REPORT

Watercare Services Ltd

Central Interceptor Project
Effect of Tunnels on Groundwater and Surface Settlement

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Executive summary

Watercare Services Ltd (Watercare) is planning to construct a new sewer tunnel to collect wastewater flows from sewer networks in the Auckland isthmus area and transfer them to the Mangere Wastewater Treatment Plant (MWWTP). Known as the Central Interceptor, the tunnel will be approximately 4.5 m internal diameter and some 13 km long.

The Central Interceptor tunnel commences in the Western Springs area and terminates at a new major pump station at the WWTP via an alignment beneath the Manukau Harbour and lowlands. Wastewater flows from key catchments along the route will be collected by various new link sewers (a further 5 km of tunnels) and pipelines connecting to the tunnel at shafts.

This report presents the findings of a study carried out to assess the potential effects of the main tunnel, link tunnels and shaft construction on groundwater and of potential surface settlement that may result.

Tunnels of similar configuration to that proposed for the Central Interceptor Project have been constructed in the Auckland isthmus in recent history without significant groundwater or surface settlement effects. Construction of Central Interceptor tunnels is therefore also likely to be possible without significant effects given the similarities in tunnel size and geological conditions in this area.

Data from the historic projects and the results of analyses carried out for this study identify that the magnitude and extent of groundwater effects and resulting surface settlements are a function of tunnel design and tunnel construction methodology.

Numerical models have been utilised to assess the potential groundwater and surface settlement effects that could arise for a range of potential tunnel and shaft construction methodologies. Primarily the analyses consider capacity that these methodologies have to directly control groundwater effects during excavation, and the potential range of effects resulting from time delay between excavation and completion of a watertight lining. The degree of tunnel liner water-tightness required to control long term groundwater and settlement effects is also assessed.

Watercare Services Limited has advised they intend to construct the tunnels using an Earth Pressure Balance capable Tunnel Boring Machine (EPB TBM), installing a low permeability concrete liner (fully gasketed segmental concrete liner). A range of methodologies are being considered for shaft construction.

The study concludes that using an EPB TBM it is possible to design and construct the tunnels and shafts for the Central Interceptor Project such that:

- There is negligible risk of tunnel and shaft construction having an effect on nearby groundwater users
- There is negligible risk of surface settlement that may result in structural damage to buildings and services resulting from tunnel or shaft excavation
- There is a low risk of measurable changes in groundwater quality immediately about the tunnel and negligible risk of any adverse effect on regional groundwater quality.

It also concludes that for significant portions of the tunnel alignment, alternative construction methodologies could be employed while achieving the above key outcomes.

1 Introduction

1.1 General

Watercare Services Ltd (Watercare) is planning to construct a new sewer tunnel to collect wastewater flows from the Auckland isthmus area and transfer them across the Manukau Harbour and the Manukau Lowlands to the Mangere Wastewater Treatment Plant (MWWTP). The route is shown on Figure 1. The Central Interceptor Project (the Project) arose out of the Three Waters Plan (2008) which identified the need to provide trunk sewer capacity to central Auckland to reduce wet weather wastewater overflows and provide capacity for growth.

The overall concept proposed for the Central Interceptor is a gravity tunnel from the Western Springs area to a major pump station at the Mangere WWTP with various link sewers and connecting pipelines connecting the existing network to the main tunnel at key locations along this route.

1.2 Study scope

This report has been prepared for Watercare to provide information required for submission with applications for all necessary RMA approvals. The report specifically addresses:

- Potential for groundwater drawdown effects associated with the construction and operation of the tunnels and shafts
- Potential for surface settlement that may be induced by tunnel construction and operational groundwater drawdown effects
- Potential for groundwater flows into tunnel and shaft excavations during construction
- Potential wastewater seepage out of the tunnel and shaft linings during operation
- Effects of groundwater drawdown on consented users of the groundwater resource.

The potential for ground settlement from non-groundwater origins (e.g. tunnel excavation ground loss) is considered by others and does not form part of the scope for this assessment. The results of the assessment by others (AECOM) are however reported here in Section 5.

1.3 Report overview

The report is set out in three parts as described below:

PART A - Provides a summary of information adopted or used in the assessment of effects. Project details, as currently understood, are outlined in Section 2. Existing geological and geological conditions are described in Section 3, and hydrogeological conditions in Section 4.

