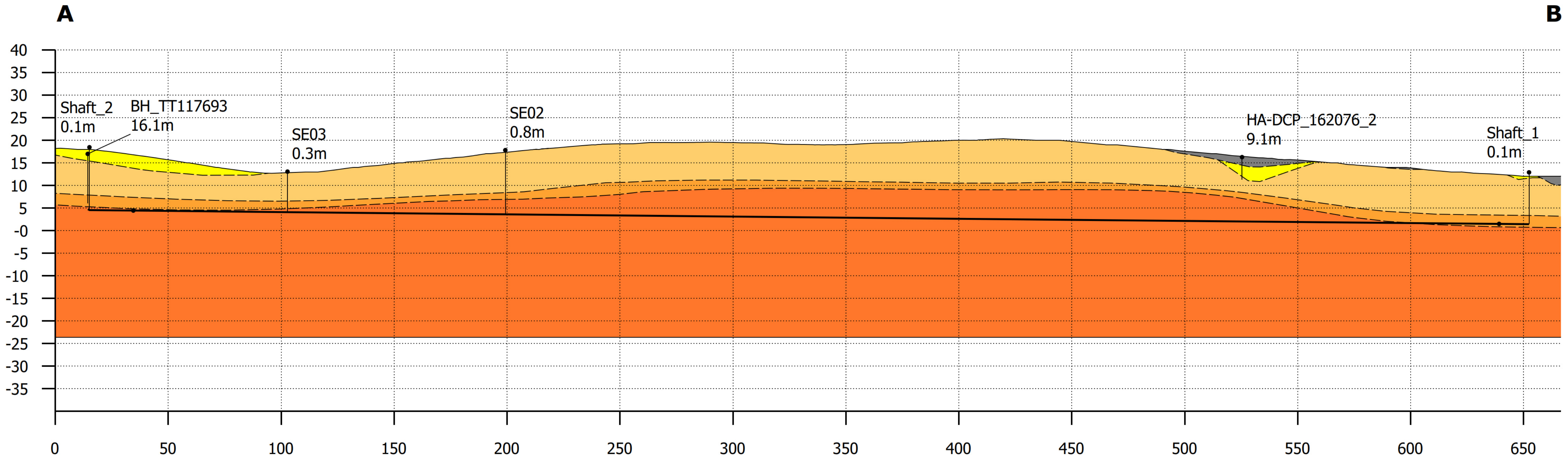
HERNE BAY TRUNK SEWER UPGRADE
HAMILTON ROAD, HERNE BAY
CONSTRUCTION PLAN – INTERCEPTION SHAFT LOCATIONS

DRAFT

CAD FILE W-SL007	DATE 16-02-2023
ORIGINAL SCALE A1 1:2000	CONTRACT No. 1
REF. No. W-SL007.01	
DWG. No. W-SL007 .008	ISSUE 1

Appendix B Geological Model

Herne Bay_CS__Shaft 1 to Shaft 2



Legend

Herne Bay_RLXB_20230116

- Fill
- Takaanini Formation
- East Coast Bays Formation (Residual soil)
- East Coast Bays Formation (Weathered)
- East Coast Bays Formation (SPT N = 50+)

NB: Geological model presented above has been developed based on limited historical ground investigation data across the tunnel alignment. The model will be updated following completion of site-specific investigations in March to May 2023

