REPORT

CI Extension - Point Erin Tunnel

Screening-level Assessment of Groundwater and Settlement Effects

Prepared for

Watercare Services Limited

Prepared by

Tonkin & Taylor Ltd

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1 Introduction

Tonkin & Taylor Ltd (T+T) has been commissioned by Watercare Services Limited (Watercare) to undertake an Assessment of Effects for Groundwater and Settlement for and extension of the Central Interceptor (CI) tunnel from Tawariki Street in Grey Lynn to Point Erin Park in Herne Bay. This report provides a screening-level settlement effects assessment. This assessment is based on published datasets and site specific available information at the time of writing and will be refined when the site specific data becomes available in March 2023, upon completion of the geotechnical investigations.

This screening exercise was undertaken to identify areas along the tunnel alignment where building, structures and/or services could be at risk to damage caused by ground settlement resulting from construction of the proposed tunnel and/or from construction of the terminal shaft and control chamber in Point Erin Park. The assessment is based on available geotechnical information and concept level scheme plans provided to us by Watercare. While the results and conclusions of this assessment may change as ground conditions are validated with ground investigations and as designs are progressed, we consider that we have an appropriate level of information at the time of writing to undertake a screening-level assessment of the potential groundwater and settlement effects for the Project and that this represents a conservative assessment[1].

Further work is required to validate the assumptions presented in this assessment and further analysis will be undertaken as required following the site-specific data becoming available in March 2023. This further assessment will be provided following completion of the site specific ground investigation and groundwater data collection and will include a detailed assessment of groundwater and associated settlement effects.

2 Project description

Watercare is proposing to extend the CI wastewater conveyance and storage tunnel from Tawariki Street in Grey Lynn to a new terminal shaft at Point Erin (the Project / Point Erin Tunnel). The Point Erin Tunnel will ensure combined overflows are picked up and conveyed to Māngere Wastewater Treatment Plant for safe treatment, reducing overflows to the environment and improving the quality of waterways and swimmable beaches by 2028.The Project involves the construction, commissioning, operation and maintenance of a wastewater interceptor tunnel and associated activities proposed at Point Erin Park in Herne Bay. The Project can be broken into two distinct parts:

- The Point Erin Tunnel which runs from Tawariki Street in Grey Lynn to Point Erin Park in Herne Bay.
- The Point Erin Park shaft site.

These are described in further detail below (as relevant to this assessment).

2.1 Point Erin Tunnel

The Point Erin Tunnel runs from Tawariki Street in Grey Lynn to Point Erin Park in Herne Bay over a length of up to approximately 1.6 km. The tunnel is located entirely below ground at depths typically between 20 m – 60 m and will reach its shallowest point of 17 m as it enters the Point Erin Park where the proposed terminal shaft is located. There are no surface works required for the tunnel.

[1] Assumptions have been made that are conservative in nature. For the effects presented in this assessment to eventuate many of the assumptions are required to be valid simultaneously, which is unlikely.

The tunnel will take a reasonably direct route from Tawariki Street to Point Erin, with the alignment broadly following the road reserve beneath Curran Street to Point Erin[2]. To allow for construction tolerances, Watercare is seeking to consent a 10 m wide corridor centred on this alignment i.e. within 5 m either side of the centreline shown for the length of the tunnel. Vertically, the tunnel will be located within a corridor of -2 m/+2 m based on the centreline and tunnel invert level. The tunnel will have an internal diameter of 4.5 m and will be concrete-lined with a HDPE (high density polyethylene) corrosion protection liner.

Excavation of the tunnel will continue using the existing CI Tunnel Boring Machine ("TBM"). As well as currently being used to construct the CI tunnel, this type of machine has been successfully used in Auckland in similar ground conditions on Project Hobson, the replacement of the Rosedale Wastewater Treatment Plant Outfall, City Rail Link and the Waterview Connection. Construction spoil from the tunnel will be taken back down the CI tunnel and removed at the existing consented/designated CI May Road construction site.

The general alignment of the tunnel is shown in Figure 2.1.

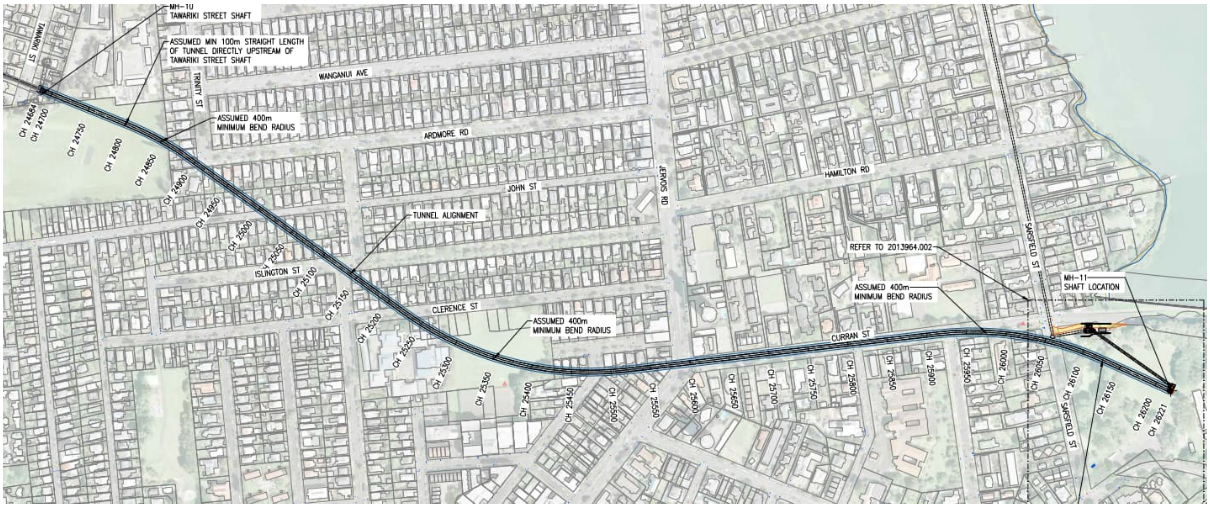


Figure 2.1: Point Erin Tunnel general alignment

2.2 Point Erin Park shaft site

The works at the Point Erin Park shaft site are proposed to occur in two discrete locations within the park:

- The terminal shaft and associated construction area is proposed to be located in the grassed area immediately to the south of the Point Erin Pools (referred to as the main construction area).
- The control chamber, plant room and associated construction area is proposed to be located towards the southwest corner of Point Erin Park near the intersection of Curran and Sarsfield Streets (referred to as the southwestern construction area).

The proposed general layout for these activities is shown on Figure 2.2 below, and in more detail in Appendix A.

[2] Refer Figure 2.1 and Appendix C of the AEE.

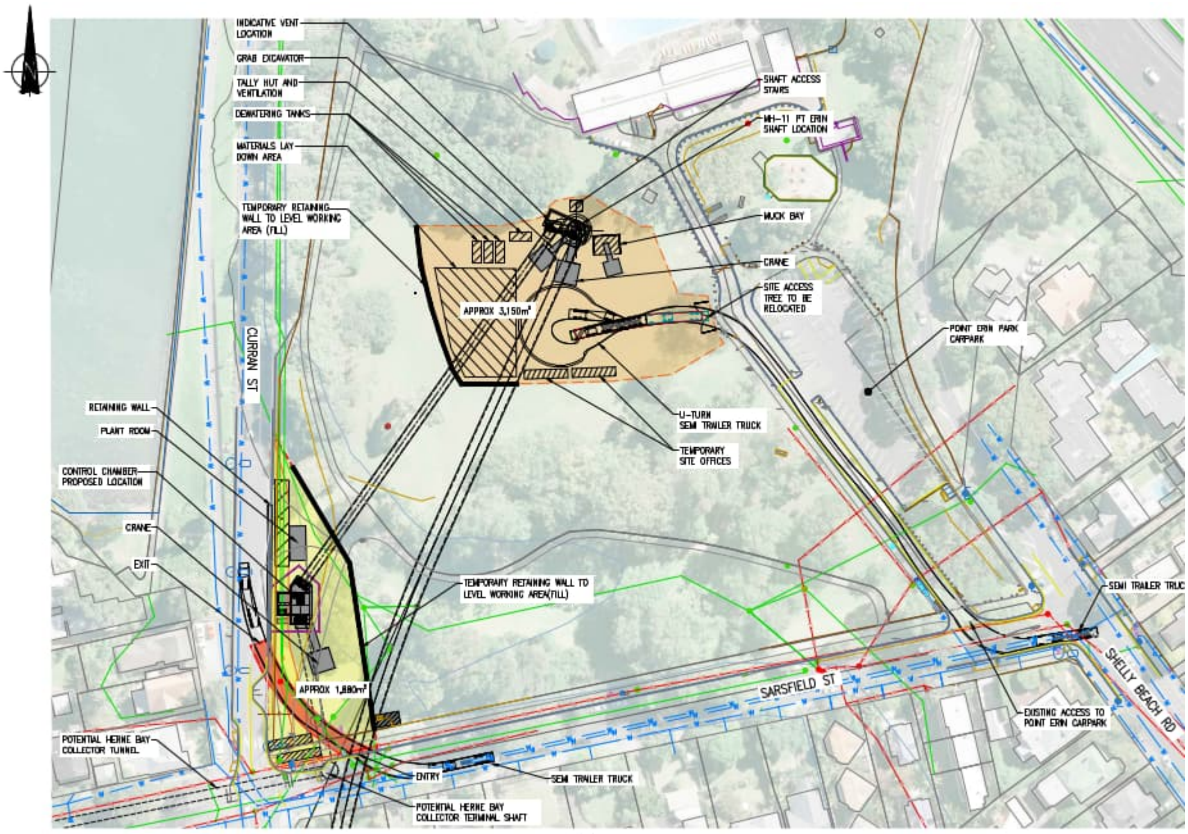


Figure 2.2: Point Erin Park general layout (main construction area shown in orange and south western construction area in yellow)

The Project works within the above mentioned locations in Point Erin Park broadly comprise:

- The construction of infrastructure including a control chamber and plant room, and a terminal shaft for removal of the CI TBM.
- Earthworks of approximately 5,000 m^2 in total across the two construction areas (approx. 3,150 m^2 in the grassed area to the south of the Point Erin Pools and approx. 1,880 m^2 in the south-western corner of the park).
- Tree works (pruning, works in the root zone, removal, relocation).
- Temporary works including retaining walls to create level working areas, site access and internal circulation, and Contractor's site compound.
- Transport movements including delivery of plant and construction materials, removal of material excavated during the construction of the shaft and control chamber, and removal of the TBM.
- Park reinstatement and landscaping following completion of construction works.

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 likely that some details will change but remain within the envelope of effects assessed in this assessment. All figures and dimensions provided are approximate and will be confirmed during the detailed design stage.

3 Construction methodology

3.1 Indicative construction programme

The TBM is expected to arrive at Tawariki Street in February 2025. The TBM advances in the order of 10 m to 20 m per day and is expected to arrive at Point Erin in May 2025 (noting that actual tunnelling progress varies from day to day and week to week and timeframes may change as the TBM progresses along the CI alignment). Ideally, construction works at Point Erin will commence at least 12 months prior to the expected arrival of the TBM at Point Erin, i.e. site establishment in the first half of 2024.

The CI terminal shaft construction is expected to occur over a 6 month period from around September 2024 to February 2025 potentially followed by a hiatus of a few months due to the time taken for the TBM to arrive at the shaft site. This will be followed by approximately 9 months of activity from May 2025 to February 2026 to remove the TBM and complete the internal structure of the main shaft.

The chamber construction is anticipated to take appropriately 9 months (indicatively from around January 2025 to October 2025).

The shaft and chamber are likely to be constructed separately; although, there is the potential there may be some cross over in the construction programme with the programming of works determined by the Contractor.

Overall construction works at Point Erin are expected to take approximately two years (i.e. around 2024 to mid-late 2026), although it may take longer depending on the TBM's progress and other factors such as supply chains and resourcing (e.g. up to three years). It is relevant to note that construction will not be continuous over this full duration, rather there is likely to be periods of more intensive or less intensive construction and then 'quieter' periods, for example when waiting for the arrival of the TBM.

The Point Erin Extension project is expected to be completed mid to late 2026, with the northern section of CI including the Point Erin Extension expected to be commissioned in 2026/2027.

3.2 Tunnelling methodology

Excavation of the tunnel will continue using the existing CI TBM, specifically a Herrenknecht TBM Earth Pressure Balance (EPB) Shield machine. As noted above, this machine is currently in operation and is excavating the southern section of the CI tunnel.

The tunnel liner segments are brought into the tunnel via the consented/designated May Road shaft and transported through the tunnel to the TBM. The segmental precast concrete tunnel liner is progressively placed behind the machine and grouted into the excavated ground opening as the TBM moves forward. The resulting 4.5 m diameter pipe has a durable lining which protects the concrete from corrosion over its 100-year lifespan.

Infiltration of groundwater into the tunnel will be primarily controlled through the design and specification of near-watertight lining systems to limit water inflow. Groundwater inflows through the tunnel lining during construction are limited to less than 0.5 litre / m^2 of tunnel lining per day (13 m^3 per day for the 1.6 km length of the tunnel) based on lining specifications. CI experience to date indicates actual leakage is much lower than this. Any observable leakage would be repaired in situ prior to tunnel commissioning.

Key inputs for this Stage 1 assessment include:

- The invert levels of the tunnel are approximately -12 m RL (24 m depth below ground) at the southern end, and -11 m RL (17 m depth) at the northern end, with a maximum depth of just over 60 m over the central portion of the tunnel alignment as shown in the Point Erin Preliminary Tunnel Longitudinal Section in Appendix A.
- Following the operational plan for the balance of the CI project the TBM will operate in open mode for this proposed alignment given the anticipated ground conditions. Where required, if weak ground material is encountered, the TBM can operate in closed mode which assists with controlling tunnelling risks.
- Where the TBM operates in open mode, some groundwater inflow into the tunnel face is expected resulting in some depressurisation of the hydrostratigraphic unit/ aquifer immediately outside the tunnel at the location of the TBM.
- In closed mode, no groundwater inflow into the tunnel or depressurisation of the hydrostratigraphic unit/ aquifer immediately outside the tunnel is expected.
- Operation of the TBM includes installation of concrete lining concurrent with progression with the advancing tunnelling. For this assessment, it has been assumed that there will be, at any one time, a 12.5 m separation between the cutting face of the tunnel and the section of TBM which installs the concrete lining. We have also conservatively assumed that tunnel progression will advance at a constant rate of 10 m per day[3].

The implication for the assessment is that where the TBM operates in open mode, 12 m section(s) of the tunnel may be 'open' and therefore subject to groundwater inflow for up to 2 days. Where the TBM operates in open mode, 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.

3.3 Construction of the terminal shaft and control chamber

The terminal shaft is required at the termination of the CI tunnel to allow for the retrieval of the CI TBM. The design depth of the 12 m diameter shaft will be -10.8 m RL (approximately 29 m deep). On completion of the shaft and tunnel excavations, the shaft will be fitted out to form a permanent lined shaft.

The shaft is expected to be excavated by conventional mechanical equipment (e.g. CAT 330 medium hydraulics excavator or similar) through overburden soils and East Coast Bay Formation (ECBF) material.

The shaft excavation will have an upper soil support system consisting of secant piles (or other support system) which will be designed to be near-watertight to limit groundwater drawdown. In weathered to fresh ECBF bedrock, excavation support once piled is anticipated to consist of a combination of rock bolts, steel mesh and/or shotcrete depending on ground conditions. The shaft lining and interior structures will be constructed of either cast-in-situ concrete or precast concrete, and potentially of other corrosion resistant materials.

The control chamber will be approximately 12 m x 12 m and 20 m deep. The construction of the chamber may require sheet piling (drained excavation), and otherwise will follow similar construction methods as the terminal shaft (e.g. mesh and bolting below upper support).

An approximately 2.5 m diameter pipe connection will be provided from the shaft to the control structure in the south west corner. This will be constructed via trenchless methods (likely pipe-jacking or alternative method).

[3] The TBM is currently advancing in the order of 15 – 20 m per day.

Key input assumptions for the assessment include:

- Terminal shaft design assumes that the shaft will be constructed using secant piles in a circular arrangement assuming the following:
  - Ground level is assumed to be 18.2 m RL.
  - Secant piles embedded to the unweathered ECBF rock depth estimated at 6.7 m RL (approx. 11.5 m depth).
  - Lining below the unweathered ECBF interface at 6.7 m RL to the design depth of -10.8 m RL (approx. 29 m depth). For the purposes of our assessment, we have conservatively assumed that this lining does not impede the flow of groundwater or provide sufficient rigidity to reduce ground deformations.
- Control chamber design assumes the chamber will be constructed using sheet piles adopting the following:
  - Ground level is assumed to be 11 m RL.
  - Sheet piles to the unweathered ECBF rock depth estimated at 0 m RL (approx. 11 m depth).
  - Lining below the unweathered ECBF interface at -0 m RL to the design depth of -10.0 m RL (approx. 21 m depth). For the purposes of our assessment, we have conservatively assumed that this lining does not impede the flow of groundwater provide sufficient rigidity to reduce ground deformations.

4 Geological and hydrogeological conceptual model

4.1 Available data

A site investigation and drilling programme is running concurrent with this screening-level assessment. The conceptual model presented below is based on information available to the T+T project team at the time of writing this letter report, including:

- Site specific (draft) bore log information along the alignment, provided by Beca.
- Site investigation and testing data obtained from other Watercare projects in the same geologic formations with similar lithology descriptions. This includes the extensive data and interpretation from Central Interceptor.
- Bore log information sourced from New Zealand Geotechnical Database (NZGD).
- Published data including geological mapping by GNS, and regional/ published groundwater levels.

The current site investigation programme including associated reporting is expected to be completed in March 2023.

Based on the information available at the time of writing we consider that we have an appropriate level of information to assess the potential groundwater and settlement effects at a high level for the project and consider that this represents a conservative assessment.

4.2 Regional geology

The project region is characterised by three major stratigraphic groups:

- Late Pleistocene basaltic deposits of the Auckland Volcanic Field (AVF).
- Late Pliocene to Holocene Tauranga Group alluvial and estuarine sediments.
- Miocene Waitemata Group and Waitakere Group marine sedimentary and volcanic rocks

Along the tunnel alignment and Point Erin Park, it is expected that the tunnel and shafts will only intercept the East Coast Bays Formation (ECBF), part of the Waitemata Group. GNS describe 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 stiff silt or clay with a variable sand content.

4.3 Local geology (LeapFrog ground model)

As outlined below, T+T developed a LeapFrog geological model based on the current available data.

4.3.1 Data sources

An alignment for the Central Interceptor – Point Erin pipeline was provided to T+T (via Jacobs on 25 November 2022) as a 2D AutoCAD file. To compile relevant site investigation data from the T+T Geotechnical Database (TTGD) and the New Zealand Geotechnical Database (NZGD) a ~300 m buffer either side of the proposed pipe alignment was extracted, and selected investigations were used for the development of the model.

Some of the available non-project specific geotechnical investigation data was 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).

While the full set of geotechnical technical information was not available, preliminary photographs from BH01 and BH06 were also considered in this assessment.

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

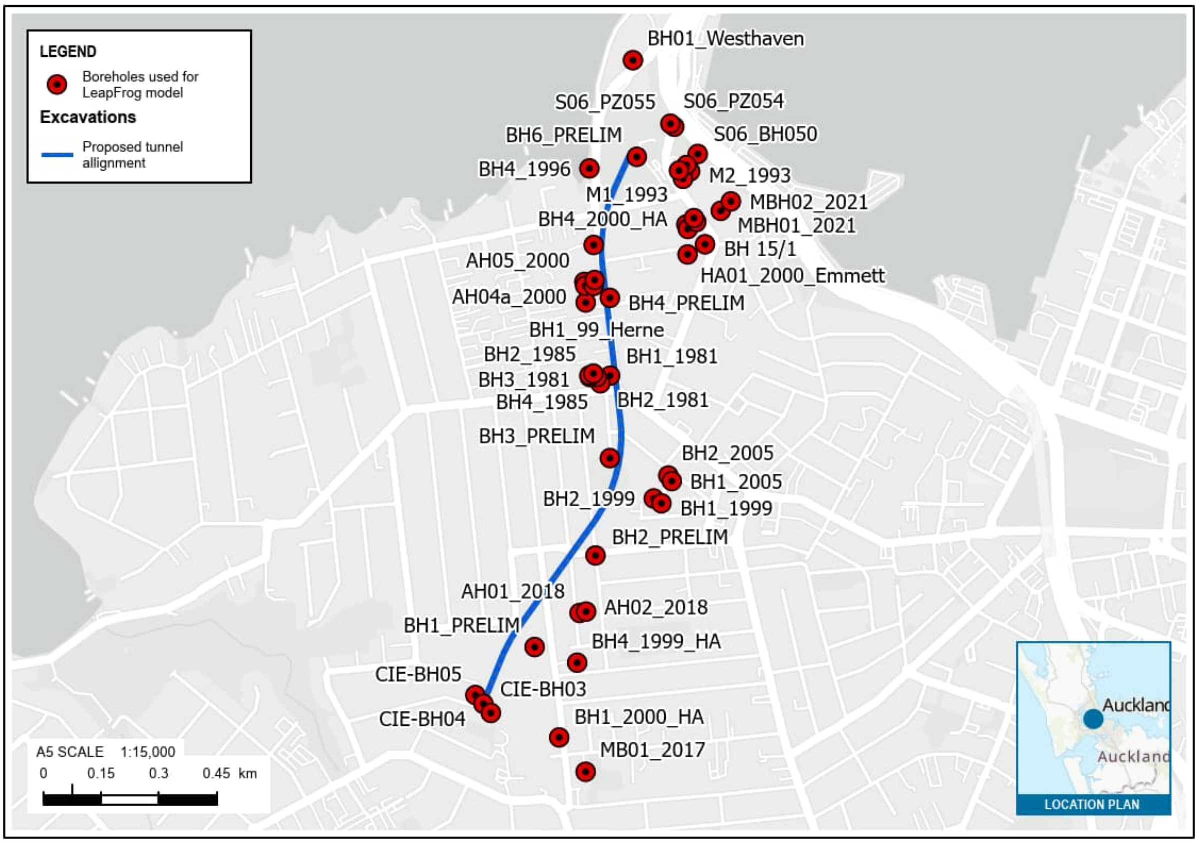


Figure 4.1: Locations of boreholes used to develop the LeapFrog model

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 form 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.

