REPORT

Watercare Services Ltd

Northern Interceptor Phase 1 Groundwater and Settlement Assessment Report

Prepared for:

Watercare Services Ltd

Prepared by:

Tonkin & Taylor Ltd

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Table of contents

List of Terms and Abbreviations

Term	Definition
Northern Interceptor	New wastewater interceptor to convey wastewater flows from the Northern Strategic Growth Area (NorSGA) and South Rodney (Kumeu/Huapai/Riverhead) via the Hobsonville Pump Station (PS) to the Rosedale Wastewater Treatment Plant (WWTP).
Phase 1	To be completed in 2020, Phase 1 transfers the existing Hobsonville Pump Station flows to Rosedale WWTP through a 710mm DN rising main crossing the Upper Harbour, and through Greenhithe, The North Shore Memorial Park, the North Shore Golf Club and Rosedale industrial areas. The majority of the construction will be open trenched.
Horizontal Directional Drilling	Is a steerable trenchless method of installing underground pipes in a shallow arc along a prescribed bore path using a surface launched drilling rig.
Micro-tunnelling	Is a technique used to construct small tunnels. The Micro-tunnel boring machine and jacking frame are installed in a shaft at the required depth. The Micro-tunnel boring machine is directed by an operator located at the surface.
Rising main	From the Hobsonville pump station to the Rosedale WWTP the Phase 1 sewer is pressurised by pumping and is termed a rising main.

Abbreviation	Definition
AEE	Assessment of Effects on the Environment
ACRP:ALW	Auckland Council Regional Plan: Air, Land and Water
Council	Auckland Council
DN	Nominal Diameter
HDD	Horizontal Directional Drill
MTBM	Micro Tunnel Boring Machine
NI	Northern Interceptor
PAUP	Proposed Auckland Unitary Plan (Notified 30 September 2013)
PS	Pump Station
RMA	Resource Management Act 1991
Watercare	Watercare Services Limited
WWTP	Wastewater Treatment Plant

Executive summary

Project description

Watercare Services Limited ("Watercare") proposes to build new wastewater pipelines and associated infrastructure to convey wastewater from north-western parts of Auckland to the Rosedale Wastewater Treatment Plant ("WWTP") in Albany. This project is known as the "Northern Interceptor". Construction of the Northern Interceptor is intended to be staged, with the timing of various stages depending on the rate of population growth.

The proposed work is likely to require various resource consents under the Resource Management Act 1991 ("RMA"). This technical report provides specialist input for the *Northern Interceptor Phase 1 – Assessment of Effects on the Environment report* ("the main AEE") prepared by MWH New Zealand Limited, which supports the resource consent application.

The proposed Northern Interceptor Phase 1 will transfer existing flows from the existing Hobsonville Pump Station ("PS") to the Rosedale WWTP via a corridor extending under State Highway 18 ("SH18"), along the northern side of the motorway causeway, under the Upper Waitemata Harbour, through Greenhithe and the commercial area of Rosedale.

Based on the preliminary design to date, the proposed pipeline will comprise:

- A single 710mm DN polyethylene ("PE") pipeline in land sections; and
- Twin 550mm DN PE pipelines at the Upper Waitemata Harbour crossing, Te Wharau Creek and at Alexandra Stream (near the Rosedale WWTP).

Investigations undertaken

The Project carried out geotechnical investigations along the entire pipeline route as part of wider geotechnical investigations and design. These comprised 12 rotary cored boreholes, 8 Cone Penetrometer Tests (CPTs) and 20 hand auger holes on land and 8 rotary cored boreholes along the route through the Upper Waitemata Harbour as well as bathymetric survey of the sea bed. The investigations were supplemented by information available from other projects located in close vicinity to the proposed alignment.

The results of the investigations were used to develop the geological model for the alignment and to inform our understanding of the range of potential effects that may impact existing services and structures.

Construction considerations and criteria for assessment of effects

The Northern Interceptor preliminary design includes pipelines constructed by a combination of open trenching, directional drilling, marine trenching and micro tunnelling. These construction methods are routinely used for the installation of pipelines similar to those proposed for the NI - stormwater, wastewater and bulk water supply pipelines are commonly constructed this way - and these methods are well understood by the engineering and contracting community.

The assessments in this report consider the potential for these construction methods to affect existing services and buildings along the alignment as a result of mechanical and groundwater drawdown induced surface settlement, based on the preliminary design.

Over most of the route the pipeline will be shallow and buildings are some distance from the construction activities, which limits the extent to which they might be significantly affected by them. Over some 6% of the total alignment however, the proximity of the construction activities to buildings and services present the potential for them to be within the zone of surface settlement, and hence to possibly be affected by it. In these specific locations, the construction methodology will be further assessed during detailed geotechnical design to confirm the magnitude of the potential effects and to confirm the particular construction methodology to control settlement to within the tolerances of the service and structures.

The following table provides a descriptive summary of the locations where this would be undertaken, based on the preliminary design. At each of these locations, it is expected that commonly used construction methodologies could be specifically employed to control the settlements to within tolerances of typical structures and services.

Locations / Structures potentially affected by settlement associated with pipeline construction

Location	Discussion
Tauhinu Road crossing	Excavation for the pipeline is expected to be approximately 5m deep, and extend 2 to 3m below groundwater level. Without specific control measures, surface settlement arising from the excavations could be 90mm close to the trench and may affect the pavement surface and the services that are within the road reserve (including an 810mm water line and a 225mm diameter stormwater line). The existing buildings are beyond the zone of predicted surface settlement (greater than 12m from the excavation) and are therefore not considered to be at risk of damage resulting from settlement associated with trench excavation.
Commercial properties near William Pickering Drive	Excavations for the pipeline will be within 5m of commercial buildings. The excavations are expected to be 3m deep, and extend some 2m below groundwater level. Without specific control measures, surface settlement arising from the excavations could be 50mm close to the trench. The buildings are likely to be on the fringes of the associated settlement trough, and specific geotechnical investigation and design would reduce the risk of buildings experiencing settlement and of damage being sustained by the buildings.
William Pickering Drive crossing	Excavations for the pipeline are expected to be up to 8m deep, and extend some 5m below groundwater level. Without specific control measures, surface settlement arising from the excavations could exceed 100mm close to the trench, and in the order of 20mm near the buildings. Specific geotechnical investigation and design of mitigation measures, such as sheet piling, would be expected to reduce the magnitude of settlement and the risk of buildings experiencing settlement or of damage.
11 Traffic Road.	Excavations for the pipeline will be within 7m of buildings, approximately 3m deep and extend some 2m below groundwater level. Without specific control measures, surface settlement arising from the excavations could be 50mm close to the trench, with buildings at the extreme edge of the associated settlement trough and unlikely to experience measureable settlement. Specific geotechnical investigation and design would confirm the magnitude of settlement and whether or not any measures are required to reduce the risk of buildings experiencing settlement and of damage being sustained by the buildings.
30 to 34 Newbury Place. 222 to 224 Schnapper Rock Road	Excavation for the pipeline will be within 4m of buildings and approximately 2.5m deep and will terminate above the groundwater level. Without specific control measures, surface settlement arising from the excavations could be 25mm close to the trench but buildings are expected to be at the extreme edge of the associated settlement trough and unlikely to experience measureable settlement.
1-13 Appleby Road.	Excavation for the pipeline will be within 7m of buildings, and approximately 5m deep and extend some 2m below the groundwater level. Without specific

Location	Discussion
	control measures, surface settlement arising from the excavations could be 70mm close to the trench and in the order of 10mm near the buildings. Specific geotechnical investigation and design of mitigation measures, such as sheet piling, would be expected to reduce the magnitude of settlement and the risk of buildings experiencing settlement or of damage
Commercial property at 327 Albany Highway	Excavation for the pipeline will be within 4m of buildings, and 4m deep, and extend some 3m below groundwater level. Without specific control measures, surface settlement arising from the excavations could be 30mm close to the trench, and in the order of 10mm near the buildings. Specific geotechnical investigation and design of mitigation measures, such as sheet piling, would be expected to reduce the magnitude of settlement and the risk of buildings experiencing settlement or of damage
Commercial development currently under construction at 325 Albany Highway and 14 John Glenn Drive.	The proposed alignment crosses a property currently under development, the proposed pipeline may be within close proximity to structures. We understand that Watercare will work with the developer to manage the impact of the construction of the NI.
169-174 Bush Road and Vector substation.	Excavation for the pipeline will be within 5m of buildings and approximately 3.5m deep, and extend 2.5m below groundwater level. Without specific control measures, surface settlement arising from the excavations could be 60mm close to the trench and in the order of 10mm near the structures. Specific geotechnical investigation and design of mitigation measures, such as sheet piling, would be expected to reduce the magnitude of settlement and the risk of buildings experiencing settlement or of damage
SH18 crossing including temporary access shafts and the trenched ramp on the northern side of the motorway to allow a straight run of pipe into the jacking pipe.	Pipeline will be micro tunnelled some 2.5m below groundwater level. Surface settlement arising from tunnelling works could be between 20 and 40mm in the vicinity of and beneath the Upper Harbour Highway. Surface settlement arising from construction of the southern access shaft could be in the order of 75mm. Surface settlement can be controlled and minimised by ensuring that an earth pressure balanced capable machine is used and that pressure balance during excavation is carefully controlled.

1 Introduction

Watercare Services Limited ("Watercare") is proposing to build new wastewater pipelines and associated infrastructure to convey wastewater from north-western parts of Auckland to the Rosedale Wastewater Treatment Plant ("WWTP") in Albany. This project is known as the "Northern Interceptor". Construction of the Northern Interceptor is intended to be staged, with the timing of various stages depending on the rate of population growth.

Tonkin & Taylor Ltd (T&T) has been commissioned by Watercare to assess the potential groundwater and surface settlement effects related to the construction, operation and maintenance of the proposed Northern Interceptor Phase 1 ("the Project").

The proposed work requires various resource consents under the Resource Management Act 1991 ("RMA"). This technical report provides specialist input for the *Northern Interceptor Phase 1 – Assessment of Effects on the Environment* report ("the main AEE") prepared by MWH New Zealand Limited, which supports the resource consent application.

This report provides the following:

- A brief overview of the proposed works (in Section 2);
- A description of the environmental baseline for the particular receiving environment(s) potentially affected by the project;
- Description of specific aspects of the project in relation to the subject area being investigated;
- A brief outline of the statutory framework relevant to groundwater and surface settlement effects;
- Description of the investigations undertaken to assess groundwater and surface settlement effects;
- An assessment of the actual or potential effects on the environment (construction, operation and maintenance), having reference to the statutory framework and any other environmental factors considered relevant. This includes the identification of activities that could result in adverse effects and, in turn, identifying design refinements or construction methodologies that could avoid, remedy or mitigate such effects;
- Recommended mitigation and management measures.

2 Proposed works

The proposed Northern Interceptor Phase 1 will transfer existing flows from the Hobsonville Pump Station to the Rosedale WWTP. The proposed route is from the existing Hobsonville Pump Station, under the State Highway 18 motorway, along the northern side of the motorway causeway, and then under the Upper Waitemata Harbour, through Greenhithe and then the commercial area of Rosedale.

Key elements of the project include:

- Upgrading of the existing Hobsonville Pump Station.
- Installation of a pipe under State Highway 18.
- Installation of pipelines in a widened section of the existing motorway causeway.
- Installation of dual pipelines across the Upper Waitemata Harbour to Greenhithe via marine trenching or horizontal directional drilling ("HDD").
- Installation of dual pipelines under Te Wharau Creek via HDD.
- Construction of a pipe bridge between Witton Place and North Shore Golf Course.
- Installation of dual pipelines under Alexandra Stream via HDD.
- Trenched construction for pipeline installation in roads, open space and other land; and installation of associated infrastructure.

With the exception noted below, the proposed works are described in detail in the main AEE, the drawings and construction methodology of works assessed in this report form part of Volume 3 of the main AEE.

Watercare is proposing some widening along the existing State Highway 18 motorway causeway near Hobsonville to provide for proposed water and wastewater infrastructure, including a section of the Northern Interceptor Phase 1 pipeline. That work forms part of Watercare's proposed Greenhithe Bridge Watermain Duplication and Causeway project. That project is part of a separate resource consent package, and is described in a report titled *Greenhithe Bridge Watermain Duplication and Causeway – Assessment of Effects on the Environment*, prepared by Aecom New Zealand.

3 Objective and scope of works

The objective of this report is to assess the effects during and following construction for the Northern Interceptor (NI) Phase 1 pipeline:

- Potential magnitude and extent of groundwater drawdown effects associated with the construction and operation of the NI pipeline;
- Potential magnitude and extent of surface settlement that might be induced by the NI construction (settlement induced by the construction methodology and through groundwater drawdown) and operational groundwater drawdown effects;
- Potential magnitude of groundwater inflows into the NI excavations during construction; and,
- Effects of groundwater pressure changes associated with the NI on consented groundwater users in the vicinity.

The potential magnitude of these construction effects is strongly dependent on the construction methodology and the local geological and hydro-geological conditions. The potential for long term groundwater effects following completion of construction is dependent primarily on the degree of water tightness achieved at the pipe joints and the permeability of the trench bedding medium.

In order to assess the potential effects during and following construction of the NI Phase 1 project, the following scope of works has been carried out:

- Review of the Construction Methodologies to identify potential risk areas for the construction.
- Identification of other nearby significant services which may have affected the groundwater table and which may be susceptible to the construction of the proposed pipeline.
- Preparation of a conceptual geological model based on the Geotechnical Investigations and identify geological boundaries.
- Create a hydrogeological model based on the groundwater monitoring which commenced on 17 December 2014.
- Estimate the resulting settlements based on the groundwater drawdown and the laboratory testing completed as part of the Geotechnical Investigations.
- Carry out preliminary seepage assessments and estimate the seepage volume.
- Compare the findings with the Permitted Activity criteria set out in the Proposed Auckland Unitary Plan (PAUP) and the Auckland Council Regional Plan: Air, Land and Water (ACRP: ALW).
- Summarise the findings and provide recommendations with the respect to the NI Phase 1 pipeline.

The design has developed in a series of stages over the course of our assessment. This report focusses on those sections of the alignment where risks are greatest but includes commentary and consideration of different vertical alignments and extent of effects which were ultimately defined but details remain in the report.

4 Construction methodologies

4.1 General

The NI Phase 1 project utilises construction methodologies that are frequently used and are well understood by the Engineering and Contracting community. Very similar construction methodologies are used for the installation of pipelines throughout Auckland and New Zealand. The methodologies are routinely used for the installation of pipelines for stormwater networks, wastewater networks, and bulk water supply pipelines. This knowledge is widely available in New Zealand.

For short sections of the route, more specialised methodologies are required. The equipment and skills required for these sections of the route are still readily available in New Zealand.

The construction methodologies are described below, with specific comment on how they will be used within the project - along with discussion on the potential effects and mitigation measures that might be associated with them that are relevant to this assessment.

In general terms, the effects that are relevant to this assessment for third parties are:

- Surface settlement that arises from construction of the pipeline. This effect is of significance when the surface settlement affects existing buildings or services;
- Groundwater lowering (drawdown) that might occur during construction, or persist after construction is complete. The groundwater drawdown may cause surface settlement;

In addition to these effects, additional effects that are relevant to the construction of the project are;

- The rate and volume of groundwater inflow into excavations during construction. High groundwater inflows have the potential to complicate construction processes.

4.2 Open excavation

4.2.1 Trenched construction

Most of the pipeline will be constructed by open excavation of trenches - approximately 7.6 km of open cut trenching is proposed for the installation of the wastewater pipeline out of the total 9.7 km. In these areas the pipeline will be constructed in the road reserves and park reserves and is therefore a considerable distance from nearby buildings or structures. Construction will involve excavation, potentially shoring of the trench and backfilling subsequent to installation of the pipeline.

The depth of trenching typically varies between 2.5m and 4m below existing ground. These sorts of excavation depths are routinely undertaken for pipeline projects. Short sections of deeper trenching is proposed where conflict with existing services has been identified, which will require more detailed consideration.

The polyethylene (PE) pipes that will be used are generally delivered in 12 m to 15 m lengths and are welded together on site to form a continuous pipe. Typically, sections of trench will be excavated and pre-welded pipe installed using excavators or cranes; where space is more limited lengths of pipe will be welded within trenches.

4.2.2 Manholes

Pipelines are typically laid in straight lines. Direction changes are made at manholes which are also used for installation of scour valves which are generally installed at topographic low points along the pipeline.

Typically the scour chambers are in the order of 2.5 to 3.5 m deep and 1-2m in diameter, similar to the manholes. Such construction is routine, and the manholes will be constructed using conventional techniques for the particular ground conditions and surrounding land use. The manhole structures are most likely to be constructed using precast riser rings.

There are two deeper passages with manholes required for scour valves along the Phase 1 alignment which may require more specific consideration:

- The Tauhinu Road/ Rame Road crossing where the pipe passes beneath the North Harbour Water Main No. 1 (NHWM1) – a 7m deep scour valve manhole is required.
- The William Pickering Drive crossing where the pipe passes beneath the North Harbour Water Main No. 2 (NHWM2) – an 8m deep scour valve manhole is proposed.

While such deeper manholes are not routine construction, they are at a depth that has been regularly constructed around Auckland for the wastewater and stormwater networks. These deeper manholes will require specific excavation support.

4.3 Potential effects of tunnelling and marine trenching

4.3.1 Micro-tunnelling

Micro-tunnelling is utilised on a short (approximately 100 m long) section of the alignment where it crosses under SH 18 near the Hobsonville PS.

The micro tunnelling would begin approximately 20 m north of the existing Hobsonville PS and travels approximately 5 to 6 m below the SH 18. The tunnelling will be completed using a Micro Tunnel Boring Machine (MTBM) with access shafts set up on either side of the motorway. A concrete jacking pipe will be installed as the MTBM traverses across the proposed area of works. A 710 DN PE pipe will then be slip-lined into the jacking pipe and the annulus grouted upon completion.

The proposed temporary access shafts are expected to be in the order of 6 m wide and 8 m long and extend approximately 0.2m below the proposed invert of the pipe. The Construction Methodology documentation prepared by MWH (Ref 1) proposes that the excavated sides are supported by timber lagging. The 710 DN PE pipe will be installed from the northern access pit via a trenched ramp to allow a straight run of pipe into the jacking pipe.

A shield in front of the pipe is used to excavate a pipe shaped tunnel in the ground (manually or mechanically) into which the pipes are pushed. Micro-tunnelling methodologies excavate a slightly larger hole than the external dimension of the pipeline segments. Without this “overcut”, the segments would be prone to binding against the excavation during installation, and only very short runs would be possible. This small overcut eventually collapses onto the pipe and, depending on ground conditions, may result in surface settlement.

It is also common in soft ground for the volume of excavated material to exceed the open volume of the excavation. As the hole is advanced, the material at the face of the excavation moves into the void created (as the unsupported ground relaxes) and additional material is therefore excavated. The potential consequence of material moving into the face of the excavation is sometimes seen in the expression of settlement at the ground surface.

Micro-tunnelling methodologies are available which employ a number of techniques specifically developed to control these sources of surface settlement. The methodologies have been widely used and all primarily involve a pressure being applied to the face and/or the annulus to stabilise it during excavation, and backfilling of the annulus before it closes, e.g. a slurry shield capable machine in closed operation mode.

The efficiency of these methods varies depending on the skill of the operator and on the particular ground conditions. Too little pressure can lead to settlement, whilst too much can lead to heave. Transitioning from one ground condition to another (such as from soft alluvial materials into more competent rock) or excavating in mixed face conditions (soft material in the upper face, stiff material in the lower) complicates the control and leads to circumstances where unexpectedly large settlement, or conversely heave, can occur.

Past project experience and international literature identify that the most likely reason for unexpectedly large or damaging surface settlement during construction result from problems maintaining the assumed construction methodology. Such an example would be loss of control of the face or annulus stability leading to over relaxation or collapse of the excavation. In some combinations of conditions this could lead to sink holes forming at the ground surface.

4.3.2 Marine trenching

Marine trenching is also being considered as a construction methodology for the Upper Waitemata Harbour crossing to Greenhithe. This construction technique is unlikely to result in a settlement effect, but rather the potential redistribution of marine silts and sands disturbed by the construction activities. These effects are therefore not considered in this report.

4.3.3 Horizontal directional drilling

Horizontal directional drilling (HDD) is proposed in three areas within the overall construction footprint to construct the pipeline underneath the sea and stream. HDD is used to avoid environmental damage caused by traditional trenched construction techniques across watercourses and intertidal/marine areas.

A total of approximately 3.8 km of 550 mm DN pipe is proposed in the three locations:

- across the Upper Waitemata Harbour from Hobsonville to Greenhithe;
- across Te Wharau Creek, north of Wainoni Park to NSMP; and
- across Alexandra Stream, west of Rosedale Park.

Two dual drill shots are proposed for the 1.1 km harbour crossing, with a pit on the Greenhithe side of the crossings. The laydown areas and construction requirements for the long drill shot across the harbour have been set out in the construction methodology and have been allowed for in the sizing of the SH18 causeway widening being consented separately. The other HDD shot for the Phase 1 project under Alexander Stream is likely to require shallow excavations at the launching and receiving ends of the HDD shots, to allow for connection to the trenched sections of the alignment and to adequately control drilling muds. The dimension of these trenches has not yet been determined but are unlikely to be significantly larger than that required for the trenched excavations preceding them.

Pipeline construction beneath the seabed or streams has the potential to result in settlement, in much the same way that it does on the dry land. The magnitude of sea floor / stream bed settlement would be expected to be similar or less than that predicted on dry land (for equivalent construction methodologies). However, the settlement that does occur is likely to be of an order that is not locally noticeable on mud flats or stream beds, and there is not expected to be any impact to the natural processes within the intertidal or sub-tidal areas.

The horizontal directional drilling will be carried out in competent ground / rock which may cause difficulties maintaining the alignment of the pipe. There has also been past experience with hydrostatic pressure release through existing defects in the rock, i.e. extrusion of liquid to the surface during the pressurised drilling operation. The effects of hydrostatic pressure release are discussed in the Environmental Assessment report for the project (Ref 1). Similarly, the effects from marine trenching are discussed in the Coastal Assessment Report (Ref 6).

4.3.4 Potential effects of trench and manhole construction

All open excavations require appropriate care during design and construction. The soil conditions should be routinely monitored during construction to check for layers of low strength material and / or high groundwater levels which can increase the risk of trench collapse posing safety risks for the construction workers and potentially affecting structures close to the trench.

In particular, the deepest sections will require specific consideration to ensure the excavations are safely undertaken.

The potential for surface settlement to arise from trench and manhole excavations is from two mechanisms:

- Groundwater drawdown settlement – where the trench excavation extends below groundwater level, and groundwater is pumped out of the trench to provide suitable conditions for pipe installation;
- Mechanical settlement - relaxation of the ground adjacent to the trench resulting from excavation.

The trench excavations act as large drains when they extend below groundwater level, with the greatest groundwater drawdown occurring immediately adjacent to the excavation and progressively recovers with distance. Groundwater drawdown causes an increase in effective stress in the soil, and in settlement prone soils may result in surface settlement.

Surface settlement arising from either or both of these mechanisms may pose a structural integrity risk for adjacent structures such as road pavement, services and possibly buildings if located near the open trench if the settlement is significant. The NI Phase 1 alignment typically runs predominately through a corridor of existing road reserves, hence the greatest impact may be surface disturbance and impacts on buried services.

The following construction methodologies are typically used where it is necessary to reduce the likelihood of these mechanisms occurring:

- <u>Trench shields</u> - comprising steel frames which support the side walls once the excavation is complete. Props can be installed across the trench to increase the robustness of the retention system.
- <u>Sheet piles</u> are likely to be installed at deeper sections of the alignment (3m plus) where potential for groundwater drawdown is large and to ensure the trench remains stable. The sheet piles are driven or vibrated in place and support the sides of the trench providing a safer work environment. Props may be required across the trench to reduce deformations that might result in mechanical surface settlement.

4.4 Key locations for effects assessments

The full length of the proposed NI Phase 1 project is being considered as part of this groundwater and settlement assessment. The corridor is typically quite shallow (2 -4m below ground level, consistent with the range of depths for much of the stormwater and wastewater networks in Auckland) and runs predominately in existing road reserves and parks, hence the greatest impact is likely to be surface disturbance and impacts on the roads and on buried services if they are close to the excavations. The potential for effects on properties and buildings adjacent to the corridor will ultimately depend on the location of the pipeline within the corridor, however it is expected that construction will primarily be within the road carriageway, with buildings typically in the order of 10m away (i.e. outside the inferred settlement influence radius).

Within the corridor, there are however particular areas where the combination of construction methodology, geological conditions and groundwater present the potential for surface effects to affect buildings and structures and our assessments consider these areas in particular. These areas are isolated to a few locations:

- The micro tunnelling operation beneath the SH 18 crossing, including temporary access shafts and the trenched ramp on the northern side of the motorway to allow a straight run of pipe into the jacking pipe.
- The Tauhinu Road crossing where the pipe passes beneath the North Harbour Water Main No. 1 (NHWM1) – a 7m deep scour valve chamber is required.
- The William Pickering Drive crossing where the pipe passes beneath the North Harbour Water Main No. 2 (NHWM2) – an 8m deep scour valve chamber is required.
- 30 and 32 Newbury Place where the proposed pipe is being installed beneath a walkway between two residential properties.
- The commercial site at 325 Albany Highway and 14 John Glenn Drive where construction of a commercial building is currently taking place near the alignment of the pipe.
- The commercial site at 327 Albany Highway which is within 4 m of the proposed alignment.
- 169-174 Bush Road and the Vector substation where the pipeline is likely to be constructed close to the buildings.

At these locations we have assessed the potential magnitude and extent of surface settlement that could arise from construction of the pipeline, as a basis for assessing the potential effects that might impact existing buildings and services and where specific attention is needed for construction methodologies to mitigate the effects. Settlement associated with the construction of the pipeline away from these locations is unlikely to impact existing structures or services.

5 Site investigations

The geological environments through which the corridor passes are generally understood from published geological maps. Subsurface investigations were carried out along the entire route to provide a better understanding of the geological environments that will be encountered during construction.

Information available from projects nearby was used to extend the picture and provide additional detail. At the locations of more specific interests (Section 4.4), specific investigations were carried out to provide information for targeted effects assessments. The site investigations are described in detail in Appendix A where the technical documentation supporting this report has been summarised.

6 Geotechnical assessment

6.1 General

The geotechnical assessment is based on published data, historic investigations, project specific investigations, review of aerial photographs and a site walkover combined with our experience of Auckland Geology. The conceptual geological model has been developed using the site investigation information and data such as the aerial photographs provide important information with respect to modern modification of the topography and use of land which may affect the NI Phase 1 development. The geological cross sections are attached in Appendix D.

Technical information including detailed site walkover observations, geological units along the alignment and soil stiffness parameters for design are described in detail in Appendix A.

6.2 Geological model

The area around the Northern Interceptor alignment is characterised by two major stratigraphic groups:

- Miocene Waitemata Group marine sedimentary rocks.
- Late Pliocene to Holocene Tauranga Group alluvial and estuarine sediments.

The Auckland Isthmus is dominated by the weak sandstones and mudstones/siltstones of the Waitemata Group, in particular the East Coast Bays Formation (ECBF) of the Warkworth Subgroup. Tauranga Group alluvium deposits are typically located within the base and flanks of present day and paleo-drainage channels.