Location

A: 1754501, 5921239

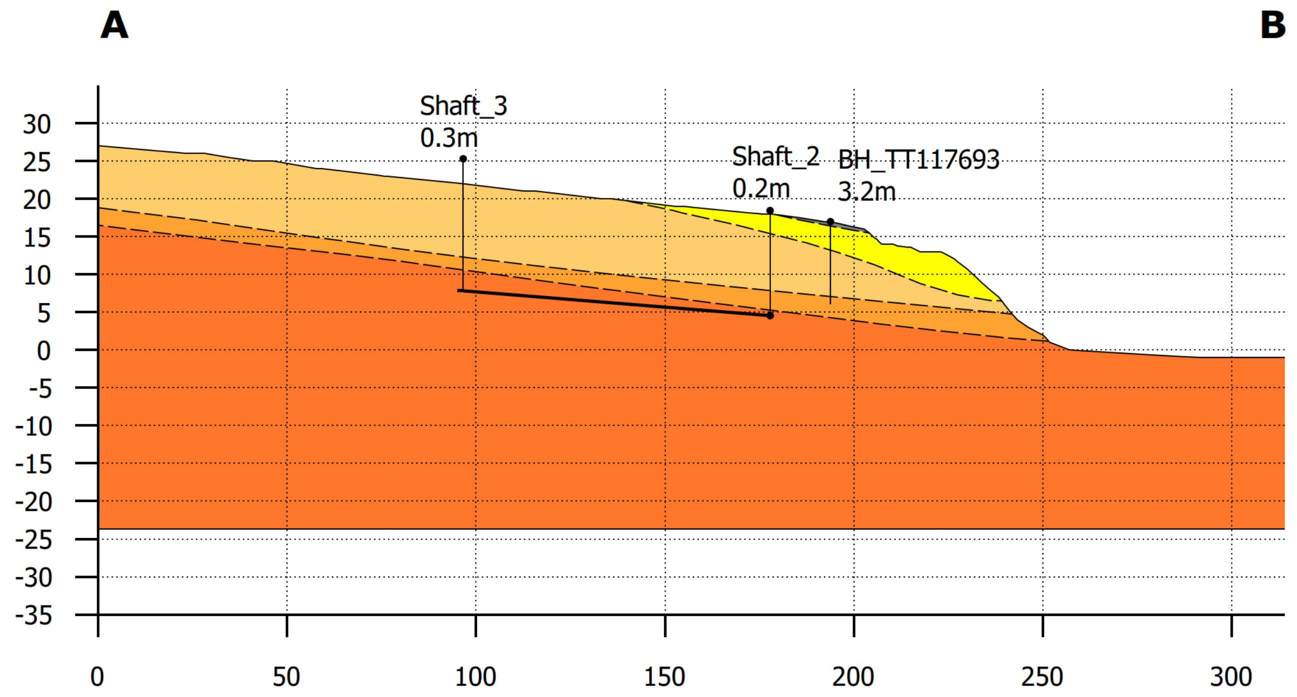
B: 1755151, 5921385

Scale: 1:2,000

Vertical exaggeration: 2x

0m 100m

Herne Bay_CS_Shaft 2 to Shaft 3



Legend

Herne Bay_RLXB_20230116

- Fill
- Takaanini Formation
- East Coast Bays Formation (Residual soil)
- East Coast Bays Formation (Weathered)
- East Coast Bays Formation (SPT N = 50+)

NB: Geological model presented above has been developed based on limited historical ground investigation data across the tunnel alignment. The model will be updated following completion of site-specific investigations in March to May 2023

Location

A: 1754531, 5921065

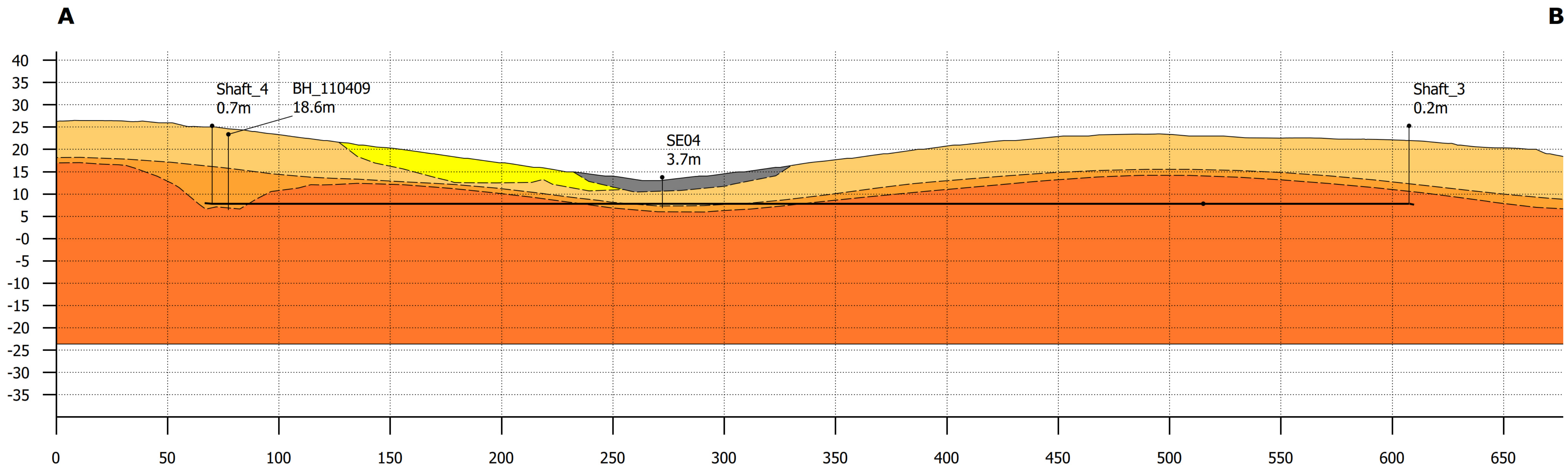
B: 1754505, 5921378

Scale: 1:2,000

Vertical exaggeration: 2x

0m 50m

Herne Bay_CS_Shaft 3 to Shaft 4



Location

A: 1753983, 5920882

B: 1754584, 5921193

Legend

Herne Bay_RLXB_20230116

- Fill
- Takaanini Formation
- East Coast Bays Formation (Residual soil)
- East Coast Bays Formation (Weathered)
- East Coast Bays Formation (SPT N = 50+)