4.3.2 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 not incorporated into the model 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 recoded 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.

4.3.3 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.

4.3.4 Model results

Five geological units have been defined for the Point Erin Shaft Site and the tunnel alignment. A summary of the units is provided in Table 4.1. Supporting information is presented in Appendix B, including a line of section (plan view), geological long cross sections along the tunnel alignment, and a tabulation of our interpreted depth to the ECBF rock geological contact along the alignment

Table 4.1: Geological Formations modelled in LeapFrog

Code	Formation	Description
FILL	Fill	Clay, silt, gravel, sand, cobbles, boulders, and refuse.
TAKAANINI	Takaanini Group Alluvium (Previously referred to as Tauranga Group 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	Sandstone/siltstone rock (SPT N 50+)

4.4 Tunnel alignment hydrogeology

4.4.1 Hydrostratigraphic units and properties

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

- HSU1, which includes 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 conductivity range (m/s)[a]	Sy (-)[b]
1	Residually weathered and weathered ECBF	Silt and clay (stiff), sand (dense), weathered mudstone and muddy sandstone	3 – 14	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)4, p.228

4.4.2 Groundwater levels and flow regime

At the time of writing, limited site specific groundwater level data was available. In the absence of the ground investigation groundwater monitoring data, a publicly available national (NZ) dataset[5] has been used to estimate the water table depth as metres below ground level (m bgl).

For the Point Erin tunnel alignment, the water table has been estimated to be shallowest toward the northern and southern extents of the alignment (0 to 0.5 m bgl) and deepest near the middle of the alignment (up to 20 m bgl).

The data utilised is generally regarded as conservative as it does not allow for an understanding of varying aquifers at depth. Rather, the assumption is made that the shallower groundwater recorded is part of a connected aquifer system.

4.4.3 Saturated compressible material

To inform the groundwater-induced settlement analysis presented below, 'grid math' functionality available in GIS software was used to evaluate the thickness of the saturated compressible material based on the following data:

- Depth to the competent ECBF layer (ECBF ROCK) assumed to incompressible (m bgl) from LeapFrog modelling presented above.
- Depth to water table using the publicly available national (NZ) dataset[4] (m bgl) presented above.

The output of processing this data is presented on Appendix C. Positive values represent areas where the water table occurs above the top of the competent ECBF rock, and negative values represent area where the water table is below the top of competent ECBF rock.

Processing of this data indicates that the water table is located below the depth of the competent ECBF rock for most of the alignment. Only a small portion of the compressible material along the tunnel alignment has been assessed as potentially saturated (area to the north as identified on Figure 1 in Appendix C). The thickness of the saturated compressible material in this identified zone is variable between 0 and approximately 7.5 m (based on currently available data).

[4] Anderson, M. P., Woessner, W. W., & Hunt, R. J. (2015). Applied groundwater modelling: simulation of flow and advective transport. Academic press.
[5] GNS Science. (2018). National water table model [Dataset]. GNS Science. https://doi.org/10.21420/KZ52-NT28

4.5 Point Erin Park hydrogeology

4.5.1 Hydrostratigraphic units and properties

Three hydrostratigraphic units and corresponding hydraulic properties have been defined for Point Erin Park, as shown on Table 4.3.

Table 4.3: Hydrostratigraphic units identified near Point Erin Park

Geological Unit	Thickness (m)	Hydraulic conductivity range (m/s)	Sy range (-)[(b)]	Comment
Takanini Group	0 - 2	10^{-7} to 10^{-6}	0.01 to 0.1	Geological unit not identified at the shaft location. Identified at the control chamber only.
Residually weathered and weathered ECBF	4.4 - 12.2	10^{-7} to 10^{-6} [(a)]	0.01 to 0.1	-
ECBF Rock	> 30	10^{-8} to 10^{-5} [(a)]	0.01 to 0.1	-

Note:

a – CI Geotechnical Design Parameters Table–b - Anderson et al. (2015)[6], p.228

c- Estimate based on site testing in similar materials (Watercare/ T+T projects) based in Auckland.

4.5.2 Groundwater levels flow regime

Groundwater level monitoring data available at the time of writing was used to inform the drawdown-induced settlement assessment at Point Erin Park. This included adopting the lowest groundwater level recorded during the monitoring period in project borehole BH6. Local topography and proximity to the coastline were used as proxies to inform our interpretation of static groundwater levels near the control chamber. Further groundwater data will be made available in March 2023, upon completion of the geotechnical investigation.

4.5.3 Saturated compressible material

Based on the LeapFrog modelling undertaken (Section 4.3) and our interpretation of the groundwater flow regime (Section 4.5.2), the thickness of saturated compressible material which may settle due to dewatering at the site is expected to be small. Our assessment of this saturated compressible material thickness at each excavation location is presented in the following report sections.

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

5 Analysis of tunnel alignment

5.1 Groundwater drawdown and related settlement

5.1.1 Assumptions

Two conservative scenarios were considered for the tunnel assessment, with differing hydraulic conductivity (K) values; adopted parameter values are tabulated in Table 5.1. Both scenarios assume the TBM has progressed 12.5 m in ‘open mode’, which is left open for a duration of two days before the lining is installed and grouted behind the TBM, effectively sealing this section off from groundwater inflow. These scenarios also assume the TBM is progressing at a rate of 10 m/day, noting that is a further conservative assumption as the current rate of the TBM is 15 – 20 m/day.

The scenarios also assume that:

- HSU1 and HSU2 are hydraulically connected and can therefore be represented as a single unit.
- The hydraulic conductivity of the material is isotropic, with the vertical hydraulic conductivity equal to the horizontal hydraulic conductivity.
- The pressure inside the tunnel is at atmospheric pressure (i.e. no partial balancing of hydrostatic pressures is applied by the TBM).
- The assumptions listed in section 5.1.2 are valid.

Table 5.1: Assessment input parameters

Parameter	Scenario 1	Scenario 2
Tunnel diameter (D); m	4.5	4.5
Length of open tunnel (L); m	12.5	12.5
Duration tunnel ‘open’ (t); days	2	2
Aquifer thickness (b); m	30	30
Tunnel depth below water table (H); m	30	30
Hydraulic Conductivity (K); m/s	$2x10^{-8}$	$1x10^{-6}$
Transmissivity (T); m^2/s	$6x10^{-7}$	$3x10^{-5}$
Specific Yield (S_y); (-)	0.05	0.05

5.1.2 Method

Steady state inflow to the open portion of the tunnel was calculated based on the Goodman equation (Goodman et al., 1965)[7]:

$$Q = \frac{2\pi KHL}{\ln\left(^{4H}/_{D}\right)}$$

The Goodman equation assumes the following:

a A tunnel of infinite length.

b Steady-state conditions.

c Soil or rock of homogeneous and isotropic permeability.

[7] Goodman, R., Moye, D., Schalkwyk, A., Javendel, I., 1965. Groundwater inflow during tunnel driving. Eng. Geol. 2 (2), 39–56.

d No drawdown of the groundwater level.

e Flow occurs to the cylindrical surface (sometimes known as the extrados) of the tunnel only; no allowance is made for the flow to the end of tunnel drive (i.e., the working face).

Transient drawdown from the open portion of the tunnel was calculated based the Theis (1935)[8] equation using AQTESOLV software[9], where *u* is the well function, *y* the variable of integration, and *Q* is the discharge rate from the Goodman equation:

$$s = \frac{Q}{4\pi T}\int_u^\infty \frac{e^{-y}}{y}\,dy$$

$$u = \frac{r^2 S}{4Tt}$$

The Theis equations makes the following assumptions:

a aquifer has infinite areal extent.

b aquifer is homogeneous, isotropic and of uniform thickness.

c control well is fully penetrating.

d flow to control well is horizontal.

e aquifer is nonleaky confined.

f flow is unsteady.

g water is released instantaneously from storage with decline of hydraulic head.

h diameter of a pumping well is very small so that storage in the well can be neglected.

5.1.3 Drawdown-induced settlement

Drawdown- induced settlement has been calculated based on:

- Drawdown calculated and presented above.
- Assumed bulk modulus of compressibility (mv) value of 0.03 m2/MN (30 mPa).
- Assumes soil consolidation occurs immediately.
- Equation below which describes settlement that arises due to reduction in pore water pressure (groundwater lowering) and increase in stress.

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

S= settlement
mv = modulus of compressibility
Δσ v = Change in stress
H = Saturated geological unit thickness

5.1.4 Results

Calculated inflow to the tunnel for each scenario was as follows:

- Scenario 1: 1.3 m^3/day.
- Scenario 2: 62 m^3/day.

[8] Theis, C.V., 1935. The relation between the lowering of the piezometric surface and the rate and duration of discharge of a well using groundwater storage, Am. Geophys. Union Trans., vol. 16, pp. 519-524.
[9] AQTESOLV v4.5--Advanced Aquifer Test Analysis Software

Drawdown results for both scenarios (1 and 2) are presented in Figure 5.1.

- Scenario 1 results show maximum drawdown of 3 m at the tunnel axis, and a 9 m zone of influence.
- Scenario 2 results show maximum drawdown of 10.4 m at the tunnel axis, and a 50 m zone of influence.

Drawdown-induced settlement results for scenarios 1 and 2 are presented in Figure 5.2, and Figure 5.3, respectively.

- Scenario 1 results show maximum drawdown-induced settlement of 7.5 mm at the tunnel axis.
- Scenario 2 results show maximum drawdown-induced settlement of 26 mm at the tunnel axis.

The maximum differential settlement assessed from both scenarios is 1:300 within 5 m of the tunnel axis (Scenario 2). The maximum distance at which settlement is less than 10 mm is 6 m from the tunnel axis (Scenario 2).

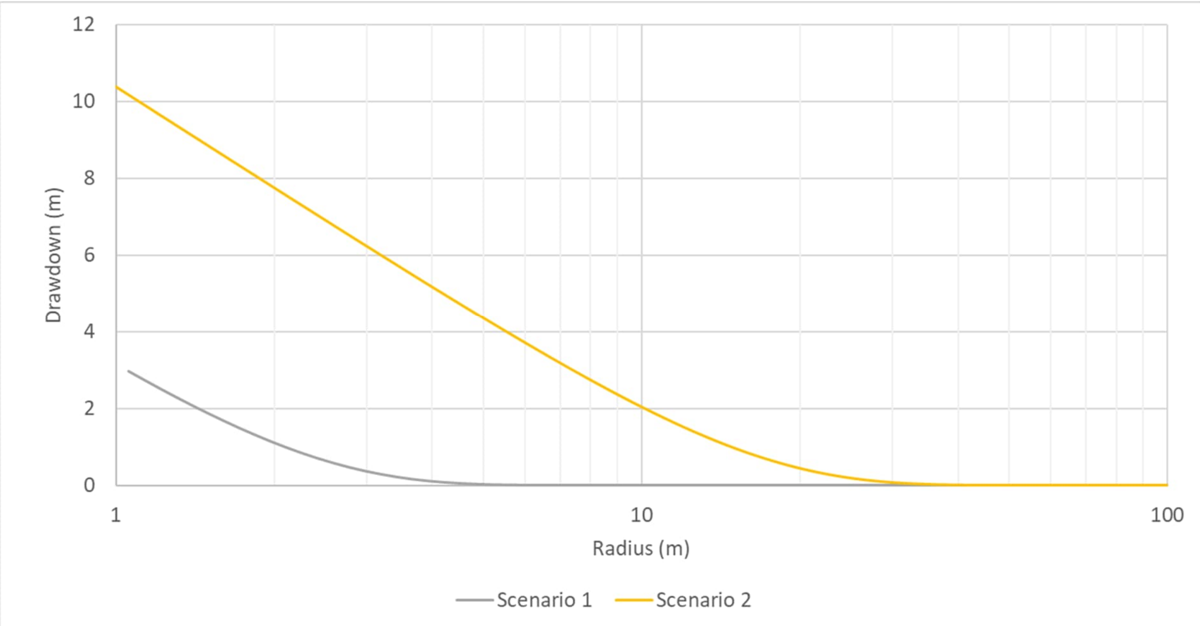


Figure 5.1: Distance-drawdown after 2 days of “open” tunnel

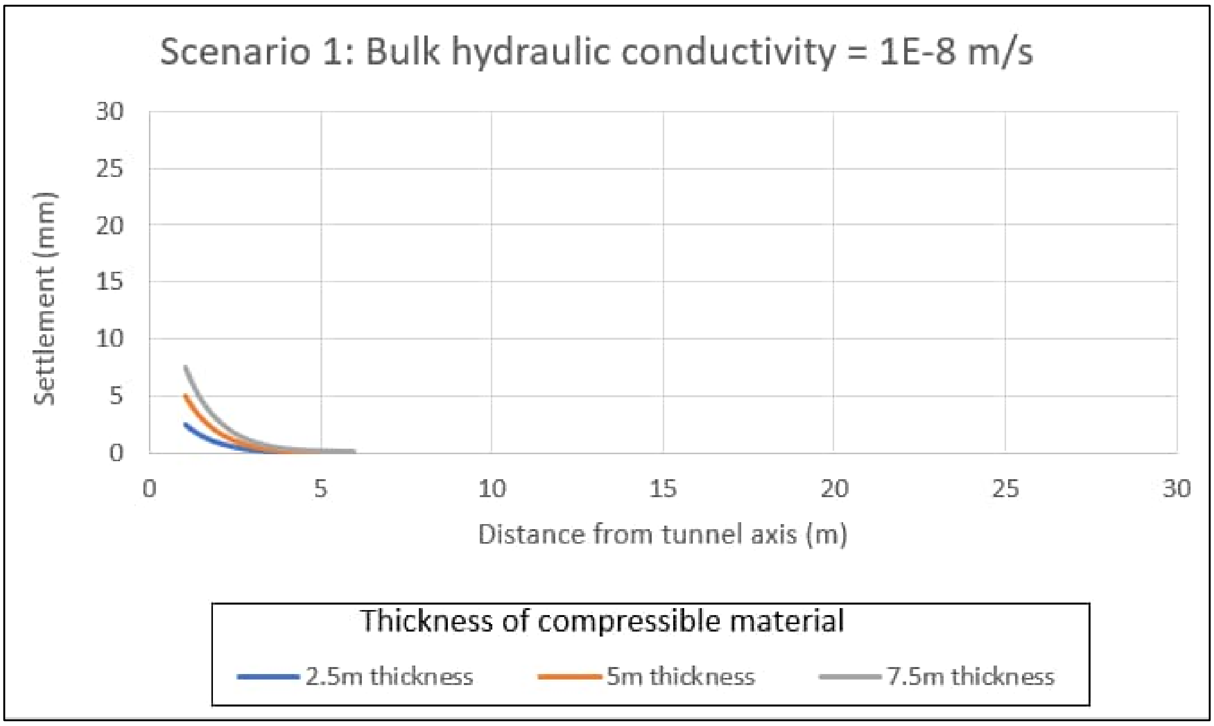


Figure 5.2: Scenario 1 (Low K) drawdown induced settlement results

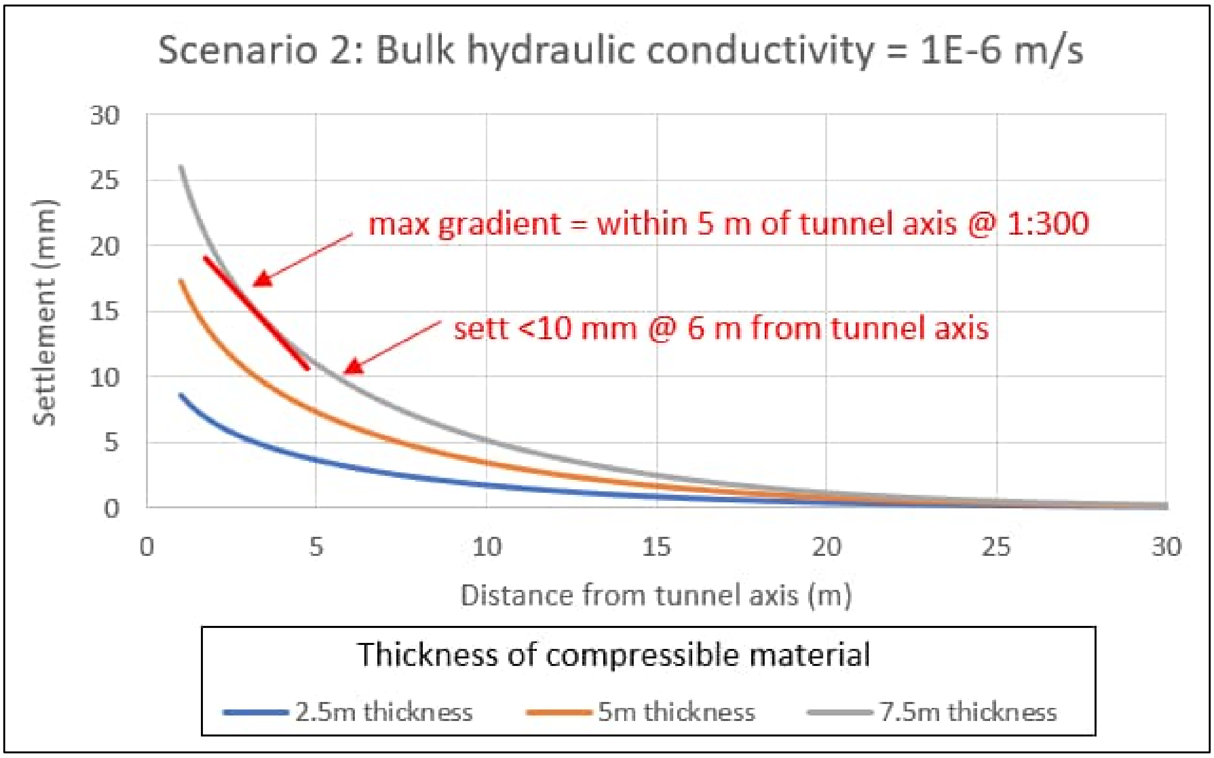


Figure 5.3: Scenario 2 (High K) drawdown induced settlement results

5.1.5 Discussion

The drawdown analysis presented above is conservative based on:

- The assumption made that the tunnelling advances at a rate of 10 m/day (conservative assumption as current rate of TBM is 15- 20 m/day).

- The assumption made that HSU1 and HSU2 are hydraulically connected (conservative assumption). Experience across the Auckland region in similar ground conditions suggests that generally if present, HSU1 and HSU2 are likely hydraulically disconnected due to the interbedded nature of the ground materials. In this scenario, while some depressurisation may occur within the unit immediately outside the tunnel (HSU2), it is unlikely that this would result in any groundwater lowering in the overlying HSU1.
- The method applied assumes confined aquifer conditions. In areas along the alignment where compressible material is saturated, the aquifer conditions are likely to be unconfined which is expected to result in less drawdown than those presented.
- The assumed isotropic hydraulic conductivity applied. It is likely that the vertical hydraulic conductivity is at least ten times less than the horizontal hydraulic conductivity.
- For the Theis analysis, the tunnel is represented as a fully penetrating bore within the aquifer, with depressurisation from advancing in a horizonal radial manner only. In reality, any depressurisation will also need to travel vertically to cause drawdown. As such, the overall radial distance would be greater, resulting in less drawdown of the water table.

The overarching conservatism inherent to this assessment is that all the assumptions listed above are required to be valid simultaneously for the drawdown presented to eventuate, which is unlikely.

Importantly, the calculated settlement is based on a long term permanent stress change that occurs over a period of time. Typical durations of full consolidation for this type of material is in the order of weeks to months. However the TBM transits over any one property in typically 2 days and at any one time only have 12 m of section open. As such, we anticipate that actual settlement during construction to be less than assessed above as the duration of dewatering is not of prolonged or of sustained length to allow for full consolidation.