The inferred surface geology is presented on geological sections in Appendix D.

6.3 Site walkover

A T&T geotechnical engineer and an Engineering Geologist completed an alignment walkover inspection on 20 October 2014. The main observations included outcrops of ECBF rock and presence of potentially settlement sensitive structures near the proposed pipe alignment.

6.4 Aerial photograph review

The historical photos available on the Auckland Council website have been reviewed with particular focus on the photos from 1959 to identify historic gullies and area of fill which may affect the NI Phase 1 development.

- An old valley was present near William Pickering Drive which has since been infilled with fill. T&T records indicate that the fill is greater than 5m in thickness in this location.
- An old valley also appears to have been infilled in the western part of Piermark Avenue.
- Two large sludge ponds are located to the south of the Rosedale WWTP. These have since been infilled and the area grassed.

6.5 Discussion

The soil investigations generally confirm the published geology along the alignment and provide additional detail for specific assessments that is not provided on the maps – the engineering properties of the soils, and the depths and layering of specific deposits.

The following sections discuss the potential for specific geological deposits to contribute to the mechanisms of surface settlement associated with pipeline construction. This highest potential hazard for surface settlement associated with each formation is discussed in this section and presented in decreasing level of assessed compressibility (risk) below:

<u>Tauranga Group alluvial soils:</u>

- These materials are generally relatively highly compressible and contribute significantly to surface settlement when groundwater levels are lowered. Where these soils are present, it may be necessary to use construction methodologies that specifically mitigate the potential for groundwater lowering such as sheet piles if existing buildings and structures could be exposed to the surface settlement that would arise (as defined in Section 4.4).
- These soils are likely to be the poorest performing with respect to trench stability along the open trench sections. Similarly, the soils may collapse adjacent the micro tunnel when constructed, leading to potential for significant surface settlement (particularly when associated with high groundwater level when running ground may result if excavations are unsupported). Practical and well understood construction methodologies are available to address this.
- These soils are generally highly variable in nature and sometimes contain large proportions of woody matter (non-decomposed trees and branches are possible), that are difficult to excavate using micro-tunnelling equipment. Where works to remove the woody matter are not subject to control, significant surface settlement can result. This should be considered for construction planning of the micro-tunnelling section beneath SH18.

<u>Historic fill:</u>

- Fill is a generic term of material placed by humans. These materials are therefore often non-uniform soils and have often been subject to variable levels of compaction. Where suitable compaction has not been employed during placement these historic fills often have high void ratio and consolidation settlements can occur if subject to groundwater drawdown.
- The variability of the fill materials provides significant uncertainty in their expected stability performance in open cut excavations.
- Depending on the degree of compaction during placement, the fill generally does not contain the same density as insitu soils. The variability of these soils can lead to uncontrolled discharge of drilling muds to ground surface in horizontal drilling operations if high mud pressures are being used. The alignment contains segments of horizontal drilling (Upper Waitemata Harbour crossing, Te Wharau Creek and Alexander Stream), but these are outside settlement sensitive areas and are not likely to affect existing structures.

<u>ECBF soils and Rock:</u>

- The ECBF soils are generally pre-consolidated and typically less susceptible to settlement although sandier layers within the soil mantle can make water during excavation. The ECBF rock is near incompressible relative to the effects likely to be associated with the NI pipeline construction works.

7 Hydrogeological assessment

The groundwater investigations suggest that the majority of the NI Phase 1 alignment is underlain by an unconfined aquifer where hydrostatic groundwater pressures are generally encountered between 1 and 3m below the ground level. The groundwater table in the Tauranga Group soils appears to be interconnected with the groundwater in the ECBF materials (both soil and rock). The groundwater monitoring between 17 December 2014 and 6 March 2015 shows that the groundwater levels have decreased by approximately 2m in places. Greater seasonal variations may be recorded as the monitoring period extends into the winter months when increased rainfall and elevated groundwater levels are expected. It is possible, that during winter months, groundwater levels may be at or close to the surface for much of the route.

The NI Phase 1 development predominately consists of open excavation trenching ranging between 2.5 and 4m depth with trench depths in isolated cases up to 8m below ground level. A search has been completed by Auckland Council to identify the groundwater users within the vicinity of the NI pipeline (200m on either side of the proposed pipeline). In general terms, groundwater is usually drawn from very deep aquifers, which are recharged across a very wide area. The NI pipelines are very shallow, so are unlikely to have an effect on groundwater users of any more significance than other stormwater and wastewater networks would in the area.

The site specific permeability assessment including groundwater level measurements are presented in Appendix A.

8 Consideration of ACRP: ALW, PAUP and other relevant guidelines

8.1 General

The Auckland Council Regional Plan: Air, Land and Water (ACRP: ALW) and the Proposed Auckland Unitary Plan (PAUP) (notified on 30 September 2013) determine the requirements for resource consent for activities involving groundwater and settlement effects. We have considered how our assessments relate to groundwater permitted activity criteria set out in the ACRP: ALW and PAUP, as set out in Section 8.2 and 8.3 below.

Section 8.4 below summarises generally accepted guidelines for settlement tolerances which vary depending on rigidity of structures and susceptibility to deformation.

8.2 The Auckland Council Regional Plan: Air, Land and Water

The relevant permitted activity rules from the ACRP: ALW applicable to the proposed NI Phase 1 development is:

- 6.5.76 The diversion of *groundwater* in an *unconfined aquifer* caused by changing the permeability of the *aquifer* at the location of the works by trenching, digging or tunnelling is a Permitted Activity, subject to the following conditions:

  (a) The diversion shall not change the water level regime or direction of flow of the *aquifer* after completion of the works; and

  (b) Any resulting settlement shall not cause adverse effects on buildings, structures and services.

A table summarising our preliminary findings in relation to permitted activity conditions is attached in Appendix C.

8.3 Proposed Auckland Unitary Plan

A preliminary assessment of the geotechnical aspects of the project with respect to the PAUP requirements for "Permitted Activities" has been undertaken as part of this report. The following sections are considered applicable for the proposed NI Phase 1 development:

- 4.17.3.1.3 Water take and use of groundwater.
- 4.17.3.1.4 Diversion of groundwater caused by any excavation, trench, tunnel up to 1m in diameter, or thrust bore.

A table summarising our preliminary findings in relation to permitted activity conditions is attached in Appendix C.

8.4 Guidance on acceptable settlements

8.4.1 New Zealand Transport Agency

Settlements associated with construction of a pipeline beneath State Highway 18 should be considered with respect to the guidelines typically provided by the New Zealand Transport Agency (NZTA). NZTA generally release a number of requirements as part of the documentation for the major road projects in New Zealand. The requirements below are reproduced from the Ngaruawahia project but T&T has sighted these conditions for a number of other large infrastructure projects:

Fill embankments and foundation treatment shall be designed such that the predicted cumulative pavement surface vertical displacements within 5 years after completion of pavement construction are not greater than:

a) 150mm total vertical displacement;

b) 1% transverse differential vertical displacement measured over the formation width either side of the median. Minimum design crossfalls shall be maintained;

c) 20mm differential vertical displacement over any 10m length;

d) In any event, settlement after pavement constructions shall not result in more than 10% change in equivalent design speed at any location.

Condition c) is considered the most applicable when assessing the potential damage along the State Highway 18 crossing.

8.4.2 Services

Services are typically relatively flexible and can typically tolerate relatively large differential settlements. A paper by O'Rourke & Trautmann (1985) recommends a maximum differential settlement of 1:140 for cast iron pipes and brittle utilities with a diameter of 200mm or greater. O'Rourke considers cast iron as the material most susceptible to damage from differential settlement. The majority of the major services along the NI Phase 1 alignment appear to be more flexible materials such as Concrete Lined Steel (CLS) pipes.

A reference slope of 1V:200H has therefore been conservatively adopted for the services along the NI Phase 1 development to assess potential for adverse effects. For any services passing above the pipeline, or within a zone of potential settlement the actual service will need to be checked during detailed design for its tolerance to the predicted settlement magnitude and shape, with specific mitigation measures developed in the instance where tolerances may be approached. Mitigation measures could include physical isolation of the service from the settling ground, relaying the service, or using a construction methodology that provides specific control of the magnitude of settlement that might occur.

8.4.3 Buildings

There are only a couple of potential locations where structures could potentially span across settlement troughs or depressions along the NI Phase 1 alignment. These are presented in Table 8.1 of this report. The NZ building code provides guidance on settlement tolerances in Appendix B B1/VM4, clause B1.0.2:

"Foundation design should limit the probable maximum differential settlement over a horizontal distance of 6m to no more than 25mm under serviceability limit state load combinations of NZS 4203:1992[updated in 2004], unless the structure is specifically designed to prevent damage under a greater settlement."

This clause effectively sets a deflection guidance limit of approximately 1:240 for tolerable differential settlements (NB differential settlement or tilt is commonly expressed as the ratio of differential vertical deflection to horizontal distance over which the differential movement occurs, as adopted within this report).

Further guidance on the tolerance of specific building types and/or uses to differential settlement is provided by Bjerrum, 1963 as summarised in Table 8.1 below. The level of settlement that is generally accepted in New Zealand as being the upper limit for buildings is a total settlement of 50mm and a differential settlement of 1:1,000.

Table 8.1 - Differential settlements and buildings

No.	Description	Limit
1	Limit for typical settlement sensitive machinery	1:750
2	Potential damage to frames with diagonals	1:600
3	Potential limit for cracking	1:500
4	Tilting of high buildings becomes noticeable	1:250
5	Structural damage likely, considerable cracking	1:150

9 Groundwater drawdown and surface settlement estimates

9.1 General

This section first presents the general range of effects that might arise from the NI construction activities, and then summarises the specific effects that are estimated for those areas of particular interest that are set out in Section 4.4.

The estimated groundwater drawdown and settlements are presented on the drawings in Appendix D. The drawings separately present the estimated mechanically induced settlements and groundwater related settlements as well as the inferred radius of influence for each source. The table in Appendix C presents the conditions when the permitted activity criteria from the PAUP and ACRP: ALW are exceeded. Pipe sections where specific geotechnical design is likely to be required have been highlighted in Tables 9.2, 9.3 and 9.5 of this report. Note that in winter months, groundwater levels may rise to close to or at ground surface level. In this instance most of the alignment would have the potential to lower groundwater levels by 2m or more. From a settlement effects perspective, the degree to which the groundwater level is, or may be lowered, below historic low levels is the controlling factor – not absolute groundwater lowering. Our assessments of settlement effects are therefore based on the lowest recorded groundwater level, not the potential highest groundwater level.

Areas where structures are within 10m of the alignment and sections where the total estimated ground settlement (both mechanically and groundwater drawdown induced) exceeds 50mm may require review during the detailed design stage. The review may include seepage analysis using Seep/W and assessments of the proposed retention system using Wallap. Such analysis is likely to reduce the estimates for both mechanically and groundwater drawdown induced settlement.

9.2 Trenches

9.2.1 Groundwater drawdown

The permitted activity criteria from the PAUP plan is exceeded if the natural groundwater levels is reduced by more than 2m. Thirteen (13 No.) standpipe piezometers have been installed as part of this assessment and provides information with respect to the groundwater levels along the alignment. The hydrogeological model is considered appropriate for preliminary design.

Table 9.1 presents the locations where the groundwater level is likely to be reduced by more than 2m. The assessment is based on the lowest recorded groundwater readings between 17 December 2014 and 6 March 2015. The drawing in Appendix D also identifies sections along the alignment where the groundwater level is likely to be reduced by approximately 2m.

Table 9.1 - Groundwater drawdown is likely to exceed 2m – trenched excavation.

Description	Geology	Potential groundwater drawdown (based on recorded low)
Tauhinu Road crossing	ECBF soils	< 3m
Commercial properties near William Pickering Drive	TGA over ECBF soils	3m
William Pickering Drive crossing	TGA over ECBF soils	5m

9.2.2 Excavation deformation settlement

The relationship between potential ground settlement with distance from excavation in soft soil, is provided in Figure 9.1. It has been developed based on the methodology outlined in Peck (1969). The calculations indicate that where relatively deep (greater than 3m) trench excavations are proposed in areas with elevated groundwater levels, specific geotechnical analysis and design would be required to limit potential mechanically induced settlements where structures or services are in the vicinity. The calculations include allowance for soft soils and complications during the construction. The estimated mechanical settlement due to the open excavation is expected to be significantly reduced following detailed design.

In all cases considered, the estimates indicate that settlements can be expected to reduce to minimal levels (5mm or less) some 10-15m from the excavations.

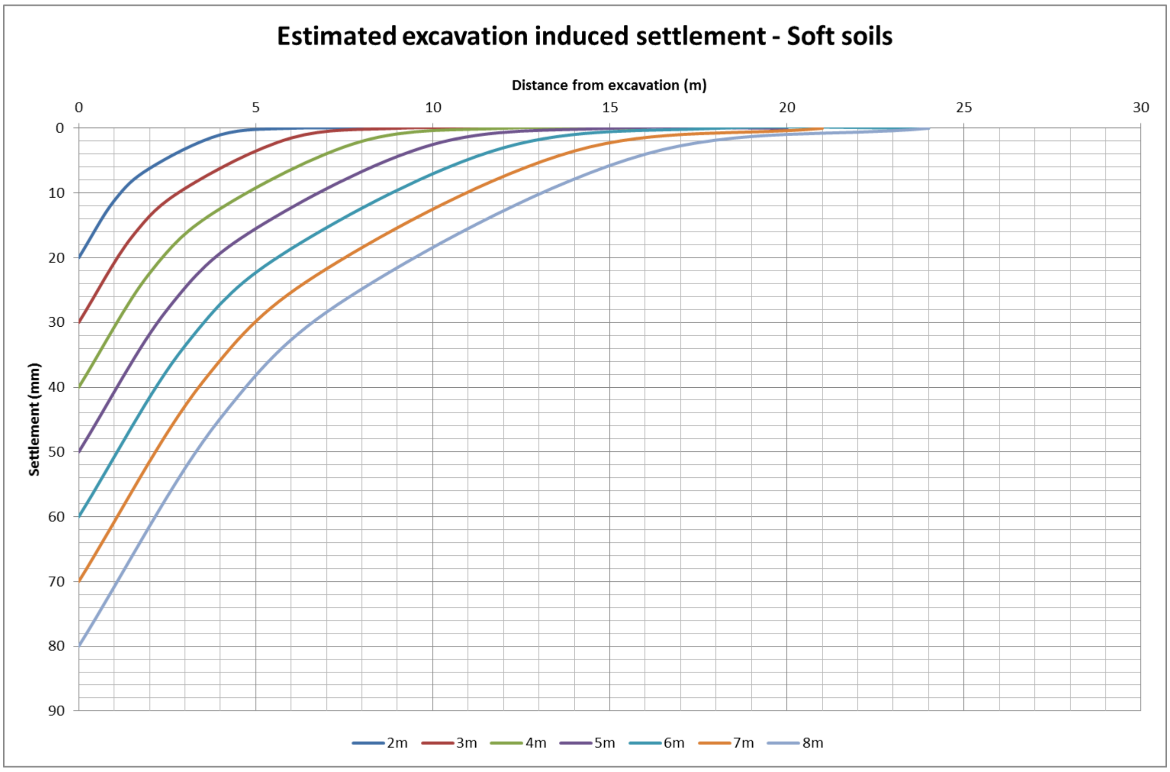

Figure 9.1 – Surface settlement estimates for soft soil for trench excavations caused by the open excavation.

9.2.3 Groundwater drawdown settlement

Construction of trenches has the potential to result in significant groundwater drawdown effects where trenches are constructed within Tauranga Group soils below groundwater level. For deep deposits of Tauranga Group material, where no controls are placed on potential dewatering effects from the open excavation construction, there is the potential for larger magnitudes of surface settlement, as identified on Figure 9.2. In general however, total settlements caused by groundwater drawdown are not expected to exceed the settlement limit of 50mm for buildings (nb – settlement limits for services are likely to be higher depending on material type). These settlements would typically be expected to reduce to minimal levels within 5m of the excavation. The groundwater settlements are based on the lowest recorded groundwater levels in the standpipe piezometers.

Trenches excavated in weathered ECBF soils are significantly less likely to result in large settlements where groundwater is drawn down as a result of construction.

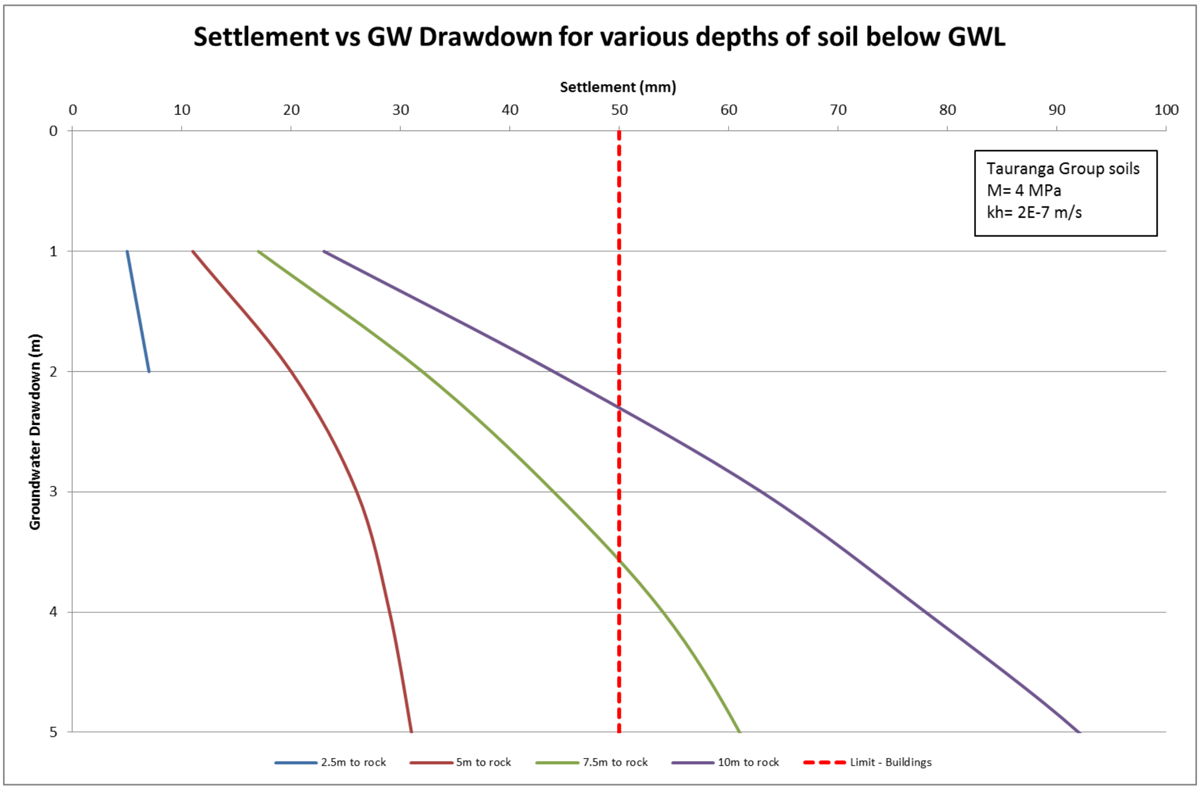

Figure 9.2 –Surface settlement estimates for trench excavations in soft soil caused by groundwater drawdown.

Typical surface differential tolerances are expected to be exceeded for some combinations of drawdown and depth of deposit as shown on Figure 9.3. The differential settlements shown on Figure 9.3 apply where the distance from the road reserve and the structure is 10m or greater. The majority of the settlement is likely to occur at a 5m distance from the trench, i.e. within the road reserve. The largest differential settlement is therefore expected within the road reserve away from any buildings. The settlement guidelines in Section 8 of this report suggest that limits are set at 1:200H for services and 1:1000H for buildings.

In addition to settlement effects due to groundwater drawdown into open trenches, the backfilling of trenches could potentially result in long term changes in groundwater flow. This may be due to the use of either high or low permeability backfill material (relative to the permeability of the natural ground). High permeability backfill may drawdown and/or redirect groundwater flow along the backfill, whereas low permeability backfill may interrupt groundwater flow, causing damming and increases in groundwater levels up-gradient and lowering of groundwater levels down-gradient. These effects can be controlled by appropriately specified backfill of the trenches and as such should be considered as part of final design.

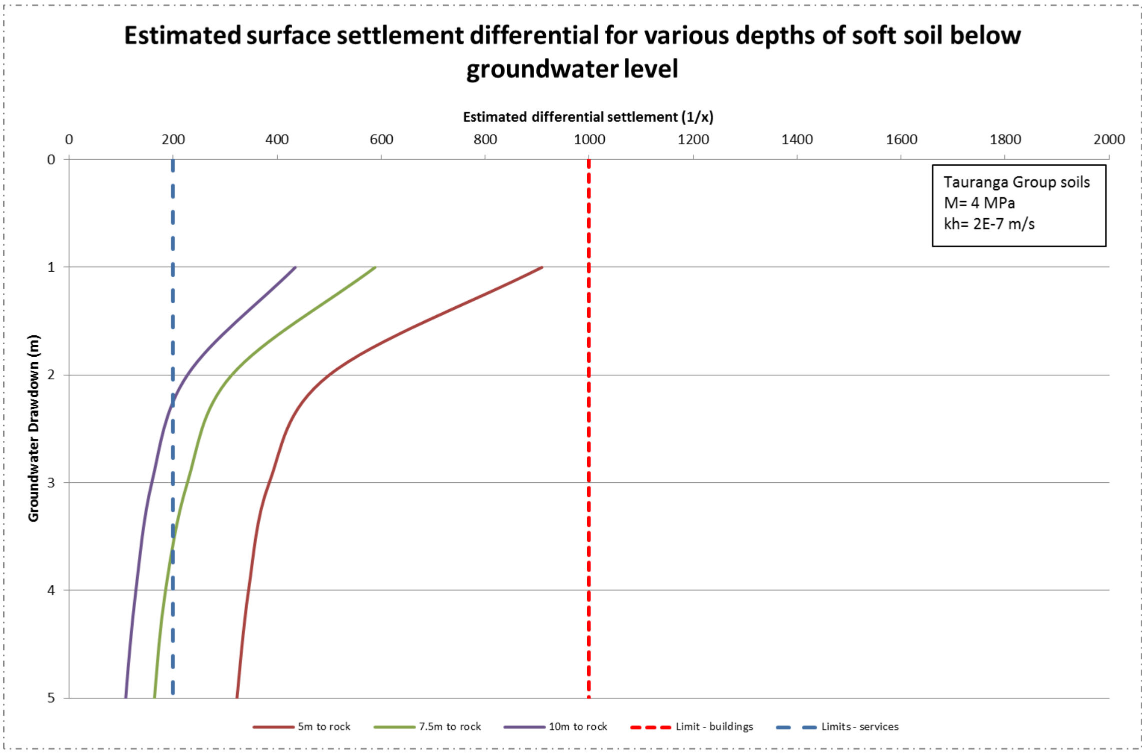

Figure 9.3 –Estimates for differential settlement for trench excavations in soft soil caused by groundwater drawdown.

9.2.4 Groundwater inflows

Potential inflows into trench excavations through Tauranga Group or ECBF materials below the water table are estimated to be up to 0.1 to 0.5 m^3/day per metre length of trench as shown in Figure 9.4.

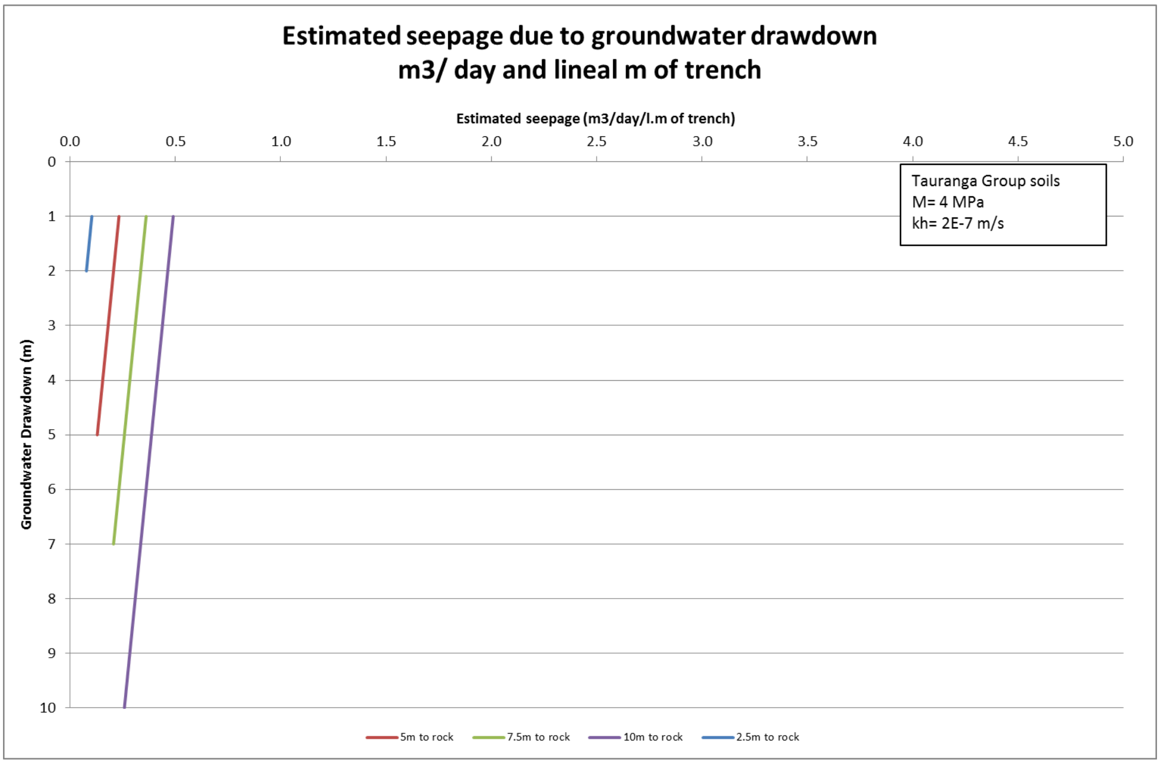

Figure 9.4 –Estimates for seepage due to groundwater drawdown caused by trench excavation.

9.2.5 Summary of settlement and areas for further assessment

The combined effect from both mechanical and groundwater induced settlement generates relatively high total estimates at the ground surface, as presented in Appendix D. The reported total estimated settlement due to trenching ranges between 30 and 110mm.

The total settlement will occur immediately adjacent to the trench and reduces with distance away from the trench. The assessment suggests that the settlement induced by groundwater drawdown is limited to a 5m distance from the trench, i.e. the radius of influence. The inferred lateral effect of mechanically induced settlement caused by the trench excavation is slightly more extensive. However, the mechanical settlement is reduced by half at 2m from the trench for a 3m deep excavation and at 5m from the trench for an 8m deep excavation.

Table 9.1 presents areas along the NI Phase 1 alignment where the expected groundwater drawdown is likely to exceed 2m. The groundwater model is based on widely spaced standpipe piezometers and there are several areas where the inferred groundwater drawdown is in the order of 2m. Longer monitoring interval of the existing piezometers and installation of supplementary boreholes may be required at the detailed design stage.