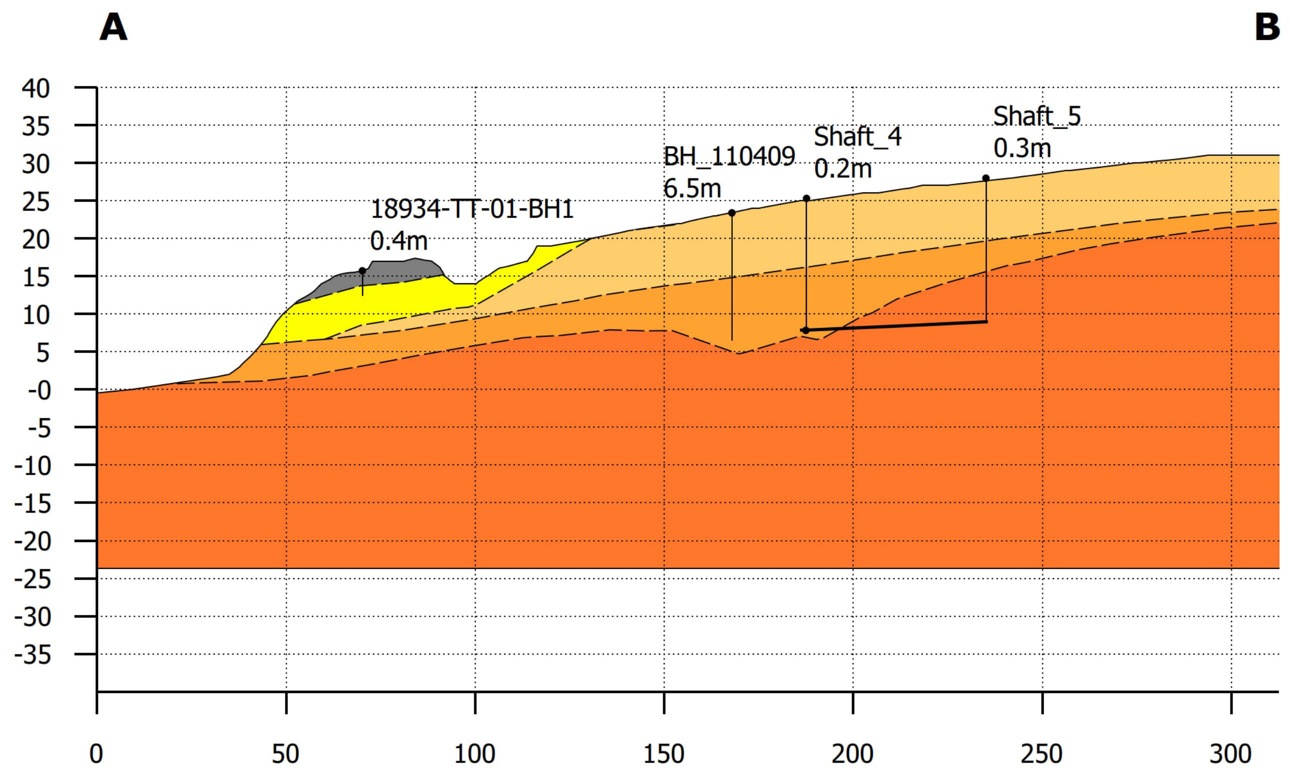
NB: Geological model presented above has been developed based on limited historical ground investigation data across the tunnel alignment. The model will be updated following completion of site-specific investigations in March to May 2023

Scale: 1:2,000

Vertical exaggeration: 2x

0m 100m

Herne Bay_CS_Shaft 4 to Shaft 5



Legend

Herne Bay_RLXB_20230116

- Fill
- Takaanini Formation
- East Coast Bays Formation (Residual soil)
- East Coast Bays Formation (Weathered)
- East Coast Bays Formation (SPT N = 50+)

NB: Geological model presented above has been developed based on limited historical ground investigation data across the tunnel alignment. The model will be updated following completion of site-specific investigations in March to May 2023