Furthermore, the TBM may be operated in “closed mode” (i.e. no drawdown of water) if required to ensure appropriate settlement levels are complied with. Tunnel settlement arrays may be used to assess this behaviour during construction, and to manage the potential risk of damage to properties to within levels considered to be “less than minor” (Category 1 on the Burland Classification).

5.2 Tunnel alignment mechanical settlement assessment

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

5.2.1 Method

The method of New and O’ Reilly (1982)[10] has been used to assess the maximum magnitude and lateral extent of mechanically induced ground settlement due to construction of the proposed wastewater tunnel.

Tunnelling publications comprising experience from thousands of projects suggest that volume loss can vary from 0% to 4% depending partly on the ground conditions, but primarily from the specific capabilities of the TBM being used and contractor performance. The tunnel is expected to be bored exclusively within unweathered ECBF rock, minimising the risk of volume loss and annulus closure. From experience the following parameters can be adopted when assessing settlement associated with tunnelling as a result of volume loss.

[10] 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 5.2: Parameters adopted for assessing settlement due to tunnelling volume loss

Parameter	Value
Volume loss in HW to UW ECBF (%)	0.5
Trough width factor, K	0.5
TBM Dia. (m)	5.44

5.2.2 Results

The surface ground settlements for different tunnel depths are presented in Table 5.3. The results indicate that surface ground settlements are expected to be less than 8 mm with differential settlement less than 1V:1300H for a tunnel centreline depth of 12 m which is expected to be the shallowest section of the alignment.

We expect this level of movement is within the natural seasonal fluctuations of the ground. We also note that this shallowest section of the alignment occurs under Point Erin Park. The remainder of the alignment beneath properties is generally at a minimum depth of 18 m at the northern end of the alignment and 30 m at the southern end of the alignment, and up to around 60 m towards the centre of the alignment under the Jervois Road ridge.

Settlements of this range are assessed to be negligible (Category 0) to very slight (Category 1) when compared against the Burland Classification and generally considered less than minor effects by Auckland Council.

Table 5.3: Summary of surface ground settlement resulting from tunnelling volume loss

Volume loss (%)	Depth to tunnel centreline (m)	Surface ground settlement (mm)	Maximum differential settlement
0.5 (unweathered ECBF)	12	<8	less than 1V:1300H
	15	<7	less than 1V:2000H
	20	<5	less than 1V:2000H

6 Analysis of terminal shaft and chamber in Point Erin Park

6.1 Terminal shaft groundwater drawdown and settlement effects

6.1.1 Method

Analytical Element Method (AEM) groundwater flow modelling software Analytical Aquifer Simulator (AnAqSim[11]) 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 competent ECBF unit was incompressible. Modulus of compressibility (mv) values adopted for analysis are shown in Table 6.2 for each geological unit identified from Leapfrog model.

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

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

- Observation points (X,Y) obtained from lines of section.
- Geological contact elevation (Z) values (m RL) obtained from 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 Python12 script.
  - Divided the geological profile (H_{total}) into incremental units for calculation, in this case 0.1 m thick.
  - Assigned assumed constrained modulus to each unit.
  - Calculated the change in pore water pressure at the centre of each incremental layer caused by the groundwater drawdown (refer Equation 2).
  - Estimated the settlement of each incremental unit layer and sum the incremental settlement (refer Equation 1).
- The following assumptions were made for the settlement assessment:
  - Initial static water levels were considered hydrostatic.
  - ECBF rock was considered incompressible.

[11] www.fittsgeosolutions.com
[12] www.python.org

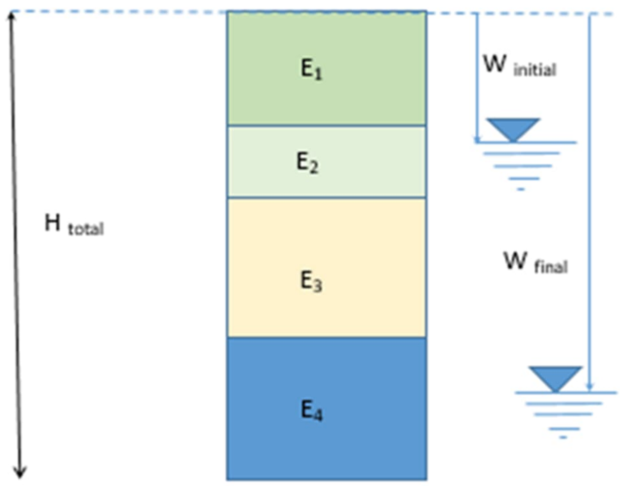


Figure 6.1: Example soil column and initial/final water level for calculating settlement using layer-wise summation method

Equation 1: Layer wise summation method:

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

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

$\Delta P = change\ in\ pore\ water\ pressure$ (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:

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

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

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

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

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

6.1.2 Model setup

The numerical model inputs and setup for the terminal shaft is presented below.

Table 6.1: Numerical model inputs

Model layer	Top elevation (m RL)	Bottom elevation (m RL)	Hydraulic head (m RL)	Kh (m/s)	Kv (m/s)	Aquifer type
1	18.2	11.2	13.0	1×10^{-6}	1×10^{-7}	Unconfined
2	11.2	6.7	13.0	1×10^{-6}	1×10^{-7}	Confined
3	6.7	-20	13.0	1×10^{-6}	1×10^{-7}	Confined

Table 6.2: Adopted compressibility (mv) values for terminal shaft

Leapfrog Mod–I - Unit ID	Modulus of compressibility (mv) kPa
Residual ECBF	30,000
Weathered ECBF	50,000
ECBF rock	Incompressible

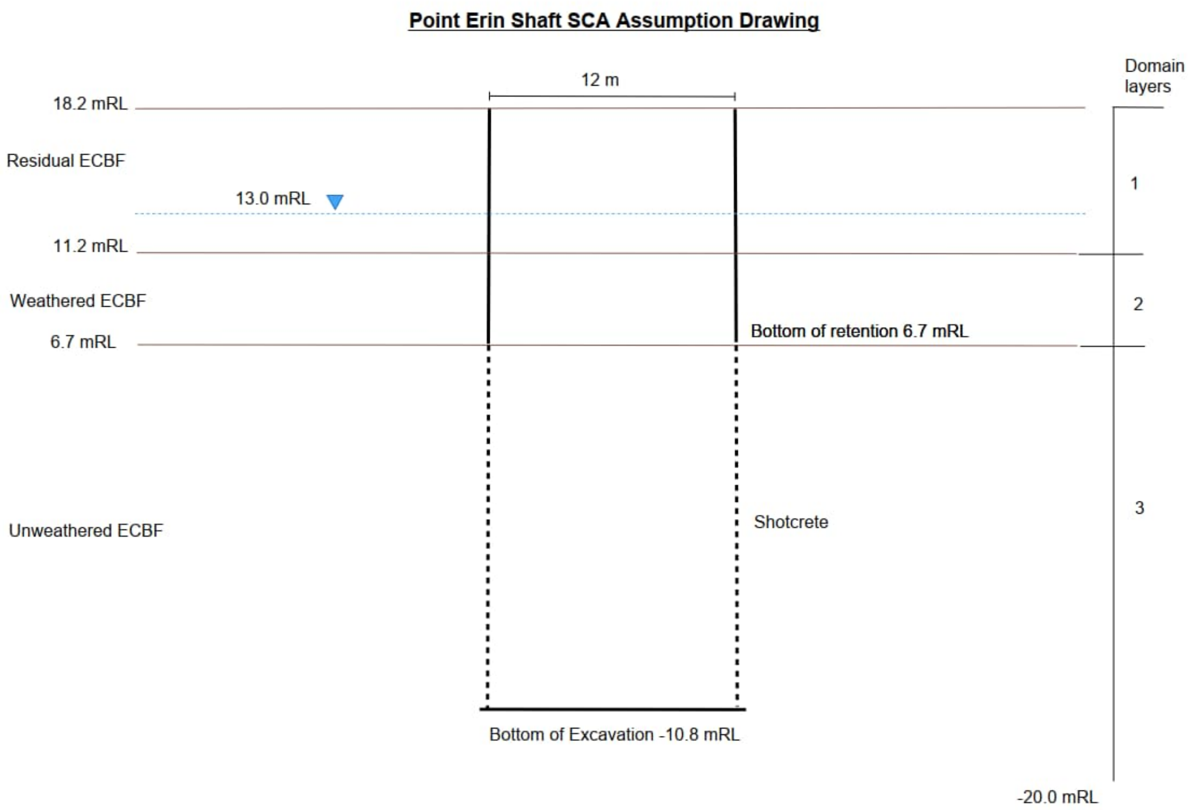


Figure 6.2: Terminal shaft assumption sketch used for AnAqSim model

Excavation retention was added to the numerical model, along the perimeter of the proposed shaft footprint to a level of 6.7 m RL. This was achieved using a leaky barrier boundary condition applied to model layers 1 and 2 with a conductance value of 1E-2 day^{-1} (this is conservative as this assumes a leaky wall).

Groundwater levels within the shaft footprint was lowered from the initial head to the base of the shaft at -10.8 m RL using head specified line boundaries.

6.1.3 Assumptions and limitations

Assumptions:

For modelling purposes we have assumed:

- Flat water table / no hydraulic gradient or direction.
- Rainfall recharge does not occur.
- Radius of AEM model set to 1 km from each shaft centre.
- Timestep 365 day selected to represent pseudo steady-state conditions[13].
- Hydrostatic conditions / groundwater levels in each geological unit are the same.
- For groundwater modelling, horizontal / uniform thickness geological units were assumed for consistency with simplifying assumption of AnAqSim software.
- Hydraulic conductivity was assumed to be anisotropic: Kv = Kh x 0.1.
- Competent ECBF rock was assumed to be incompressible.
- The settlement results documented reflect summer groundwater conditions. It is assumed that material above the recorded summer low has already consolidated and will not settle further with fluctuating groundwater due to dewatering in the shafts.

Limitations:

The dewatering analysis did not account for:

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

Implications:

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

- Design:
  - If the excavation level adopted for design is shallower the inflows and drawdowns estimates are expected to be less, and vice-versa.
  - If the retention embedment depth adopted for design is shallower the inflows and drawdowns estimates are expected to be greater, and vice-versa.

[13] 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.

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

Based on the information available at the time of writing, we consider that we have an appropriate level of information and conservatism in the approach to the analysis.

6.1.4 Output format

The transient model outputs are provided at a single 365-day timestep, selected to represent pseudo steady-state conditions (i.e. long term conditions). The model outputs are presented at drawdown observation points along four lines of section (north: N, south: S, east: E, west: W) as shown in Appendix D.

The total (drawdown plus mechanical) settlement results are presented as contour plots as shown in Appendix E. The method applied includes drawdown-induced settlement contours were generated using Surfer software applying the Kriging interpolation method.

6.1.5 Results

The groundwater model drawdown and drawdown induced settlement results along the four lines of section (north, south, east west) are presented in Appendix D. These results show that the modelled drawdown is generally even in all directions around the proposed excavations. Maximum groundwater drawdown levels of up to 2.5 m are predicted next to the excavations, reducing to less than 0.5 m at approximately 25 m distance, and to near zero to within 100 m distance.

Modelled groundwater induced ground settlement immediately outside the excavation ranges from approximately 5 mm to 8 mm and reduces toward zero at increasing distance from the excavation. We expect this level of movement is within the natural seasonal fluctuations of the ground.

Settlements of this range are assessed to be negligible (Category 0) to very slight (Category 1) when compared against the Burland Classification and generally considered less than minor effects by Auckland Council.

6.2 Terminal shaft mechanical settlement

This section assesses the surface ground settlement resulting from the excavation of the secant piled shaft. The assessed construction methodology for the shaft is a circular arrangement secant pile retention system, these structures are inherently rigid by design as the structural system goes into compression to resist the lateral earth pressures, rather than deforming or relying on a “toe embedment” as seen with conventional cantilevered or tied-back retention systems. This results in very small to negligible deformations.

We have previously assessed multiple secant piles shafts adopting a similar methodology including those located at Tawariki Street for Central Interceptor which are of similar depth in a similar geological setting. Based on this experience, circular secant pile shafts result in very limited mechanical settlement (less than 5 mm at the edge of excavation, the effects of which would be negligible), and a detailed numerical assessment is assessed to not be required.

6.3 Control chamber groundwater drawdown and related settlement

The methodology, assumptions and limitations, and output format presented for the terminal shaft above (Sections 6.1.1, 6.1.3, and 6.1.4 respectively) is consistent with those adopted for the control chamber and is therefore not repeated in this section of the report.

6.3.1 Model setup

The numerical model inputs and setup for the control chamber is presented below.

Table 6.3: Numerical model inputs

Model layer	Top elevation (m RL)	Bottom elevation (m RL)	Hydraulic head (m RL)	Kh (m/s)	Kv (m/s)	Aquifer type
1	11.0	4.0	5.5	1×10^{-6}	1×10^{-7}	Unconfined
2	4.0	0.0	5.5	1×10^{-6}	1×10^{-7}	Confined
3	0.0	-10	5.5	1×10^{-6}	1×10^{-7}	Confined

Table 6.4: Adopted compressibility (mv) values for terminal shaft

Leapfrog Mod–I - Unit ID	Modulus of compressibility (mv) kPa
Fill	25,000
Takanini	5,000
Residual ECBF	50,000
ECBF rock	Incompressible

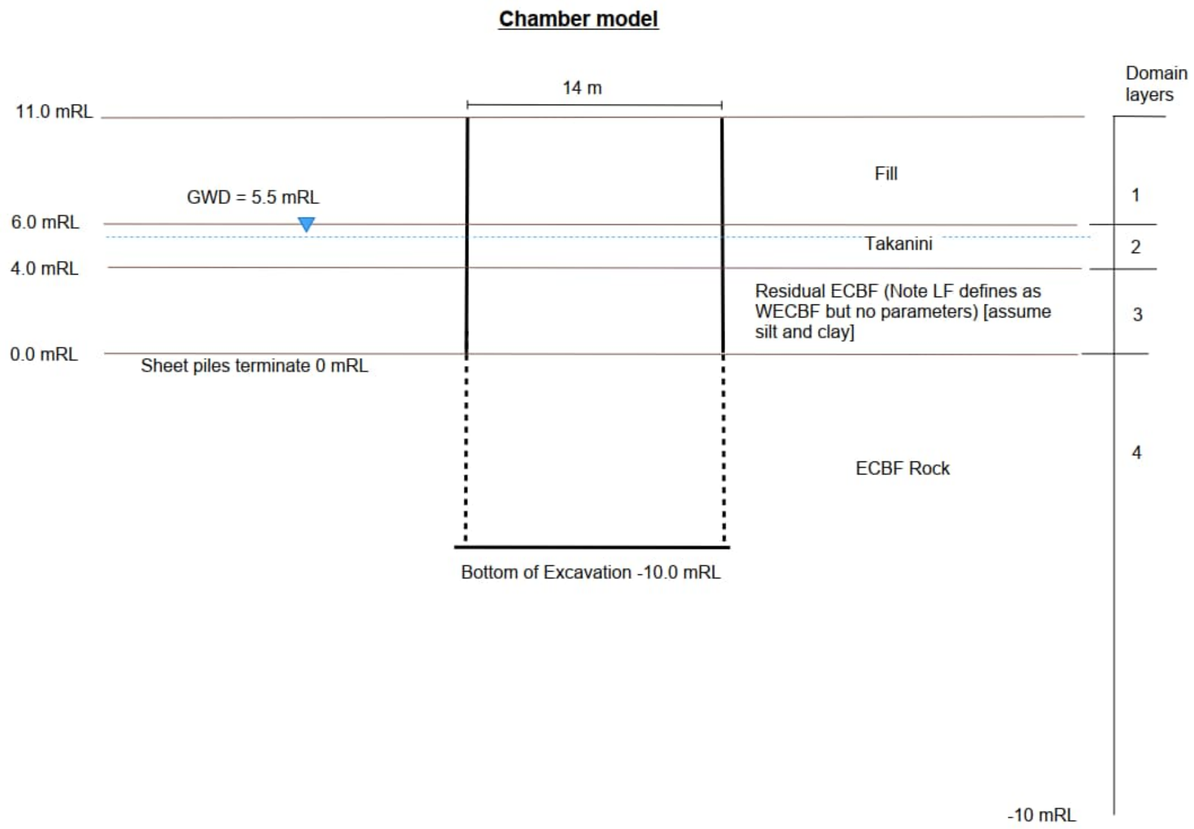


Figure 6.3: Control chamber assumption sketch used for AnAqSim numerical model

Excavation retention added to the numerical model, along the perimeter of the proposed shaft footprint to a level of 0.0 m RL. This was achieved using a leaky barrier boundary condition applied to model layers 1 and 2 with a conductance value of 6E-2 day^{-1} (this is conservative as this assumes leaky walls).

Groundwater levels within the shaft footprint was lowered from the initial head to the base of the shaft at -10.0 m RL using head specified line boundaries.

6.3.2 Results

The groundwater model drawdown and drawdown induced settlement results along the four lines of section (north, south, east west) are presented in Appendix D. These results show that the modelled drawdown is generally even in all directions around the proposed excavations. Maximum groundwater drawdown levels of up to 2.8 m are predicted next to the excavations, reducing to less than 0.5 m at approximately 25 m distance, and to near zero to within 100 m distance.

Modelled groundwater induced ground settlement immediately outside the excavation ranges from approximately 4 mm to 5 mm and reduces toward zero at increasing distance from the excavation.

We expect this level of movement is within the natural seasonal fluctuations of the ground. Settlements of this range are assessed to be negligible (Category 0) to very slight (Category 1) when compared against the Burland Classification and generally considered less than minor effects by Auckland Council.

6.4 Control chamber mechanical settlement

6.4.1 Method

Analysis of the vertical movements resulting from the excavation of the control chamber has been undertaken using the finite-element geotechnical software PLAXIS[14] 2D. PLAXIS 2D allows for a staged construction sequence to be modelled to simulate the proposed construction conditions. The analysis was carried out in a plane strain model. The settlement has been extracted directly from PLAXIS for use in the assessment of settlement effects.

A preliminary ground model has been developed based on the available geological and groundwater information to estimate potential impacts from the chamber construction.

The ground model adopted for the analyses is presented in Figure 6.4 below.

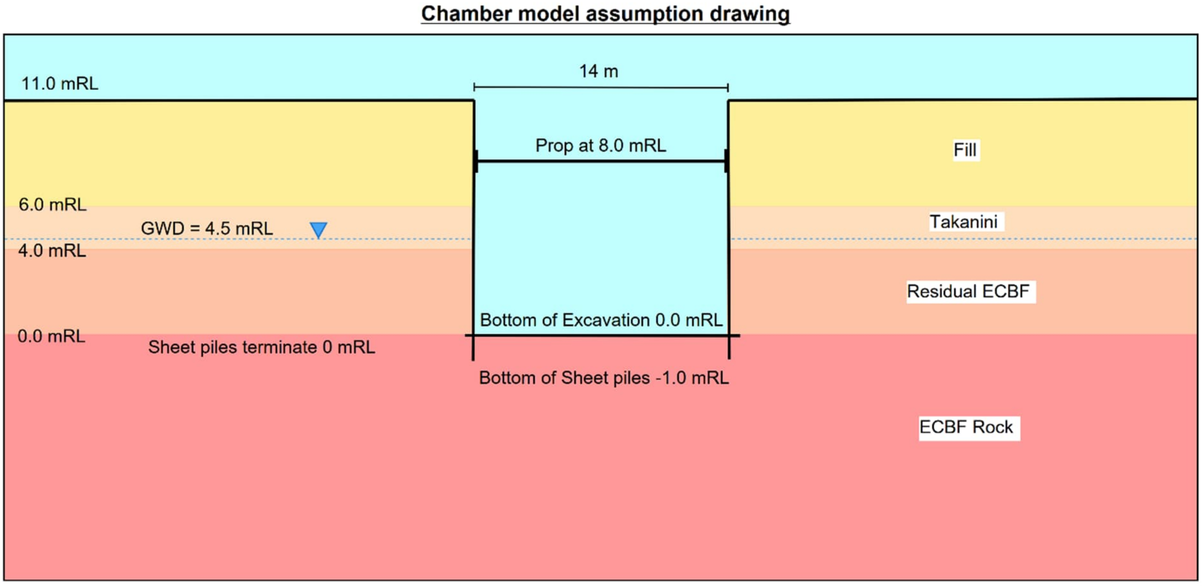


Figure 6.4: Chamber model

6.4.2 Structural properties of sheet pile

No structural details on the sheet piles have been provided at the time of this assessment. For the purpose of this assessment, and subject to contractor design, the following sheet pile retaining wall parameter assumptions have been adopted (similar to other CI sites):

- Sheet pile type adopted is STU2700 manufactured by Steel and Tube.
- Prop type adopted is 310UC manufactured by Steel and Tube.
- Sheet piles extend to the soil/rock interface (approximately 12 m long).
- The effective moment of inertia, I_{eff} is assumed to be 100% of the gross value under construction condition for the sheet piles.
- The effective moment of inertia, I_{eff} is assumed to be 50% of the gross value under construction condition for the props.
- An installation reduction factor has not been considered.
- Sheet piles extend to the soil/rock interface (approximately 12 m long).