The majority of the settlement induced by the trench excavation will therefore occur within the road reserve away from any buildings. Table 9.2 presents areas of restricted access where the proposed pipe is close to buildings or other significant assets. These locations are not likely to exceed the criteria set by the PAUP and / or ACRP: ALW plans with the potential exception of the identified commercial development near William Pickering Drive, 169-174 Bush Road and the Vector substation near Bush Road where buildings are located closer than 4m from the trench. However, further analysis during the detailed design stage is likely to significantly reduce the mechanical settlements adjacent the trench.

Table 9.2 - Areas of restricted access, trenched excavation

Description	Geology	Potential groundwater drawdown (based on recorded low)	Approximate radius of settlement / influence	Approximate distance from road reserve to asset
11 Traffic Road. Residential	ECBF soils	< 2m	8m	6m
30 to 34 Newbury Place. 222 to 224 Schnapper Rock Road	Engineered fill and ECBF soil	0m	8m	4 to 8m
1-13 Appleby Road. Residential.	TGA over ECBF soils	~ 2m	8m	8 to 10m
Commercial property at 327 Albany Highway[1]	TGA over ECBF soils	3m	8m	N/A
Commercial development currently under construction at 325 Albany Highway and 14 John Glenn Drive[2].	TGA over ECBF soils	3m	8m	N/A
169-174 Bush Road and Vector substation. Commercial[1].	TGA over ECBF soils	< 2m	8m	4 to 8m

[1] The proposed alignment is located closer than 4m to an existing building and the excavation extends below the groundwater level. The permitted activity according to the PAUP plan is therefore exceeded.

[2] The proposed alignment crosses a property currently under development. Watercare will work with the developer to manage the impact of NI – Phase 1 construction on the proposed development.

Buried services are likely to be exposed to relatively large settlement when they are located close to the open trench excavation. In such situations, specific attention will need to be paid to construction methodology to ensure that the excavations do not affect the service, irrespective of the theoretical assessment of settlement affects. Disturbance of the services is just as likely to be as a result of accidental over excavation, or other such factors in these areas. However, accepting this, the buried services are considered to be more ductile and have greater tolerance of differential settlements. A reference slope of 1V:200H has been conservatively adopted for the services along the NI Phase 1 development to assess potential for adverse effects. The services are also likely to experience lower settlements at depth as the settlement effect observed at the surface are cumulative and tend to be greater. Higher settlement tolerances can therefore be expected within a 5m wide zone of the trench – subject to detailed assessment of the final pipe alignment and adjacent services.

9.3 Micro tunnelling

9.3.1 Groundwater drawdown

The micro tunnelling assessment is conservative as it assumes groundwater is allowed to fully respond to tunnelling i.e. groundwater locally drawn down to tunnel excavation invert. The groundwater data suggests that the groundwater level can be reduced by up to 3m at the southern end of the micro tunnel as presented in Table 9.3. The pipe level rises towards the northern side of the tunnel while the groundwater depth decreases resulting in negligible groundwater drawdown in this location.

During construction, the potential for groundwater drawdown is linked to the length of time portions of the excavation are open, prior to the final pipeline being installed. Typical construction operations restrict this period as a matter of course, hence minimising the potential for drawdown to fully develop. The estimated groundwater drawdown in Table 9.3 is therefore conservative.

Table 9.3 - Groundwater drawdown is likely to exceed 2m – micro tunnel

Description	Geology	Potential groundwater drawdown (based on recorded low)
Southern access shaft.	TGA over ECBF soils	< 3m
Southern end of State Highway 18 crossing.	TGA over ECBF soils	< 3m

9.3.2 Ground loss settlement

The estimated mechanical settlement for the proposed micro tunnel below the Upper Harbour Highway is presented in Table 9.4. The calculations suggest that the mechanical settlement induced by the micro tunnelling is within acceptable limits provided that groundwater drawdown is controlled during the construction. The mechanical settlement is likely to increase if the cover to crown is reduced or the pipe diameter is increased. A final design check should be completed as part of the detailed design.

Table 9.4 - Estimated mechanical settlement - micro tunnel

Location	Pipe diameter	Approximate cover to crown	Inferred soil class	Estimated mechanical settlement
Southern end (Chainage 122m)	710mm	5.5	I	5mm
Northern end (Chainage 212m)	710mm	4.5	III/II	20mm

9.3.3 Groundwater drawdown settlement

Potential for groundwater drawdown, resulting from micro tunnelling, to result in surface settlement is primarily limited to the more compressible Tauranga Group soils which are likely to consolidate as a result. Table 9.5 presents the estimated settlement which may be induced due to groundwater drawdown during the tunnelling. Preliminary analysis suggests that approximately 15 to 20mm of settlement may occur at the southern end of the micro tunnel as a result of groundwater drawdown if the micro tunnel operation is complete within 12 weeks and the pipeline is made water tight upon completion. More direct means of settlement control, such as operation of the slurry TBM in closed mode, further reduce the potential for substantial groundwater drawdown to develop.

The settlement estimates are considered to be applicable for the temporary access shafts installed on either side of the SH18 Crossing to accommodate the micro tunnelling equipment. The Hobsonville PS is supported on driven steel Universal Column (UC) piles and is located sufficiently far away (approximately 15m) from the proposed southern access shaft to be affected by the estimated groundwater drawdown.

Table 9.5 - Estimated groundwater settlement - micro tunnel.

Location	Pipe diameter	Potential groundwater drawdown (based on recorded low)	Thickness of Tauranga Group soils	Estimated groundwater settlement
Southern end	710mm	< 3m	8m	35mm
Northern end	710mm	< 1m	6m	5mm

9.3.4 Groundwater inflows

Groundwater inflows, based on CIRIA C515, into the proposed 710mm diameter pipe have been estimated assuming no lining and full groundwater drawdown. This approach is expected to provide conservative (over) estimate of potential inflows. Estimated groundwater inflows into the pipe are between approximately 0.1 and 0.5 m^3/day per metre length of tunnel, depending on tunnel depth and groundwater level.

The temporary access shafts for the micro tunnel machinery have also been assessed in accordance with the recommendations in CIRIA C515. The assessment indicates that the inflow at the southern access shaft is likely to be in the order of 6 to 7m^3/day. The inflow volume is based on the access shaft area being 6m x 8m and unlined.

The base of the northern access shaft is located approximately 1.4m higher than the southern access shaft and the groundwater table is approximately 1.7m lower in this location. The inferred groundwater drawdown is therefore less than 1m and the expected inflow volume less than 1m^3 per day.

9.3.5 Summary and recommendations for detailed assessment

The total surface settlement is estimated at the northern side of the SH18 crossing is likely to be less than 25mm while around 40mm of total settlement is expected at the southern side as presented in Appendix D. The NZTA settlement guidelines require less than 20mm of differential vertical displacement over any 10m length (Section 8.4.1 of this Report) which the settlement estimates on the southern side of the SH18 crossing exceeds. The estimated settlement at the northern side of the SH18 crossing marginally exceeds the NZTA guidelines.

The largest settlement is expected directly above the micro tunnel and reduces with distance away from the tunnel. The radius of the mechanical settlement is relatively large and is not likely to cause damage induced by differential settlement. However, the estimated settlement trough caused by groundwater drawdown is steep and the differential settlements are likely to cause some damage to the SH18 if not controlled. Consolidation settlement is time dependant and can be managed by either restricting the construction programme. Alternatively, the micro tunnel machine can operate in closed mode which significantly reduces the potential for groundwater drawdown.

The estimated total settlement at the southern access shaft for the micro tunnel operation is 75mm while the total settlement is likely to be 50mm at the southern access shaft. Both shafts are located more than 15m from any structure but are likely to require detailed retention design due to their excavation depth and potential hazard to construction workers and machinery. Table 9.3 presents areas along the micro tunnel where the expected groundwater drawdown is likely to exceed 2m while Table 9.6 presents areas where additional design is recommended to provide a safe work environment for the temporary construction design case.

Table 9.6 - Areas likely to be subject to detailed design - micro tunnel

Description	Geology	Potential groundwater drawdown (based on recorded low)
Southern access shaft.	TGA over ECBF soils	< 3m
Northern access shaft.	TGA over ECBF soils	< 1m

10 Discussion

10.1 Open trench excavation

Most of the route will be constructed by open trenching. In particular, open trenching is the primary construction method through built up areas. It therefore is the method that is most likely to affect developed land, if effects do develop. The alignment runs predominately in existing road reserves, hence the greatest impact is likely to be surface disturbance and impacts on buried services with little to no impact on buildings located approximately 10m from the trench.

For the open trench excavation, the most significant hazard relates to Tauranga Group soils and high groundwater levels. Where trenches are constructed in deep deposits of Tauranga Group material, and construction methodologies are adopted that could lower the groundwater level, there is potential for surface settlement of 40 to 50mm. Mechanical settlements caused by relaxation of soil and construction methodology are likely to further increase the total settlement resulting in the upper limit of 50mm of settlement being exceeded. Specific design will need to consider this potential, and utilise appropriate construction methodologies that limit expected settlements to acceptable levels.

Open trench excavations constructed in weathered ECBF soils are significantly less likely to result in large settlements where groundwater is drawn down as a result of construction, but are likely to be affected by large mechanical settlements.

In locations where the ground is sensitive to groundwater drawdown effects, methodologies such as those employing the following techniques are likely to be required to manage drawdown and surface settlement to acceptable levels (i.e. less than a "minor effect") in surrounding geology;

- <u>Trench shields</u> comprising steel frames which support the side walls once the excavation is complete. Props can be installed across the trench to increase the robustness of the retention system.
- <u>Sheet piles</u> are likely to be installed at deeper sections of the alignment (3m plus) where potential for groundwater drawdown is large or if locally poor ground conditions are identified. The sheet piles are driven or vibrated into place and the excavation is completed subsequently providing a safer work environment and limiting groundwater inflows. Props may be required across the trench to further reduce deformations. These may be required to mitigate settlement/construction risks in areas such as those identified in Table 9.1 and Table 9.2.

The groundwater monitoring between 17 December 2014 and 6 March 2015 shows that seasonal variations can be expected along the pipe alignment. The construction methodology should include allowance for elevated groundwater levels if the pipe is installed during the winter months.

Alternatively, and by careful positioning it may be possible to distance the excavations sufficiently far away that settlement effects at structures are nil, or reduced sufficiently to mitigate effects.

In addition to settlement effects due to groundwater drawdown into open trenches, the backfilling of trenches could potentially result in long term changes in groundwater flow. These effects can be controlled by appropriately specified backfill of the trenches which will act as groundwater cut-offs and will not need to be considered as part of final design.

10.2 Micro tunnelling

Micro tunnelling is only planned for relatively short lengths of the pipeline at the SH18 Crossing. The potential for effects associated with the micro tunnelling, should they develop, is therefore localised.

The current model assumptions are generally conservative and result in high estimates of settlement for the combination of potential groundwater drawdown related settlement and mechanical settlement. The analysis suggests that the groundwater drawdown portion of the settlement can be managed by adopting a short construction period during which the groundwater can be drawn down and settlement occur. Alternatively the micro tunnel machine can be operated in closed mode which reduces the potential for groundwater drawdown.

Approximately 35mm of settlement may be generated at the southern end of the micro tunnel due to groundwater drawdown and approximately 5mm due to mechanical settlement. This is above the guidelines issued by NZTA (section 8.4.1). The combined settlement at the northern side of the SH18 crossing is likely to be less than 25mm, i.e. marginally above the NZTA guidelines. The pump station is also likely to be located at sufficient distance from the temporary shaft not to be affected by the inferred groundwater drawdown. The retention design for both access shafts are, however, likely to be subject to specific geotechnical design to provide a safe work environment for the construction workers and machinery.

Previous construction experience, and a review of international literature relating to pipe jacking, highlights that the most likely reason for unexpected large or damaging settlements relates to non-adherence to the assumed construction methodology (e.g. maintenance of appropriate balancing pressures). No allowance has been made for potential additional settlement that may arise during construction relating to operational difficulties of the excavation, jacking and pressurisation equipment. Such additional settlements could be minor (a small percentage of the overall settlement) or extreme - the formation of a sinkhole - depending on the circumstances. Such effects can be managed through appropriate monitoring and construction control.

Combined with installation of watertight seals at pipe joints, the potential for short and long term groundwater drawdown in the Tauranga Group and uncontrolled fill materials can be readily mitigated, such that settlement estimates are within acceptable limits.

10.3 Horizontal drilling/ marine trenching effects

HDD tunnelling / marine trenching is primarily utilised for constructing the pipeline under streams, and the harbour. The potential for effects to property and buildings associated with the HDD tunnelling, should they develop, is therefore very low.

10.4 Effect on buried services

The buried services are likely to be exposed to relatively large total and differential settlements as the services are located much closer to the open trench excavation than any of the buildings along the alignment. Where a service is offset from the NI pipe, it may also experience horizontal displacement associated with ground loss at the excavation face. Services running perpendicular to the trench are considered to be at the highest risk of damage. Services parallel to the pipe are also likely to experience similar total settlement but with gentler settlement slope, i.e. differential settlement.

Damage typically manifests as either opening of joints (for jointed pipes/ducts) or cracks in the cables or ducts. An allowable slope of 1V:200H has been conservatively adopted for the services along the NI Phase 1 development. For any services passing above the pipeline, the actual service will need to be checked during detailed design for its tolerance to the predicted settlement magnitude and shape.

10.5 Consideration of ACRP: ALW and PAUP

The Auckland Council Regional Plan: Air, Land and Water (ACRP: ALW) and Proposed Auckland Unitary Plan (PAUP) provide rules with respect to groundwater take and groundwater drawdown as a consequence of a development. A review of the relevant permitted activity provisions of these plans has been completed in relation to our assessment of groundwater and surface settlement and is summarised in the Tables in Appendix C.

Settlement assessments generally conclude that accepted settlement tolerances for buildings can be met along the alignment, provided that the mechanical settlements are controlled, i.e. the settlement induced by groundwater drawdown alone is less than 50mm immediately adjacent the trench. However, permitted activity criteria are likely to be exceeded at locations along the NI corridor, as identified in Appendix D.

In reaching this conclusion we note that our assessments draw on a limited monitoring dataset from the 2014 / 2015 summer (low) groundwater levels. In winter months, groundwater levels may rise to close to or at ground surface level. In these conditions excavations in most of the alignment would have the potential to lower groundwater levels by 2m or more. From a settlement effects perspective, the degree to which the groundwater level is, or may be lowered, below historic low levels is the controlling factor – not absolute groundwater lowering.

The groundwater investigations suggest that the majority of the NI Phase 1 alignment is underlain by an unconfined aquifer where hydrostatic groundwater pressures are generally encountered between 1 and 3m below the ground level. The excavations therefore extend more than 2m below the groundwater level at several locations along the alignment and long term dewatering needs to be considered as outlined above.

11 Monitoring programme

11.1 General

A monitoring programme should be implemented to monitor construction of the NI Phase 1 lines and manholes, specifically targeting those areas where significant settlement effects have the potential to affect existing buildings and services as presented in Table 9.1, Table 9.2 and Table 9.6. It would measure the effect that construction has on the groundwater system and confirm that any associated surface settlement is within acceptable limits. Buildings within the zone of expected settlement effects (as outlined in Section 4.4) should be subject to a building condition survey prior to construction. Building surveys are carried out by structural engineers and typically include visual inspection of pre-existing cracks and photographs.

A well scoped monitoring programme also provides advanced warning of the potential for effects to vary from those estimated from pre-construction assessments. On the basis of the advanced warning, construction can take account of the variation as necessary to control effects.

11.2 Approach to monitoring

The recommended approach for monitoring is to prepare a programme which records groundwater and surface level changes and compares the field measurements with the calculated estimates. Trigger levels should be set so that additional monitoring (frequency and/or locations) is required in the event that alert levels are approached. The alert levels are typically set to 75% of the maximum acceptable settlements, the so-called alarm levels.

In areas where the settlement hazard is estimated to be low (such as in low compressibility geology where little groundwater drawdown is expected), monitoring locations would be more widely spaced than in areas where settlement hazard is estimated to be higher (such as large groundwater drawdown and presence of low strength Pleistocene deposits).

Contingency measures would be developed for implementation if alarm levels were threatened.

11.3 Baseline data

For groundwater and surface level monitoring, a clear understanding of seasonal behaviour and survey repeatability is of key importance in interpreting the response of monitoring installations during the construction period. A clear understanding of seasonal behaviour can be achieved from baseline monitoring records that extend for at least 12 months prior to commencement of construction activities.

Similarly, the degree of survey repeatability (variance in surface levels at a given point between successive survey rounds) should be established by a repeat survey at the locations identified in Table 9.1, Table 9.2 and Table 9.6 prior to commencing construction.

11.4 Monitoring installations

Monitoring of NI Phase 1 construction could include:

- A network of surface level monitoring marks installed on representative, or critical, cross sections to the pipeline alignment.
- Additional surface level monitoring marks located on or near settlement sensitive structures and building condition surveys including photographs should be completed prior to construction.
- Piezometers installed in close proximity to the pipelines or manholes to monitor groundwater level response to construction within geological units with potential to consolidate. The

standpipe piezometers installed as part of the Factual Geotechnical Report (Ref 2) should be maintained where possible and monitoring continued for approximately 12 months.

12 Conclusions and recommendations

The Northern Interceptor Phase 1 utilises construction methodologies that are frequently used and are well understood by the Engineering and Contracting community. Very similar construction methodologies are used for the installation of pipelines throughout Auckland and New Zealand. The methodologies are routinely used for the installation of pipelines for stormwater networks, wastewater networks, and bulk water supply pipelines.

In general terms, the effects that are relevant to this assessment for third parties are:

- Surface settlement that arises from construction of the pipeline. This effect is of significance when the surface settlement affects existing buildings or services;
- Groundwater lowering (draw down) that might occur during construction, or persist after construction is complete. The groundwater draw down may cause surface settlement;
- Significant groundwater lowering has the potential to affect users of groundwater that might be relying on the groundwater resource.

Additional effects that are relevant to the construction of the project are:

- The rate and volume of groundwater inflow into excavations during construction. High groundwater inflows have the potential to complicate construction processes, and are typically controlled by specific conditions in the Consent.

The full length of the proposed NI Phase 1 project is being considered as part of this groundwater and settlement assessment. The corridor is typically quite shallow (2 -4m below ground level, consistent with the range of depths for much of the stormwater and wastewater networks in Auckland) and runs predominately in existing road reserves, hence the greatest impact is likely to be surface disturbance and impacts on the roads and on buried services if they are close to the excavations. The potential for effects on properties and buildings adjacent to the corridor will ultimately depend on the location of the pipeline within the corridor, however construction will primarily be within the road carriageway as shown on the Drawings. Large sections are also in parks and open space.

Settlement assessments generally conclude that accepted settlement tolerances for buildings can be met along the alignment, provided that the mechanical settlements are controlled, i.e. the settlement induced by groundwater drawdown alone is less than 50mm immediately adjacent the trench. The drawings in Appendix D present the groundwater drawdown and settlement assessments carried out as part of this report.

The construction methodology should include allowance for elevated groundwater levels if the pipeline is installed during the winter months.

HDD tunnelling / marine trenching is primarily utilised for constructing the pipeline under streams, and the harbour. The potential for effects to property and buildings associated with the HDD tunnelling, should they develop, is therefore very low.

A monitoring programme should be implemented to monitor construction of the NI Phase 1 lines and manholes, specifically targeting areas where potential impacts on existing buildings and services (as presented in Table 9.1, Table 9.2 and Table 9.6) have been identified. The proposed monitoring programme is outlined in Section 11.

13 References

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14 Applicability

This report has been prepared for the benefit of Watercare Services Ltd with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose without our prior review and agreement.

Tonkin & Taylor Ltd

Environmental and Engineering Consultants

Report prepared by:	Authorised for Tonkin & Taylor Ltd by:
....	....
Karin Speight/Claudia Harford	Peter Roan
Geotechnical Engineers	Project Director

Report reviewed by:

....

Graeme Twose

Senior Geotechnical Engineer

Karin Speight/KJ
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Appendix A: Technical assessment

- Site investigations
- Geotechnical assessment
- Hydrogeological assessment
- Settlement assessment methodology

1 Site investigations

1.1 Project specific investigations

1.1.1 Terrestrial investigations

Subsurface investigations were carried out between 19 November 2014 and 18 December 2014. The investigations comprised 12 rotary cored boreholes, 8 Cone Penetrometer Tests (CPTs) and 20 hand auger holes. The investigation locations are illustrated on 28773.210-F1 to F6. The full test results are appended to the Factual Geotechnical Report (Ref 2).

The rotary cored boreholes were completed using a tracked rotary machine drill rig operated by McMillan Drilling Ltd and the CPTs were carried out using a Truck mounted CPT rig by Ground Investigation Ltd. The hand auger boreholes were completed by T&T staff. Upon completion, all machine boreholes were fitted with 32mm diameter PVC standpipe piezometers to allow for groundwater level monitoring and groundwater sampling. Downhole pressure transducers (Solinst level loggers) were installed in selected boreholes and the reading frequency was set to every 15 minutes. A barologger was also installed to allow for barometric correction of the water level loggers.

The following laboratory testing was completed as part of the investigations; 3 No. 1D consolidation tests, 5 No. Triaxial (undrained consolidated, SUP) tests, 4 No. Atterberg Limit tests, 4 No. Particle Size Distribution tests and 9 No. Uniaxial compression tests. The interpretation of the data is presented in Appendix C and the full test results are appended to the Terrestrial Factual Geotechnical Report (Ref 2).

1.1.2 Marine investigations

T&T carried out marine investigations at the Upper Harbour Crossing between 24 July and 15 August 2014. The field work included 8 rotary cored boreholes and bathymetric survey of the sea bed. The machine boreholes were completed by Drill Force Ltd and the bathymetric survey by Scantec. The following laboratory testing was completed, as part of the investigations; 3 No. Triaxial (undrained consolidated, SUP) tests, 6 No. Particle Size Distribution tests and 16 No. Uniaxial compression tests. The full test results are appended to the Marine Factual Geotechnical Report (Ref 3).

Opus carried out supplementary investigations at the Upper Harbour Crossing and at Lucas Creek between 30 October and 5 November 2014. The machine boreholes were completed by Drill Force Ltd. 10 No. Uniaxial compression tests were completed as part of the investigations. The full test results are appended to the Marine Factual Geotechnical Report, Opus (Ref 4).

1.1.3 Potholing investigations

Potholing investigations were completed by Drilltech 1996 Ltd in October 2014 (Ref 5). The investigations entailed excavation and uncover of selected services in the road reserve along the NI Phase 1 alignment. Drilltech then confirmed the pipe diameter, material and noted the type of pipe, i.e. watermain, sewer etc. The depth to top of pipe and horizontal distance to kerb were recorded by tape measure as part of the assessment and photographs taken of the excavated pit and the reinstatement of the sealed pavement.

1.1.4 Site walkover

A T&T geotechnical engineer and a T&T engineering geologist carried out a site walkover on 20 October 2014 to identify areas which may be susceptible to settlement or groundwater drawdown. A further walkover was carried out by a T&T geotechnical engineer along John Glen Avenue between

Albany Highway and William Pickering Drive on 15 May 2015. Existing developments such as buildings and services which may be susceptible to groundwater and settlement effects were recorded along with rock exposures at the bottom of gullies and streams.

1.2 Information available from other projects and sources

1.2.1 Review of aerial photographs

T&T has completed a review of available photographs on the Auckland Council GIS website for the purposes of the settlement and groundwater drawdown study. The focus has been to identify areas of historic gullies where elevated groundwater levels and historic fill may be encountered.

1.2.2 Hobsonville sewer pump station

T&T carried out investigations for the Hobsonville Sewer Pump Station upgrade in 2009. The investigations included drilling of 1 No machine borehole to 19.5m depth (BH1) using rotary drilling and 3 No machine boreholes to 6m depth (BH2, BH3, BH3A) using percussion techniques. Standpipe piezometers were fitted in the boreholes and divers were installed at 5.5m bgl in BH1 and at 0.8 to 4m bgl in BH3. Consolidation tests for settlement estimate purposes were carried out at 4.85m and 11.4m bgl in BH1.

The ground investigations also included 4 No. CPT tests with dissipation tests at selected depths. The CPTs extended between 11.9m and 13.3m bgl. The investigation locations are illustrated on 28773.210-F1 to F6 but are not attached to this report.

1.2.3 SH18 investigation data

The New Zealand Transport Agency (NZTA) commissioned ground investigation information for the approaches and pier locations for the new SH18 upgrade. Watercare and NZTA have an agreement in place to make that historic borehole information available to the Northern Interceptor project to supplement the specific ground investigation data that has been obtained as part of the marine investigation package.

The locations of historic ground investigation boreholes are illustrated in Appendix A as guidance, although that information is not collated in this document.

1.2.4 Schnapper Rock development

T&T was involved in the development of the Schnapper Rock sub-division and carried out numerous hand auger boreholes and construction observations across the sub-division area. The area has been subject to large scale earthworks to form level platforms for the lots.

1.2.5 North Shore Golf Course

The North Shore Golf Course holds an existing Consent to extract groundwater from the Waitemata aquifer. The consent allows the golf course to extract a combined total 50,500m^3/year from three boreholes. Bore 1 (ref no 32432) is located near Appleby Road and the proposed Northern Interceptor alignment, as shown on the Borehole Location plans in Appendix A.

2 Geotechnical assessment

2.1 Structural geology

The structural geology within the study area is limited to faults, folds, joints and bedding within the underlying ECBF only. The overlying Pleistocene age Tauranga Group material typically postdates the faulting within the ECBF and so these faults do not extend through the Tauranga Group.

Bedding in the ECBF within the Auckland area is often orientated sub horizontally, however the presence of faulting or folding results in steeper orientations often being developed. Cliff sequences around the foreshore often provide guidance on the variability in rock structure. Bedding orientations indicted on maps of the foreshore at either end of the proposed harbour crossings indicate a range of orientations between 30° and 60° from the horizontal.

Large scale normal faulting is illustrated parallel to and underlying Te Wharau Creek. Smaller scale normal faulting and small scale folds have also been mapped in the foreshore cliffs of both Greenhithe and Hobsonville, these are common within the ECBF and typically disrupt the laterally continuous sandstone and siltstone units.

2.2 Site walkover

A T&T geotechnical engineer and an Engineering Geologist completed alignment walkover inspections on 20 October 2014 (full alignment) and 15 May 2015 (Albany Highway to William Pickering Drive). Relevant observations made at the time of the inspections from the western to northern ends of the Phase 1 project are summarised below.

- The works will occur at the edge of a stream gully adjacent to the Hobsonville PS.
- An existing stormwater pond is located near the proposed thrust pit to the north of the SH 18 Crossing.
- A Summerset rest home is being developed on the north side of the Squadron Drive onramp; earthworks, construction, and production-bore drilling was occurring at the time of the drive-by.
- ECBF rock is exposed in the wave cut platform at the bottom of Rahui Road where the directionally drilled pipe will emerge.
- The creek at the Greenhithe Road / Sunnyview Road is eroded down into ECBF rock.
- The alignment passes through largely grassed/unpaved areas of Wainoni Park to the Te Wharau Creek shore line. A stream cuts across the park with a pedestrian access footbridge. Tauranga Group materials were encountered at the base of the stream. Similar soils were identified at the southern edge of Te Wharau Creek.
- The alignment skirts around the North Shore Cemetery and the south-eastern corner of their crematorium. ECBF rock is exposed in the floor of the creek within the cemetery grounds. ECBF rock also outcrops in Te Wharau Creek foreshore in this area.
- The alignment crosses the North Shore Golf Club driving range and carpark. ECBF rock is exposed in the creek between the Schnapper rock subdivision and the golf club.
- The alignment crosses 325 Albany Highway just south of a building on 327 Albany highway.
- A range of commercial retail and light industrial properties such as mechanics and engineering workshops are located along Piermark Drive. The properties are well separated from the road reserve.
- On the east side of Bush Road, the alignment runs down an access way between a Kea rental depot and a Vector substation. The Vector substation comprises a gravelled switchyard, with two sets of transformers and circuit breakers, and an equipment shed.