Location

A: 1754081, 5921098

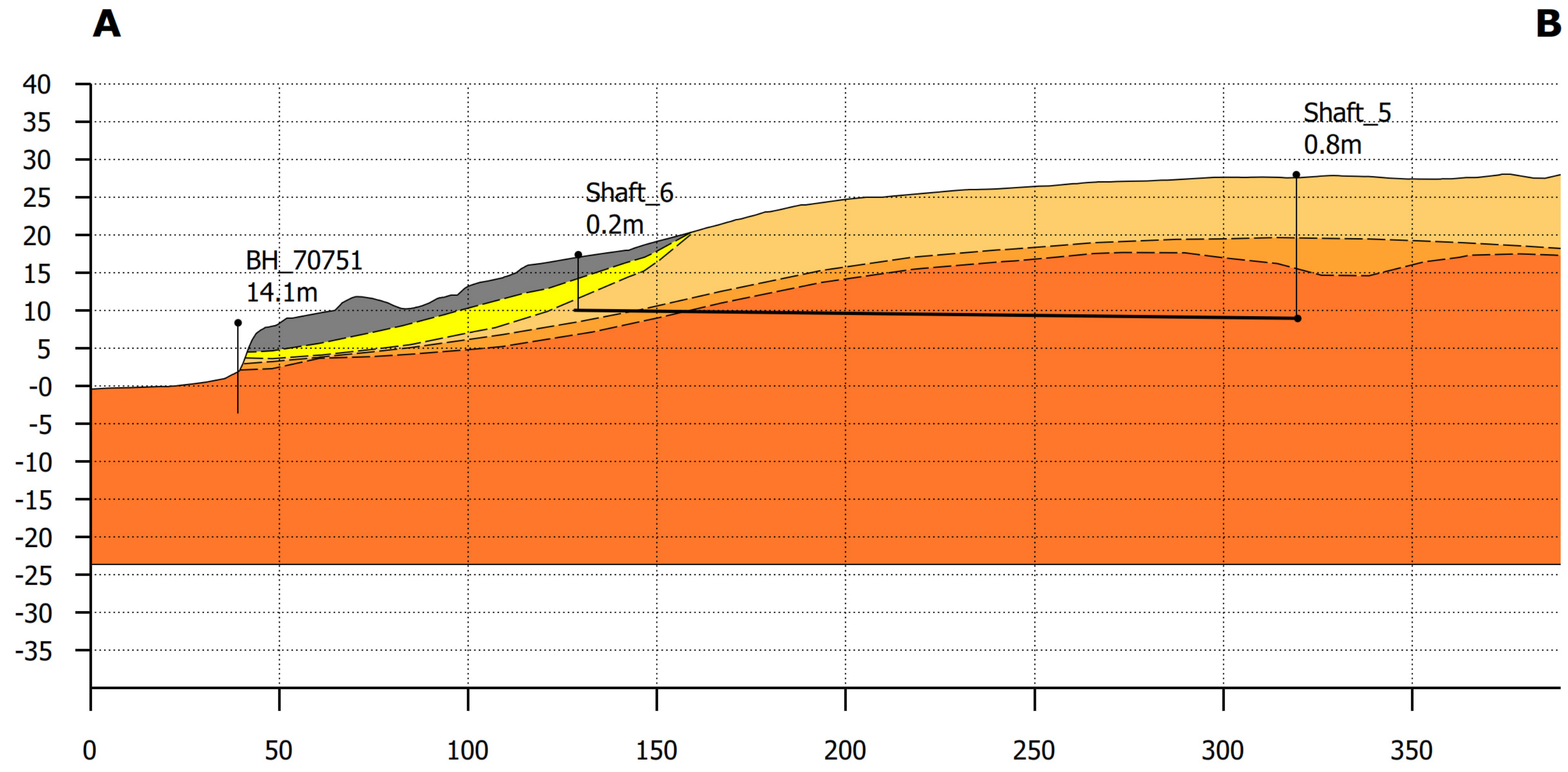
B: 1754022, 5920791

Scale: 1:2,000

Vertical exaggeration: 2x

0m 50m

Herne Bay_CS_Shaft 5 to Shaft 6



Legend

Herne Bay_RLXB_20230116

- Fill
- Takaanini Formation
- East Coast Bays Formation (Residual soil)
- East Coast Bays Formation (Weathered)
- East Coast Bays Formation (SPT N = 50+)

NB: Geological model presented above has been developed based on limited historical ground investigation data across the tunnel alignment. The model will be updated following completion of site-specific investigations in March to May 2023

Location

A: 1753718, 5920847

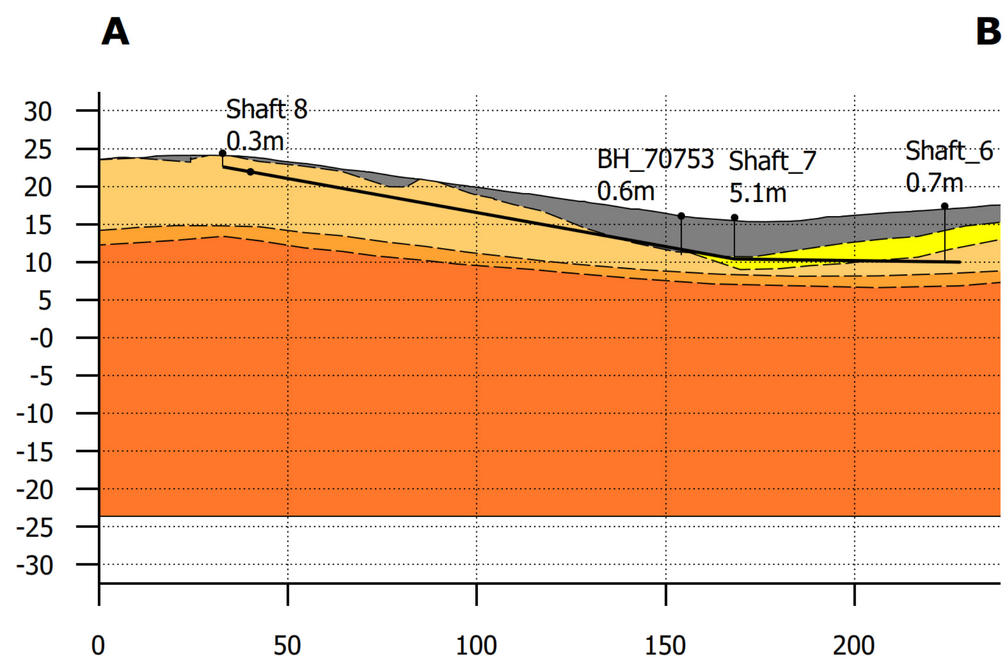
B: 1754106, 5920872

Scale: 1:2,000

Vertical exaggeration: 2x

0m 50m

Herne Bay_CS_Shaft 6 to Shaft 8



Legend

Herne Bay_RLXB_20230116

- Fill
- Takaanini Formation
- East Coast Bays Formation (Residual soil)
- East Coast Bays Formation (Weathered)
- East Coast Bays Formation (SPT N = 50+)

NB: Geological model presented above has been developed based on limited historical ground investigation data across the tunnel alignment. The model will be updated following completion of site-specific investigations in March to May 2023