[14] PLAXIS 2D: Version 20, PLAXIS BV, Delft, Netherlands

- The effective moment of inertia, I_{eff} is assumed to be 100% of the gross value under construction condition for the sheet piles.
- The effective moment of inertia, I_{eff} is assumed to be 50% of the gross value under construction condition for the props.
- An installation reduction factor has not been considered.

6.4.3 Construction sequence

The adopted construction sequence in our PLAXIS model is summarised below:

- Initial phase, K0 option used to generate initial stress status in the model.
- Activate plate structure to represent the installation of the perimeter sheet piles.
- Excavation stage 1, deactivate the soil in the foundation pit to represent the excavation to 4 m bgl.
- Install horizontal props at 3 m bgl.
- Complete excavation to the design level of -10 m RL (approximately 21 m bgl)

6.4.4 Other assumptions and analysis limitations

The following assumptions have been made as part of our assessment to simplify our models:

- "Soil Hardening" model is adopted in this assessment to capture the stiffness changes during excavation.
- Self-weight of the prop is not considered in the model however, it should be considered during detailed design. The propping arrangement may be altered and/or stiffeners added to compensate for these effects.
- Groundwater drawdown within the foundation pit is assumed to be completed immediately after the excavation.
- The deadman anchor is assumed to be rigid and fixed (i.e. will not deform or displace). Depending on the final propping configuration and loads, a cast in-situ beam in lieu of the deadman sheet pile may be required to limit deformations.

6.4.5 Results

Model results are presented for mechanical settlement on Figure 6.5 and Figure 6.6. The findings of the assessment are summarised below:

- The analysis indicated that under the construction methodology assumed, a maximum vertical ground settlement of 36 mm may occur at the edge of the control chamber.
- Settlement of up to 10 mm (generally considered less than minor effects by Auckland Council) is estimated to occur at 6 m from the chambers edge.
- The effects of mechanical settlement are estimated to be negligible beyond 12 m from the chamber's edge.

Settlement profiles have been combined with the calculated groundwater drawdown settlement and extrapolated onto a plan view as presented in Appendix E.

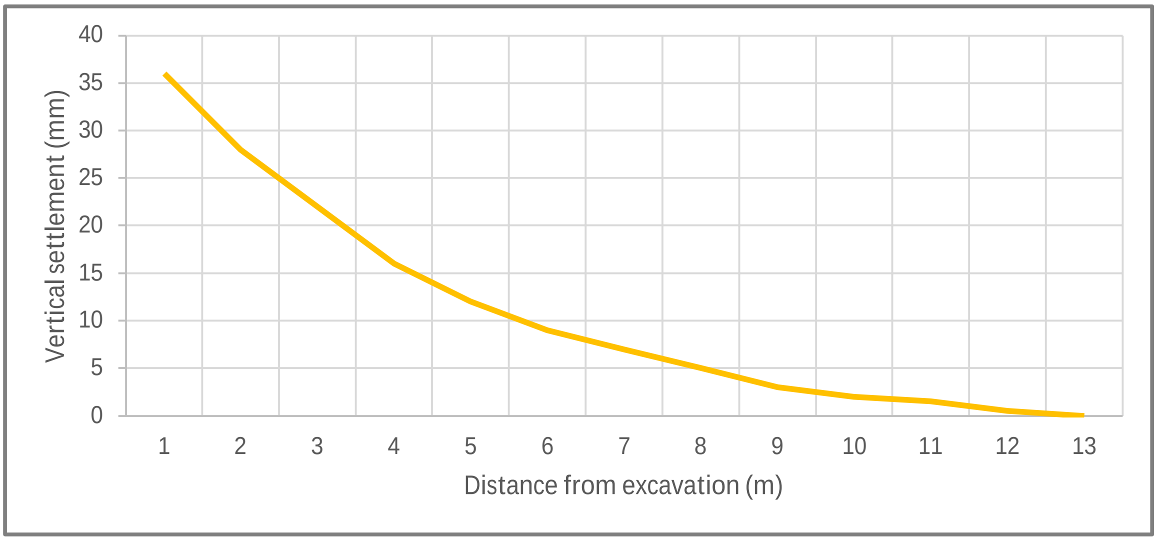


Figure 6.5: Control chamber mechanical settlement plot

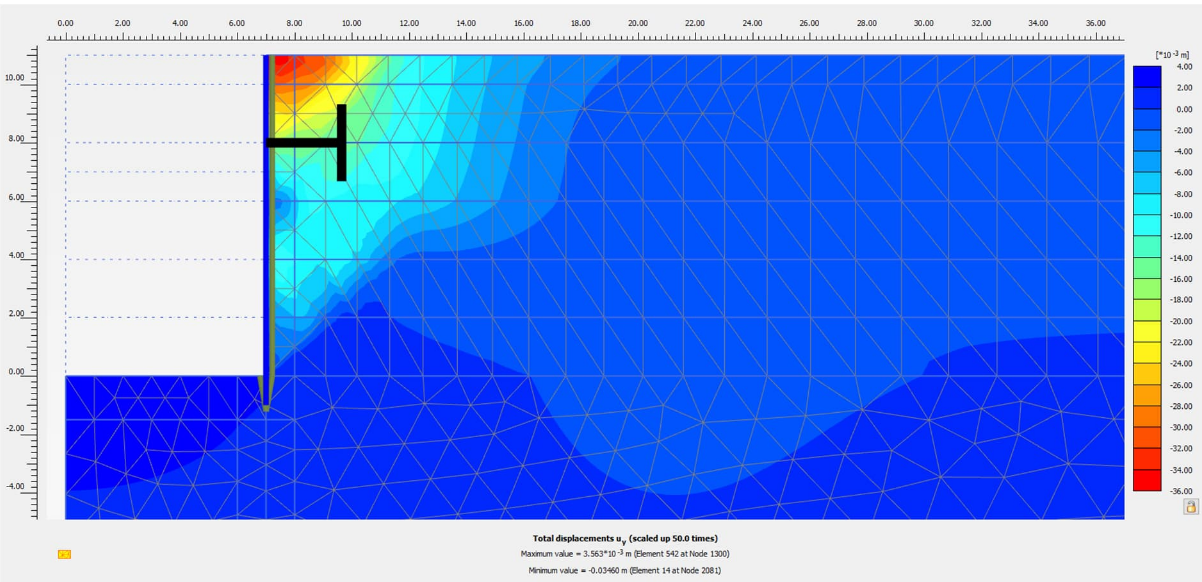


Figure 6.6: Control chamber mechanical settlement visualisation

6.5 Risk of damage to existing buildings, structures and services

The Burland (2012)[15] building damage correlation is commonly referred to when specifying trigger levels and damage risk classifications. We have therefore used this in our assessment and as the basis for preliminary trigger levels specified in Table 6.6 below (e.g. risk category 1) . However as it is generic criteria it does not capture more site-specific details, for example differences in the type and robustness of neighbouring buildings, pre-existing settlement/damage, and broader project risk implications (e.g. impact of construction delays if damage is observed, or the risk perception and acceptance of neighbouring building owners). Therefore the preliminary trigger levels currently specified in Table 6.6 below should be considered as a starting point and further assessment could

[15] Chapter 26 Building response to ground movements, John B.Burland, ICE manual of geotechnical engineering: Volume I. January 2012, 281-296

result in these values being updated (either raised or lowered) in the Groundwater Settlement Monitoring and Contingency Plan (GSMCP).

All dwellings and building structures are outside the proposed 5 mm settlement contour line assessed for the shaft and chamber excavations at Point Erin Park. This indicates that the risk of damage to these structures is very slight to negligible if adopting the Burland Classification (Table 6.6).

The only structures predicted to be subject to higher settlements are public assets located near the Point Erin Park south western construction area i.e. near the intersection of Curran and Sarsfield Streets. These are listed below in Table 6.5. These assets require specific detailed review and consultation with the asset owner. As such the specific type of damage that may occur cannot be qualified for these assets at this stage.

Table 6.5: Structures which may be subject to 10 mm or greater of settlement

Structure	Material	Asset Owner	Total Settlement (mm)	Differential Settlement (1V: x H, mm)
Road – Curran Street, Herne Bay	Sealed	Auckland Council	33	350
Stormwater pipe – gravity main (1050 mm internal diameter)[16]	Reinforced concrete	Auckland Council	35	335
Stormwater pipe – rising main (160 mm external diameter)[17]	Polyethylene	Auckland Council	35	350

Table 6.6: Burland 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 1mm.	
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.	

[16] Sarsfield overflow collector
[17] St Mary's Bay pressure line

Risk Category	Maximum settlement of building (mm)	Maximum differential settlement	Description of risk	General Category
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

7 Summary and key conclusions

This report identifies areas of the alignment where building, structures and/or services could be at risk to damage caused by ground settlement resulting from construction of the proposed tunnel and/or from construction of the terminal shaft and control chamber in Point Erin Park.

This assessment is based on published datasets and site specific available information at the time of writing and will be refined when the site specific data becomes available in March 2023, upon completion of the geotechnical investigations. While the results and conclusions of this assessment may change as ground conditions are validated with ground investigations and as designs are progressed, we consider that we have an appropriate level of information to assess the potential groundwater and settlement effects at a high level for the project and that this represents a conservative assessment.

A summary of our assessment is set out as follows:

Tunnel alignment

- Two conservative scenarios were considered for the tunnel assessment, with an overarching conservatism in that all of the assumptions are required to be valid simultaneously for the drawdown presented to eventuate (which is unlikely). Hence this assessment is considered to represent a broad envelope of effects that will be refined once the geotechnical data is available.
- Scenario 1 results show maximum drawdown of 3 m at the tunnel axis and a 9 m zone of influence. Scenario 1 results show maximum drawdown-induced settlement of 7.5 mm at the tunnel axis.
- Scenario 2 results show maximum drawdown of approximately 10m at the tunnel axis and a 50 m zone of influence. Scenario 2 results show maximum drawdown-induced settlement of 26 mm at the tunnel axis.
- The maximum differential settlement assessed from both scenarios is 1:300 within 5 m of the tunnel axis (Scenario 2). The maximum distance at which settlement is less than 10 mm is 6 m from the tunnel axis (Scenario 2).
- Surface ground settlements results from mechanical settlement are expected to be less than 8 mm with differential settlement less than 1V:1300H for a tunnel centreline depth of 12 m which is expected to be the shallowest section of the alignment which is under Point Erin Park. The remainder of the alignment where it traverses beneath properties is generally at a minimum depth of 18 m at the northern end of the alignment and 30 m at the southern end of the alignment, and up to around 60 m towards the centre of the alignment under the Jervois Road ridge.

We expect this level of movement is within the natural seasonal fluctuations of the ground. We also note that the calculated settlement is based on a long term permanent stress change that occurs over a period of time. While typical durations of full consolidation for this type of material is in the order of weeks to months, the TBM transits under any one property in typically 2 days and at any one time only has 12.5 m of section open. Hence we anticipate that actual settlement during construction will be less than assessed above.

Furthermore, we understand the TBM may be operated in “closed mode” (i.e. no drawdown of water) if required to ensure appropriate settlement levels are complied with. Tunnel settlement arrays may be used to assess this behaviour during construction, and to manage the potential risk of damage to properties to within levels considered to be “less than minor” (i.e. Category 1 on the Burland Classification presented above).

Terminal shaft

- The modelled drawdown is generally even in all directions around the proposed shaft excavation. Maximum groundwater drawdown levels of up to 2.5 m are predicted next to the excavations, reducing to less than 0.5 m at approximately 25 m distance, and to near zero to within 100 m distance.
- Modelled groundwater induced ground settlement immediately outside the excavation ranges from approximately 5 mm to 8 mm and reduces towards zero at increasing distance from the excavation. We expect this level of movement is within the natural seasonal fluctuations of the ground.
- Based on previous assessments of secant piles shafts adopting a similar methodology and which are of similar depth in a similar geological setting, circular secant pile shafts result in very limited mechanical settlement (less than 5 mm at the edge of excavation). The effects of this are negligible.

Control chamber

- The modelled drawdown is generally even in all directions around the proposed control chamber excavation. Maximum groundwater drawdown levels of up to 2.8 m are predicted next to the excavations, reducing to less than 0.5 m at approximately 25 m distance, and to near zero to within 100 m distance.
- Modelled groundwater induced ground settlement immediately outside the excavation ranges from approximately 4 mm to 5 mm and reduces towards zero at increasing distance from the excavation. As above, we expect this level of movement is within the natural seasonal fluctuations of the ground.
- Based on the proposed construction methodology (sheet piling), a maximum vertical ground settlement of 36 mm can be expected at the edge of the control chamber. Settlement of up to 10 mm (Category 1 on the Burland Classification, generally considered less than minor effects by Auckland Council) is estimated to occur at 6 m from the edge of the chamber. The effects of mechanical settlement are estimated to be negligible beyond 12 m from the chamber's edge.
- The use of an alternative construction methodology (e.g. secant piles) will result in effects within (less than) the envelope of those assessed above.

Terminal shaft and control chamber in Point Erin Park: Risk of damage to existing buildings, structures and services

- All dwellings and building structures are outside the proposed 5 mm settlement contour line for the works being undertaken in Point Erin Park, indicating that the risk of damage to these structures is negligible based on the Burland Classification.
- The only structures predicted to be subject to higher settlements are public assets located near the corner of Curran and Sarsfield Streets. Healthy Waters has provided written approval and therefore, the only asset that requires detailed review and consultation with the asset owner is the road adjacent to the south-western construction area.
- Adopting the Burland Classification for risk of damage, the combined settlement effects associated with the terminal shaft is assessed to be within categories 0 to 1 (negligible to very slight).

Our initial assessment summarised above indicates that the ground settlement and dewatering effects arising for the tunnel, and shaft and chamber excavations can be managed to within levels typically accepted by Auckland Council, provided standard 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 consider this can be achieved through:

Tunnel from Tawariki Street to Point Erin Park

- A management plan that incorporates sufficient groundwater monitoring to assess groundwater drawdown impacts upon aquifers within the compressible materials and tunnel settlement array along the tunnel alignment to monitor effects. If monitoring results indicate a likely exceedance of the consented trigger levels, the Contractor can operate the TBM in "closed mode" ceasing groundwater drawdown associated with tunnelling.

Point Erin Park shaft excavation[18]

- A contingency and management plan which includes measures to quickly address any defects in the secant pile wall which may result in significant leakage if groundwater drawdown is being observed. 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.

Control chamber

- A contingency and management plan including measures to add additional propping if required to manage effects to within consented levels.

[18] And control chamber if a secant pile construction methodology is used.

8 Proposed monitoring

Ground settlement and groundwater drawdown monitoring during the construction works will be undertaken to assess if the response of the surrounding buildings and structures is within acceptable tolerances of damage risk. This process allows for the geotechnical effects to be monitored and can act as a trigger for mitigation measure to be implemented if required.

The purpose of the monitoring programme is to monitor actual settlements and establish alert and alarm levels below levels that can be expected to result in the onset of minor damage to structures under worst-case assumptions. Predicted settlements at monitored structures in many instances are too small to accurately measure and well below the threshold of damage. As such, we recommend that the potential for the onset of minor damage (Burland Risk Classification of 1) under worst-case assumptions equates to the Alarm Trigger Level, and the Alert Trigger Level is set at 80% of the Alarm for ground deformations. These preliminary trigger levels can then be reviewed and confirmed through the GSMCP.

Monitoring and surveying of the following is recommended:

- Building and Ground Settlement Monitoring Points via survey markers.
- Groundwater level monitoring via standpipe piezometers.
- Tunnel settlement survey arrays spaced every 500 m along the length of the tunnel.

A proposed monitoring plan identifying monitoring type and locations is presented in Appendix F.

Furthermore, baseline monitoring or a review of InSAR data can be undertaken to further understand the level of natural seasonal fluctuations of the ground and structures proposed to be monitored during construction.

9 Applicability

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

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.

Tonkin & Taylor Ltd

Report prepared by:	Report reviewed by:
Kevin Ledwith	
....	....
Kevin Ledwith Hydrogeologist	Eduard Mandru Geotechnical Engineer

Authorised for Tonkin & Taylor Ltd by:

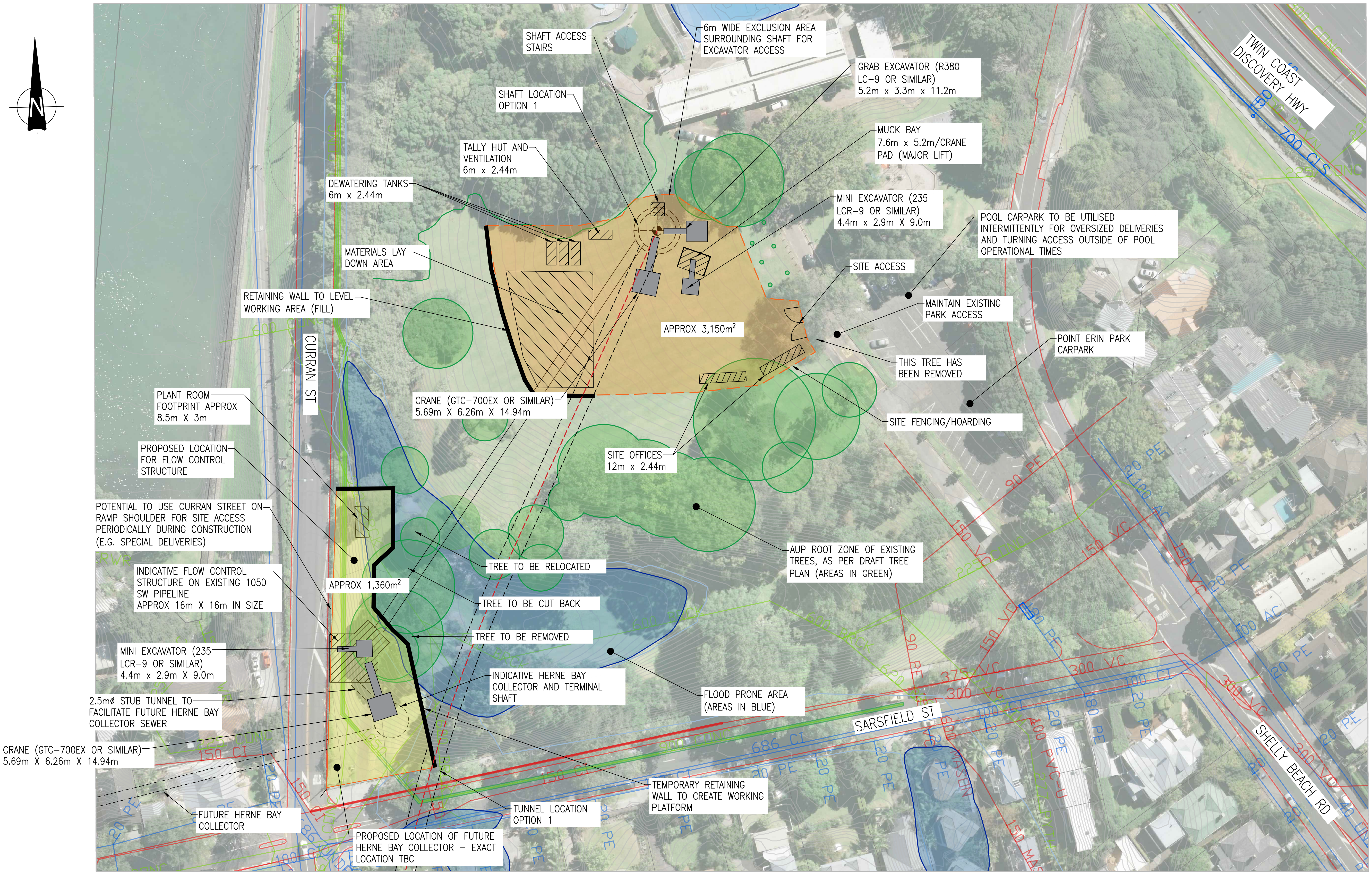
KL Baverstock

....