- ECBF rock is present at shallow depth in the base of Alexandra Stream. Geological maps indicate outcrop to the south and north of the crossing on the alignment, but these have not been able to be verified by filed mapping.
- A reserve/sports field is located just west of the Rosedale wastewater plant. The alignment primarily cuts through the parkland/reserve area and a large pond is located to the north of this area.

2.3 Geological units

2.3.1 Fill

Fill material was encountered in many of the hand auger and machine boreholes on the alignment. This material typically comprised re-worked soft to stiff, clay/silt mixtures derived from natural Tauranga Group or ECBF soils. The fill material is often underlain by a thin layer of buried topsoil at the contact with natural underlying material.

Locally around the Hobsonville Pump Station (BH t01) the fill material comprises a mixture of construction debris (concrete, steel, timber) and silt/clay soils. The Causeway fill has typically been logged as cohesive and appears to comprise re-worked materials of the Tauranga Group and East Coast Bays Formation.

2.3.2 Marine mud

The Marine Mud is encountered at the Upper Harbour crossing and at Lucas Creek. The marine mud typically comprises very soft to soft clayey silts and loose silty sands.

2.3.3 Tauranga Group alluvium

Tauranga Group materials are typically light grey to grey brown clay/silt or silty sand. Cohesive units have been described as soft through to stiff, while sand dominated units are described as loose through to dense. Organic material is disseminated through the Tauranga Group soils, with some boreholes exhibiting thicker units of soft organic clay or peat, these are primarily located in the area near the Hobsonville PS.

2.3.4 Residually to weathered East Coast Bays Formation (ECBF)

The weathered ECBF consists of two geological units subjectively differentiated by the degree of in-situ weathering. These units are defined as residual soil and weathered ECBF. The residual soil comprises either a firm to stiff, orange brown to brownish grey silt/clay mixture, or a loose, grey mottled orange brown silty sand. The weathered ECBF is typically either a very stiff to hard grey silt or a medium dense to very dense grey sand. This material usually retains some relict rock structure (bedding and jointing) but behaves as a soil material.

2.3.5 ECBF rock

The underlying basement rock along the proposed Northern Interceptor alignment is an extremely weak to weak interbedded sandstone and siltstone. Most of the boreholes carried out for the investigation programme encountered the unweathered ECBF rock at varying depths between 3.5m and 15m deep. Exceptions to this were in BH-t5 (>15.5m), BH-t6 (>15.5m) and BH-t15 (>21m) where the underlying rock mass was beyond the depth of the finished investigation boreholes.

2.4 Alignment geology

2.4.1 General

This section provides a brief summary of the expected geology along the alignment. Section 6.6 above summarises the considerations which will need to be considered for the respective geological formations. The Geological Sections in Appendix A provides further information and the full test results are appended to the Factual Geotechnical Report (Ref 2 to 4).

2.4.2 Hobsonville PS and SH 18 crossing

This section is underlain by 5 to 15m of Tauranga Group soils over ECBF. The area near the Hobsonville PS to the south of the SH 18 is likely to be underlain by organic lenses and peat. Borehole BH-T1 suggests that the base of the proposed access shaft for the micro tunnel will be formed in firm to stiff organic clay of high plasticity.

The Tauranga Group soils appear to become more competent to the north of the Upper Harbour highway where stiff to very stiff interlayered silts, clays and sands have been identified. The sandier materials appear to be located below the proposed invert of the micro tunnel.

Groundwater (summer) is encountered between 1 and 3m below ground surface in this low lying area.

2.4.3 SH18 to Causeway

This section forms part of the causeway for the SH18 where between 3 and 4m of fill overlies a thin layer of Tauranga Group in turn overlying ECBF. The fill is generally derived from Tauranga Group and ECBF soils and has been compacted in place. The causeway fill has likely been placed over a number of years but appears to be of reasonable quality.

The groundwater level is likely to coincide with the sea level (approximately RL 0m) and the top of the highway is located at RL 4m to RL 5m.

2.4.4 Upper Waitemata Harbour crossing

This part of the alignment will be constructed using horizontal drilling techniques or by marine trenching. The sea bed typically comprises marine mud which consists of very soft to soft silts over ECBF. ECBF rock (SPT N>50) is encountered at sea bed level in BH8 (RL -8.5m), BH7 (RL -9m) and BH6 (RL -10.5m).

2.4.5 Rahui Road to Greenhithe Road

This area is underlain by EBCF materials and ECBF rock is exposed in the wave cut platforms where proposed open trenching commences. The weathering profile increase in thickness as the ground rises towards Tauhinu Road and the depth to ECBF rock is greater than 15m at the top of the ridge.

Groundwater (summer) is encountered between 1 and 3m below ground surface.

2.4.6 Greenhithe Road to South Wainoni Park

This area is underlain by up to 5m of Tauranga Group Material over ECBF materials. The alluvium is typically stiff to very stiff and comprises interlayered clay, silt and sands. The ECBF rock is exposed in the stream at the intersection of Greenhithe Road and Sunnyview Road.

Groundwater (summer) is encountered between 3 and 5m below ground surface.

2.4.7 Wainoni Park (South and North)

The geotechnical investigations have not been completed through Wainoni Park at the time of writing. On the basis of published geological maps, field mapping and nearby investigations we anticipate that the park is largely underlain by Tauranga Group materials. ECBF is anticipated to be at shallow depth beneath the Tauranga Group, but we do not anticipate encountering rock within the excavations. This will need to be verified by ground investigation as the design proceeds.

Groundwater (summer) is expected at 1 and 3m below ground surface.

2.4.8 Te Wharau Creek crossing

This part of the alignment is proposed to be constructed using horizontal drilling techniques below Te Wharau stream, a North South trending tributary of Lukas Creek. The borehole investigations suggest that 1 to 3m of marine mud is located over ECBF materials. Extremely weak ECBF rock is encountered at approximately RL 2.3m in borehole BHL1 completed by Opus (Ref 4).

2.4.9 North Shore Memorial Park

This area is underlain by up to 10m of Tauranga Group soils over ECBF materials. The alluvium is typically stiff to very stiff and comprises interlayered clay, silt and sands. The ECBF rock is exposed in the stream running through the park.

Groundwater (summer) is encountered between 1 and 3m below ground surface.

2.4.10 North Shore Memorial Park to North Shore Golf Club

The Schnapper Rock subdivision was constructed during an earthworks operation around 2006 and any fill is expected to be of engineered fill standard. The area is underlain by up to 5m of Tauranga Group soils over ECBF materials.

Groundwater (summer) is expected between 3 and 5m below ground surface.

2.4.11 North Shore Golf Course to Albany Highway

This area is underlain by up to 10m of Tauranga Group soils over ECBF materials. The alluvium is typically stiff to very stiff and comprises interlayered clay, silt and sands.

Groundwater (summer) is encountered between 2 and 4m below ground surface.

2.4.12 Albany Highway to William Pickering Drive

The geotechnical investigations have not been completed between Albany Highway and William Pickering Drive at the time of writing.

Historic aerial photographs suggest a short section of this area originally comprised a valley which subsequently was infilled in the 1960's. Borehole logs held in the T&T database suggest that approximately 5m of fill has been encountered in the vicinity. Boreholes BH-t13 and BH-t14 appear to be located at the edges of the former valley (approximately 100 m to 140 m to the south).

On the basis of topographical information and nearby investigations we anticipate that this part of the alignment is underlain by Tauranga Group soils over ECBF materials with a localised pocket of fill at the location of the infilled valley. The lense of Tauranga Group soils is thin in BH-t14 which appears to be located on a local high point of ECBF materials.

Groundwater (summer) is expected at between 1 and 4 m below ground surface.

These assumptions will need to be verified by ground investigation as the design proceeds.

2.4.13 Piermark Drive to Bush Road

This area is underlain by between 0 and 15m of Tauranga Group Material over ECBF materials. The alluvium is typically stiff to very stiff and comprises interlayered clay, silt and sands.

Groundwater (summer) is encountered between 3 and 4m below ground surface.

2.4.14 Rosedale Park to Rosedale WWTP (including Alexandra Stream crossing)

This part of the alignment is proposed to be constructed using horizontal drilling techniques below Alexandra Stream. Ground investigations data indicates the ECBF is present beneath a shallow cover of recent alluvium in the Alexandra Stream.

Groundwater (summer) is encountered between 2 and 3m below ground surface.

2.5 Stiffness characterisation of soils

Understanding the stiffness of the various rock and soil units is of importance in assessing the potential settlement effects due to groundwater changes and ground loss associated with the tunnel construction methodologies. Very stiff soils are unlikely to result in settlement, even when groundwater lowering is significant – conversely, very soft soils may settle even when groundwater level is lower a small amount. The stiffness of specific soils (such as Tauranga Group alluvial soils) varies from location to location. Natural variability in stiffness is large, but it is possible to statistically define a range within which the stiffness typically falls, and assessments are undertaken across this range.

In order to assign a range of parameters for use in the assessment, both material testing from the Northern Interceptor boreholes, as well experience from previous projects have been utilised to provide a larger sample for assessment.

The NI Phase 1 site investigations have been tailored to provide information with respect to the stiffness of the soils and susceptibility to subsidence following groundwater drawdown. The laboratory testing has been carried out on Tauranga Group materials as these soils are considered to be more compressible and problematic than the ECBF soils along the alignment. The field investigation and laboratory testing data is presented in detail in the Terrestrial Geotechnical Factual Report for the Phase 1 development (Ref 2), but has briefly been discussed in this section for completeness. A plot of the test data is attached in Appendix C.

The 1D consolidation test results are considered to represent the most applicable parameters with respect to the compressibility of the soils. The laboratory results suggests that the test samples are slightly over-consolidated (OCR=2 to 3) and moderately compressible (M= 4 to 5 MPa). An estimate of the modulus has also been obtained by analysing the undrained consolidated triaxial test results. This modulus is considered a lower bound assessment as the samples are assessed at the point of shearing and are likely to be slightly softened, i.e. not peak stiffness (E= 1.4 to 13MPa).

The CPT test results have been assessed based on the soil behaviour index, Ic, and the cone resistance, qc. This assessment method is considered to generate an upper bound estimate of the constrained modulus (M=5 to 30MPa). The CPT results provide assessments of compressibility for the Tauranga Group and ECBF materials.

Material property values adopted for the purposes of this study are summarised in Table 2.1 while Table 2.2 presents design parameters from other projects in the wider Auckland area.

Table 2.1- Summary of material stiffness parameters adopted for this study

Geological Unit	**Deformation Modulus, "E" (MPa)**
Tauranga Group (above 12m depth)	4
Weathered ECBF soil[1]	10

Table 2.2- Comparison of material stiffness parameters with other Auckland projects

Geological Unit	**Deformation Modulus, "E" (MPa)**					
	Central Interceptor[1]	**Waterview Connection[2]**	**Rosedale Tunnel[3]**	**Hobson Bay Tunnel[4]**	**Vector Tunnel[4]**	**Britomart[4]**
Tauranga Group (above 12m depth)	6	1.5 to 10 (stress range dependant)	3	8 to 10	N/A	N/A
Weathered ECBF soil	15	2.5 to 10 (stress range dependant)	10	11	N/A	N/A
ECBF Rock	500	150 to 1050	N/A	450	560	670

1 – T&T (2012); 2 – T&T (2012); 3 - Maunsell Limited (2004); 4 - T&T (2004)

3 Hydrogeological assessment

3.1 Regional groundwater

The Auckland Region is broadly characterised by perched transient groundwater levels within near surface deposits and a deeper more stable groundwater level within the ECBF. The ECBF groundwater level is typically a subdued reflection of surface topography, with gradients of the order of 2-5% from the coast.

Within the ridges, groundwater seepage is typically dominated by vertical seepage patterns, (including cascading perched systems) percolating to the deeper regional level. In gullies seepage from ECBF rock supports stream base flow, or where historic gullies have been in filled by more recent alluvial or volcanic deposits combine in directional seepage along the ancient drainage paths.

3.2 Groundwater level measurements

Each machine borehole has been fitted with a shallow and deep standpipe piezometer for groundwater monitoring purposes. Electronic level loggers, which continuously record the groundwater levels at selected time intervals, have been installed in selected boreholes. Table 3.1 presents manually recorded groundwater levels in each monitoring location including important information such as the screen depth and the geological unit for which the groundwater level has been recorded.

The shallow groundwater levels are expected to fluctuate, potentially by metres in response to rainfall, and across the seasons. Currently, groundwater measurements are available from 17 December 2014 to 13 February 2015, so this fluctuation has not necessarily been fully reflected in records to date.

The deeper groundwater in the ECBF is also expected to fluctuate, but potentially to a lessor magnitude than the shallow groundwater. As for the shallow groundwater, records are available from 17 December 2014 to 13 February 2015.

Table 3.1 - Groundwater readings in the standpipe piezometers

ID	RL (m) AUCK 1946	Screen bgl (m)	Geological Unit	Diver Installed	17/12/ 2014	22/12/ 2014	14/01/ 2015	27/01/ 2015	06/03/ 2015
					depth below ground level (m)				
BH1	6.3	3.0 - 6.0	TGA					1.29	
		9.0 - 12.0	TGA					0.6	
BH2	6.4	4.0 - 7.0	TGA	23/12/14 @ 6.8m		2.29	2.4	2.45	
		10.0 - 11.5	ECBF rock[1]			2.28			
BH3	3.2	0.5 - 2.0	Fill				1.47		1.6
		5.5 - 7.0	ECBF rock[1]			1.33			1.64
BH5	22.4	3.5 - 5.0	ECBF soil			1.45			1.93
		13.0 - 14.5	ECBF soil			1.56			2.56
BH6	33.9	4.0 - 7.0	ECBF soil			2.86			4.88
		11.5 - 14.5	ECBF soil			3.64			5.68

ID	RL (m) AUCK 1946	Screen bgl (m)	Geological Unit	Diver Installed	17/12/ 2014	22/12/ 2014	14/01/ 2015	27/01/ 2015	06/03/ 2015
					depth below ground level (m)				
BH10	13.4	2.5 - 4.0	ECBF soil			0.97			2.8
		8.5 - 10.0	ECBF rock[1]			1.27			3.15
BH12	39.9	3.0 - 6.0	TGA	17/12/14 @ 5m	3.2		2.9	3.11	
		13.0 - 14.5	ECBF soil		6.26			6.36	
BH13	42	2.5 - 5.5	TGA			1.21			1.39
		8.0 - 11.0	ECBF soil			2.59			3.15
BH14	34.2	3.0 - 6.0	ECBF soil				3.66	3.7	
		12.0 - 15.0	ECBF rock[1]				3.56	3.6	
BH15	39.2	3.0 - 4.5	TGA	17/12/14 @ 4.4m	2.5		2.55	3.58	4.15
		5.5 - 7.0	TGA	17/12/14 @ 6m	2.72		2.53	3.32	3.92
BH16	38.6	2.0 - 4.0	ECBF soil	17/12/14 @ 3.5m	1.77		1.89	2.5	
		9.0 - 12.0	ECBF soil	17/12/14 @ 11.4m	1.68		1.9	2.4	
BH17	25	2.5 - 4.0	ECBF soil			1.31			2.86
HA28	26.2	2.4-6.2	ECBF soil					4.3	4.65

3.3 Permeability assessment

The machine boreholes were cleaned out prior to construction of the standpipe piezometers and each well was purged dry prior to installation of the divers. The groundwater levels had been monitored for approximately 4 weeks and appeared to have stabilised when a slug test was completed to assess the insitu permeability in selected boreholes. The slug test involved removal of 1m of water head and the recharge was recorded in the divers at 1 minute intervals. The rising head data was analysed using the Hvorslev method. The results are relatively consistent and are in general accordance with permeabilities observed from assessments completed for other projects in the Auckland region.

The permeability was also assessed based on the Coefficient of Consolidation (Cv) as part of the 1D consolidation tests. Historic site investigations and local knowledge suggests that the horizontal permeability is ten times greater than the vertical permeability of soils. This is predominately attributed to horizontal layering during the deposition and presence of more permeable layers in the soil matrix. The evaluated permeability as part of the consolidation testing has therefore been multiplied by 10 to obtain the horizontal permeability component.

The permeability has also been assessed by carrying out dissipation tests in the CPT locations, i.e. the CPT probe is held at a particular depth while the pore pressure is measured over time as equilibrium is reached. CPT dissipation tests are commonly used to complement more robust test data, but can generate variable results depending on saturation of the pore pressure filter or whether the operator allows sufficient time for the test to be completed.

The horizontal permeability assessment results for the tests carried out on the Tauranga Group materials are presented in Table 3.2. The test results range between 1×10^{-7} and 2×10^{-9} m/s. Typically adopted parameters in the Auckland area for Tauranga Group range between $2x\ 10^{-6}$ and

2 x 10^{-8} m/s, i.e. slightly more permeable. Historically, similar horizontal permeabilities are attributed to the weathered ECBF soils. For the purposes of analysis a horizontal permeability of kh=2 x 10^{-7} m/s has been adopted for both soils.

Table 3.2 - Horizontal permeability assessment results

Type of Analysis	Slug Test	1D Consolidation Test	CPT Dissipation
	Horizontal permeability, kh (m/s)		
Min	1.2 x 10^{-8}	1.8 x 10^{-9}	2.7 x 10^{-9}
Max	1.5 x 10^{-7}	2.6 x 10^{-8}	2.6 x 10^{-8}
Mean	8.0 x 10^{-8}	1.3 x 10^{-8}	2.3 x 10^{-8}

3.4 Groundwater model

Groundwater levels measured in surface deposits and underlying ECBF are typically hydrostatic with groundwater levels typically close to the surface as shown on Figure 3.1. The upper perched groundwater table in the Tauranga Group soils appear to be interconnected with the groundwater in the ECBF materials (both soil and rock).

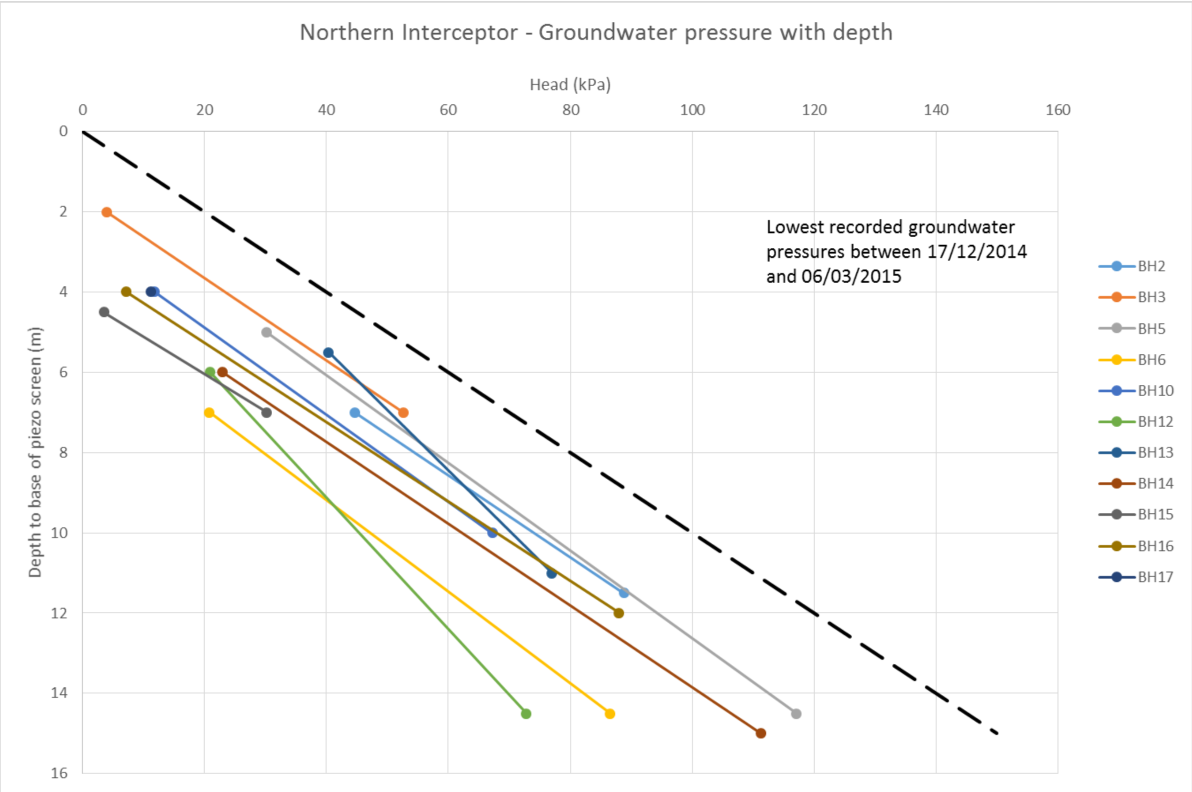


Figure 3.1. Measured groundwater levels which suggest hydrostatic conditions.

4 Settlement assessment methodology

4.1 General

The following sections describe the methodologies adopted to assess the potential surface settlement that might arise in areas where surface settlement could affect existing buildings and services (Section 4.4 of the main report). The methodologies adopted are consistent with methodologies used for the assessment of effects on other recent projects (such as the Central interceptor Project). We consider that the approaches are appropriate for this assessment as they are normally expected to provide conservative assessments of effects (where conservative means they are more likely to provide an over estimate of effects, rather than an under estimate). The settlement estimates include both total and differential settlement where the latter often is considered the most damaging. The assessments assume that the adopted construction methods are carried out such that they achieve their aim and settlement estimates are made on this basis.

The total settlements at the ground surface will result from a combination of values from the two settlement sources:

- Mechanically induced settlement which generates relatively quick settlements following their trigger mechanisms (excavation of the trench or micro tunnelling).
- Consolidation settlement caused by groundwater drawdown which takes a comparatively longer time to occur.

The combination of settlement effects has been assessed as a long term case, i.e. assuming full settlement has occurred for all sources. The method used to combine the settlements is a simple superposition of the settlement values from each individual source.

4.2 Trenching

4.2.1 Mechanically induced settlement

The potential magnitude and extent of ground movement settlements behind sheet piled, or similarly supported trench excavations have been estimated using the method described in Peck (1969). This simplified method is empirical and based on actual recorded settlements adjacent to excavations in soft clay. The data are from excavations using standard ground retention techniques such as sheet piles, and the results suited to excavations where walls are able to deflect inwards towards the excavation. The estimates adopted here for the NI Phase 1 are considered for excavation depths of up to 8m.

The estimates are based on the potential settlement hazards arising in the absence of specific design or construction measures to minimise settlement. For example, excavations supported by un-propped cantilever sheet piles may result in settlements close to the potential settlements estimated. Specific design considering minimisation of surface settlement as a key criteria (potentially by increasing the stiffness of the wall and prop members) can substantially lessen actual settlement. Such methodologies would be developed by the Contractor during final design.

In estimating settlements associated with trenching, it has been assumed that the length of trench that will be excavated and open at one time will typically be in excess of 10m. Should the trenching operations be advanced in shorter sections, the predicted settlements presented here can be expected to represent upper bound estimates.

4.2.2 Groundwater drawdown and inflow

The potential magnitude and extent of temporary groundwater drawdown behind sheet piled trench excavations, or adjacent to pipe jacking operations have been estimated using the method in

Somerville, 1988. The magnitude of settlement has been predicted from the estimated drawdown using the soil stiffness adopted in Section 6.9.

The pipe bedding has the potential to act as a drain long term unless groundwater cutoffs (i.e. less permeable layers) are incorporated in the bedding. Once the trench is backfilled to an appropriate standard it is considered unlikely that further groundwater drawdown will occur. The settlement analyses for the NI Phase 1 are therefore considered conservative as they do not incorporate time dependant settlement.

4.3 Micro tunnelling

4.3.1 Mechanically induced settlement

The potential magnitude and extent of settlements resulting from ground loss associated with tunnelling operations has been estimated using the methods described in Sinclair et al (1988). In following the methods described in this paper, the site soils have been categorised using the system described in Andrews et al (1984). The categorisation is summarised below:

Table 4-1 Assumed tunnel conditions for mechanical settlement estimates

Geological Unit	Adopted Tunnelling Conditions
Tauranga Group Alluvium/ Upper Puketoka Formation	I – Tunnel predominantly in soft soils
Weathered ECBF	II – Tunnel predominantly in medium stiff to hard soils
ECBF	IV – Tunnel predominantly in rock
Mixed Face TGA/UPF and ECBF	III/I Top of tunnel in sand, tunnel remains in soft soil[1]

1. Chosen as worst case example mixed face conditions.

To allow for face stability control in earth pressure balanced excavations, a reduction in the total ground volume loss has been built into the model to the methods described by Sinclair.

Face loss as a volume per metre advance of the excavation has been assumed to reduce 1% as recommended in Sinclair and Norfolk (2001). Hence, to allow for the effects of Micro Tunnel equipment, (be it mechanical or slurry), the percentage of lost ground has been reduced by an absolute value of 1% in the assessment of total lost ground (i.e. a calculated lost ground percentage of 7% would reduce to 6%). In terms of lost ground, the effect of the filling of the over-break annulus with slurry has been assumed to be a reduction in maximum estimated settlement by approximately 50%.

In instances where the inferred geology indicates mixed face conditions may be encountered (i.e. transitioning from one geological unit across the face of the excavation) worst case conditions for settlement have been assumed.

4.3.2 Groundwater drawdown settlement

The potential magnitude and extent of temporary groundwater drawdown adjacent to tunnelling/pipe jacking operations have been estimated using the method described by Somerville, 1988. The magnitude of settlement has been predicted from the estimated drawdown using the soil stiffness adopted in Section 2.5.

The effect of controlled pressurised slurry in the jacked pipe annulus is assumed to be a reduction in the effect of the pipe jacking operations on the surrounding groundwater system by balancing the groundwater pressures around the excavations, and limiting the magnitude of drawdown effects.

In theory, the slurry could be pressurised to exactly balance the in situ groundwater pressures, to effectively maintain existing groundwater pressures in the surrounding soils and prevent any drawdown related ground settlement.