Location

A: 1753716, 5920673

B: 1753855, 5920868

Scale: 1:2,000

Vertical exaggeration: 2x

0m 50m

Appendix C Tunnel Alignment Settlement Contours

C:\Users\ADSC\OneDrive - Tonkin + Taylor Group Ltd\Documents\ArcGIS\Projects\230419 Herne Bay\Herne Bay.aprx Layout: Shaft 2023-Apr-21 3:56 pm Drawn by ADSC

NOTES:
Basemap NZ Hybrid Reference (Vector): Eagle Technology, LINZ, StatsNZ, NIWA, Natural Earth, © OpenStreetMap contributors..
Address: public. NZ Navigation Map: Eagle Technology, LINZ, StatsNZ, NIWA, Natural Earth, © OpenStreetMap contributors.. NZ
Imagery: Eagle Technology, Land Information New Zealand, GEBCO, Community maps contributors

REV	DESCRIPTION	GIS	CHK	DATE
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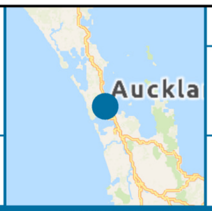

LOCATION PLAN

PROJECT No.	1090120	
DESIGNED	ADSC	APR.23
DRAWN	ADSC	APR.23
CHECKED	RXSW	APR.23
APPROVED		DATE

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CLIENT WATER CARE SERVICES LTD
PROJECT HERNE BAY CONNECTOR
TITLE SHAFT AND TUNNEL CONSTRUCTION SETTLEMENT CONTOUR PLAN

SCALE (A3) 1:1,500 FIG No. 1 REV 0

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REV	DESCRIPTION	GIS	CHK	DATE
0	First version	ADSC	RXSW	21/04.23

LOCATION PLAN: Auckla

PROJECT No.	1090120	
DESIGNED	ADSC	APR.23
DRAWN	ADSC	APR.23
CHECKED	RXSW	APR.23
APPROVED		DATE

CLIENT WATER CARE SERVICES LTD

PROJECT HERNE BAY CONNECTOR

TITLE SHAFT AND TUNNEL CONSTRUCTION SETTLEMENT CONTOUR PLAN

SCALE (A3) 1:1,500 FIG No. 2 REV 0

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REV	DESCRIPTION	GIS	CHK	DATE
0	First version	ADSC	RXSW	21/04.23

LOCATION PLAN: Auckla

PROJECT No.	1090120	
DESIGNED	ADSC	APR.23
DRAWN	ADSC	APR.23
CHECKED	RXSW	APR.23
APPROVED		DATE

CLIENT: WATER CARE SERVICES LTD

PROJECT: HERNE BAY CONNECTOR

TITLE: SHAFT AND TUNNEL CONSTRUCTION SETTLEMENT CONTOUR PLAN

SCALE (A3): 1:1,500   FIG No. 3   REV 0

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REV	DESCRIPTION	GIS	CHK	DATE
0	First version	ADSC	RXSW	21/04.23

LOCATION PLAN

PROJECT No.	1090120	
DESIGNED	ADSC	APR.23
DRAWN	ADSC	APR.23
CHECKED	RXSW	APR.23
APPROVED		DATE

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PROJECT HERNE BAY CONNECTOR
TITLE SHAFT AND TUNNEL CONSTRUCTION SETTLEMENT CONTOUR PLAN