Karen Baverstock
Project Director

7-Feb-23
p:\30552\30552.9081 ci extension point erin - consenting\workingmaterial\geotechnical\reporting\stage1_v2 report\07.02.23 ci extension - point erin tunnel - stage 1 groundwater assessment (002) - report.docx`

Appendix A Project Figures and Plans provided by Watercare Service Limited



PLAN
SCALE. 1:500

SCALE 1:500 (A1) 10 8 6 4 2 0 10 20 30 40 50m

SCALE 1:2500 (A1) 50 25 0 50 100 150 200 250m

PRELIMINARY

FOR INFORMATION

JACOBS AECOM McMILLEN JACOBS ASSOCIATES

ISSUE	DATE	AMENDMENT
A		ISSUED FOR INFORMATION

Watercare

CENTRAL INTERCEPTOR (DSCIN)
POINT ERIN TERMINAL SHAFT AND NETWORK CONNECTIONS
PRELIMINARY SITE LAYOUT

CAD FILE DSCIN-DEL-SKT-C-J-300010.DWG
REF No.
ORIGINAL SCALE A1 AS SHOWN
SKETCH No. DSCIN-DEL-SKT-C-J-30015
ISSUE A

Plot Date: Nov 04, 2022 - 10:46am C:\pw_work\anthony.doherty@jacobs.com\dms51944\DSCIN-DEL-SKT-C-J-300010.dwg

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SCALE 1:2500 (A1) 50 25 0 50 100 150 200 250m

PRELIMINARY

FOR INFORMATION

JACOBS AECOM McMILLEN JACOBS ASSOCIATES

ISSUE	DATE	AMENDMENT
A		ISSUED FOR INFORMATION

Watercare

CENTRAL INTERCEPTOR (DSCIN)

POINT ERIN TERMINAL SHAFT AND NETWORK CONNECTIONS

PRELIMINARY TUNNEL ALIGNMENT

DRAFT

CAD FILE DSCIN-DEL-SKT-C-J-300010.DWG

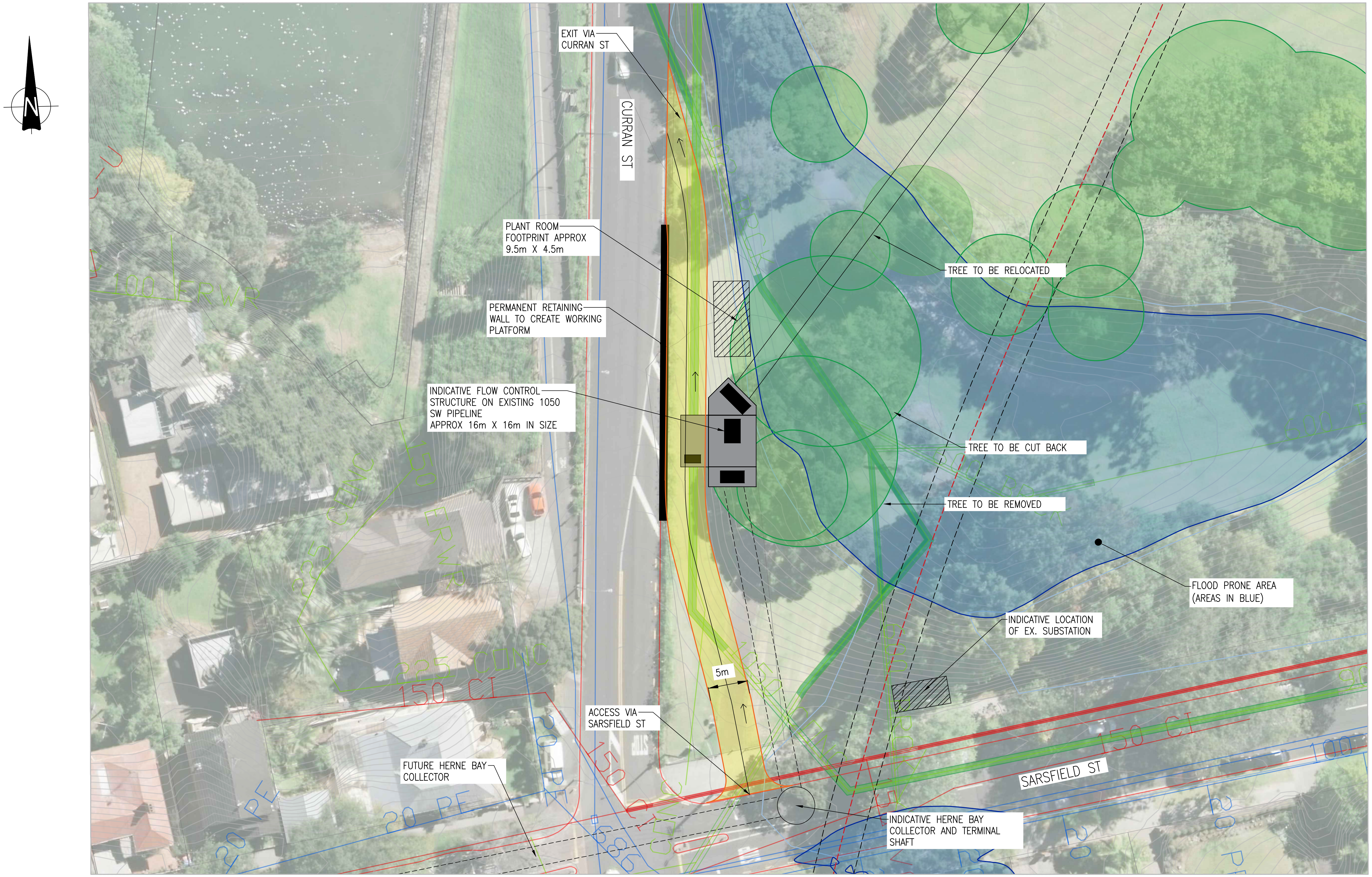
REF No.

ORIGINAL SCALE A1 AS SHOWN

SKETCH No. DSCIN-DEL-SKT-C-J-30016

ISSUE A

Plot Date: Nov 04, 2022 - 11:00am C:\pw_work\anthony.doherty@jacobs.com\dms51944\ DSCIN-DEL-SKT-C-J-300010.dwg



PLAN
SCALE. NTS

SCALE 1:500 (A1) 10 8 6 4 2 0 10 20 30 40 50m

SCALE 1:2500 (A1) 50 25 0 50 100 150 200 250m

PRELIMINARY
FOR INFORMATION

JACOBS AECOM McMILLEN JACOBS ASSOCIATES

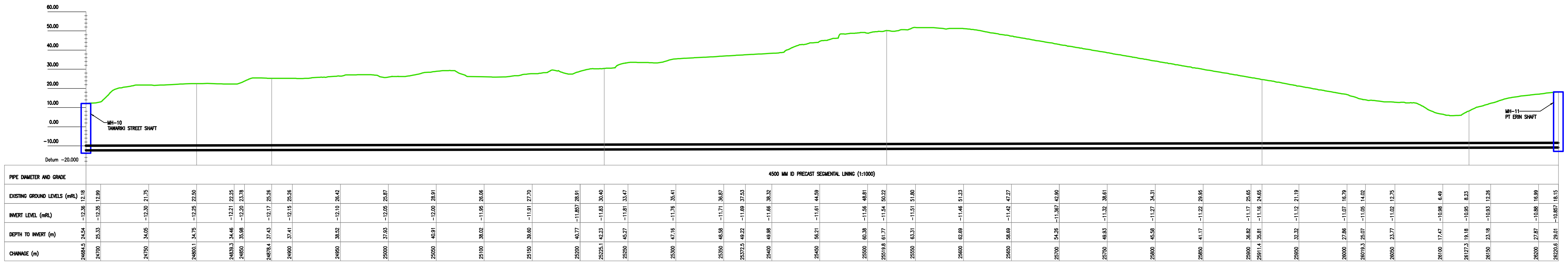
ISSUE	DATE	AMENDMENT
A		ISSUED FOR INFORMATION

Watercare

CENTRAL INTERCEPTOR (DSCIN)
POINT ERIN TERMINAL SHAFT AND NETWORK CONNECTIONS
PRELIMINARY SITE LAYOUT

CAD FILE DSCIN-DEL-SKT-C-J-300010.DWG
REF No.
ORIGINAL SCALE A1 AS SHOWN
SKETCH No. DSCIN-DEL-SKT-C-J-30017
ISSUE A

Plot Date: Nov 23, 2022 - 1:10pm C:\pw_work\anthony.doherty@jacobs.com\dms51944\DSCIN-DEL-SKT-C-J-300010.dwg



LONGITUDINAL SECTION
SCALE. 1:2500H 1:500V

SCALE 1:2500 (A1) 50 25 0 50 100 150 200 250m

SCALE 1:500 (A1) 10 8 6 4 2 0 10 20 30 40 50m

PRELIMINARY

FOR INFORMATION

JACOBS AECOM McMILLEN JACOBS ASSOCIATES

ISSUE	DATE	AMENDMENT
A		ISSUED FOR INFORMATION

Watercare

CENTRAL INTERCEPTOR (DSCIN)
82 GRAVITY SEWER INCLUDING MANHOLES
TAWARIKI ST TO PT ERIN PRELIMINARY TUNNEL LONGITUDINAL SECTION

DRAFT

CAD FILE DSCIN-DEL-SKT-C-J-300020.DWG

REF No.

ORIGINAL SCALE A1 AS SHOWN

SKETCH No. DSCIN-DEL-SKT-C-J-30020

ISSUE A

Appendix B Leapfrog Geological Model

TT LeapFrog model XsecID	Distance (left to right)	ECBF rock contact (m RL)	Elevation ground level (m RL)	Depth to ECBF rock (m)
A	0	6.8	12.4	5.6
A	40	5.1	16.4	11.3
A	80	9.0	21.2	12.2
A	120	12.3	22.0	9.7
A	160	13.5	22.0	8.5
B	0	13.6	22.0	8.4
B	40	14.2	24.2	10.0
B	80	14.2	25.0	10.8
B	120	14.8	26.9	12.1
B	160	16.2	26.0	9.8
B	200	17.1	28.0	10.9
C	0	28.3	33.3	5.0
C	40	26.7	32.7	6.0
C	80	24.9	29.3	4.4
C	120	22.8	28.3	5.5
C	160	20.5	26.0	5.5
C	200	18.7	26.0	7.3
C	240	17.3	28.4	11.1
D	0	35.7	46.3	10.6
D	40	32.8	38.6	5.8
D	80	31.3	37.0	5.7
D	120	29.8	35.6	5.8
D	160	28.3	33.3	5.0
E	0	36.2	47.2	11.0
E	40	38.4	49.3	10.9
E	80	39.9	51.0	11.1
E	120	40.4	51.3	10.9
E	160	40.1	50.0	9.9
E	200	38.4	47.0	8.6
E	240	36.1	43.5	7.4
E	260	34.9	41.7	6.8
F	0	34.2	41.0	6.8
F	40	31.6	37.4	5.8
F	80	28.8	34.0	5.2
F	120	23.7	30.5	6.8
F	160	18.2	27.0	8.8
F	200	13.9	23.6	9.7
F	240	9.4	19.6	10.2
F	260	7.4	18.0	10.6
G	0	5.6	16.4	10.8
G	40	1.8	12.5	10.7
G	80	0.6	10.9	10.3
G	120	-0.5	6.0	6.5
G	160	1.8	13.5	11.7
G	200	5.8	16.7	10.9
G	220	7.2	17.6	10.4

Depth to ECBF rock (m) stats	
min=	4.4
max=	12.2

ASSUMED MIN 50m STRAIGHT LENGTH OF TUNNEL DIRECTLY DOWNSTREAM OF PROPOSED POINT ERIN SHAFT

PRELIMINARY POINT ERIN SHAFT LOCATION

PLAN
SCALE. 1:125

SCALE 1:500 (A1) 10 8 6 4 2 0 10 20 30 40 50m

SCALE 1:2500 (A1) 50 25 0 50 100 150 200 250m

PRELIMINARY
FOR INFORMATION

JACOBS AECOM McMILLEN JACOBS ASSOCIATES

ISSUE	DATE	AMENDMENT
A		ISSUED FOR INFORMATION

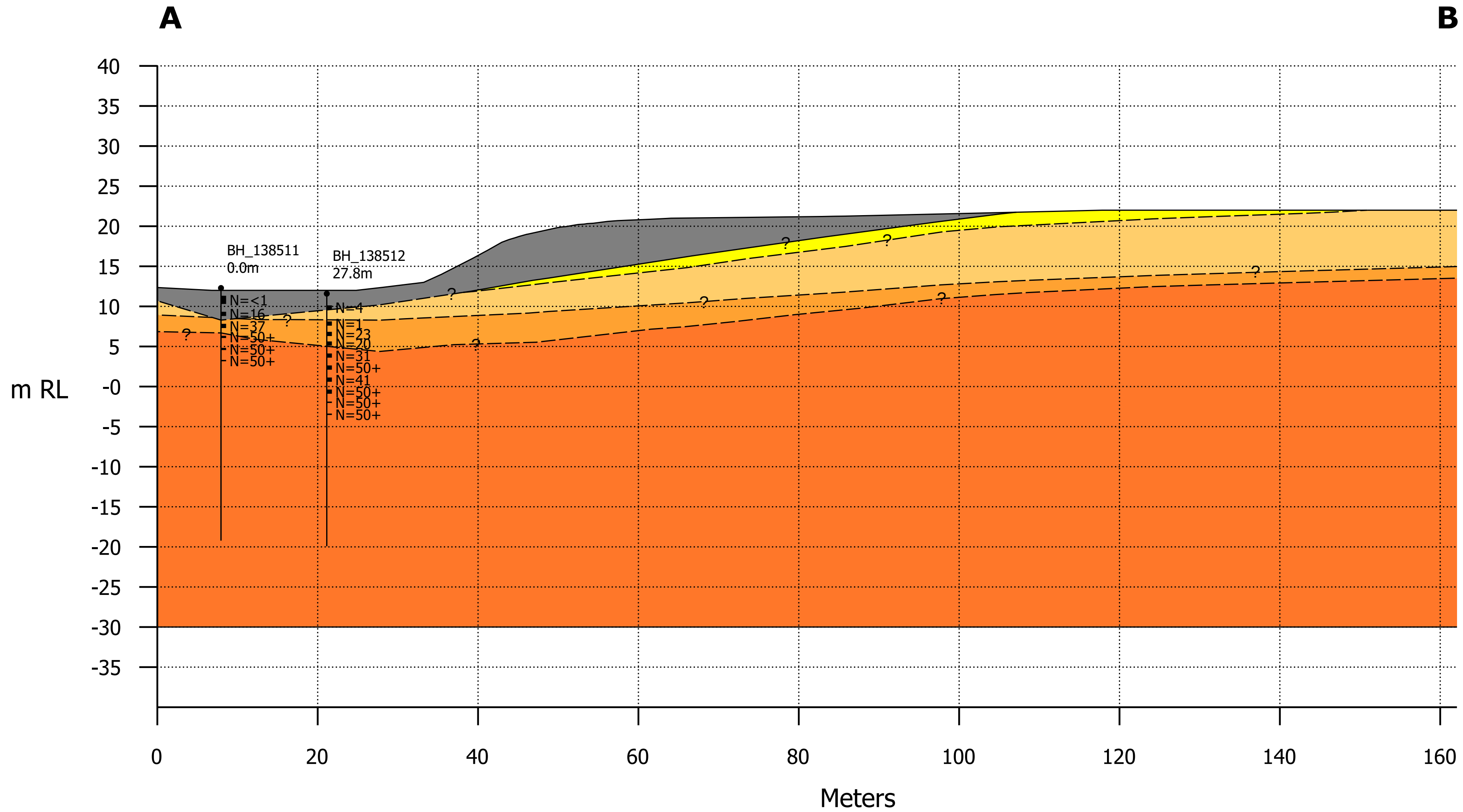
Watercare

CENTRAL INTERCEPTOR (DSCIN)
POINT ERIN TERMINAL SHAFT AND NETWORK CONNECTIONS
PRELIMINARY TUNNEL ALIGNMENT

DRAFT

CAD FILE DSCIN-DEL-SKT-C-J-300010.DWG
REF No.
ORIGINAL SCALE A1 AS SHOWN
SKETCH No. DSCIN-DEL-SKT-C-J-30016
ISSUE A

Pt Erin_Cross Section_A



NOTES:
1. ALL DIMENSIONS ARE IN METERS UNLESS NOTED OTHERWISE.
2. EXISTING GROUND PROFILE BASED ON 1M LIDAR SOURCED FROM OPENTOPOGRAPHY.ORG (LIDAR CAPTURED FOR AUCKLAND COUNCIL BY AERIAL SURVEYS BETWEEN 2016 AND 2018).
3. GEOLOGICAL BOUNDARIES ARE INFERRED ONLY AND BASED ON THE INTERPRETATION OF GEOLOGICAL INVESTIGATIONS AT DISCRETE LOCATIONS. VARIATION OF ACTUAL GEOLOGICAL BOUNDARIES AND GROUND CONDITIONS BETWEEN INVESTIGATION LOCATIONS SHOULD BE EXPECTED.
4. GEOLOGICAL CONDITIONS BENEATH THE BASE OF GEOLOGICAL INVESTIGATIONS ARE NOT KNOWN.
5. BOREHOLE OFFSET FROM CROSS SECTION ALIGNMENT STATED BENEATH INVESTIGATION ID ON CROSS SECTION IE. '27.8m' REFERS TO 27.8M HORIZONTAL OFFSET FROM CROSS SECTION ALIGNMENT.

Legend

TT_Pt Erin_Leapfrog model_20230120

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

Location

A: 1754811, 5920085

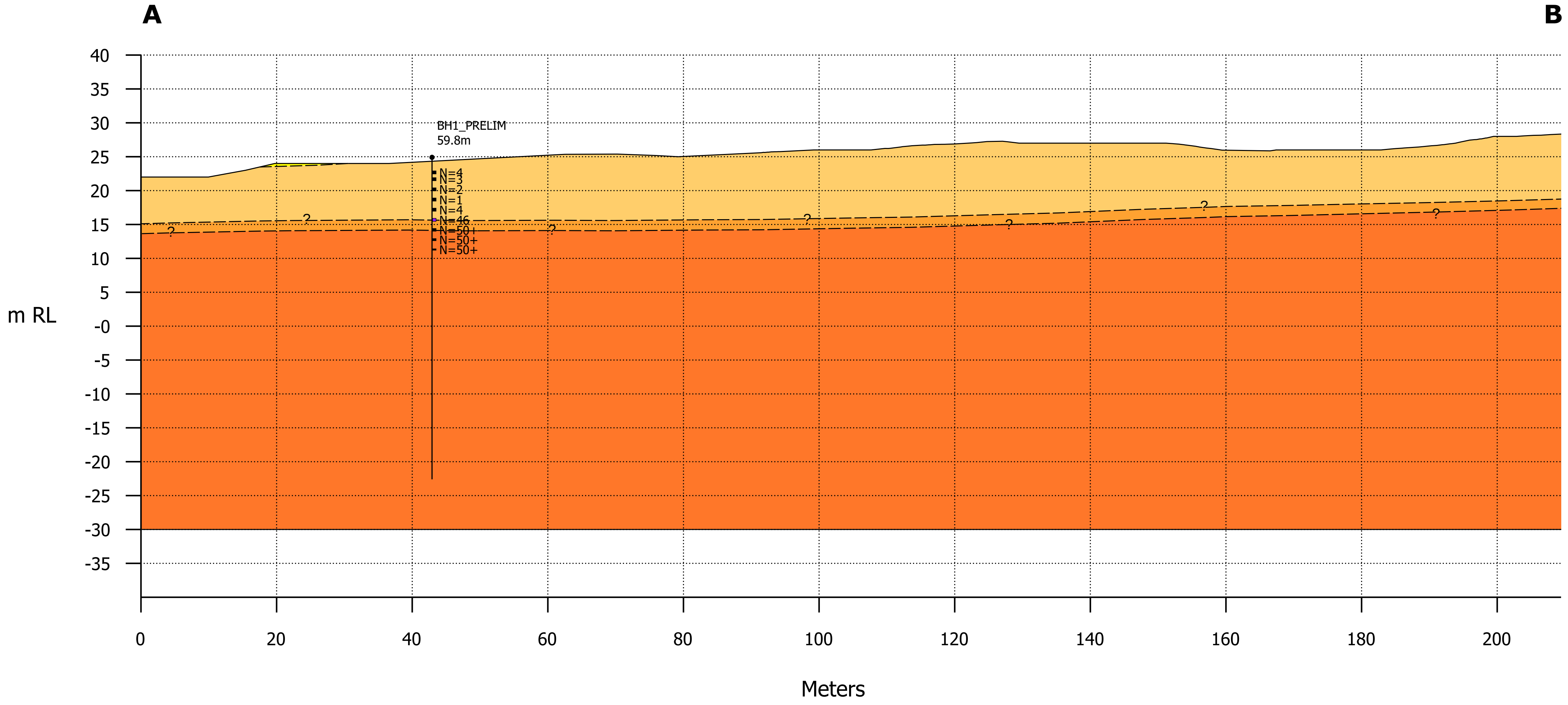
B: 1754872, 5920235

Scale: 1:750

Vertical exaggeration: 1x

0m 50m

Pt Erin_Cross Section_B



NOTES:
1. ALL DIMENSIONS ARE IN METERS UNLESS NOTED OTHERWISE.
2. EXISTING GROUND PROFILE BASED ON 1M LIDAR SOURCED FROM OPENTOPOGRAPHY.ORG (LIDAR CAPTURED FOR AUCKLAND COUNCIL BY AERIAL SURVEYS BETWEEN 2016 AND 2018).
3. GEOLOGICAL BOUNDARIES ARE INFERRED ONLY AND BASED ON THE INTERPRETATION OF GEOLOGICAL INVESTIGATIONS AT DISCRETE LOCATIONS. VARIATION OF ACTUAL GEOLOGICAL BOUNDARIES AND GROUND CONDITIONS BETWEEN INVESTIGATION LOCATIONS SHOULD BE EXPECTED.
4. GEOLOGICAL CONDITIONS BENEATH THE BASE OF GEOLOGICAL INVESTIGATIONS ARE NOT KNOWN.
5. BOREHOLE OFFSET FROM CROSS SECTION ALIGNMENT STATED BENEATH INVESTIGATION ID ON CROSS SECTION IE. '59.8m' REFERS TO 59.8M HORIZONTAL OFFSET FROM CROSS SECTION ALIGNMENT.