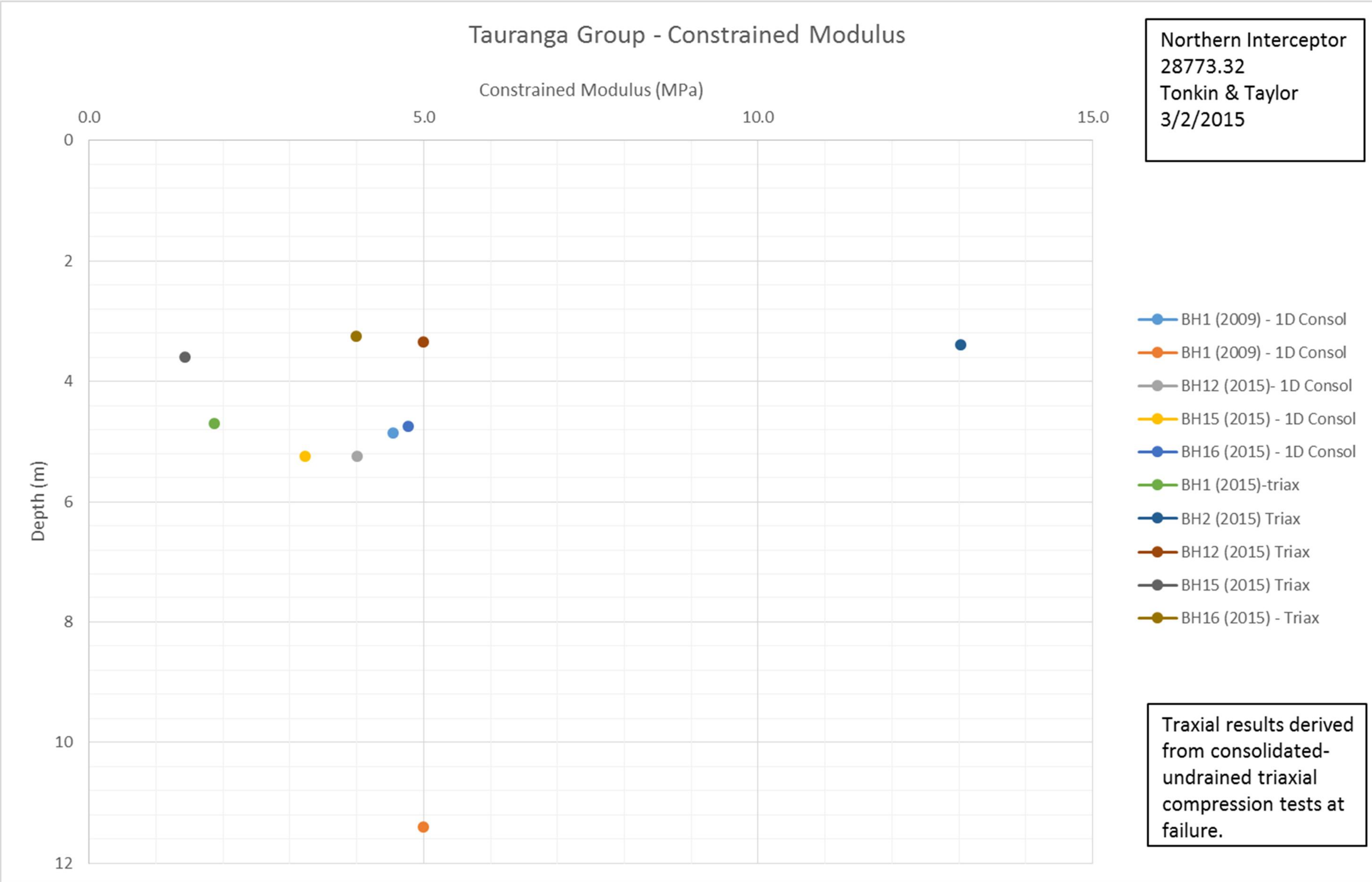
In practice however, it is unlikely that this level of precision and control in the construction process could be consistently and reliably maintained. There is a counter risk of ground heave caused by excess pressure. Therefore the pressure controls are typically targeted at less than the existing ground pressure. The practical effect of the pressurised annulus slurry is therefore expected to slow the development of, and reduce the absolute magnitude of, the groundwater response in the short term.

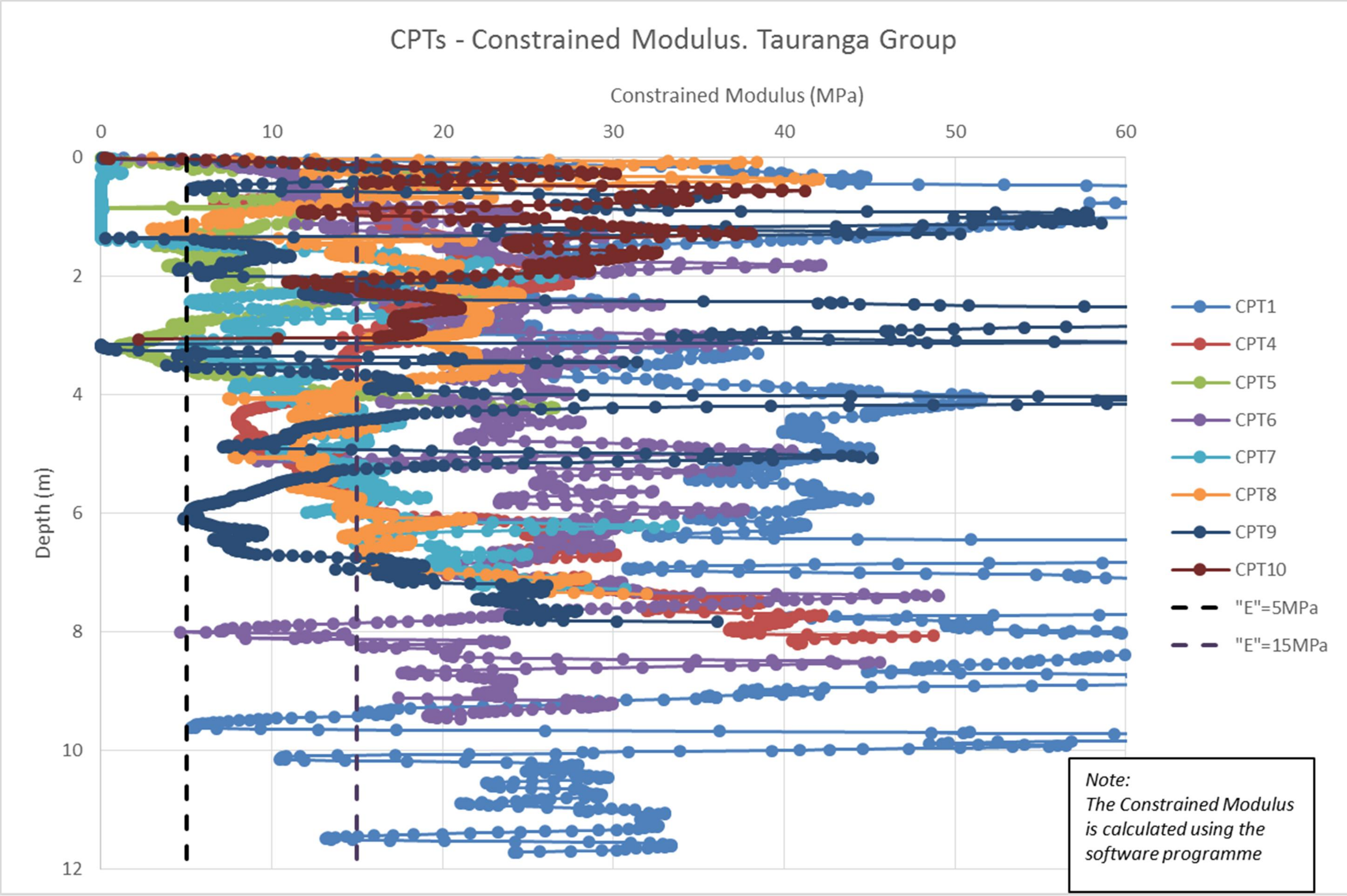
It is expected that, with reliable slurry pressure control, the potential effect on local groundwater pressures (and hence surface settlement) can be reduced by some 75% of that expected without the controlled slurry pressurisation.

In terms of settlement extents, more conservative assessments of soil stiffness, (i.e. assuming softer soils) would reduce the predicted extent of surface effects (albeit while increasing the associated magnitude of maximum settlement).

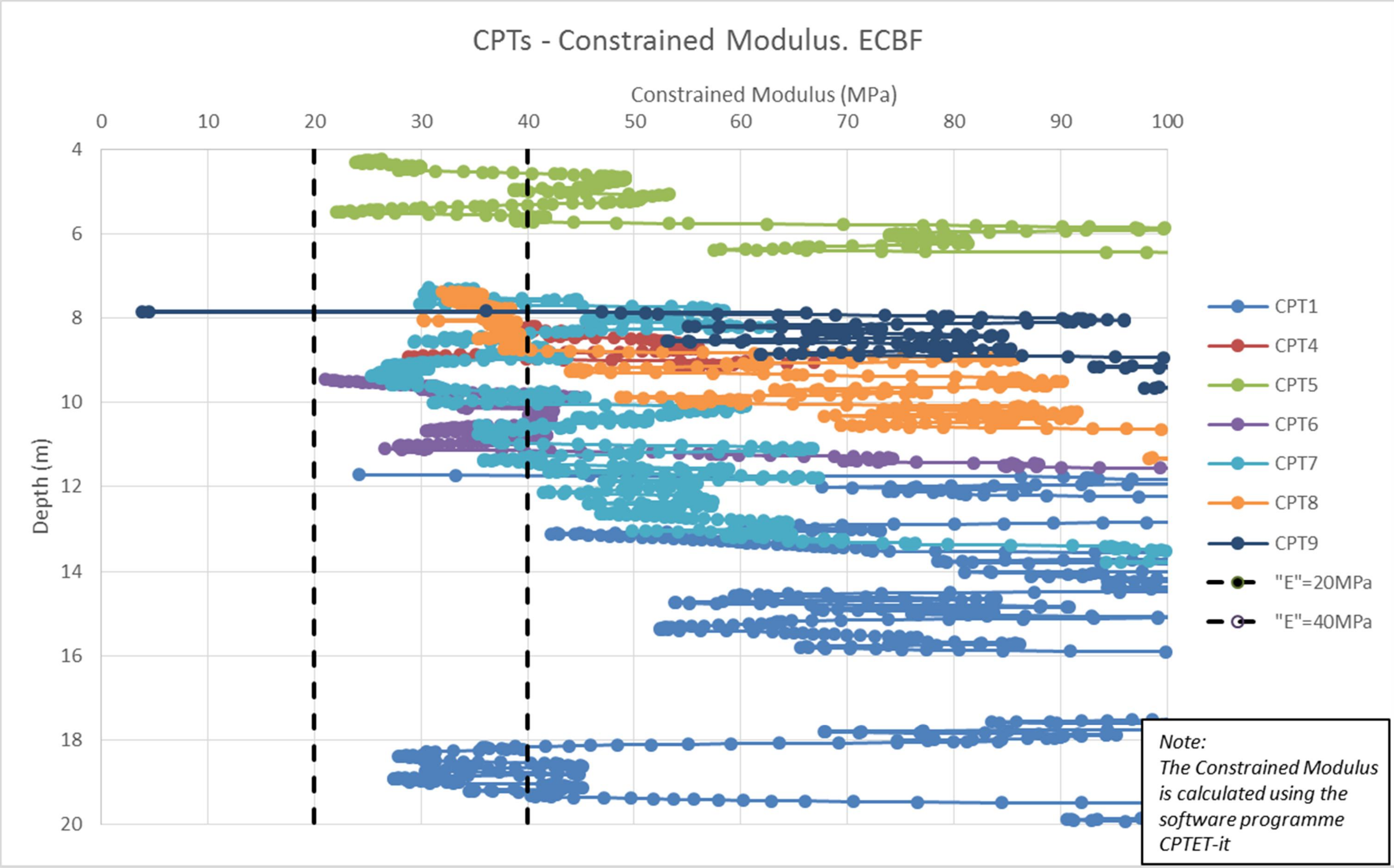
Appendix B: Stiffness and permeability data

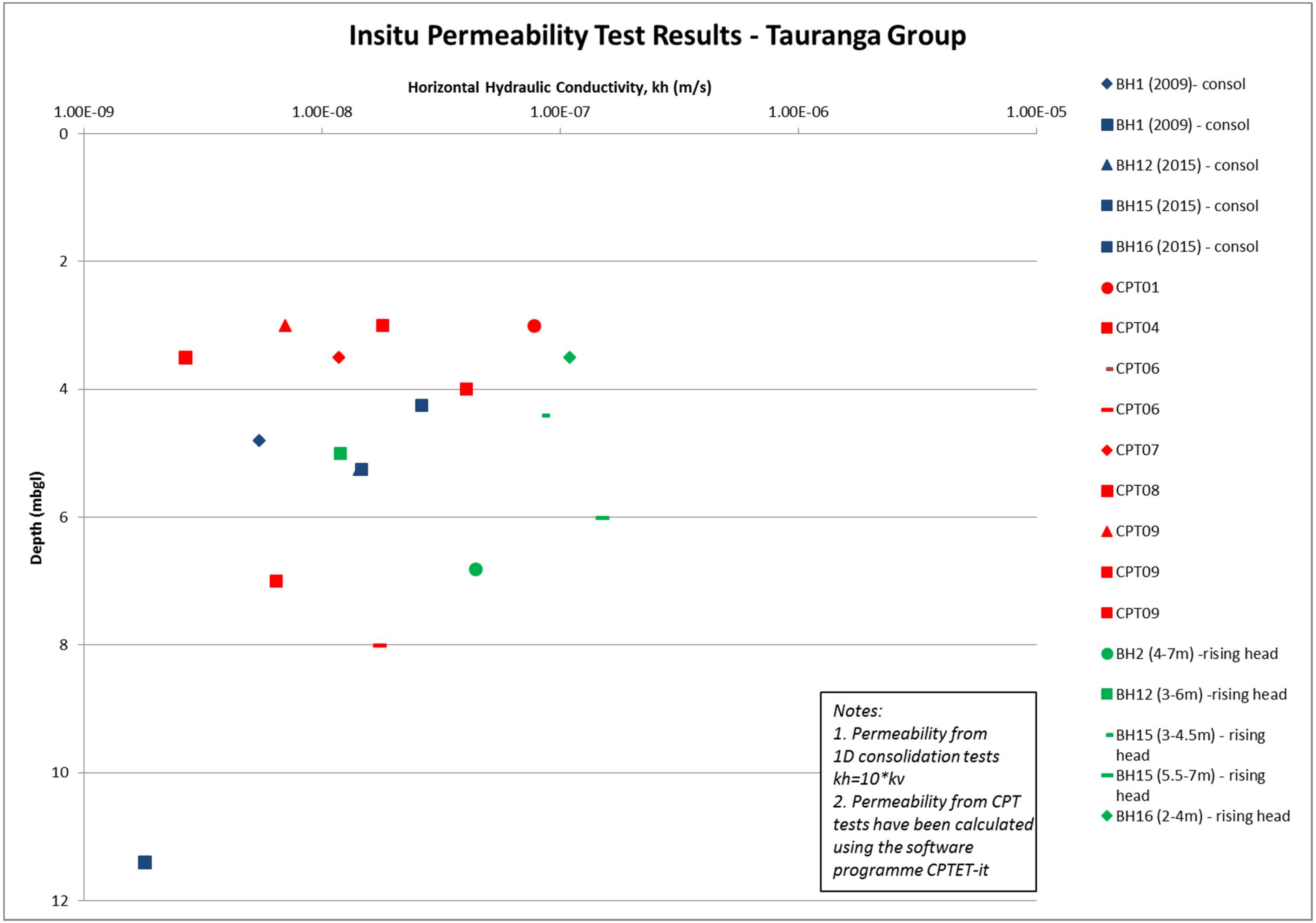
- Interpretation of test data

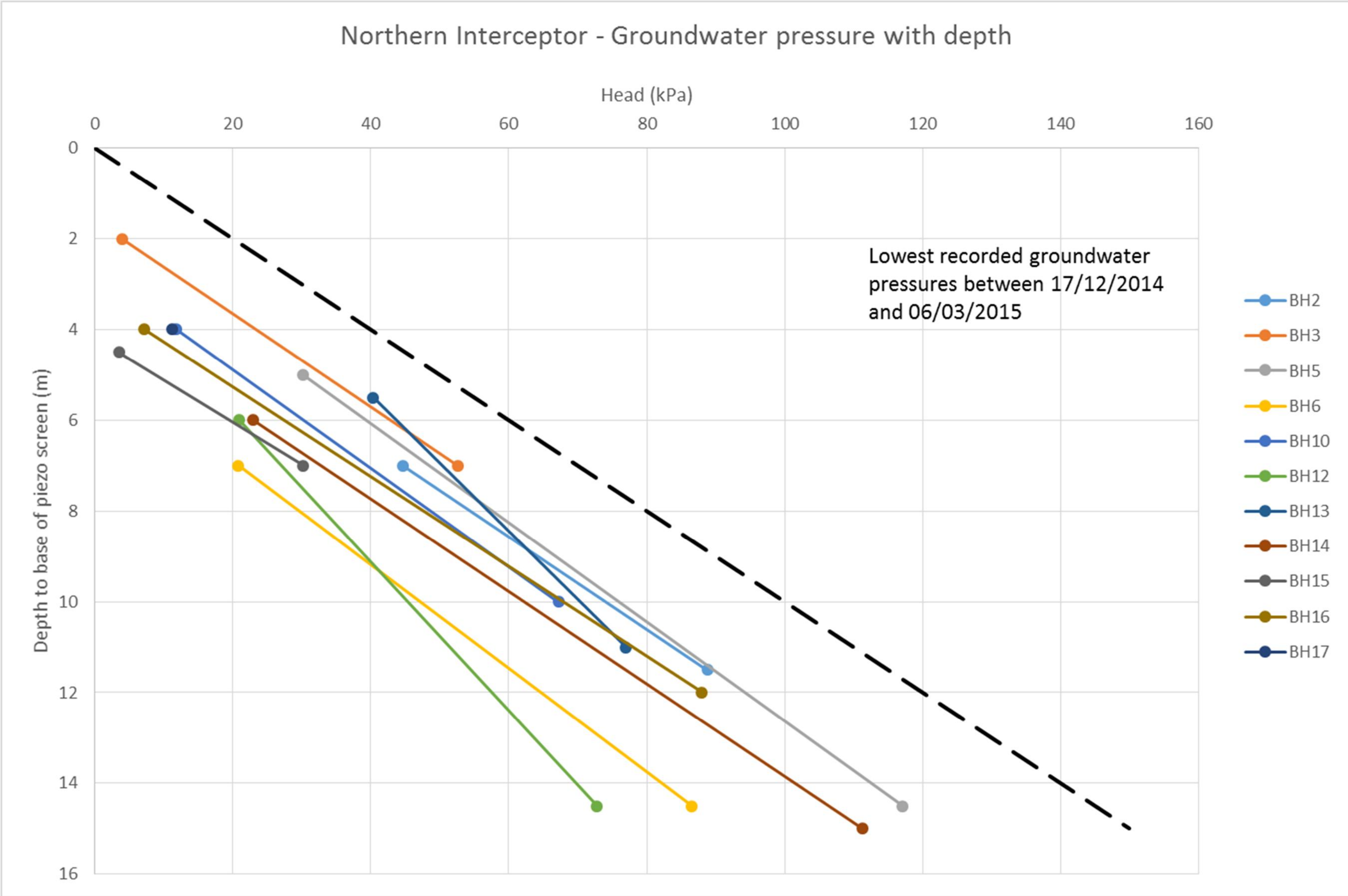

Tauranga Group - Constrained Modulus
Constrained Modulus (MPa)
0.0
5.0
10.0
15.0
0
2
4
6
8
10
12
Depth (m)
Northern Interceptor
28773.32
Tonkin & Taylor
3/2/2015
BH1 (2009) - 1D Consol
BH1 (2009) - 1D Consol
BH12 (2015)- 1D Consol
BH15 (2015) - 1D Consol
BH16 (2015) - 1D Consol
BH1 (2015)-triax
BH2 (2015) Triax
BH12 (2015) Triax
BH15 (2015) Triax
BH16 (2015) - Triax
Traxial results derived from consolidated-undrained triaxial compression tests at failure.

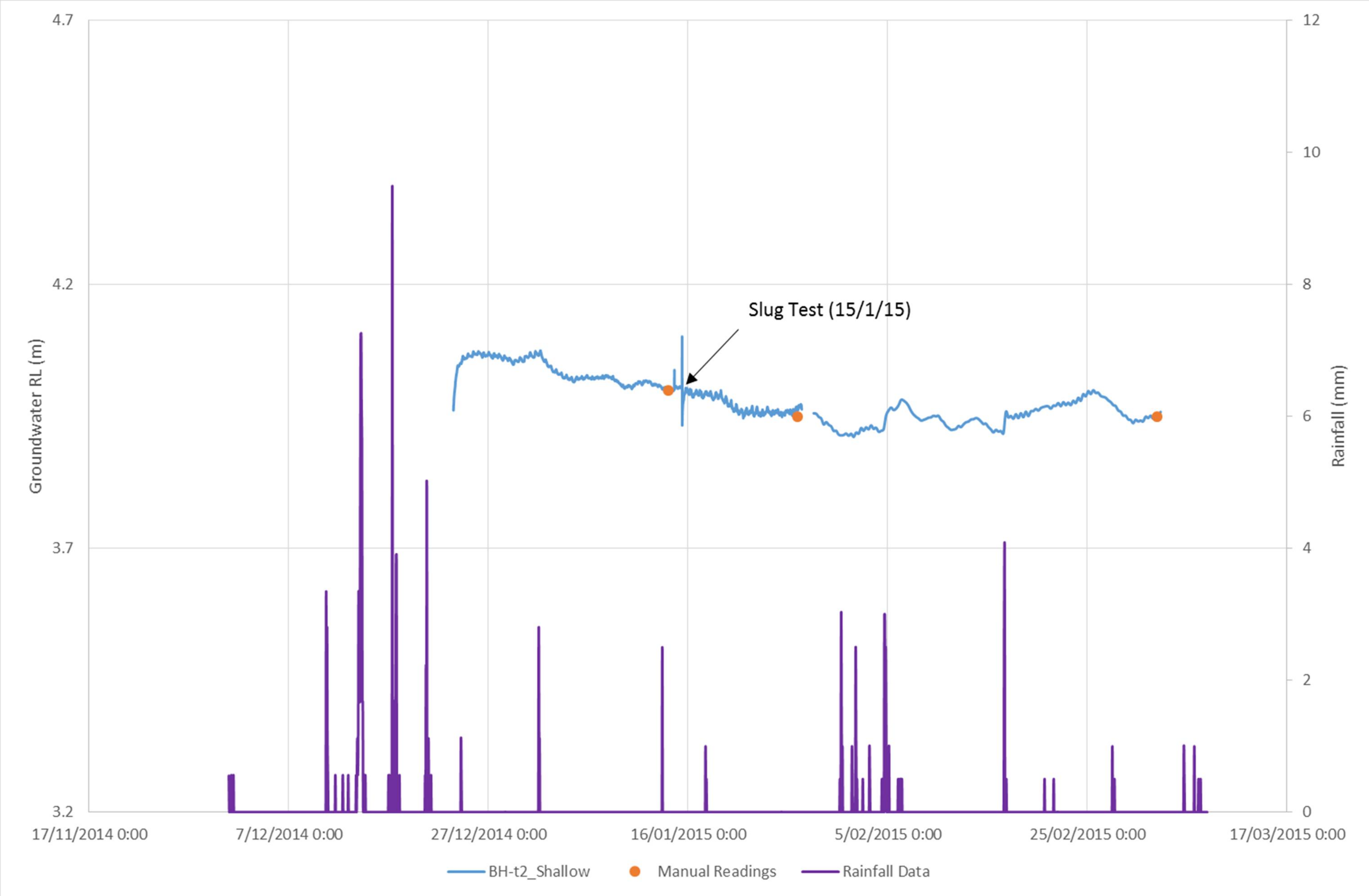


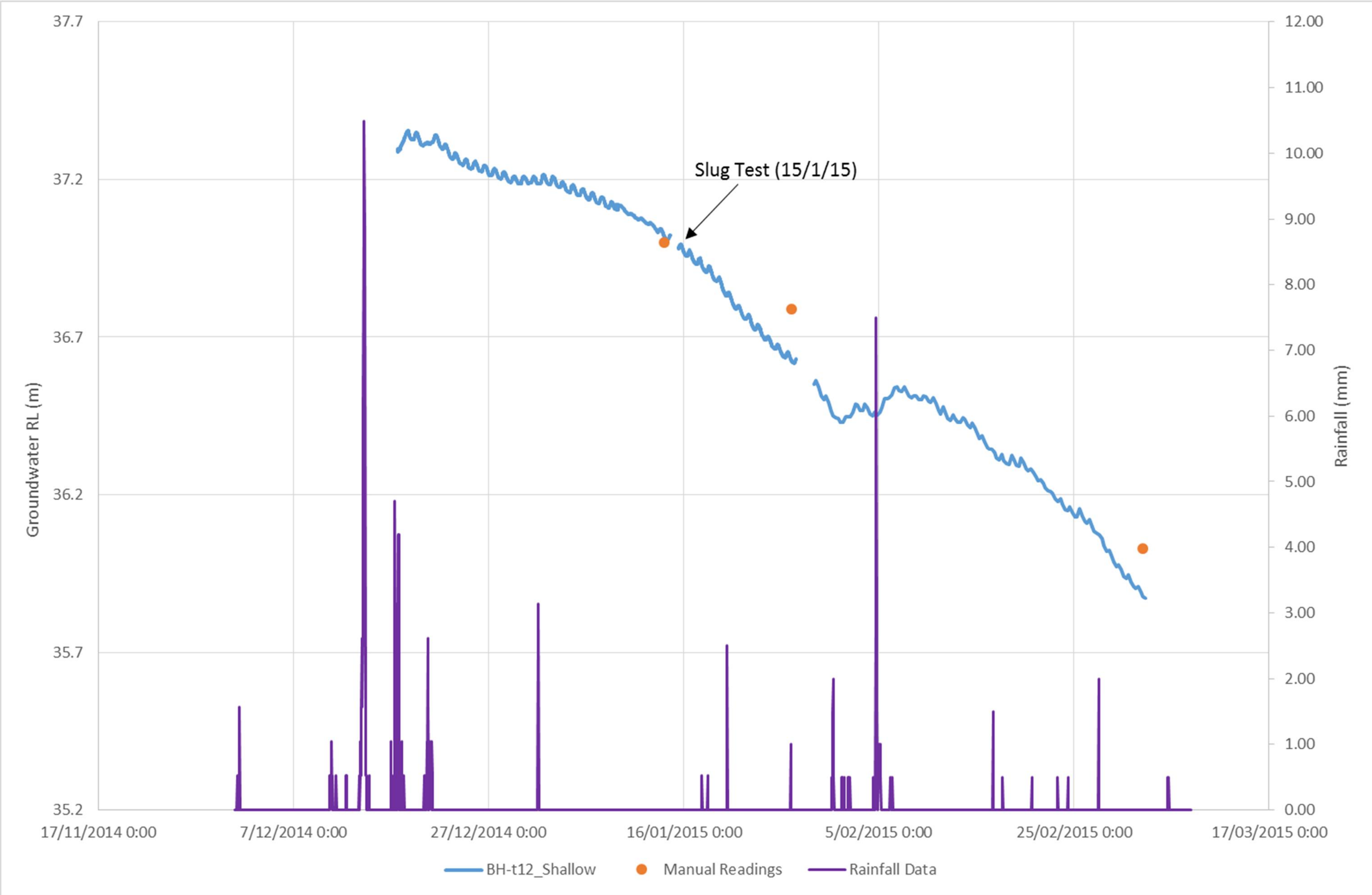
CPTs - Constrained Modulus. Tauranga Group
Constrained Modulus (MPa)
0
10
20
30
40
50
60
Depth (m)
0
2
4
6
8
10
12
CPT1
CPT4
CPT5
CPT6
CPT7
CPT8
CPT9
CPT10
"E"=5MPa
"E"=15MPa
Note:
The Constrained Modulus
is calculated using the
software programme