SCALE (A3) 1:1,000 FIG No. 4 REV 0

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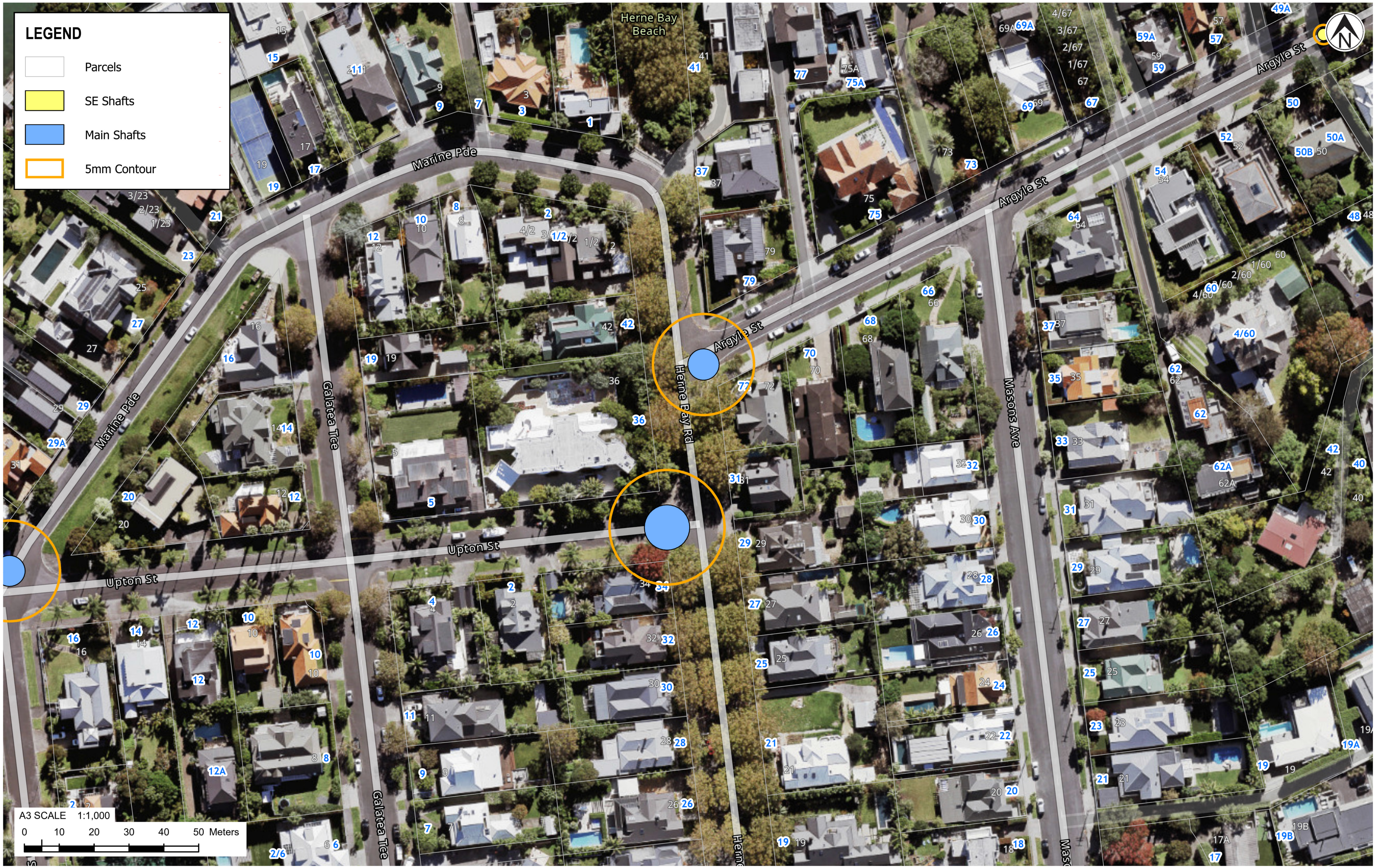

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REV	DESCRIPTION	GIS	CHK	DATE
0	First version	ADSC	RXSW	21/04.23

LOCATION PLAN: Auckla

PROJECT No.	1090120	
DESIGNED	ADSC	APR.23
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CHECKED	RXSW	APR.23
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PROJECT HERNE BAY CONNECTOR

TITLE SHAFT AND TUNNEL CONSTRUCTION SETTLEMENT CONTOUR PLAN

SCALE (A3) 1:1,000   FIG No. 5   REV 0

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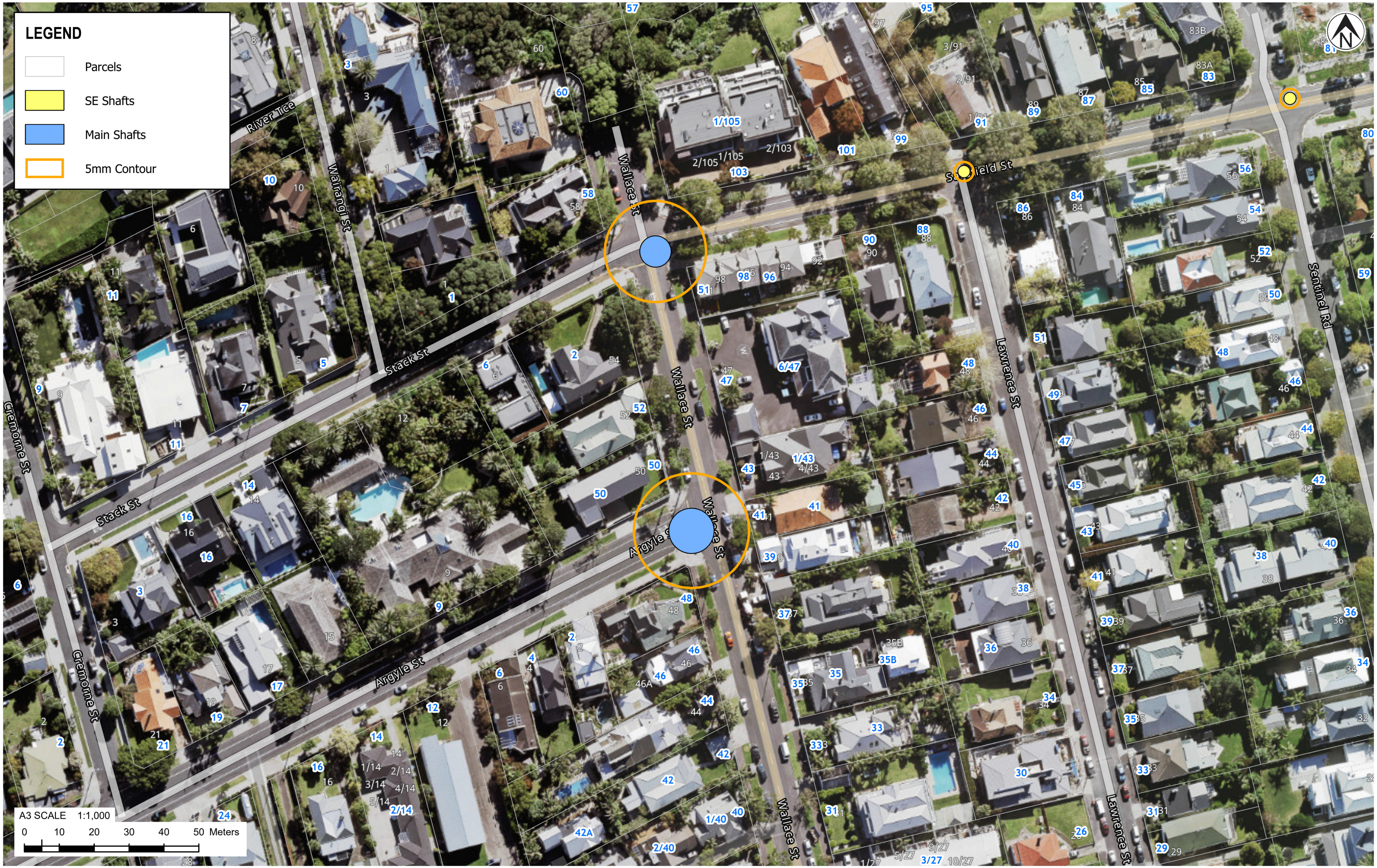