Legend

TT_Pt Erin_Leapfrog model_20230120

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

Location

A: 1754874, 5920239

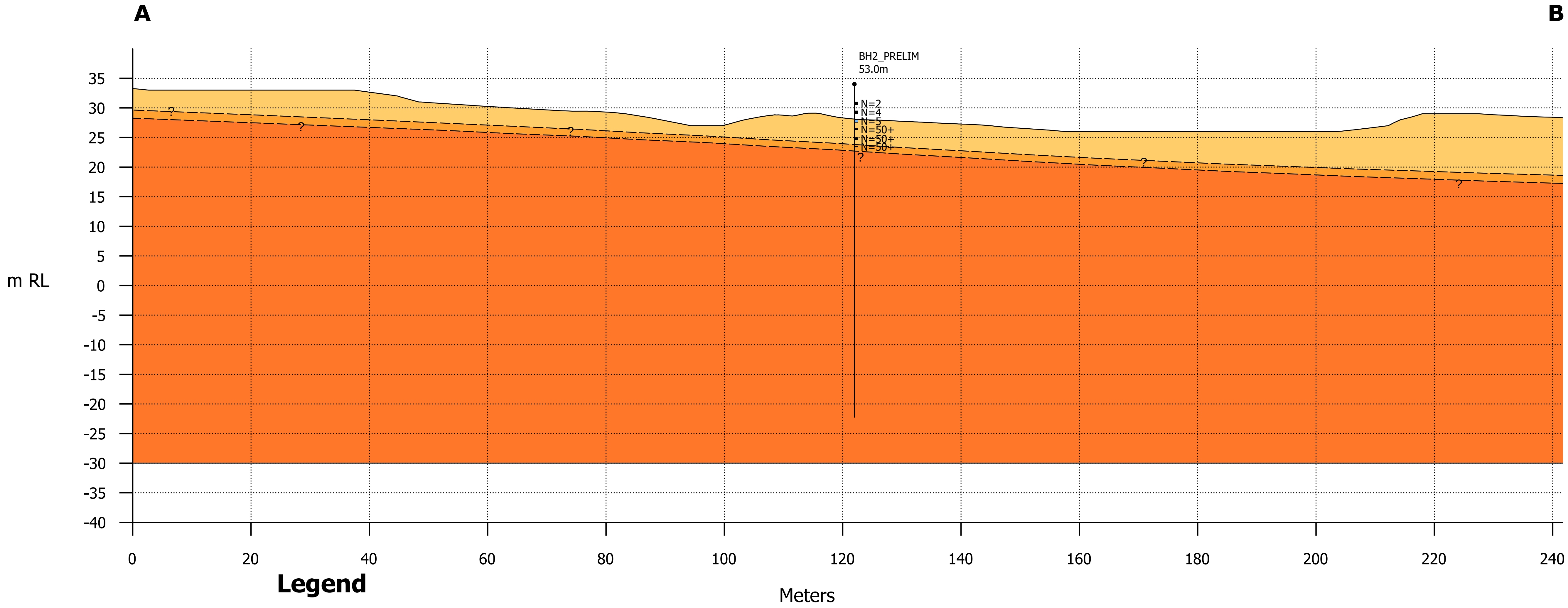
B: 1754993, 5920411

Scale: 1:750

Vertical exaggeration: 1x

0m 50m

Pt Erin_Cross Section C



Legend

TT_Pt Erin_Leapfrog model_20230120

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

NOTES:
1. ALL DIMENSIONS ARE IN METERS UNLESS NOTED OTHERWISE.
2. EXISTING GROUND PROFILE BASED ON 1M LIDAR SOURCED FROM OPENTOPOGRAPHY.ORG (LIDAR CAPTURED FOR AUCKLAND COUNCIL BY AERIAL SURVEYS BETWEEN 2016 AND 2018).
3. GEOLOGICAL BOUNDARIES ARE INFERRED ONLY AND BASED ON THE INTERPRETATION OF GEOLOGICAL INVESTIGATIONS AT DISCRETE LOCATIONS. VARIATION OF ACTUAL GEOLOGICAL BOUNDARIES AND GROUND CONDITIONS BETWEEN INVESTIGATION LOCATIONS SHOULD BE EXPECTED.
4. GEOLOGICAL CONDITIONS BENEATH THE BASE OF GEOLOGICAL INVESTIGATIONS ARE NOT KNOWN.
5. BOREHOLE OFFSET FROM CROSS SECTION ALIGNMENT STATED BENEATH INVESTIGATION ID ON CROSS SECTION IE. '53.0m' REFERS TO 53.0 M HORIZONTAL OFFSET FROM CROSS SECTION ALIGNMENT.

Location

A: 1755132, 5920609

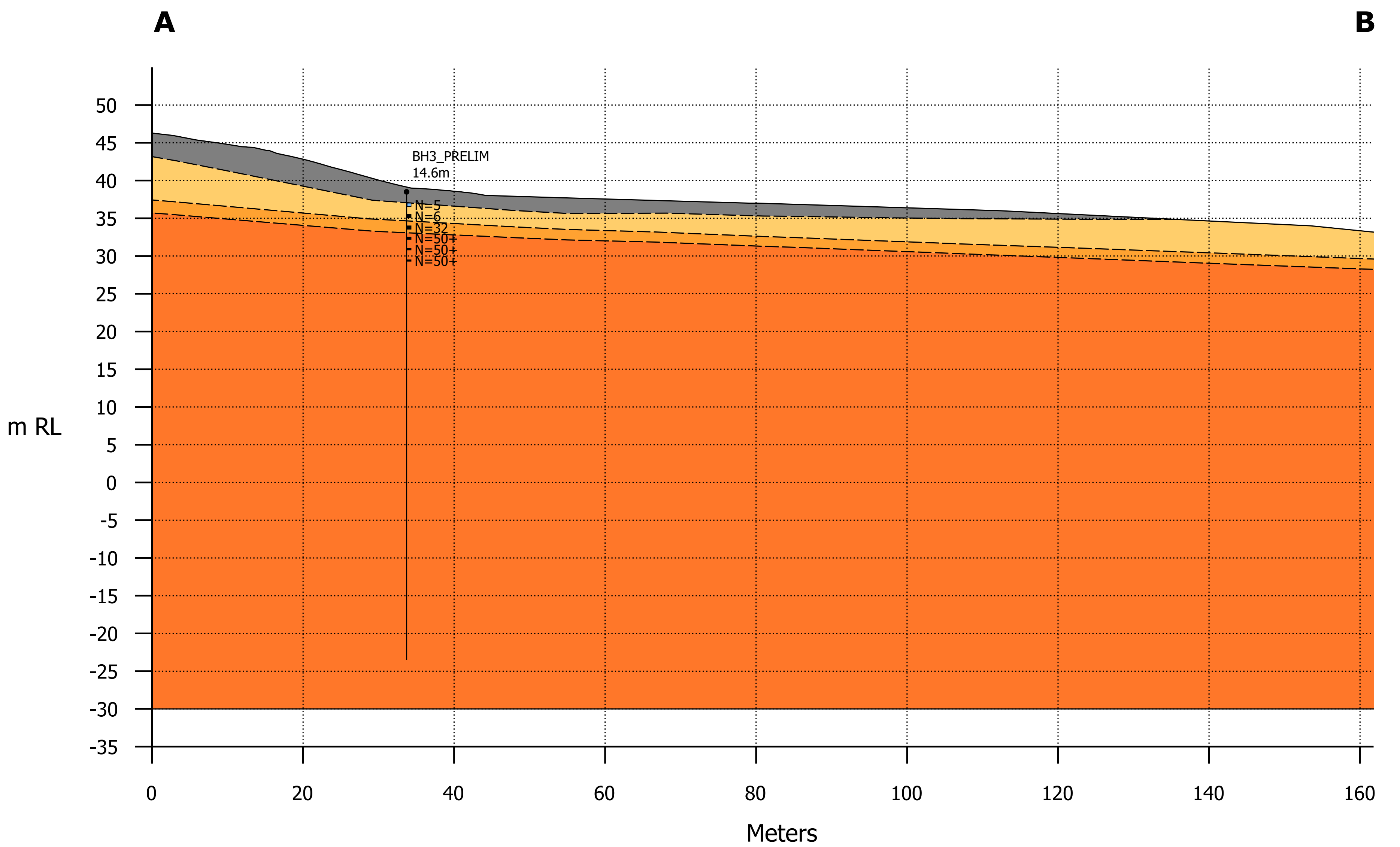
B: 1754993, 5920411

Scale: 1:750

Vertical exaggeration: 1x

0m 50m

Pt Erin_Cross Section_D



NOTES:
1. ALL DIMENSIONS ARE IN METERS UNLESS NOTED OTHERWISE.
2. EXISTING GROUND PROFILE BASED ON 1M LIDAR SOURCED FROM OPENTOPOGRAPHY.ORG (LIDAR CAPTURED FOR AUCKLAND COUNCIL BY AERIAL SURVEYS BETWEEN 2016 AND 2018).
3. GEOLOGICAL BOUNDARIES ARE INFERRED ONLY AND BASED ON THE INTERPRETATION OF GEOLOGICAL INVESTIGATIONS AT DISCRETE LOCATIONS. VARIATION OF ACTUAL GEOLOGICAL BOUNDARIES AND GROUND CONDITIONS BETWEEN INVESTIGATION LOCATIONS SHOULD BE EXPECTED.
4. GEOLOGICAL CONDITIONS BENEATH THE BASE OF GEOLOGICAL INVESTIGATIONS ARE NOT KNOWN.
5. BOREHOLE OFFSET FROM CROSS SECTION ALIGNMENT STATED BENEATH INVESTIGATION ID ON CROSS SECTION IE. '14.6m' REFERS TO 14.6M HORIZONTAL OFFSET FROM CROSS SECTION ALIGNMENT.

Legend

TT_Pt Erin_Leapfrog model_20230120

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

Location

A: 1755179, 5920763

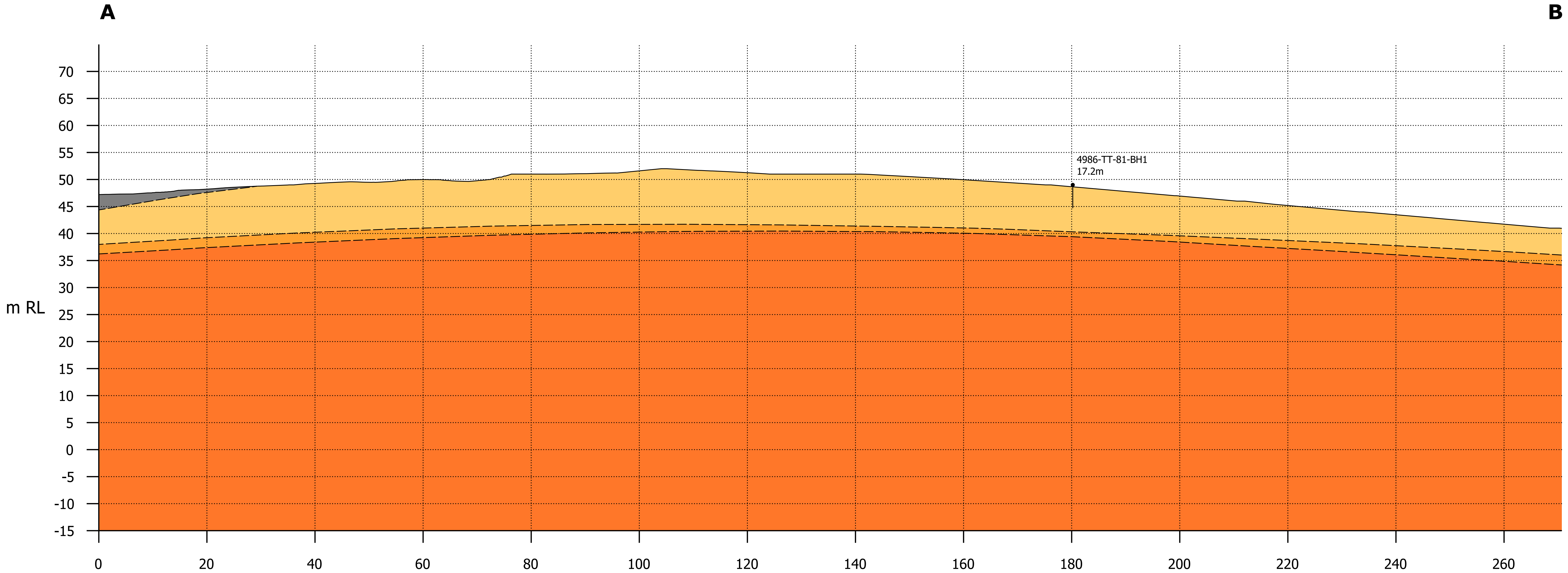
B: 1755132, 5920608

Scale: 1:750

Vertical exaggeration: 1x

0m 50m

Pt Erin_Cross Section_E



NOTES:
1. ALL DIMENSIONS ARE IN METERS UNLESS NOTED OTHERWISE.
2. EXISTING GROUND PROFILE BASED ON 1M LIDAR SOURCED FROM OPENTOPOGRAPHY.ORG (LIDAR CAPTURED FOR AUCKLAND COUNCIL BY AERIAL SURVEYS BETWEEN 2016 AND 2018).
3. GEOLOGICAL BOUNDARIES ARE INFERRED ONLY AND BASED ON THE INTERPRETATION OF GEOLOGICAL INVESTIGATIONS AT DISCRETE LOCATIONS. VARIATION OF ACTUAL GEOLOGICAL BOUNDARIES AND GROUND CONDITIONS BETWEEN INVESTIGATION LOCATIONS SHOULD BE EXPECTED.
4. GEOLOGICAL CONDITIONS BENEATH THE BASE OF GEOLOGICAL INVESTIGATIONS ARE NOT KNOWN.
5. BOREHOLE OFFSET FROM CROSS SECTION ALIGNMENT STATED BENEATH INVESTIGATION ID ON CROSS SECTION IE. '17.2m' REFERS TO 17.2M HORIZONTAL OFFSET FROM CROSS SECTION ALIGNMENT.

Legend

TT_Pt Erin_Leapfrog model_20230120

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

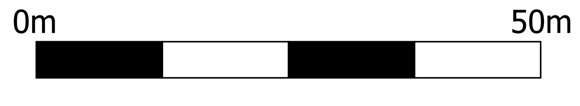
Location

A: 1755181, 5920769

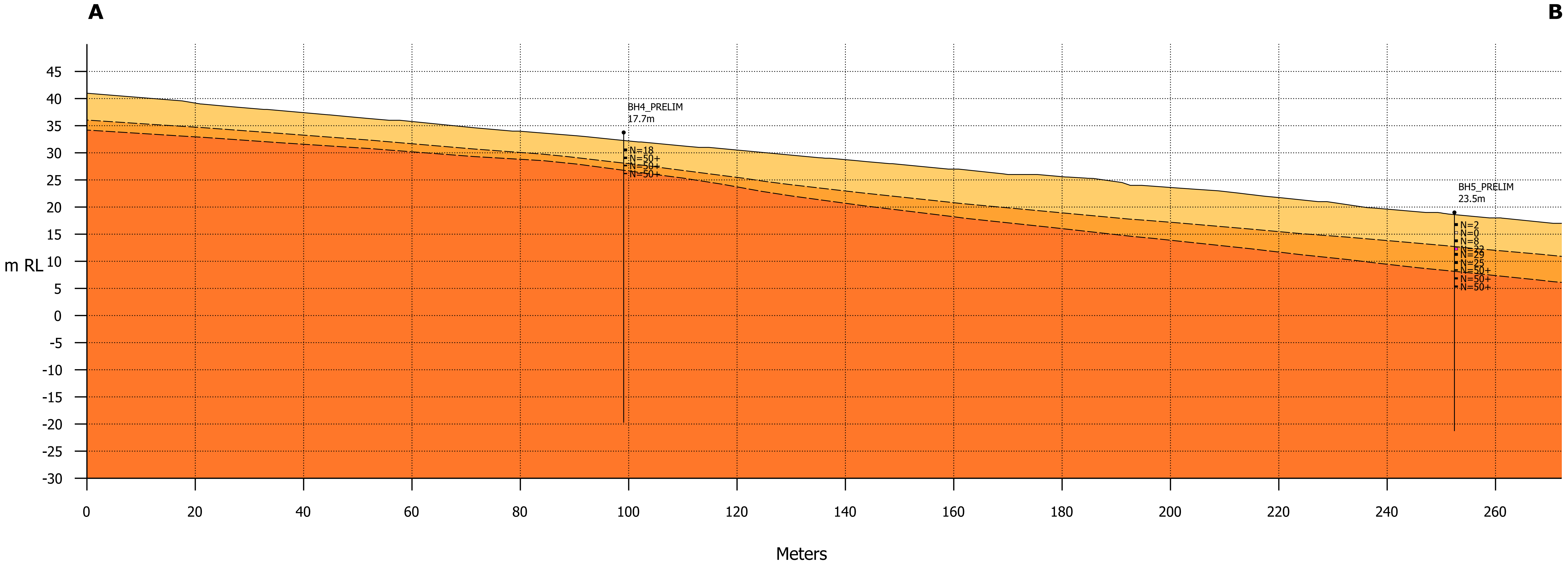
B: 1755151, 5921038

Scale: 1:750

Vertical exaggeration: 1x



Pt Erin_Cross Section F



NOTES:
1. ALL DIMENSIONS ARE IN METERS UNLESS NOTED OTHERWISE.
2. EXISTING GROUND PROFILE BASED ON 1M LIDAR SOURCED FROM OPENTOPOGRAPHY.ORG (LIDAR CAPTURED FOR AUCKLAND COUNCIL BY AERIAL SURVEYS BETWEEN 2016 AND 2018).
3. GEOLOGICAL BOUNDARIES ARE INFERRED ONLY AND BASED ON THE INTERPRETATION OF GEOLOGICAL INVESTIGATIONS AT DISCRETE LOCATIONS. VARIATION OF ACTUAL GEOLOGICAL BOUNDARIES AND GROUND CONDITIONS BETWEEN INVESTIGATION LOCATIONS SHOULD BE EXPECTED.
4. GEOLOGICAL CONDITIONS BENEATH THE BASE OF GEOLOGICAL INVESTIGATIONS ARE NOT KNOWN.
5. BOREHOLE OFFSET FROM CROSS SECTION ALIGNMENT STATED BENEATH INVESTIGATION ID ON CROSS SECTION IE. '17.7m' REFERS TO 17.7M HORIZONTAL OFFSET FROM CROSS SECTION ALIGNMENT.