CPTs - Constrained Modulus. ECBF
Constrained Modulus (MPa)
0
10
20
30
40
50
60
70
80
90
100
Depth (m)
4
6
8
10
12
14
16
18
20
CPT1
CPT4
CPT5
CPT6
CPT7
CPT8
CPT9
"E"=20MPa
"E"=40MPa
Note:
The Constrained Modulus is calculated using the software programme CPTET-it

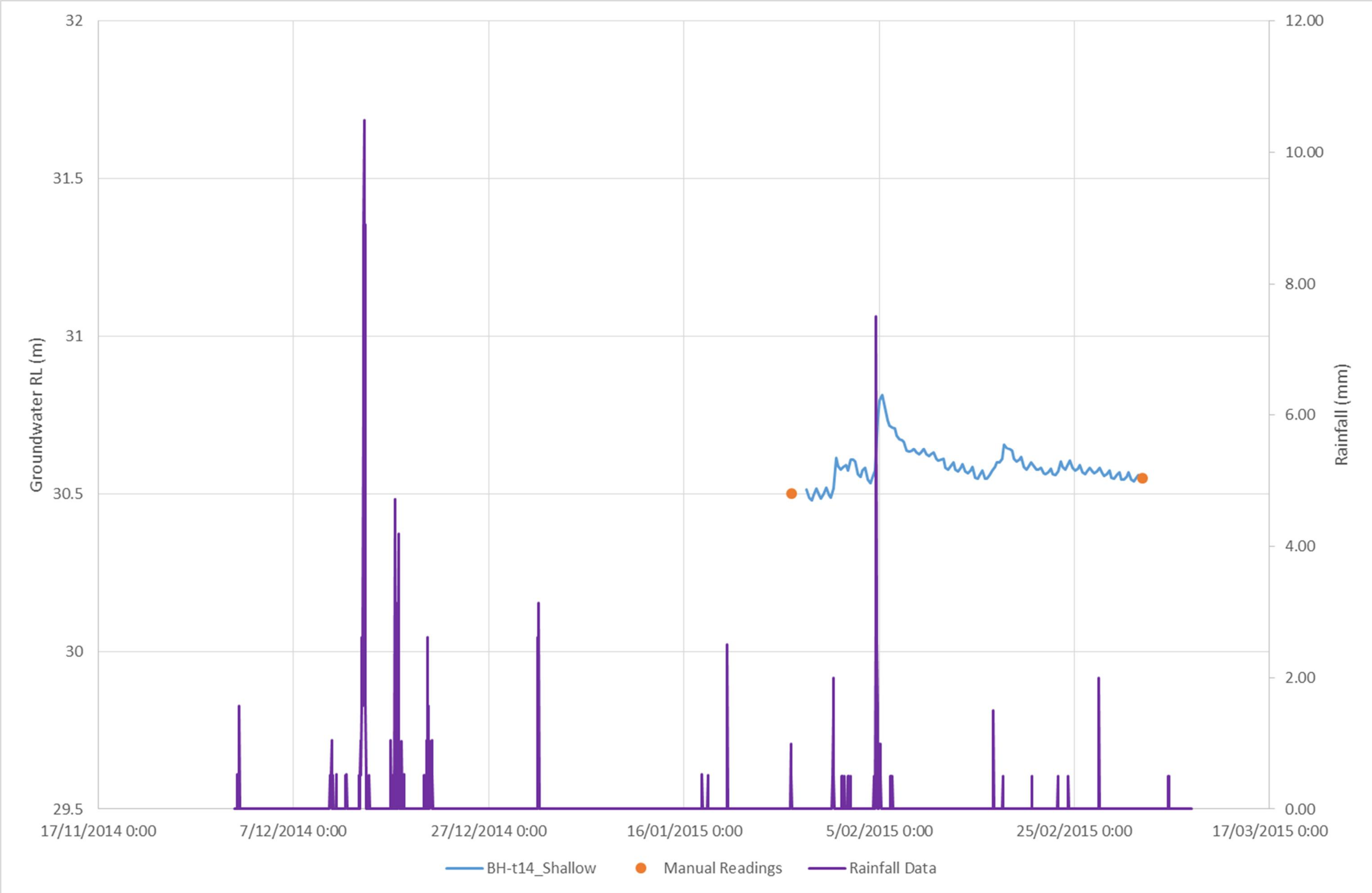

Insitu Permeability Test Results - Tauranga Group
Horizontal Hydraulic Conductivity, kh (m/s)
1.00E-09
1.00E-08
1.00E-07
1.00E-06
1.00E-05
Depth (mbgl)
0
2
4
6
8
10
12
BH1 (2009)- consol
BH1 (2009) - consol
BH12 (2015) - consol
BH15 (2015) - consol
BH16 (2015) - consol
CPT01
CPT04
CPT06
CPT06
CPT07
CPT08
CPT09
CPT09
CPT09
BH2 (4-7m) -rising head
BH12 (3-6m) -rising head
BH15 (3-4.5m) - rising head
BH15 (5.5-7m) - rising head
BH16 (2-4m) - rising head
Notes:
1. Permeability from 1D consolidation tests kh=10*kv
2. Permeability from CPT tests have been calculated using the software programme CPTET-it

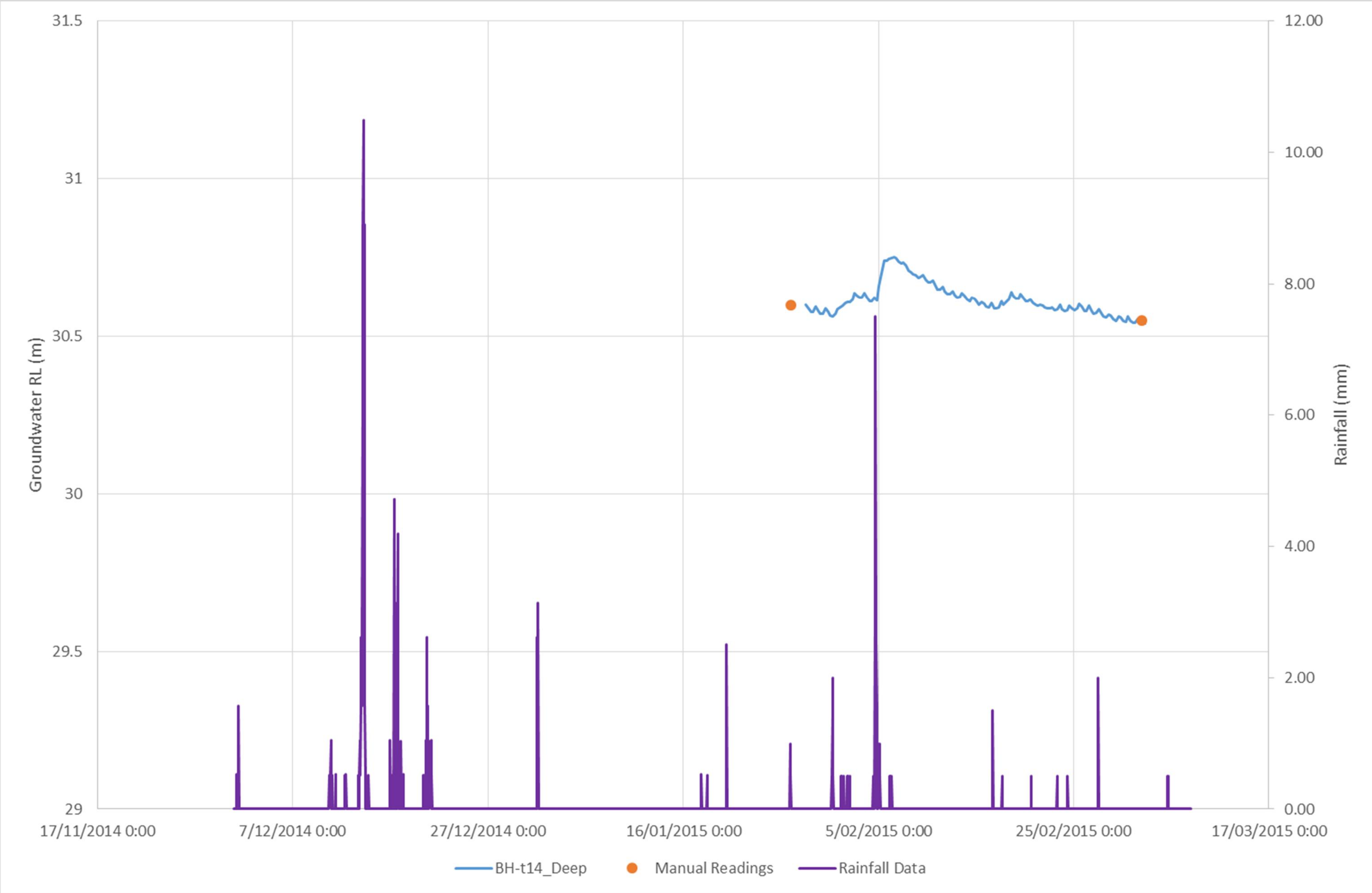

Northern Interceptor - Groundwater pressure with depth
Head (kPa)
0
20
40
60
80
100
120
140
160
Depth to base of piezo screen (m)
0
2
4
6
8
10
12
14
16
Lowest recorded groundwater pressures between 17/12/2014 and 06/03/2015
BH2
BH3
BH5
BH6
BH10
BH12
BH13
BH14
BH15
BH16
BH17


4.7
4.2
3.7
3.2
Groundwater RL (m)
12
10
8
6
4
2
0
Rainfall (mm)
Slug Test (15/1/15)
17/11/2014 0:00
7/12/2014 0:00
27/12/2014 0:00
16/01/2015 0:00
5/02/2015 0:00
25/02/2015 0:00
17/03/2015 0:00
BH-t2_Shallow
Manual Readings
Rainfall Data

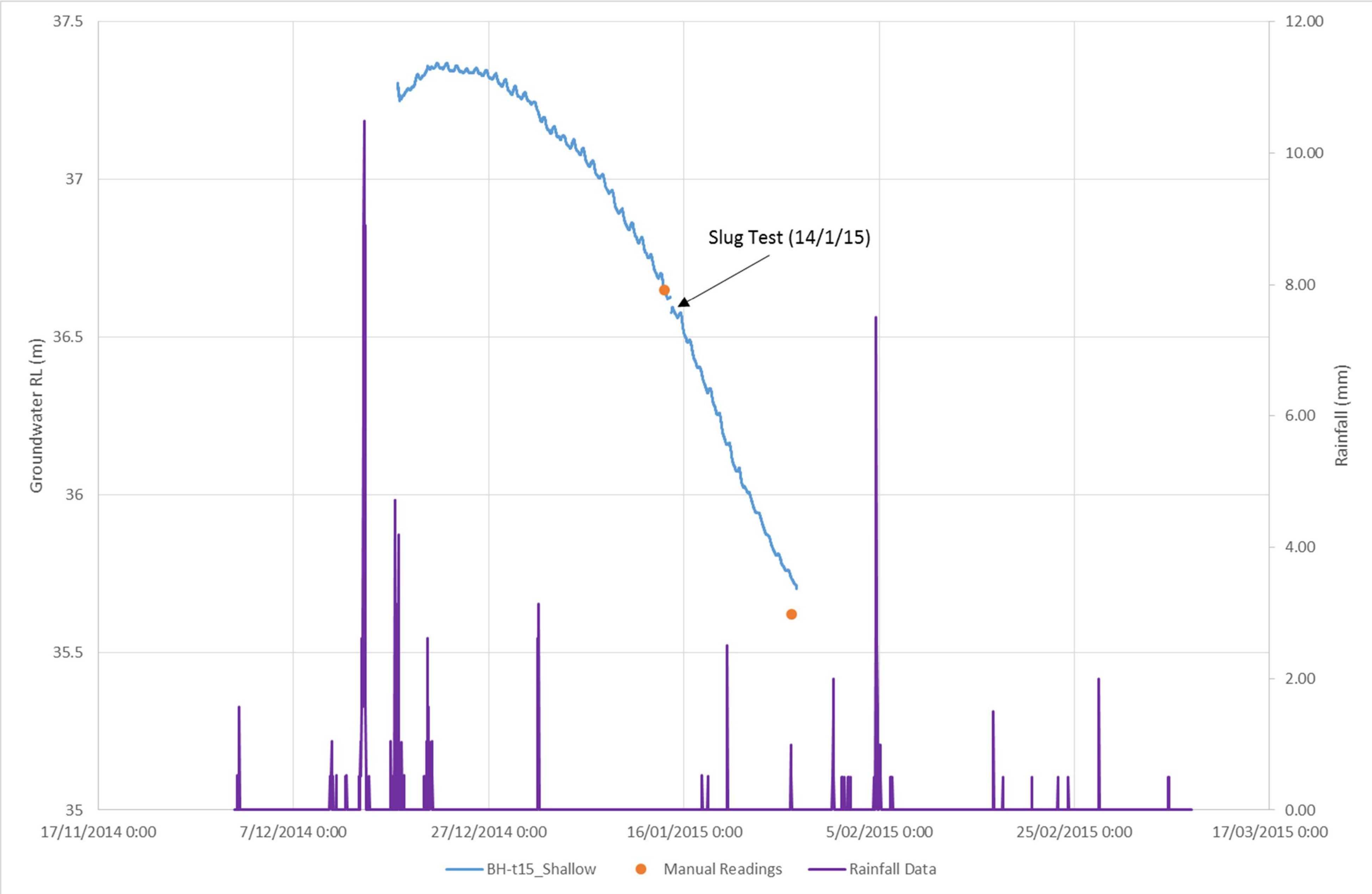


Slug Test (15/1/15)
Groundwater RL (m)
Rainfall (mm)
37.7
37.2
36.7
36.2
35.7
35.2
12.00
11.00
10.00
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
17/11/2014 0:00
7/12/2014 0:00
27/12/2014 0:00
16/01/2015 0:00
5/02/2015 0:00
25/02/2015 0:00
17/03/2015 0:00
BH-t12_Shallow
Manual Readings
Rainfall Data

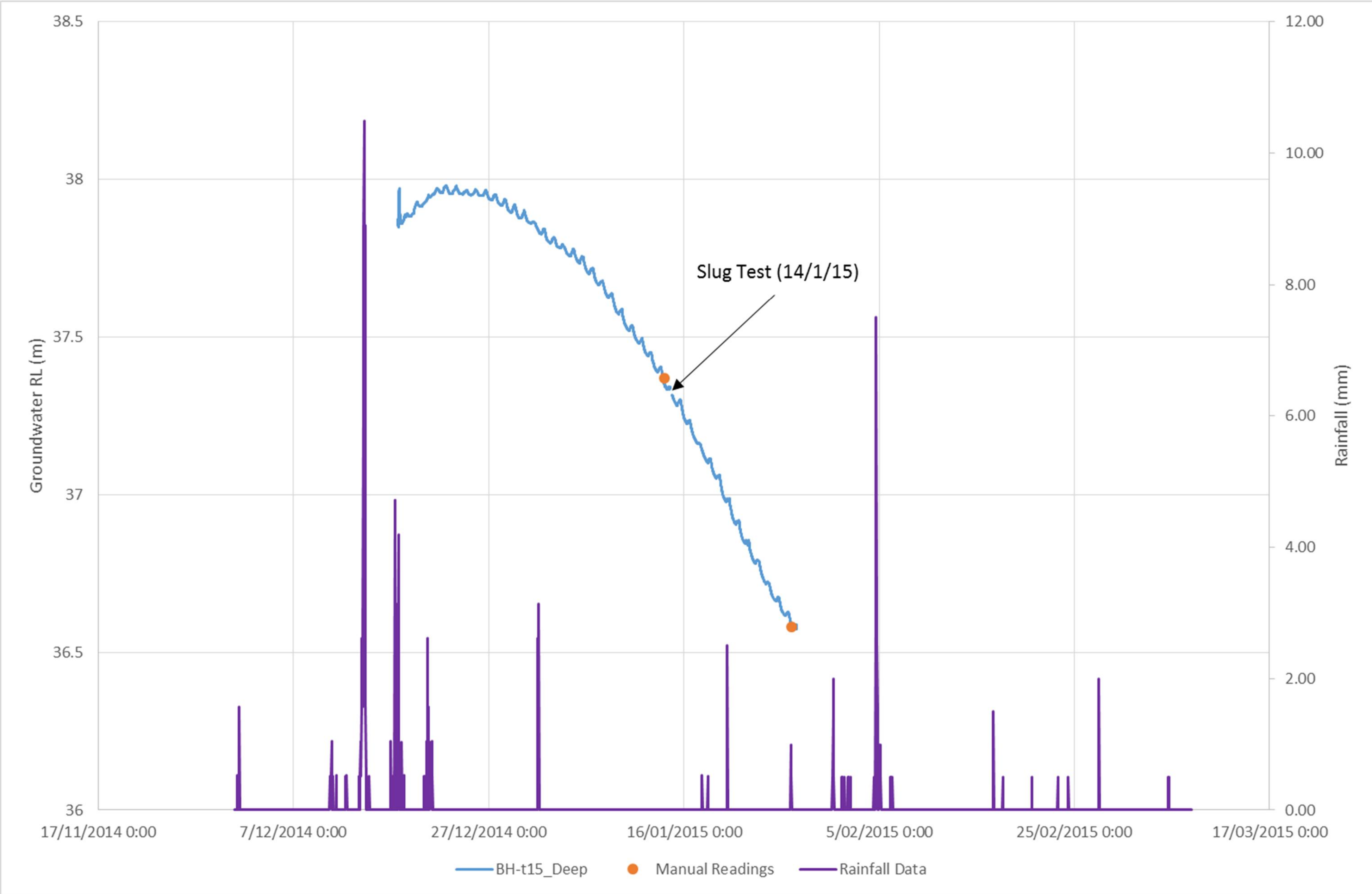

32
31.5
31
30.5
30
29.5
Groundwater RL (m)
12.00
10.00
8.00
6.00
4.00
2.00
0.00
Rainfall (mm)
17/11/2014 0:00
7/12/2014 0:00
27/12/2014 0:00
16/01/2015 0:00
5/02/2015 0:00
25/02/2015 0:00
17/03/2015 0:00
BH-t14_Shallow
Manual Readings
Rainfall Data



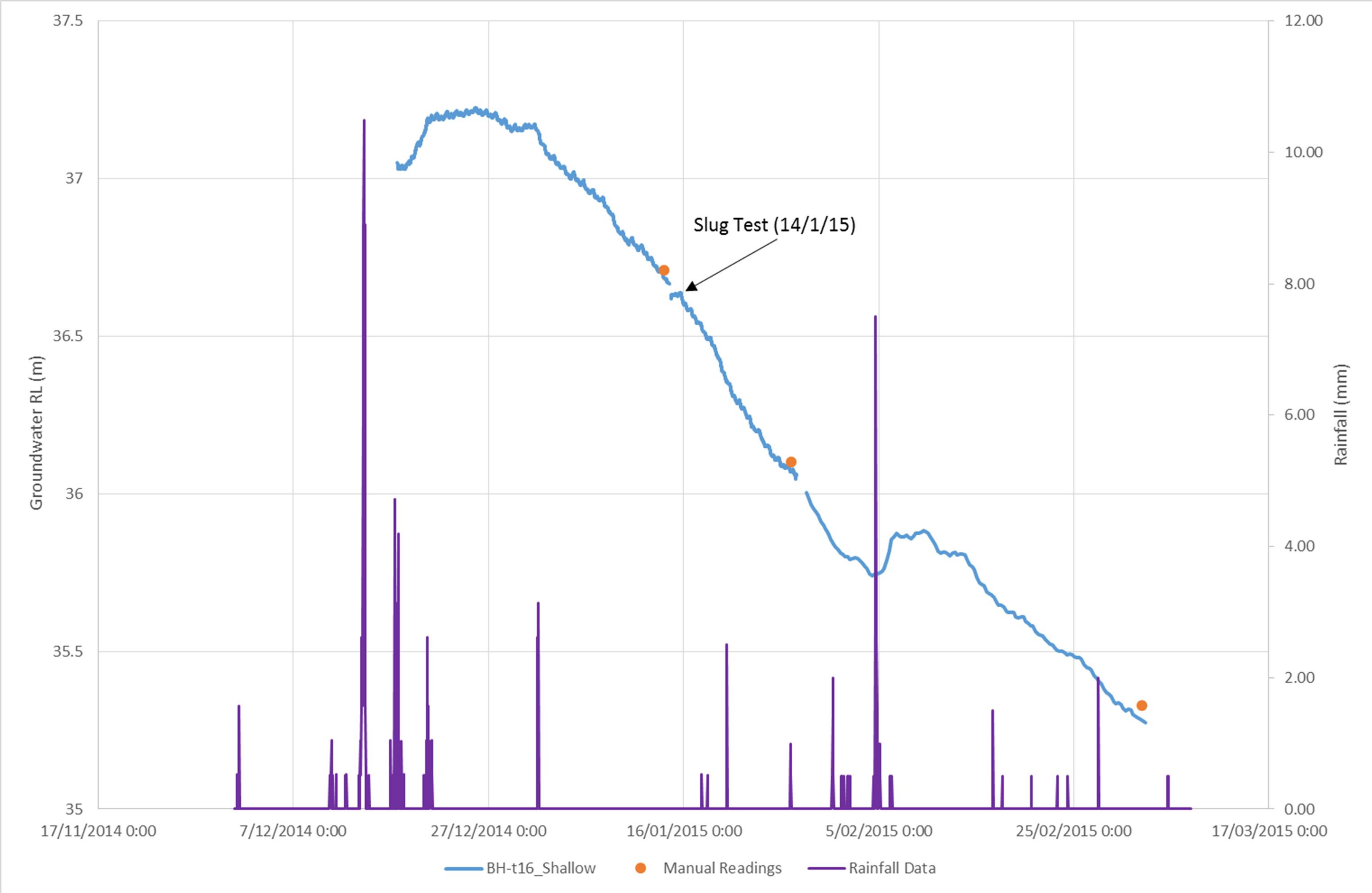
31.5
31
30.5
30
29.5
29
12.00
10.00
8.00
6.00
4.00
2.00
0.00
Groundwater RL (m)
Rainfall (mm)
17/11/2014 0:00
7/12/2014 0:00
27/12/2014 0:00
16/01/2015 0:00
5/02/2015 0:00
25/02/2015 0:00
17/03/2015 0:00
BH-t14_Deep
Manual Readings
Rainfall Data



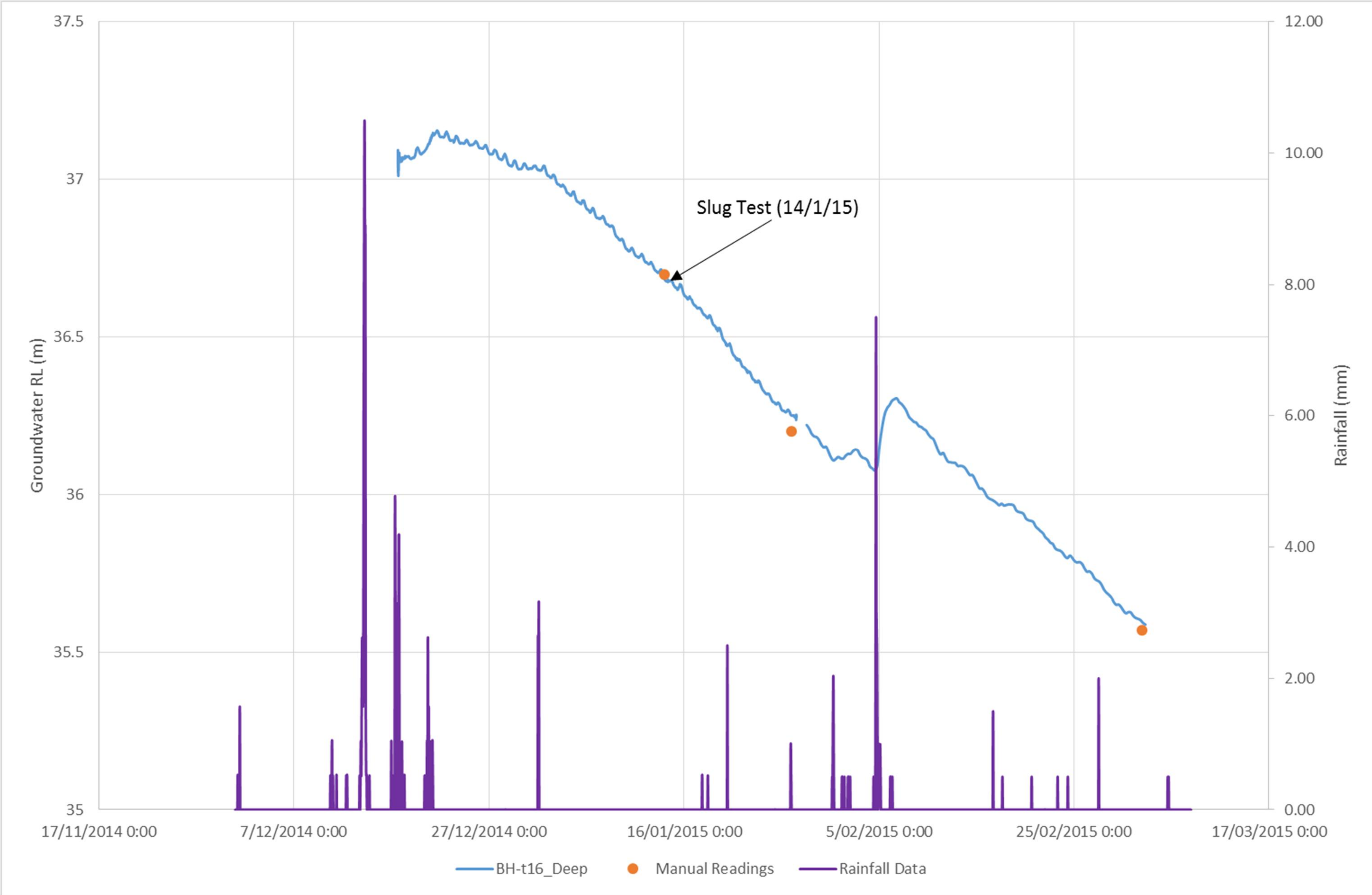
37.5
37
36.5
36
35.5
35
Groundwater RL (m)
12.00
10.00
8.00
6.00
4.00
2.00
0.00
Rainfall (mm)
Slug Test (14/1/15)
17/11/2014 0:00
7/12/2014 0:00
27/12/2014 0:00
16/01/2015 0:00
5/02/2015 0:00
25/02/2015 0:00
17/03/2015 0:00
BH-t15_Shallow
Manual Readings
Rainfall Data



Groundwater RL (m)
Rainfall (mm)
38.5
38
37.5
37
36.5
36
12.00
10.00
8.00
6.00
4.00
2.00
0.00
17/11/2014 0:00
7/12/2014 0:00
27/12/2014 0:00
16/01/2015 0:00
5/02/2015 0:00
25/02/2015 0:00
17/03/2015 0:00
Slug Test (14/1/15)
BH-t15_Deep
Manual Readings
Rainfall Data



Slug Test (14/1/15)
Groundwater RL (m)
Rainfall (mm)
37.5
37
36.5
36
35.5
35
12.00
10.00
8.00
6.00
4.00
2.00
0.00
17/11/2014 0:00
7/12/2014 0:00
27/12/2014 0:00
16/01/2015 0:00
5/02/2015 0:00
25/02/2015 0:00
17/03/2015 0:00
BH-t16_Shallow
Manual Readings
Rainfall Data