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Imagery: Eagle Technology, Land Information New Zealand, GEBCO, Community maps contributors

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LOCATION PLAN: Auckla

PROJECT No.	1090120	
DESIGNED	ADSC	APR.23
DRAWN	ADSC	APR.23
CHECKED	RXSW	APR.23
APPROVED		DATE

CLIENT: WATER CARE SERVICES LTD
PROJECT: HERNE BAY CONNECTOR
TITLE: SHAFT AND TUNNEL CONSTRUCTION SETTLEMENT CONTOUR PLAN
SCALE (A3): 1:1,000   FIG No. 6   REV 0

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C:\Users\ADSC\OneDrive - Tonkin + Taylor Group Ltd\Documents\ArcGIS\Projects\230419 Herne Bay\Herne Bay.aprx Layout: Shaft 2023-Apr-21 3:56 pm Drawn by ADSC

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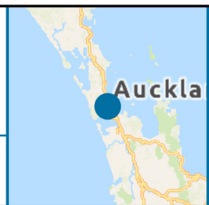
REV	DESCRIPTION	GIS	CHK	DATE
0	First version	ADSC	RXSW	21/04.23



LOCATION PLAN

PROJECT No.	1090120	
DESIGNED	ADSC	APR.23
DRAWN	ADSC	APR.23
CHECKED	RXSW	APR.23
APPROVED		DATE

CLIENT WATER CARE SERVICES LTD

PROJECT HERNE BAY CONNECTOR

TITLE SHAFT AND TUNNEL CONSTRUCTION SETTLEMENT CONTOUR PLAN

SCALE (A3) 1:1,000 FIG No. 7 REV 0

Appendix D AUP Considerations

Permitted Standards		Requirement	Geotechnical Interpretation of compliance	Compliance
E7.6.1.6		Dewatering or groundwater level control associated with a groundwater diversion permitted under Standard E7.6.1.10, all of the following must be met:		
		(1) The water take must not be geothermal water;	No geothermal water expected to be present.	Complies.
		(2) The water take must not be for a period of more than 10 days where it occurs in peat soils, or 30 days in other types of soil or rock; and	No peat soils expected to be present. Water take in non-peat soils likely to exceed 30 days.	Does not comply.
		(3) The water take must only occur during construction.	Tunnel and shaft will be sealed on completion with no permanent water take.	Complies.
E7.6.1.10 Diversion of groundwater caused by any excavation, (including trench) or tunnel	**All of the following activities are exempt from the Standards E7.6.1.10(2)-(6):**			
	(1)	(a) pipes cables or tunnels including associated structures which are drilled or thrust and are less than 1.2m in external diameter;	Drilled / thrust pipe sections will be no greater than 1.2m diameter.	Horizontally drilled pipes are compliant.
		(b) pipes including associated structures up to 1.5m in external diameter where a closed faced or earth pressure balanced machine is used;	TBM drilled tunnel sections in excess of 1.5m proposed.	TBM tunnel sections are not compliant.
		(c) piles up to 1.5m in external diameter are exempt from these standards;	Pile diameters for the shafts are to be confirmed, however anticipated to comprise 750 to 900mm diameter secant piles.	Shaft excavations are compliant.
		(d) diversions for no longer than 10 days; or	Shafts excavations will exceed 10 days.	Shaft excavations are not compliant.

Permitted Standards		Requirement	Geotechnical Interpretation of compliance	Compliance
		(e) diversions for network utilities and road network linear trenching activities that are progressively opened, closed and stabilised where the part of the trench that is open at any given time is no longer than 10 days.	Open trenched sections are likely to remain open for greater than 10 days	Open trenched sections are not compliant.
	(2)	Any excavation that extends below natural groundwater level, must not exceed:		
		(a) 1ha in total area; and	Maximum shaft diameter of 13.5m (<200m^2)	Complies
		(b) 6m depth below the natural ground level.	Shaft excavations will extend greater than 6m below natural groundwater level.	Does not comply.
	(3)	The natural groundwater level must not be reduced by more than 2m on the boundary of any adjoining site.	Secant wall installed to limit drawdown of groundwater level.	Complies.
	(4)	Any structure, excluding sheet piling that remains in place for no more than 30 days, that physically impedes the flow of groundwater through the site must not:		
		(a) impede the flow of groundwater over a length of more than 20m; and	Maximum external diameter of 13.5m at shaft locations.	Complies
		(b) extend more than 2m below the natural groundwater level.	Secant walls to extend >2m below natural water level. Holes will be formed in secant wall on completion of construction to minimise risk of long-term groundwater mounding	Does not comply
	(5)	The distance to any existing building or structure (excluding timber fences and small structures on the boundary) on an adjoining site from the edge of any:		