Legend

TT_Pt Erin_Leapfrog model_20230120

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

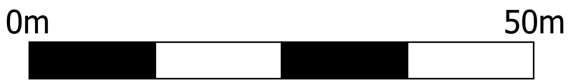
Location

A: 1755151, 5921038

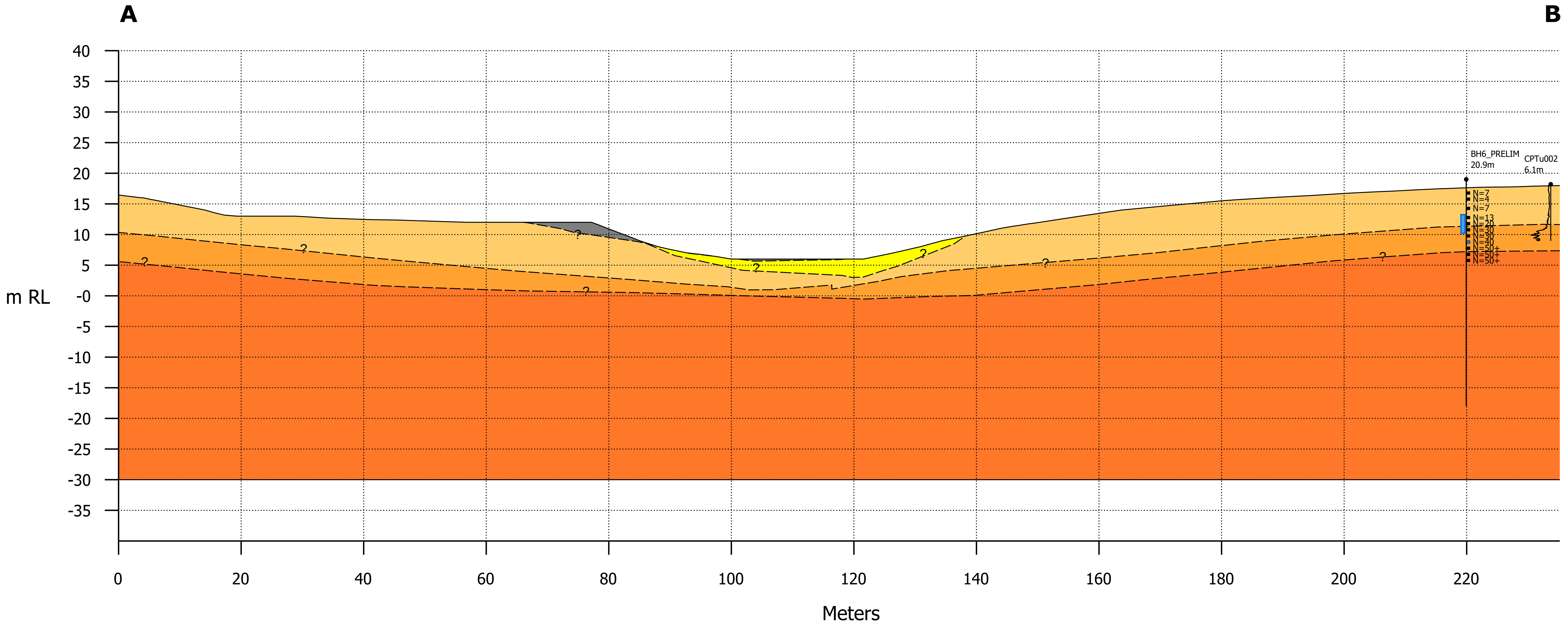
B: 1755122, 5921309

Scale: 1:750

Vertical exaggeration: 1x



Pt Erin_Cross Section_G



NOTES:
1. ALL DIMENSIONS ARE IN METERS UNLESS NOTED OTHERWISE.
2. EXISTING GROUND PROFILE BASED ON 1M LIDAR SOURCED FROM OPENTOPOGRAPHY.ORG (LIDAR CAPTURED FOR AUCKLAND COUNCIL BY AERIAL SURVEYS BETWEEN 2016 AND 2018).
3. GEOLOGICAL BOUNDARIES ARE INFERRED ONLY AND BASED ON THE INTERPRETATION OF GEOLOGICAL INVESTIGATIONS AT DISCRETE LOCATIONS. VARIATION OF ACTUAL GEOLOGICAL BOUNDARIES AND GROUND CONDITIONS BETWEEN INVESTIGATION LOCATIONS SHOULD BE EXPECTED.
4. GEOLOGICAL CONDITIONS BENEATH THE BASE OF GEOLOGICAL INVESTIGATIONS ARE NOT KNOWN.
5. BOREHOLE OFFSET FROM CROSS SECTION ALIGNMENT STATED BENEATH INVESTIGATION ID ON CROSS SECTION IE. '20.9m' REFERS TO 20.9M HORIZONTAL OFFSET FROM CROSS SECTION ALIGNMENT.

Legend

TT_Pt Erin_Leapfrog model_20230120

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

Location

A: 1755123, 5921314

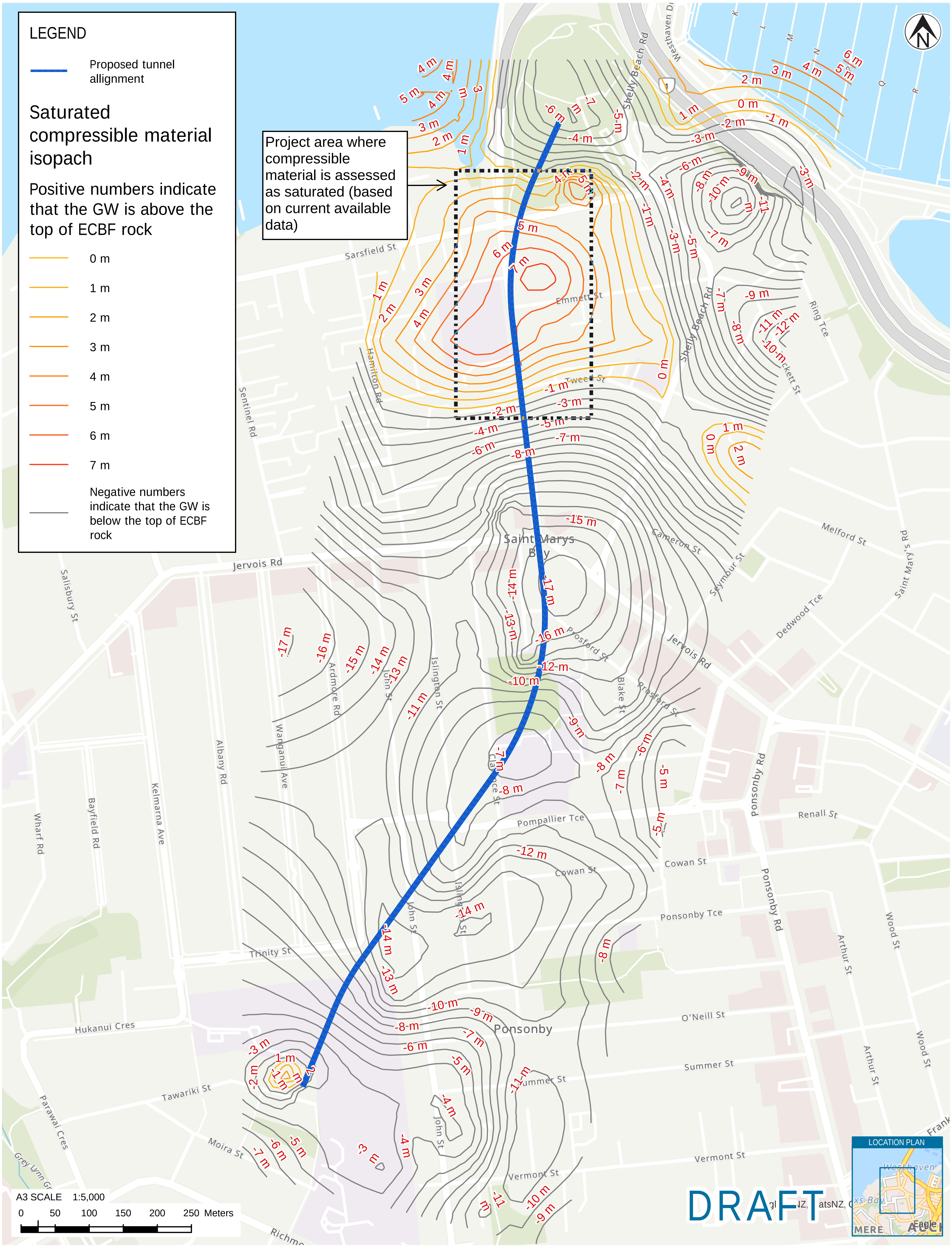
B: 1755198, 5921537

Scale: 1:750

Vertical exaggeration: 1x

0m 50m

Appendix C Tunnel alignment saturated compressible material thickness

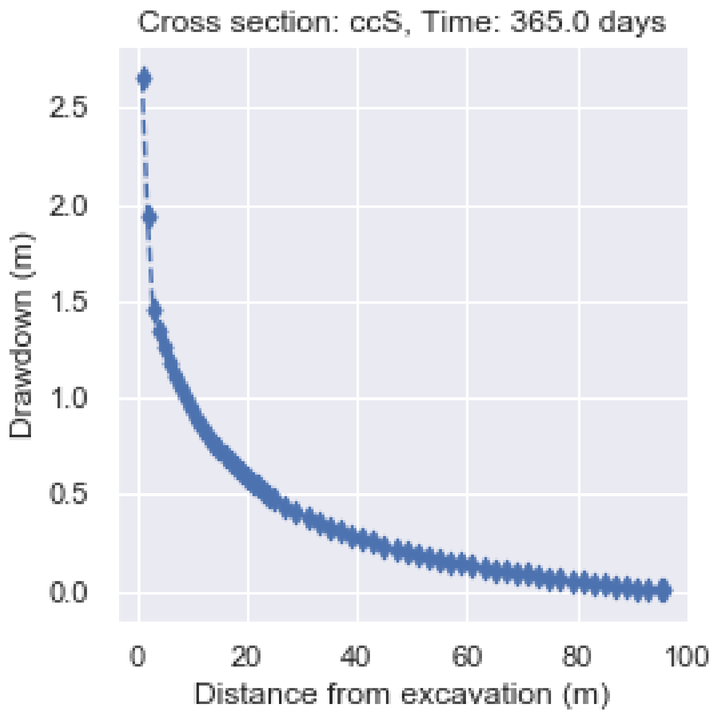


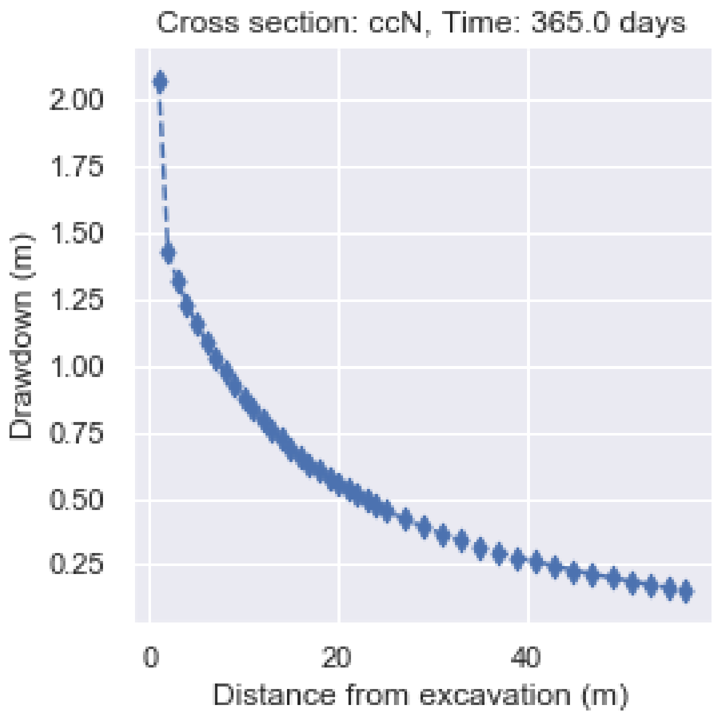
Tonkin+Taylor www.tonkintaylor.co.nz *Exceptional thinking together*	NOTES: Initial assessment based on indicative information, including: - Groundwater level surface (m bgl) from GNS Science (2018) National water table model. - Depth to competent ECBF rock estimated using LeapFrog modelling software undertaken by T+T (Dec 2022) based on available drilling information.	PROJECT No. 30552.9081	CLIENT **WATERCARE SERVICES LIMITED**
		DESIGNED KELE DEC.22	PROJECT **PT ERIN/ CI TUNNEL**
		DRAWN KELE DEC.22	TITLE SATURATED COMPRESSIBLE MATERIAL ISOPACH INITIAL ASSESSMENT BASED ON INDICATIVE INFO
		CHECKED	
	REV 0 / DESCRIPTION First version / GIS XXXX / CHK YYYY / DATE D/M/Y	APPROVED / DATE	SCALE (A3) 1:5,000 / FIG No. FIGURE 1. / REV 0

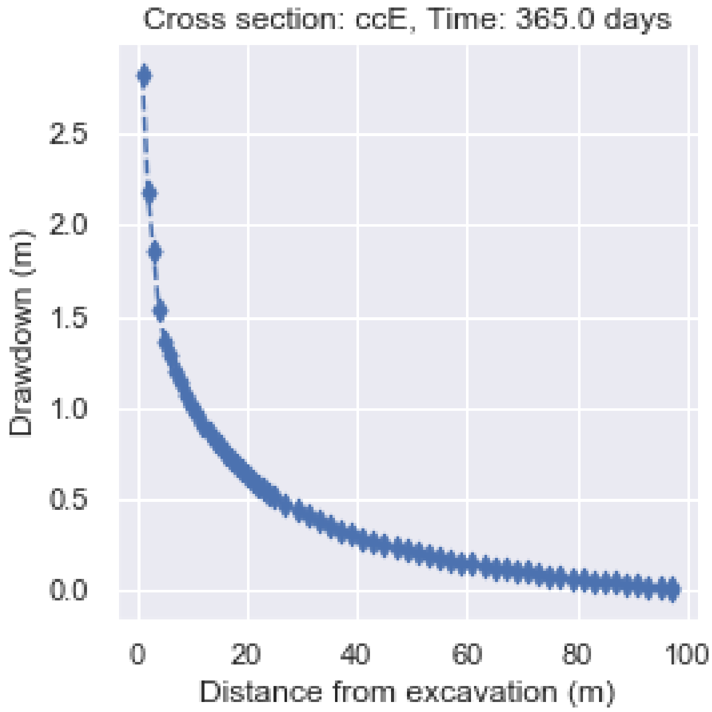
2022-Dec-07 11:57 am Drawn by KELE

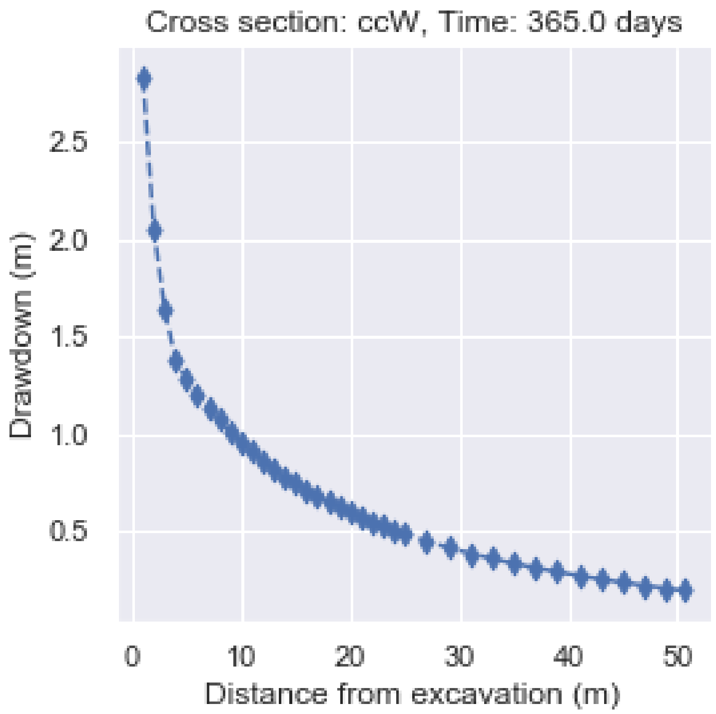
Appendix D AnAqSim model results

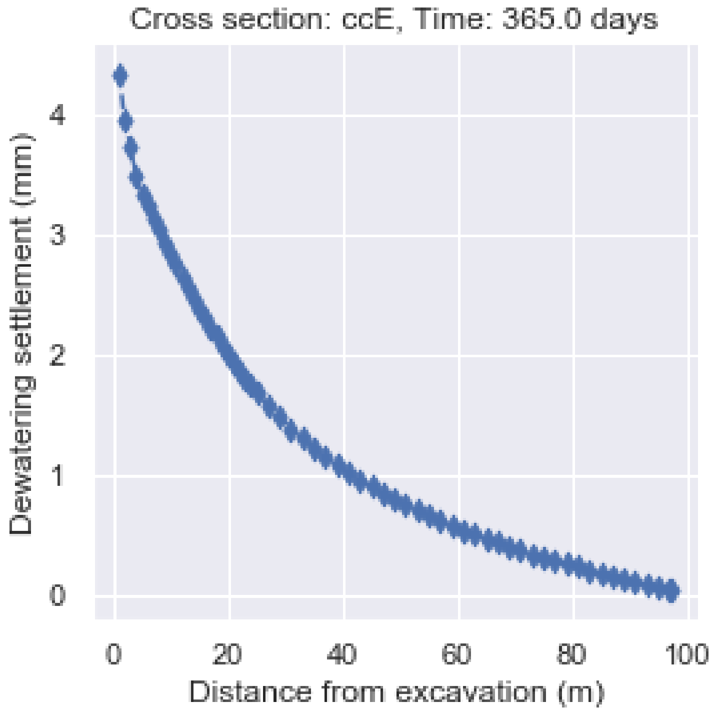
Control chamber: drawdown and related settlement numerical model results



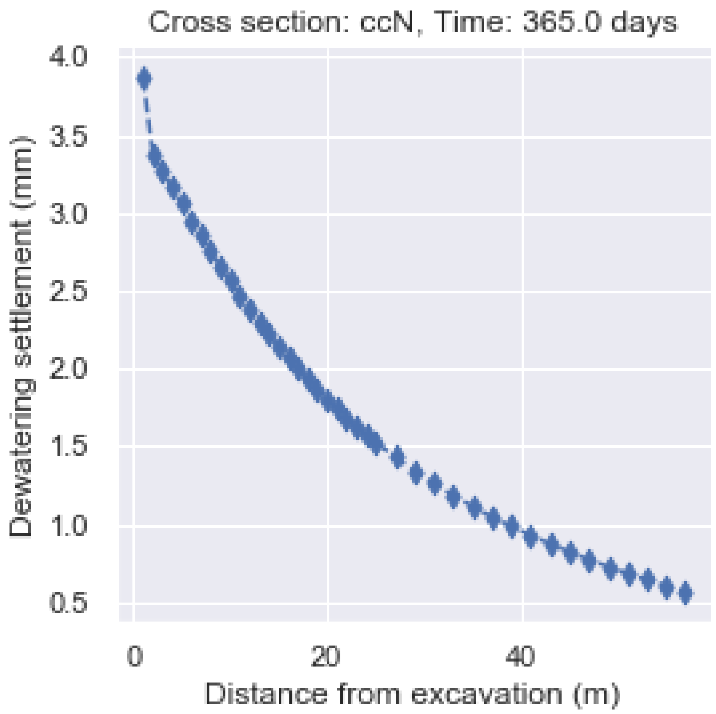



Cross section: ccE, Time: 365.0 days
Drawdown (m)
Distance from excavation (m)
0.0
0.5
1.0
1.5
2.0
2.5
0
20
40
60
80
100

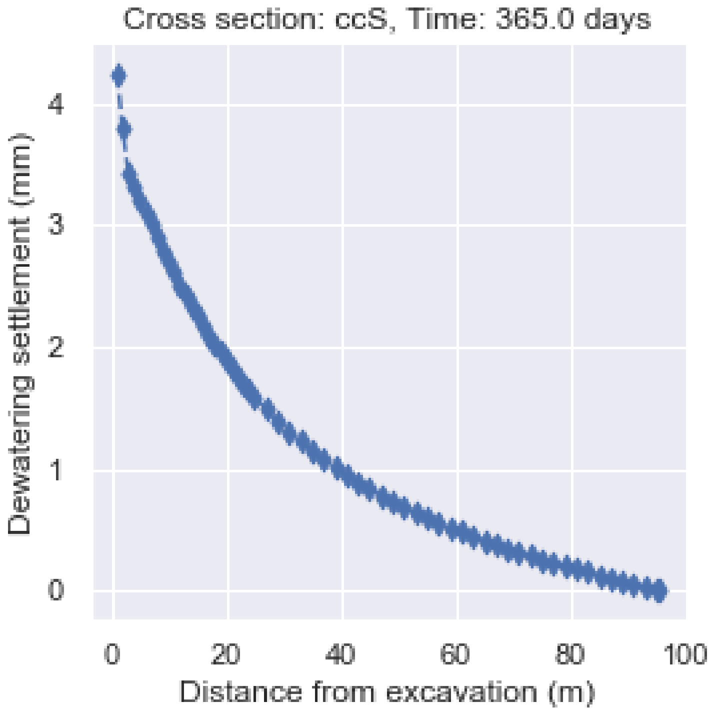

Cross section: ccW, Time: 365.0 days
Drawdown (m)
Distance from excavation (m)
0.5
1.0
1.5
2.0
2.5
0
10
20
30
40
50

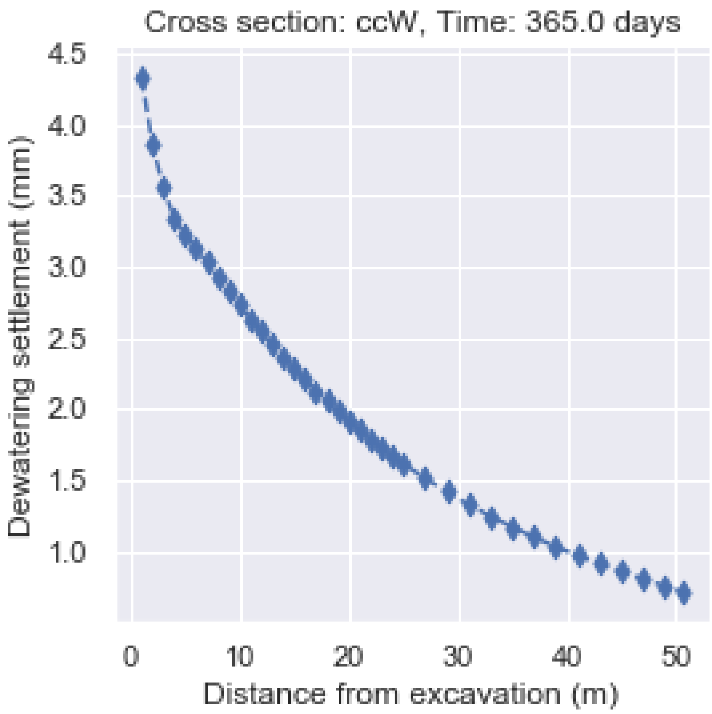


Cross section: ccE, Time: 365.0 days
Dewatering settlement (mm)
0
1
2
3
4
0
20
40
60
80
100
Distance from excavation (m)