37.5
37
36.5
36
35.5
35
Groundwater RL (m)
12.00
10.00
8.00
6.00
4.00
2.00
0.00
Rainfall (mm)
Slug Test (14/1/15)
17/11/2014 0:00
7/12/2014 0:00
27/12/2014 0:00
16/01/2015 0:00
5/02/2015 0:00
25/02/2015 0:00
17/03/2015 0:00
BH-t16_Deep
Manual Readings
Rainfall Data

Appendix C Preliminary assessment – PAUP and ACRP: ALW

DRAFT assessment of geotechnical aspects of proposed development with respect to the Auckland Regional Plan: Air Land Water

6.5.76 The diversion of *groundwater* in an *unconfined aquifer* caused by changing the permeability of the *aquifer* at the location of the works by trenching, digging or tunnelling is a Permitted Activity, subject to the following conditions:		
	(a) The diversion shall not change the water level regime or direction of flow of the *aquifer* after completion of the works;	Geotechnical Comment – the works will change the water level regime in some locations
	(b) Any resulting settlement shall not cause adverse effects on buildings, structures and services.	Geotechnical Comment– the works could result in adverse settlement effects in locations if not appropriately managed

DRAFT assessment of geotechnical aspects of proposed development with respect to the Proposed Auckland Unitary Plan (Notified 30 September 2013)

PART 3 - REGIONAL AND DISTRICT RULES»Chapter H: Auckland-wide rules»4 Natural resources»4.17 Taking, using, damming and diversion of water and drilling»3. Controls»3.1 Permitted activities

»3.1.3 Water take and use of groundwater

Condition	Geotechnical comment
1. Up to 5m3/day when averaged over any consecutive 20-day period: *a. the water take must not be geothermal water, unless it is for a purpose specified in s. 14(3)(c) of the RMA.* *b. the water take must not be from the Kumeû Waitematâ or Omaha Waitematâ High-Use Aquifer Management Areas* *c. the water take must not be for the purpose of dewatering or groundwater level control** *d. notice on the prescribed form must be received by the council 15 working days before undertaking this permitted activity.*	1. Water take expected to be required during the construction stage only. Groundwater take is exepcted to exceed 5m3/day at the southern access shaft for the micro tunnelling and at deeper sections of the pipeline (Tauhinu Rd crossing and William Pickering Drive crossing. a. Water unlikely to be geothermal based on site location b. This item is not related to geotechnical aspects of the project and should be assessed by Planners c. Dewatering likerly required in locations during construction d. Noted
2. Up to 20m3/day, when averaged over any consecutive five-day period, and no more than 5000m3/year: *a. the water take must not be geothermal water unless it is for a purpose specified in s. 14(3)(c) of the RMA* *b. the water take must not be from a High-Use Aquifer Management Area* *c. the water take must not be for the purpose of dewatering or groundwater level control* *d. the water take must be located at least 100m from any other existing lawfully established groundwater take from the same aquifer* *e. notice on the prescribed form must be received by the council 15 working days before undertaking this permitted activity.*	The water take is expected to be less than 20m3/day.
3. For the purpose of a pumping test from a bore for up to seven days at an average rate of no more than 1000m3/day: *a. the water take must not be geothermal water.*	3. Proposal is not a pumping test from a bore
4. Dewatering or groundwater level control associated with a groundwater diversion permitted under clause 3.1.4 below: *a. the water take must not be geothermal water* *b. the water take must not be for a period of more than 30 days* *c. the water take must only occur during construction of the excavation, trench, tunnel or thrust bore.*	a. Water unlikely to be geothermal based on site location b. Depends on construction method but likely required in some locations c. Yes, infiltration and leakage assumed to be negligible
5.Water take or use of geothermal water for communal benefit of Mana Whenua for purposes specified in s. 14(3)(c) of the RMA *a.the water take or use must not be for commercial purposes.* *b.the water take or use does must not have an adverse effect on the environment.*	These items are not related to geotechnical aspects of the project and should be assessed by Planners
6. Infiltration and leakage into stormwater and wastewater pipes, manholes, catchpits and lined channels: *a. the water take must not be for the purpose of dewatering or groundwater level control.*	a. Not applicable. Only to manage groundwater during construction.
7. Land drainage: *a. the water take, and any associated diversion, must not be in a Natural Stream or Wetland Management Area* *b. the drainage measures must be situated less than 2m below natural ground level.*	a. Noted b. Depends on construction method but likely required in some sections. Can be mitigated by seepage collars at regular intervals.

PART 3 - REGIONAL AND DISTRICT RULES»Chapter H: Auckland-wide rules»4 Natural resources»4.17 Taking, using, damming and diversion of water and drilling»3. Controls»3.1 Permitted activities

»3.1.4 Diversion of groundwater caused by any excavation, trench, tunnel up to 1m in diameter, or thrust bore

Condition	Geotechnical comment
1.The diversion must not be for the purpose of taking groundwater.	1. Noted
2.Any excavation that extends below natural groundwater level, including any staging of the same proposal, must not exceed: *a.1ha in total area for development, operation, maintenance or upgrading of a network utility.* *b.0.5ha in total area and 4m depth below the natural ground level.*	a. See b. b. The development is greater than 0.5 ha (9708m length x 0.75 diameter pipe = 0.7 ha). The excavation is likely to extend 4m below the natural ground in a number of locations. There are also portions which are installed using horizontal drilling techniques and are likely to extend 4m below the natural ground level.
3.The natural groundwater level must not be reduced by more than 2m.	3. The groundwater level could be reduced by 2m or more in a number of locations .
4.Any structure that physically impedes the flow of groundwater must not: *a.exceed 20m in length, including any staging of the same proposal; or* *b.extend more than 2m below the natural groundwater level.*	a. The pipe is longer than 20m and is likely to impede the groundwater flow to some extent. b. The pipeline is expected to extend 2m below the groundwater level as outlined in Condition 3 above.
5.The distance to any existing building or structure from the edge of any: *a.trench or open excavation that extends below natural groundwater level must be 4m or greater* *b.tunnel with a diameter of 0.2-1.0m that extends below natural groundwater level must be 2m or greater*	a. The open excavation is less than 4m away from development at William Pickering Drive and the Kea Rental Facility near Bush Road. b. The proposed tunnel section runs close to the residential dwelling at 12 Kerema Way.
6.The distance from the edge of any excavation, including any staging of the same proposal, must not be less than 50m from any: *a.Wetland Management Area* *b.scheduled historic heritage place or scheduled sites and places of significance to Mana Whenua* *c.surface water body* *d.lawful groundwater take.*	Noted
7.For activities other than the development, operation, maintenance or upgrading of a network utility, the length of any excavation, trench, tunnel, or thrust bore, including any staging of the same proposal, must be no greater than 50m.	7. The works are for development fo a network utility
8.For the development, operation, maintenance or upgrading of a network utility, including any staging of the same proposal, any backfilled *trench must be designed and constructed with impenetrable seepage collars / barriers installed at intervals of no greater than 50m along the alignment.*	8. Tbc from the designer.

Appendix D Preliminary assessment – Settlement estimates

- Preliminary settlement assessment - Sheets 1 to 7 prepared by Tonkin & Taylor (DWG No 28773.3200-01 to 28773.3200-09 Rev 0)

NOTES:

1. The Geological Long Section drawings for the proposed Northern Interceptor alignment, Phase 1 have been prepared by MWH on behalf of Watercare. The drawings are entitled "Northern Interceptor Preliminary Design, Phase 1, Existing Hobsonville Pumping Station to Rosedale – Geological Long Section: Sheet 1 of 7 to Sheet 7 of 7, (7 drawings in total). These Tonkin & Taylor drawings (Dwg 28773.3200–01 to 28773.3200–08 Rev 0) have been prepared to illustrate the preliminary groundwater and settlement assessment results along the proposed NI Phase 1 pipe alignment.

2. The geological section is based on geotechnical investigations along the proposed NI route. The stratigraphy has been interpolated between the individual investigation locations and may vary from what is shown.

3. The hydrogeological model is based on monitoring in 13 No. piezometer standpipes between 17 December 2014 and 6 March 2015. Extended monitoring may identify further seasonal effects and variations in groundwater levels, including elevated winter conditions. The groundwater levels are shown as RL(m).

4. These drawings should be reviewed in conjunction with the geotechnical report entitled "Northern Interceptor Phase 1 – Groundwater and Settlement Assessment Report", refer 28773.32 prepared by Tonkin & Taylor Ltd.

5. The estimated groundwater drawdown has been assessed between the recorded low/summer ground water levels and the proposed pipe trench depth level, as defined by MWH.

6. The estimated groundwater drawdown and mechanically induced settlement is the inferred maximum settlement, immediately adjacent to the pipeline. The inferred settlement decreases away from the pipeline over distance refered to as "radius of influence" at which point estimated settlement is negligible. The settlement estimates have been assessed at the ground surface and may decrease with depth.

7. The total settlement combine the groundwater drawdown and mechanically induced settlement by superposition. The combination of the settlement effects have been assessed as the long term case, i.e. assuming full settlement has occurred for all sources.

8. The estimated seepage could occur long–term unless the trench is fitted with seepage collars or similar. Higher estimated seepage may occur if the pipeline is constructed during winter.

Aerial photo sourced from Auckland Council GIS Website

KEY PLAN
NOT TO SCALE

I:\28773\28773.3200\workingmaterial\CAD\DWG\28773.3200-01.dwg, 01, 18/05/2015 8:38:23 a.m., jzm, 1:2

DRAWING STATUS: PRELIMINARY DRAFT

REVISION	DESCRIPTION	BY	DATE
0	First Issue		

DESIGNED :	KJ	May. 15
DRAWN :	JATG	May. 15
DESIGN CHECKED :		
DRAFTING CHECKED :		

CADFILE : \\28773.3200–01.dwg

APPROVED :
NOT FOR CONSTRUCTION
This drawing is not to be used for construction purposes unless signed as approved

NOTES :

REFERENCE :

Tonkin & Taylor
Environmental and Engineering Consultants
105 Carlton Gore Road, Newmarket, Auckland
Tel. (09) 355 6000 Fax. (09) 307 0265
www.tonkin.co.nz

CLIENT, PROJECT
Watercare
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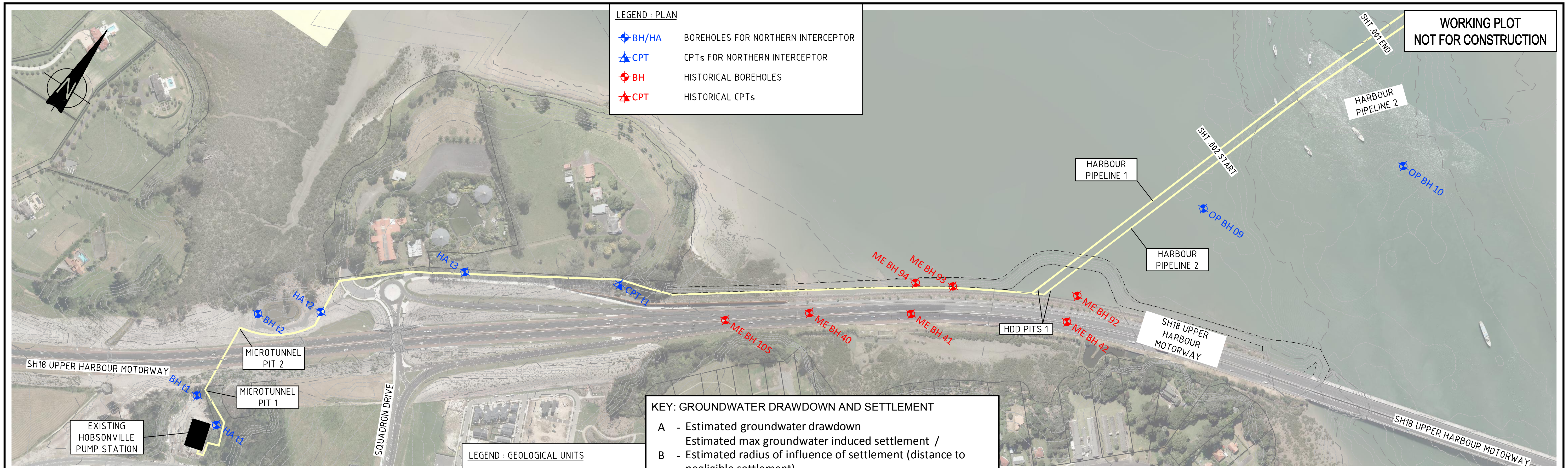
NORTHERN INTERCEPTOR PRELIMINARY DESIGN

TITLE
PHASE 1 EXISTING HOBSONVILLE PUMPING STATION TO ROSEDALE
Preliminary Settlement Assessment: Key Plan and Notes

SCALES (AT A1 SIZE)	DWG. No.	REV.
NTS	28773.3200–01	0



WORKING PLOT
NOT FOR CONSTRUCTION

LEGEND : PLAN

BH/HA	BOREHOLES FOR NORTHERN INTERCEPTOR
CPT	CPTs FOR NORTHERN INTERCEPTOR
BH	HISTORICAL BOREHOLES
CPT	HISTORICAL CPTs

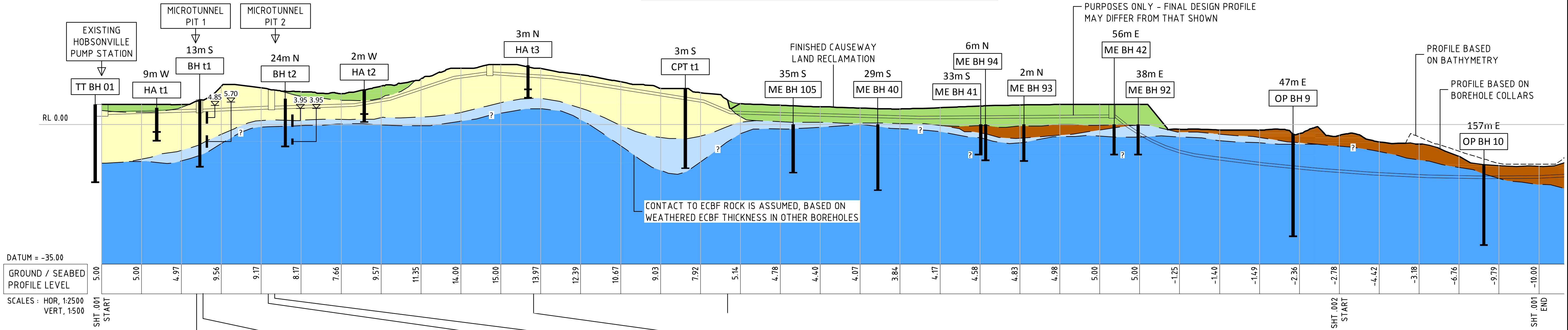
LEGEND : GEOLOGICAL UNITS

- FILL
- MARINE MUDS
- TAURANGA GROUP
- WEATHERED ECBF
- ECBF ROCK (SPT N 50+)

KEY: GROUNDWATER DRAWDOWN AND SETTLEMENT

- A - Estimated groundwater drawdown
- B - Estimated max groundwater induced settlement / Estimated radius of influence of settlement (distance to negligible settlement)
- C - Estimated max mechanical induced settlement / Estimated radius influence of settlement
- D - Estimated total max settlement / Estimated max radius influence at settlement (total)
- E - Estimated seepage

Refer to Dwg 28773.3200-01 for Notes 1 to 8



A	≈2m	<3m	<3m	<1m	<1m	<1m	0m
B	45mm/<5m	40mm/<5m	35mm /<5m	<5mm/<5m	<5mm/<5m	<5mm/<5m	<5mm/5m
C	35mm/8m	35mm/8m	5mm/10m	20mm/15m	50mm/12m	30mm/8m	30mm/8mm
D	80mm/8m	75mm/8m	40mm/10m	<25mm/15m	55mm/12m	<35mm/8m	35mm/8mm
E	0.5m³/lm trench	6-7m³/24hr	0.5m³/lm trench/day	<0.1m³/lm trench/day	<1m³/24hr	0.2m³/lm trench/day	<0.1m³/lm trench/day

Out of scope
Part of Greenhithe Bridge Watermain Duplication and Causeway project

Upper Waitemata Harbour crossing
The settlement caused by the pipe construction is likely to be of an order that is not locally noticeable on the mud flats or stream beds, and there is not expected to be any impact on the natural processes within the intertidal or sub-tidal areas

REVISION	DESCRIPTION	BY	DATE
0	First Issue		

DESIGNED :	KJ	May. 15
DRAWN :	JATG	May. 15
DESIGN CHECKED :		
DRAFTING CHECKED :		
CADFILE : \\28773.3200-02_08.dwg		
APPROVED :		

NOT FOR CONSTRUCTION
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NOTES :
1. Refer to Dwg 28773.3200-01 for notes.

REFERENCE :

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TITLE
PHASE 1 EXISTING HOBSONVILLE PUMPING STATION TO ROSEDALE
Preliminary Settlement Assessment: Sheet 1 of 7

SCALES (AT A1 SIZE) AS SHOWN | DWG. No. 28773.3200-02 | REV. 0

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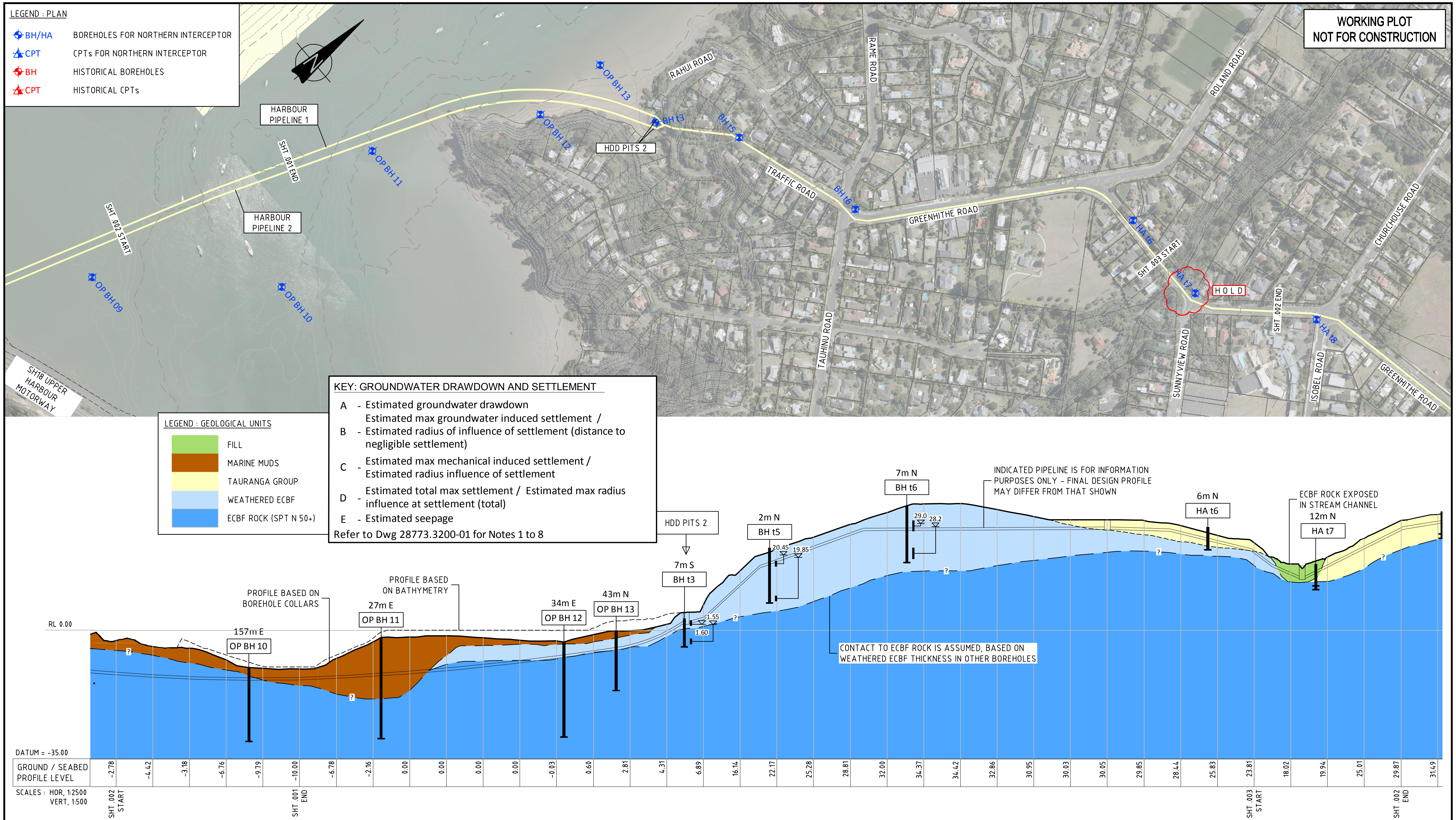

A	Upper Waitemata Harbour crossing The settlement caused by the pipe construction is likely to be of an order that is not locally noticeable on the mud flats or stream beds, and there is not expected to be any impact on the natural processes within the intertidal or sub-tidal areas	<2m	<3m	<2m
B		20mm/<5m	25mm/<5m	20mm/<5m
C		30mm/8m	50mm/12m	30mm/8m
D		50mm/8m	75mm/12m	50mm/8m
E		0.4m^3/lm trench/day	0.4m^3/lm trench/day	0.2m^3/lm trench/day

DESIGNED :	KJ	May. 15	
DRAWN :	JATG	May. 15	
DESIGN CHECKED :			
DRAFTING CHECKED :			
CADFILE : \\28773.3200-02_08.dwg			
APPROVED : NOT FOR CONSTRUCTION			

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0 First Issue

REVISION DESCRIPTION | BY | DATE

NOTES :
1. Refer to Dwg 28773.3200-01 for notes.

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TITLE
PHASE 1 EXISTING HOBSONVILLE PUMPING STATION TO ROSEDALE
Preliminary Settlement Assessment: Sheet 2 of 7

SCALES (AT A1 SIZE) AS SHOWN | DWG. No. 28773.3200-03 | REV. 0

I:\28773\28773.3200\workingmaterial\CAD\DWG\28773.3200-02_08.dwg, 03, 18/05/2015 8:39:13 a.m., jzm, 1:2

LEGEND : PLAN

BH/HA	BOREHOLES FOR NORTHERN INTERCEPTOR
CPT	CPTs FOR NORTHERN INTERCEPTOR
BH	HISTORICAL BOREHOLES
CPT	HISTORICAL CPTs

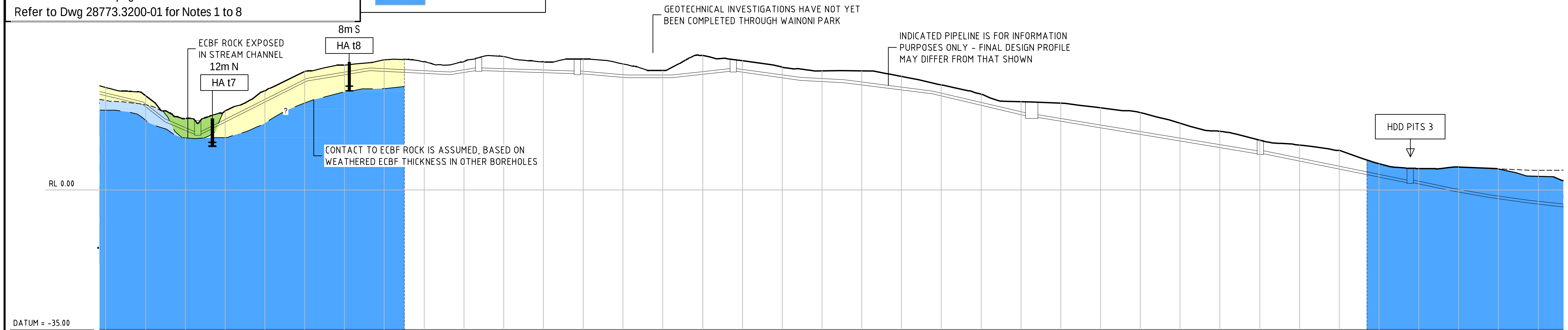
KEY: GROUNDWATER DRAWDOWN AND SETTLEMENT

- A - Estimated groundwater drawdown
- B - Estimated max groundwater induced settlement / Estimated radius of influence of settlement (distance to negligible settlement)
- C - Estimated max mechanical induced settlement / Estimated radius influence of settlement
- D - Estimated total max settlement / Estimated max radius influence at settlement (total)
- E - Estimated seepage

Refer to Dwg 28773.3200-01 for Notes 1 to 8

LEGEND : GEOLOGICAL UNITS

- FILL
- MARINE MUDS
- TAURANGA GROUP
- WEATHERED ECBF
- ECBF ROCK (SPT N 50+)



GROUND / SEABED PROFILE LEVEL																																			
	25.85	23.81	18.02	19.94	25.01	29.87	31.49	32.77	32.30	32.37	33.64	32.26	32.94	31.77	30.10	33.91	32.58	30.95	30.04	30.02	28.61	26.50	24.14	22.28	21.92	20.75	19.51	16.82	14.82	12.62	11.37	10.00	6.66	5.48	5.83	5.42	5.01

SCALES : HOR, 1:2500
VERT, 1:500

A	<2m (TBC when geotechnical investigations are complete)	Te Wharau Creek crossing
B	20mm/<5m	The settlement caused by the pipe construction is likely to be of an order that is not locally noticeable on the mud flats or stream beds, and there is not expected to be any impact on the natural processes within the intertidal or sub-tidal areas
C	30mm/8m	
D	50mm/8m	
E	0.2m³/lm trench/day	

0	First Issue		
REVISION	DESCRIPTION	BY	DATE

DESIGNED :	KJ	May. 15
DRAWN :	JATG	May. 15
DESIGN CHECKED :		
DRAFTING CHECKED :		
CADFILE : \\28773.3200-02_08.dwg		
APPROVED :		

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NOTES :
1. Refer to Dwg 28773.3200-01 for notes.