Permitted Standards		Requirement	Geotechnical Interpretation of compliance	Compliance
		(a) trench or open excavation that extends below natural groundwater level must be at least equal to the depth of the excavation;	Shaft excavations may extend up to approximately 25.5m below ground level	Does not comply.
		(b) tunnel or pipe with an external diameter of 0.2 - 1.5m that extends below natural groundwater level must be 2m or greater; or	Horizontally drilled shafts achieve >2m offset from neighbouring structures	Complies
		(c) a tunnel or pipe with an external diameter of up to 0.2m that extends below natural groundwater level has no separation requirement.		Complies
	(6)	The distance from the edge of any excavation that extends below natural groundwater level, must not be less than:		
		(a) 50m from the Wetland Management Areas Overlay;		Complies
		(b) 10m from a scheduled Historic Heritage Overlay;	Shaft 4 and 5 are within 10m of overlay. Shaft 2 greater than 10m from overlay	Does not comply
		(c) 10m from a lawful groundwater take.		To be confirmed by Planner

Rule	Assessment Criteria		
7.8.2 Assessment Criteria	(4)	Whether the proposal to take and use groundwater from any aquifer demonstrates that: (a) the take is within the water availabilities and levels for the aquifer in Table 1 Aquifer water availabilities and Table 2 Aquifer groundwater levels, in Appendix 3 Aquifer water availabilities and levels and: (i) recharge to other aquifers is maintained; (ii) aquifer consolidation and surface subsidence is avoided (b) the taking will avoid, remedy or mitigate adverse effects on surface water flows, including: (i) base flow of rivers, streams and springs; (ii) any river or stream flow requirements; (c) the taking will avoid, remedy or mitigate adverse effects on terrestrial and freshwater ecosystem habitat; (d) the taking will not cause saltwater intrusion or any other contamination; (e) the taking will not cause adverse interference effects on neighbouring bores to the extent their owners are prevented from exercising their lawfully established water takes; (f) E7.8.2(5)(e) above will not apply in the following circumstances: (i) where it is practicably possible to locate the pump intake at a greater depth within the affected bore; (ii) where it can be demonstrated that the affected bore accesses, or could access, groundwater at a deeper level within the same aquifer, if drilled or cased to a greater depth; (g) the proposed bore is capable of extracting the quantity of groundwater applied for; (h) the proposal avoids, remedies or mitigates any ground settlement that may cause distress, including reducing the ability of an existing building or structure to meet the relevant requirements of the Building Act 2004 or the New Zealand Building Code, to existing: (i) buildings; (ii) structures; and (iii) services including roads, pavements, power, gas, electricity, water supply and wastewater networks and fibre optic cables.	(a) (i) Refer to Section 9.1 of GW assessment report (a) (ii) Refer to Section 8 GW assessment report (b) (i) Refer to Section 9.2 of GW assessment report. (b) (ii) Refer to Section 9.2 of GW assessment report. (c) N/A (d) Refer to Section 9.3 of GW assessment report. (e) N/A (f) N/A (g) N/A (h) (i to iii) Refer to Section 8 of GW Assessment report.

	(6)	Whether the proposal to take and use surface water and groundwater will monitor the effects of the take on the quality and quantity of the freshwater resource to: (a) measure and record water use and rate of take; (b) measure and record water flows and levels; (c) sample and assess water quality and freshwater ecology; and (d) measure and record the movement of ground, buildings and other structures.	Refer to GSMCP detailing monitoring and alert / alarm levels to monitor effects during construction.
	(10)	Whether the proposal to divert groundwater will ensure that: (a) the proposal avoids, remedies or mitigates any adverse effects on: (i) scheduled historic heritage places and scheduled sites; and (ii) people and communities; (b) the groundwater diversion does not cause or exacerbate any flooding; (c) monitoring has been incorporated where appropriate, including: (i) measurement and recording of water levels and pressures; and (ii) measurement and recording of the movement of ground, buildings and other structures; (d) mitigation has been incorporated where appropriate including: (i) minimising the period where the excavation is open/unsealed; (ii) use of low permeability perimeter walls and floors; (iii) use of temporary and permanent systems to retain the excavation; and (iv) re-injection of water to maintain groundwater pressures.	(a) (i to ii) Refer to Section 8 of GW assessment report. (b) N/A (c) (i to ii) Refer to GSMCP detailing monitoring and alert / alarm levels to monitor effects during construction. (d) (i) N/A (d) (ii) Secant pile walls have been adopted for shaft locations to provide groundwater cut-off. (d) (iii) Secant piles are proposed to retain the shaft excavations in the temporary condition. Permanent manholes will be installed within the shafts and sealed at completion (refer to Section 3.2 of GW assessment report). (d) (iv) Refer to contingency options presented with GSMCP to be implemented during construction.