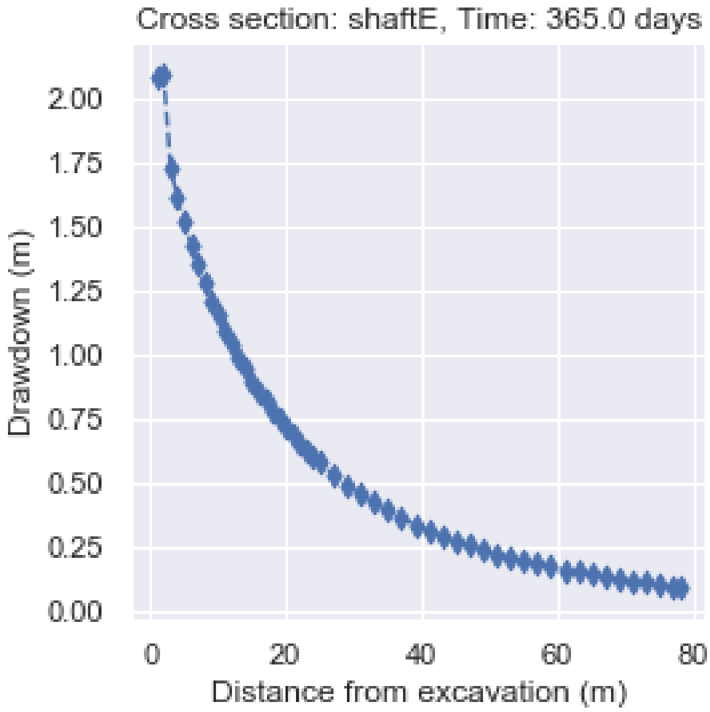


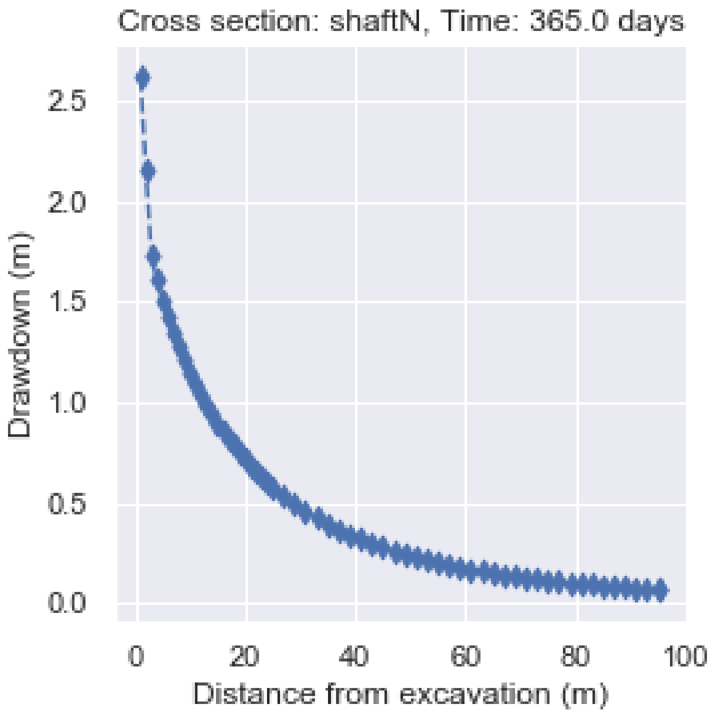
Cross section: ccN, Time: 365.0 days
Dewatering settlement (mm)
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
0
20
40
Distance from excavation (m)

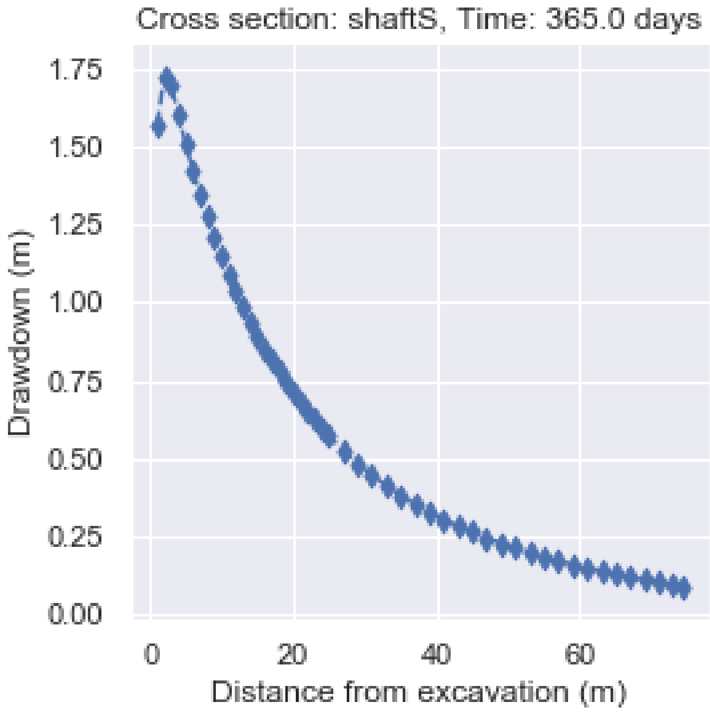

Cross section: ccS, Time: 365.0 days
Dewatering settlement (mm)
0
1
2
3
4
0
20
40
60
80
100
Distance from excavation (m)

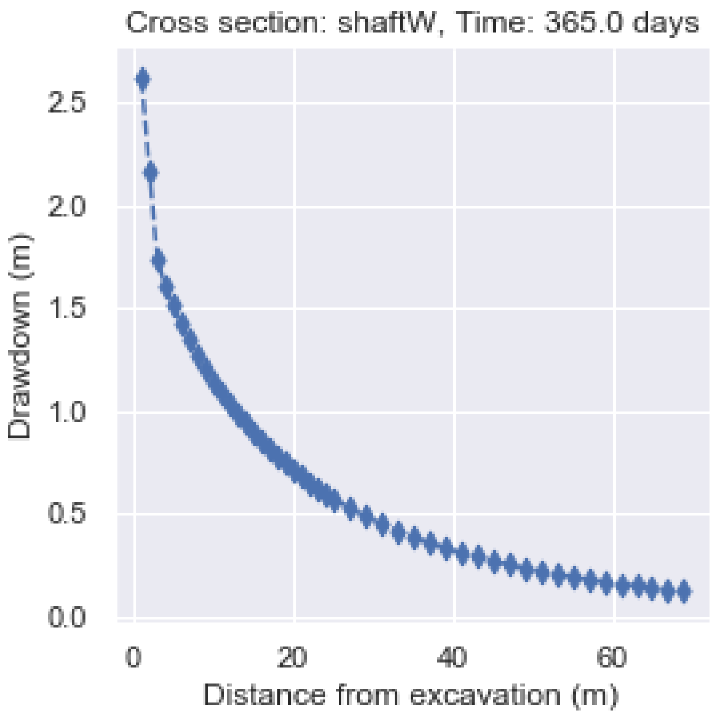

Cross section: ccW, Time: 365.0 days
Dewatering settlement (mm)
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
0
10
20
30
40
50
Distance from excavation (m)

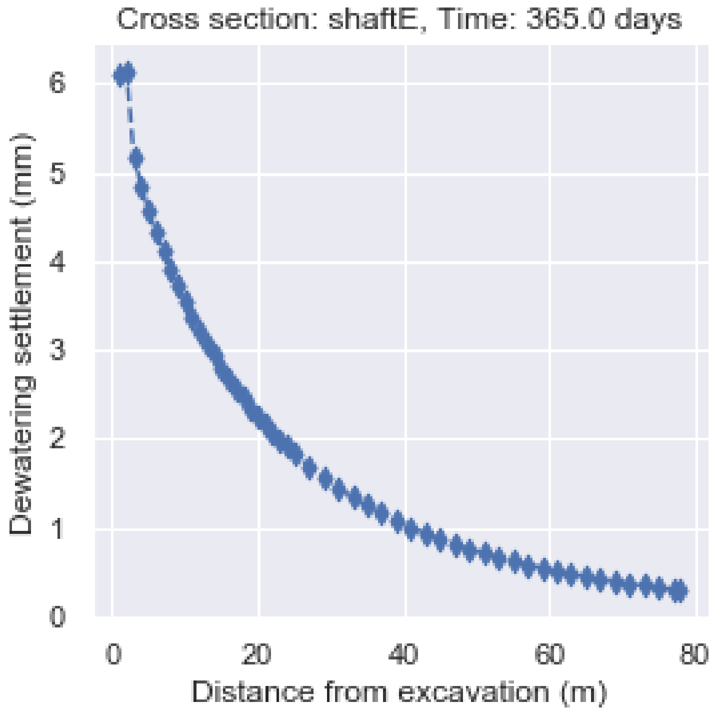
Terminal shaft: drawdown and related settlement numerical model results

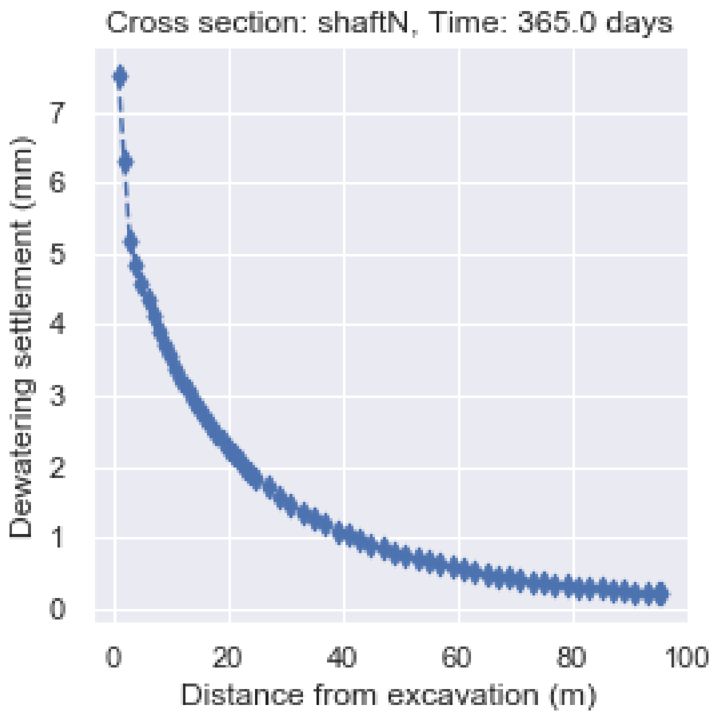


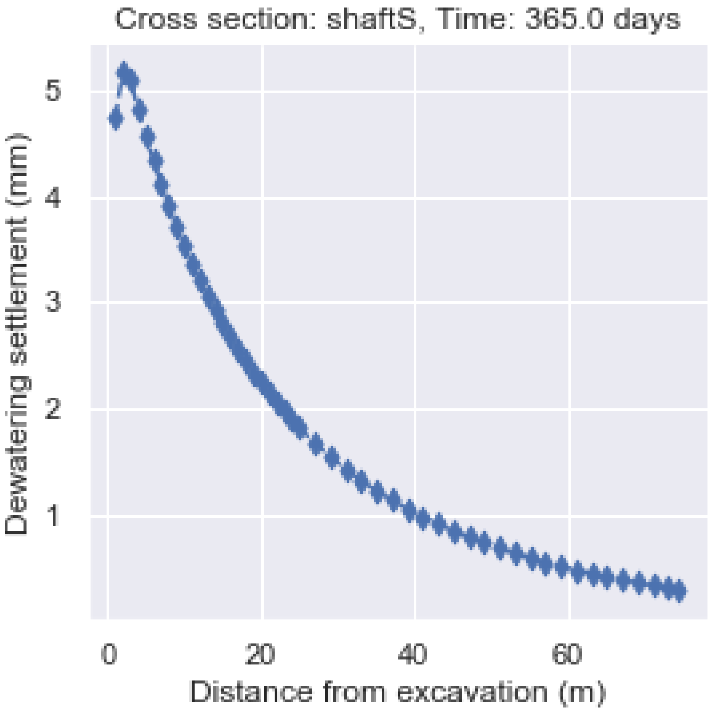


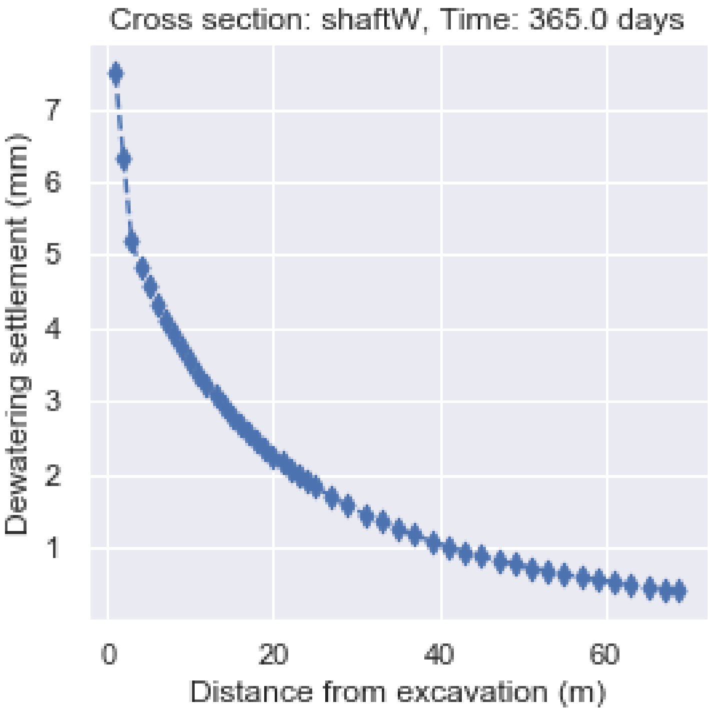

Cross section: shaftS, Time: 365.0 days
Drawdown (m)
1.75
1.50
1.25
1.00
0.75
0.50
0.25
0.00
0
20
40
60
Distance from excavation (m)


Cross section: shaftW, Time: 365.0 days
Drawdown (m)
2.5
2.0
1.5
1.0
0.5
0.0
0
20
40
60
Distance from excavation (m)

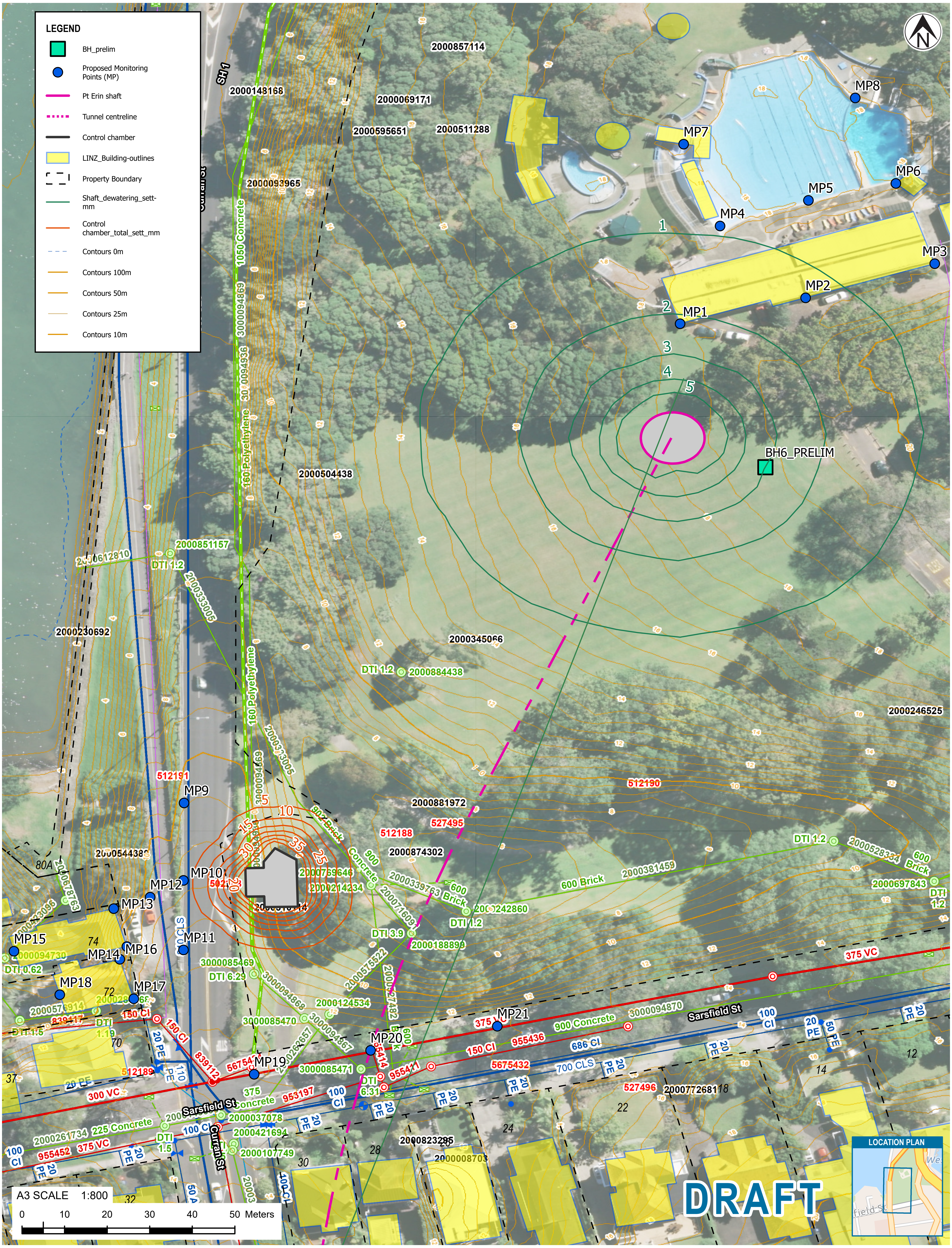

Cross section: shaftE, Time: 365.0 days
Dewatering settlement (mm)
0
1
2
3
4
5
6
0
20
40
60
80
Distance from excavation (m)


Cross section: shaftN, Time: 365.0 days
Dewatering settlement (mm)
0
1
2
3
4
5
6
7
0
20
40
60
80
100
Distance from excavation (m)


Cross section: shaftS, Time: 365.0 days
Dewatering settlement (mm)
1
2
3
4
5
0
20
40
60
Distance from excavation (m)


Cross section: shaftW, Time: 365.0 days
Dewatering settlement (mm)
1
2
3
4
5
6
7
0
20
40
60
Distance from excavation (m)

Appendix E Tunnel alignment saturated compressible material thickness



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NOTES:
Basemap UndergroundServices: public. Auckland Contours (GeoMaps): Auckland Council. 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
0	First version	XXXX	YYYY	D/M/Y

PROJECT No.	30552.9081	
DESIGNED	TRMC	FEB.23
DRAWN	TRMC	FEB.23
CHECKED		
APPROVED		DATE

CLIENT **WATERCARE SERVICES LIMITED**

PROJECT **CENTRAL INTERCEPTOR POINT ERIN EXTENSION**

TITLE POINT ERIN PARK : TOTAL GROUND SETTLEMENT ANALYSIS

SCALE (A3) 1:800 FIG No. FIGURE 1. REV 0

Appendix F Monitoring Plan

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0	First version	XXXX	YYYY	D/M/Y

PROJECT No.	30552.9081	
DESIGNED	KELE	JAN.23
DRAWN	KELE	JAN.23
CHECKED		
APPROVED		DATE

CLIENT **WATERCARE SERVICES LIMITED**
PROJECT **CENTRAL INTERCEPTOR POINT ERIN EXTENSION**
TITLE POINT ERIN PARK : PROPOSED MONITORING PLAN

SCALE (A3) 1:800 FIG No. FIGURE 1. REV 0

C:\Users\kele\dev\260123_AppendixB_figures\AppendixB_figures.aprx Layout: Tunnel alignment_Proposed monitoring 2023-Jan-27 3:24 pm Drawn by KELE

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REV	DESCRIPTION	GIS	CHK	DATE	APPROVED	DATE
0	First version	XXXX	YYYY	D/M/Y		

PROJECT No.	30552.9081	
DESIGNED	KELE	JAN.23
DRAWN	KELE	JAN.23
CHECKED		

CLIENT **WATERCARE SERVICES LIMITED**
PROJECT **CENTRAL INTERCEPTOR POINT ERIN EXTENSION**
TITLE TUNNEL ALIGNMENT: PROPOSED MONITORING PLAN
SCALE (A3) 1:4,270 FIG No. FIGURE 2. REV 0