REFERENCE :

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TITLE
PHASE 1 EXISTING HOBSONVILLE PUMPING STATION TO ROSEDALE
Preliminary Settlement Assessment: Sheet 3 of 7

SCALES (AT A1 SIZE)	DWG. No.	REV.
AS SHOWN	28773.3200-04	0

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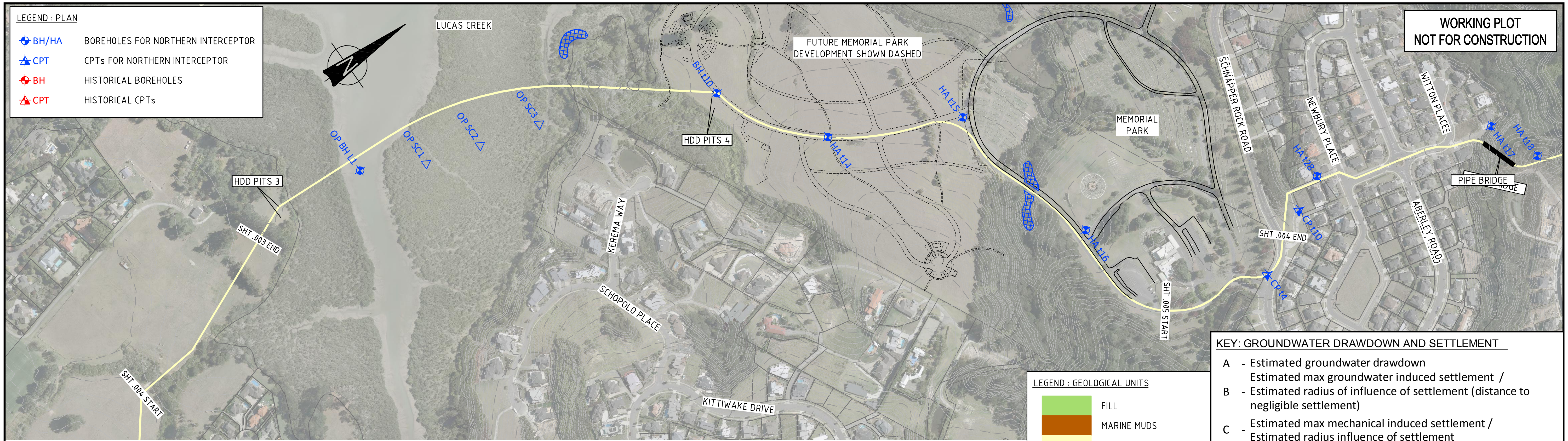


LEGEND : PLAN

BH/HA	BOREHOLES FOR NORTHERN INTERCEPTOR
CPT	CPTs FOR NORTHERN INTERCEPTOR
BH	HISTORICAL BOREHOLES
CPT	HISTORICAL CPTs

WORKING PLOT
NOT FOR CONSTRUCTION

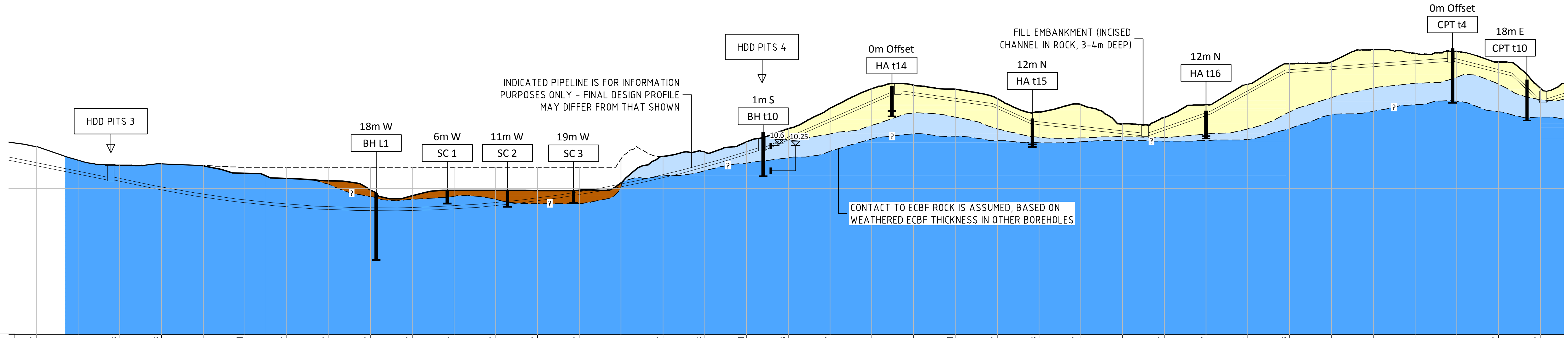
LEGEND : GEOLOGICAL UNITS

- FILL
- MARINE MUDS
- TAURANGA GROUP
- WEATHERED ECBF
- ECBF ROCK (SPT N 50+)

KEY: GROUNDWATER DRAWDOWN AND SETTLEMENT

- A - Estimated groundwater drawdown
- B - Estimated max groundwater induced settlement / Estimated radius of influence of settlement (distance to negligible settlement)
- C - Estimated max mechanical induced settlement / Estimated radius influence of settlement
- D - Estimated total max settlement / Estimated max radius influence at settlement (total)
- E - Estimated seepage

Refer to Dwg 28773.3200-01 for Notes 1 to 8



GROUND / SEABED PROFILE LEVEL: 10.00, 6.66, 5.48, 5.83, 5.42, 5.01, 5.00, 5.00, 5.00, 5.00, 5.00, 5.00, 5.00, 5.00, 6.99, 7.60, 8.63, 11.01, 14.18, 19.05, 23.72, 24.62, 23.61, 21.40, 18.18, 19.97, 15.36, 16.90, 19.95, 24.84, 29.78, 30.22, 33.12, 32.22, 32.69, 30.20, 23.90

SCALES : HOR, 1:2500
VERT, 1:500

SHT .004 START — SHT .003 END — SHT .005 START — SHT .004 END

A	Te Wharau Creek crossing The settlement caused by the pipe construction is likely to be of an order that is not locally noticeable on the mud flats or stream beds, and there is not expected to be any impact on the natural processes within the intertidal or sub-tidal areas	<1m	≈2m	<1m
B		<5mm/<5m	20mm/<5m	<5mm/<5m
C		30mm/8m	30mm/8m	30mm/8m
D		35mm/8m	50mm/8m	35mm/8m
E		<0.1m^3/ lm trench/day	0.2m^3/lm trench/day	<0.1m^3/1m trench/day

REVISION	DESCRIPTION	BY	DATE
0	First Issue		

DESIGNED :	KJ	May. 15
DRAWN :	JATG	May. 15
DESIGN CHECKED :		
DRAFTING CHECKED :		
CADFILE : \\28773.3200-02_08.dwg		
APPROVED : NOT FOR CONSTRUCTION		

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NOTES :
1. Refer to Dwg 28773.3200-01 for notes.

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TITLE
PHASE 1 EXISTING HOBSONVILLE PUMPING STATION TO ROSEDALE
Preliminary Settlement Assessment: Sheet 4 of 7

SCALES (AT A1 SIZE)	DWG. No.	REV.
AS SHOWN	28773.3200-05	0

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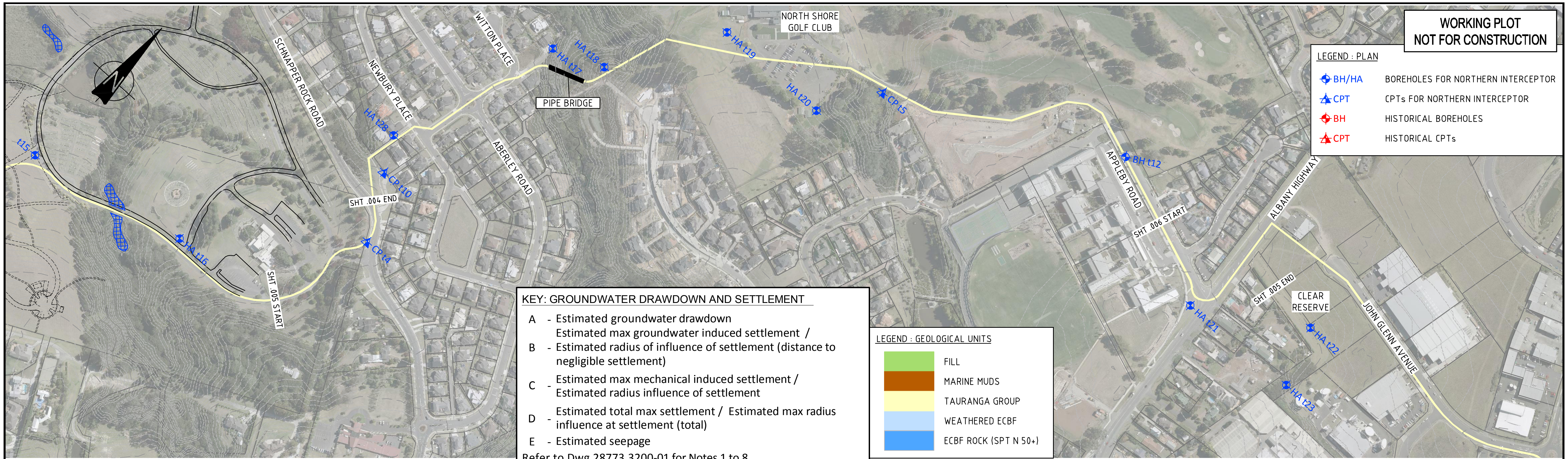
WORKING PLOT
NOT FOR CONSTRUCTION



LEGEND : PLAN

BH/HA	BOREHOLES FOR NORTHERN INTERCEPTOR
CPT	CPTs FOR NORTHERN INTERCEPTOR
BH	HISTORICAL BOREHOLES
CPT	HISTORICAL CPTs

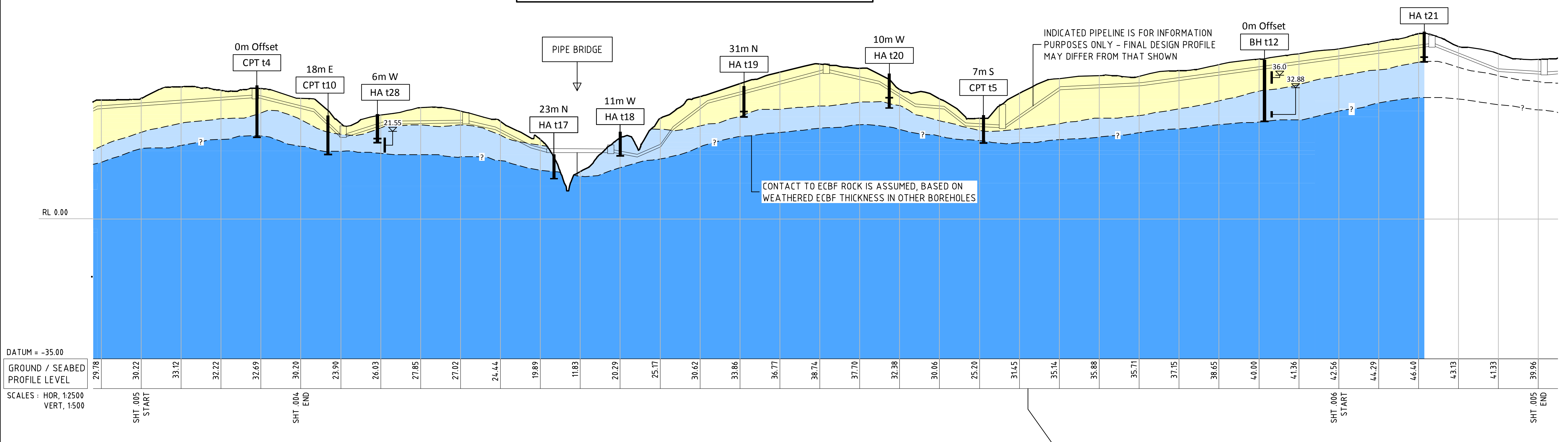
KEY: GROUNDWATER DRAWDOWN AND SETTLEMENT

A - Estimated groundwater drawdown
B - Estimated max groundwater induced settlement / Estimated radius of influence of settlement (distance to negligible settlement)
C - Estimated max mechanical induced settlement / Estimated radius influence of settlement
D - Estimated total max settlement / Estimated max radius influence at settlement (total)
E - Estimated seepage

Refer to Dwg 28773.3200-01 for Notes 1 to 8

LEGEND : GEOLOGICAL UNITS

- FILL
- MARINE MUDS
- TAURANGA GROUP
- WEATHERED ECBF
- ECBF ROCK (SPT N 50+)



A	<1m	≈2m	<1m
B	<5mm/<5m	20mm/<5m	<5mm/5m
C	30mm/8m	50mm/12m	30mm/8m
D	35mm/8m	70mm/12m	35mm/8m
E	<0.1m³/lm trench/day	0.2m³/lm trench/day	<0.1m³/lm trench/day

			DESIGNED :		KJ	May. 15
			DRAWN :		JATG	May. 15
			DESIGN CHECKED :			
			DRAFTING CHECKED :			
			CADFILE : \\28773.3200-02_08.dwg			
0	First Issue		APPROVED : NOT FOR CONSTRUCTION This drawing is not to be used for construction purposes unless signed as approved			
REVISION	DESCRIPTION	BY	DATE			

NOTES :
1. Refer to Dwg 28773.3200-01 for notes.

REFERENCE :

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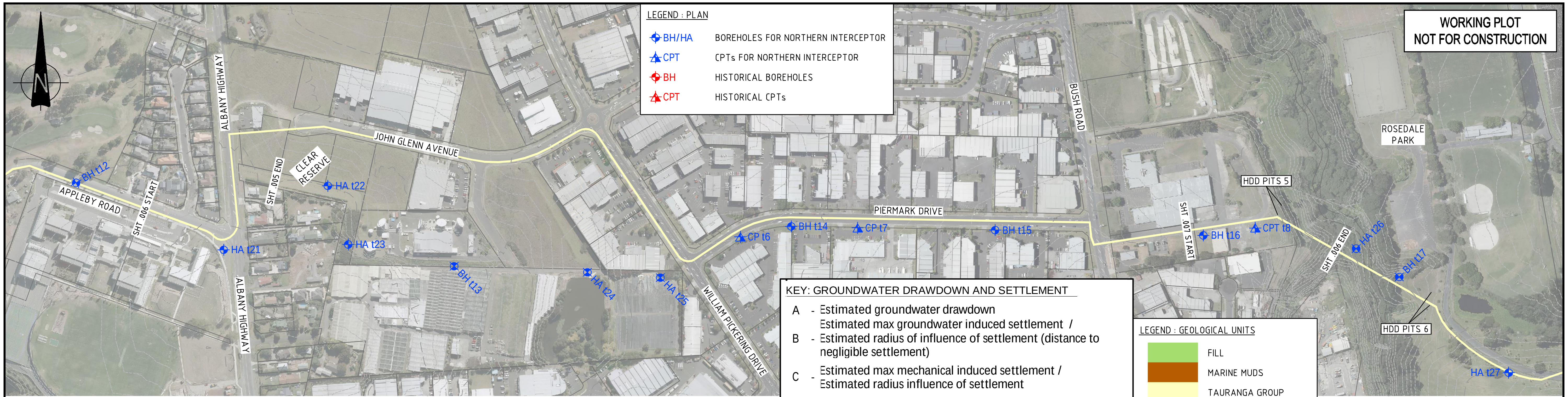
TITLE
PHASE 1 EXISTING HOBSONVILLE PUMPING STATION TO ROSEDALE
Preliminary Settlement Assessment: Sheet 5 of 7

SCALES (AT A1 SIZE) AS SHOWN | DWG. No. 28773.3200-06 | REV. 0



KEY: GROUNDWATER DRAWDOWN AND SETTLEMENT

A - Estimated groundwater drawdown

B - Estimated max groundwater induced settlement / Estimated radius of influence of settlement (distance to negligible settlement)

C - Estimated max mechanical induced settlement / Estimated radius influence of settlement

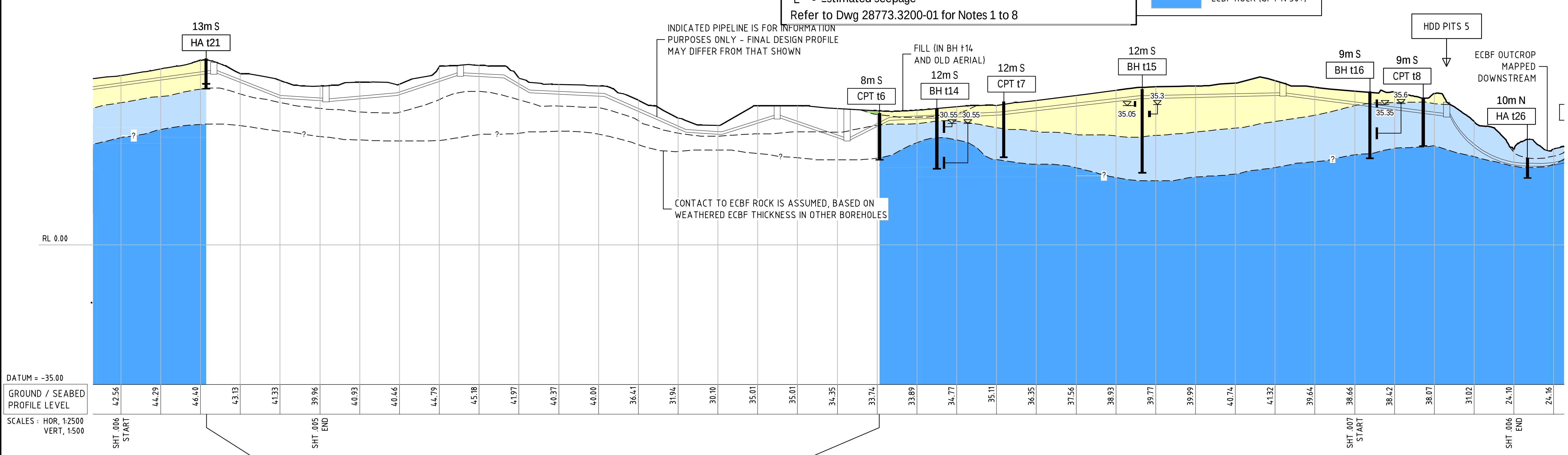
D - Estimated total max settlement / Estimated max radius influence at settlement (total)

E - Estimated seepage

Refer to Dwg 28773.3200-01 for Notes 1 to 8

LEGEND : GEOLOGICAL UNITS

- FILL
- MARINE MUDS
- TAURANGA GROUP
- WEATHERED ECBF
- ECBF ROCK (SPT N 50+)



A	<1m	3.5m (TBC when geotechnical investigations are complete)	5m	1m	<2m	Alexandra Stream crossing
B	<5mm/<5m	25mm/<5m	30mm/<5m	10mm/<5m	25mm/<5m	
C	30mm/8m	30mm/8m	80mm/20m	35mm/8m	35mm/8m	
D	35mm/8m	55mm/8m	110mm/10m	45mm/8m	60mm/8m	
E	<0.1m³/lm trench/day	0.2m³/lm trench/day	0.2m³/lm trench/day	0.2m³/lm trench/day	0.2m³/lm trench/day	

0	First Issue		
REVISION	DESCRIPTION	BY	DATE

DESIGNED : KJ Jul. 15

DRAWN : JATG Jul. 15

DESIGN CHECKED :

DRAFTING CHECKED :

CADFILE : \\28773.3200-02_08.dwg

APPROVED :

NOT FOR CONSTRUCTION

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NOTES :
1. Refer to Dwg 28773.3200-01 for notes.

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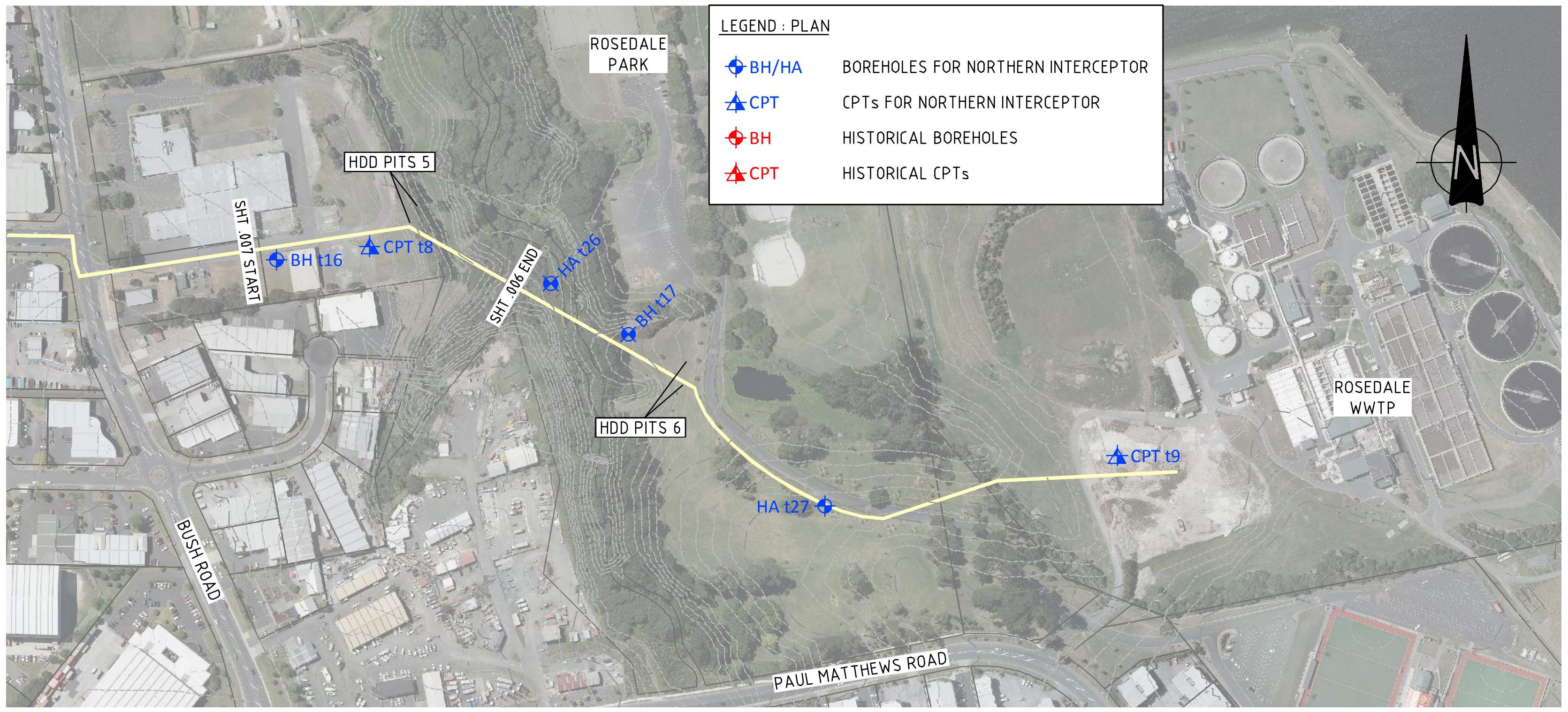
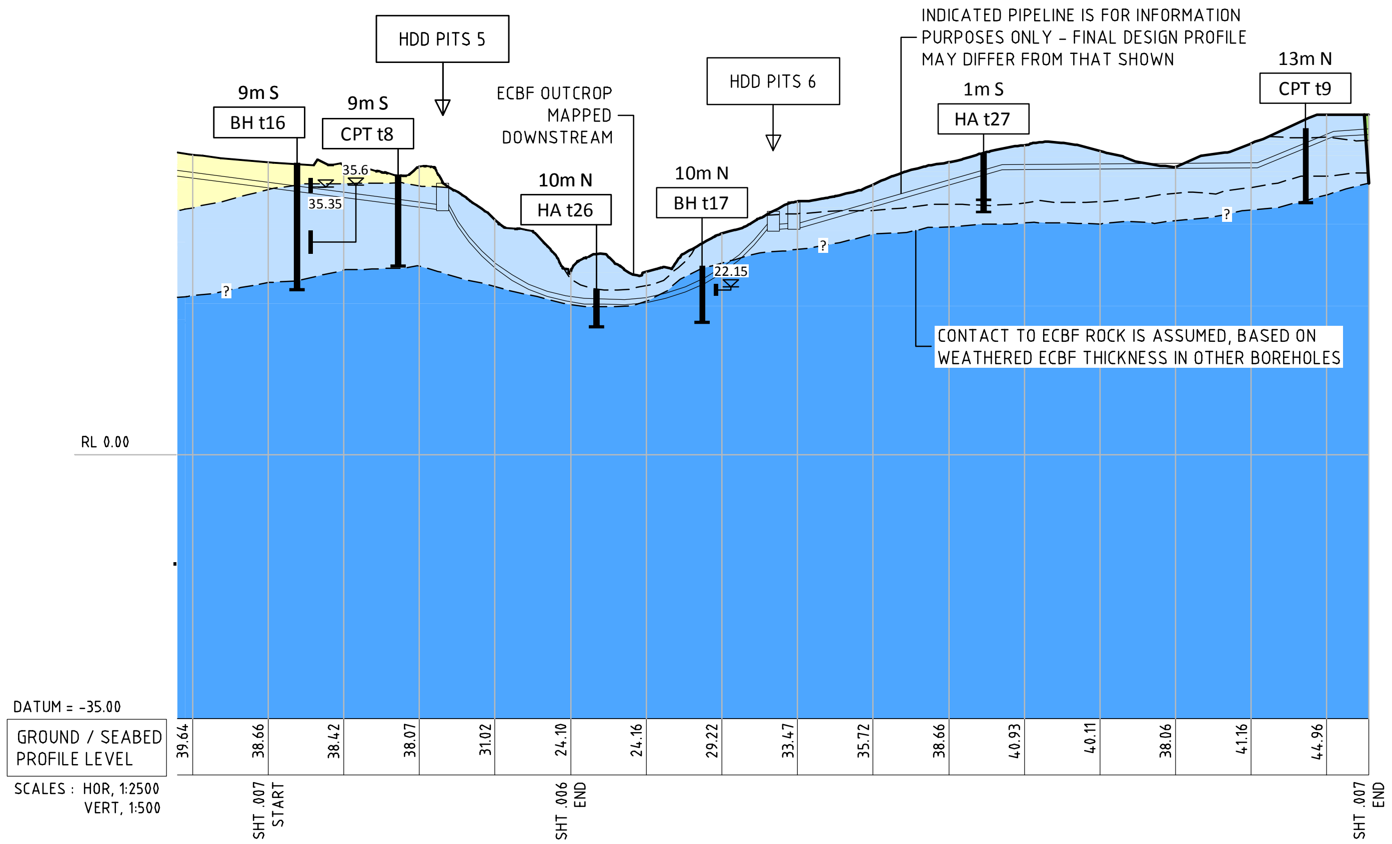
TITLE
PHASE 1 EXISTING HOBSONVILLE PUMPING STATION TO ROSEDALE
Preliminary Settlement Assessment: Sheet 6 of 7

SCALES (AT A1 SIZE) AS SHOWN
DWG. No. 28773.3200-07
REV. 0

WORKING PLOT
NOT FOR CONSTRUCTION





KEY: GROUNDWATER DRAWDOWN AND SETTLEMENT

A - Estimated groundwater drawdown
B - Estimated max groundwater induced settlement / Estimated radius of influence of settlement (distance to negligible settlement)
C - Estimated max mechanical induced settlement / Estimated radius influence of settlement
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E - Estimated seepage

Refer to Dwg 28773.3200-01 for Notes 1 to 8

LEGEND : GEOLOGICAL UNITS

- FILL
- MARINE MUDS
- TAURANGA GROUP
- WEATHERED ECBF
- ECBF ROCK (SPT N 50+)

	Alexandra Stream crossing	
A	Alexandra Stream crossing	1m
B	The settlement caused by the pipe construction is likely to be of an order that is not locally noticeable on the stream bed, and there is not expected to be any impact on the natural processes within the intertidal or sub-tidal areas	10mm/<5m
C		35mm/8m
D		45mm/8m
E		0.2m^3/lm trench/day

0	First Issue		
REVISION	DESCRIPTION	BY	DATE

DESIGNED :	KJ	May. 15
DRAWN :	JATG	May. 15
DESIGN CHECKED :		
DRAFTING CHECKED :		
CADFILE : \\28773.3200-02_08.dwg		
APPROVED : NOT FOR CONSTRUCTION		

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NOTES :
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TITLE
PHASE 1 EXISTING HOBSONVILLE PUMPING STATION TO ROSEDALE
Preliminary Settlement Assessment: Sheet 7 of 7

SCALES (AT A1 SIZE)	DWG. No.	REV.
AS SHOWN	28773.3200-08	0

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