PART B - Discusses the potential effects of tunnelling on groundwater (Section 5) and of surface settlement in response to the changes (Section 5). Observations/ experience and data from similar projects (Section 5) and theoretical models (Section 6 and 7) are utilised to provide settlement estimates (Section 8). Monitoring of construction to verify that actual settlements are within tolerable limits is discussed in Section 9.

PART C - Presents conclusions and recommendations arising from the study.

PART A

2 Overview of Central Interceptor Project

An overview of the Central Interceptor Project is presented in this section as background to preliminary tunnel design, construction and operation. The design information forms part of the input to the assessment of potential effects. The main tunnel will be approximately 4.5 m in diameter providing gravity flow for collected sewer flows from Western Springs to the Mangere Treatment Plant. Four small diameter (2.5 m) branch sewers connect to the main tunnel bringing flows from other catchments. A number of deep shafts connect the tunnel to the ground surface and provide access for construction and long term maintenance. Tunnel boring machine(s) will be used to excavate the tunnel(s), with segmental concrete lining constructed behind the earth pressure balancing capable cutter head.

The tunnel will operate under partial flow conditions for the majority of the time. Design storm events will see the tunnel surcharged by up to 30 m head internally.

A number of tunnels of similar size, in similar geology and utilising similar construction methodologies have been successfully constructed within consent allowances within the Auckland area in the last 20 years. Observations from construction monitoring from these projects are provided as context to this assessment.

2.1 Tunnel configuration

The Central Interceptor project involves approximately 18 km of tunnels, to construct a wastewater sewer extending from Western Springs Park, at the upstream extent, to the Mangere Wastewater Treatment Plant (WWTP). Refer to Figure 1.

The tunnels will connect to the existing Watercare network at key points, each sited to collect flows from, and reduce load on, the existing network. At the connection sites, structures are required for connecting to the existing network, and also for grit removal, odour treatment and access at some sites (shafts labelled AS1 to AS7 and WS1 to WS3). The Consolidation Sewer Overflow (CSO) tunnels are the subject of a separate report (Tonkin & Taylor 2011b).

At the WWTP a new major pump station (constructed in a shaft - WS3) will pump wastewater out of the tunnel and to the treatment plant.

The key elements of the project include:

- An approximately 13 km long 4.5 m diameter main tunnel from Western Springs to Mangere WWTP, up to 110 m below ground.
- Four smaller diameter link sewers (2.5 m diameter) at up to 80 m below ground level connecting the main tunnel to the existing wastewater network. Three of the link sewers are to be constructed by tunnelling.
- Associated connections to existing sewers.
- Associated structures at key sites along the route and at connections. At each site facilities include access shafts, drop shafts, and flow control structures. Grit traps, air intakes, air vents, or air treatment facilities are proposed at some sites.
- A limited number of overflow structures in nearby watercourses to enable the safe discharge of occasional overflows from the tunnel.
- A pump station located at the Mangere WWTP.

- Other associated works at and in the vicinity of the Mangere WWTP, including a rising main to connect to the WWTP and an emergency pressure relief structure to enable the safe discharge of flows in the event of pump station failure.

The main tunnel, link sewers, connection pipes and many of the associated structures will be underground. The tunnel and three of the four link sewers will be constructed by tunnelling methods, with access provided from around 20 surface construction sites.

Resource consents are being sought for a corridor within which the tunnels will be finally located. Horizontally, the tunnels will be located within a 40 m wide corridor centred on the concept design alignment shown in Figure 1 (i.e. 20 m either side of the alignment shown). At Kiwi Esplanade, consent is sought for two alternative alignments. For the concept design selected and the assessments made for this report, the width of the corridor and the location of the alternative alignments have no significant impact on the findings.

Vertically, the tunnels will be located within a 20 m high corridor, with the level of the vertical corridor varying along the alignment due to the required hydraulic grade of the tunnels. The main tunnel will be located within a vertical corridor that extends approximately from the top of the concept design tunnel alignment (shown on Figures A5 to A9) to 15 m below the bottom of this alignment. At the Western Springs site the vertical corridor extends from approximately -9 m RL to -29 m RL, while at the Mangere Pump Station site it extends from approximately -23 m RL to -43 m RL. Link Sewers 1, 2 and 3 will be located within a vertical corridor that extends approximately from 2 m above the top of the tunnel invert in the concept design to 15 m below the bottom of the tunnel. For the purposes of this report the effects of the tunnels are based on the shallowest tunnel alignment within the corridor. If deeper alignments were to be selected for the final design then the surface effects are expected to be lesser than those presented here.

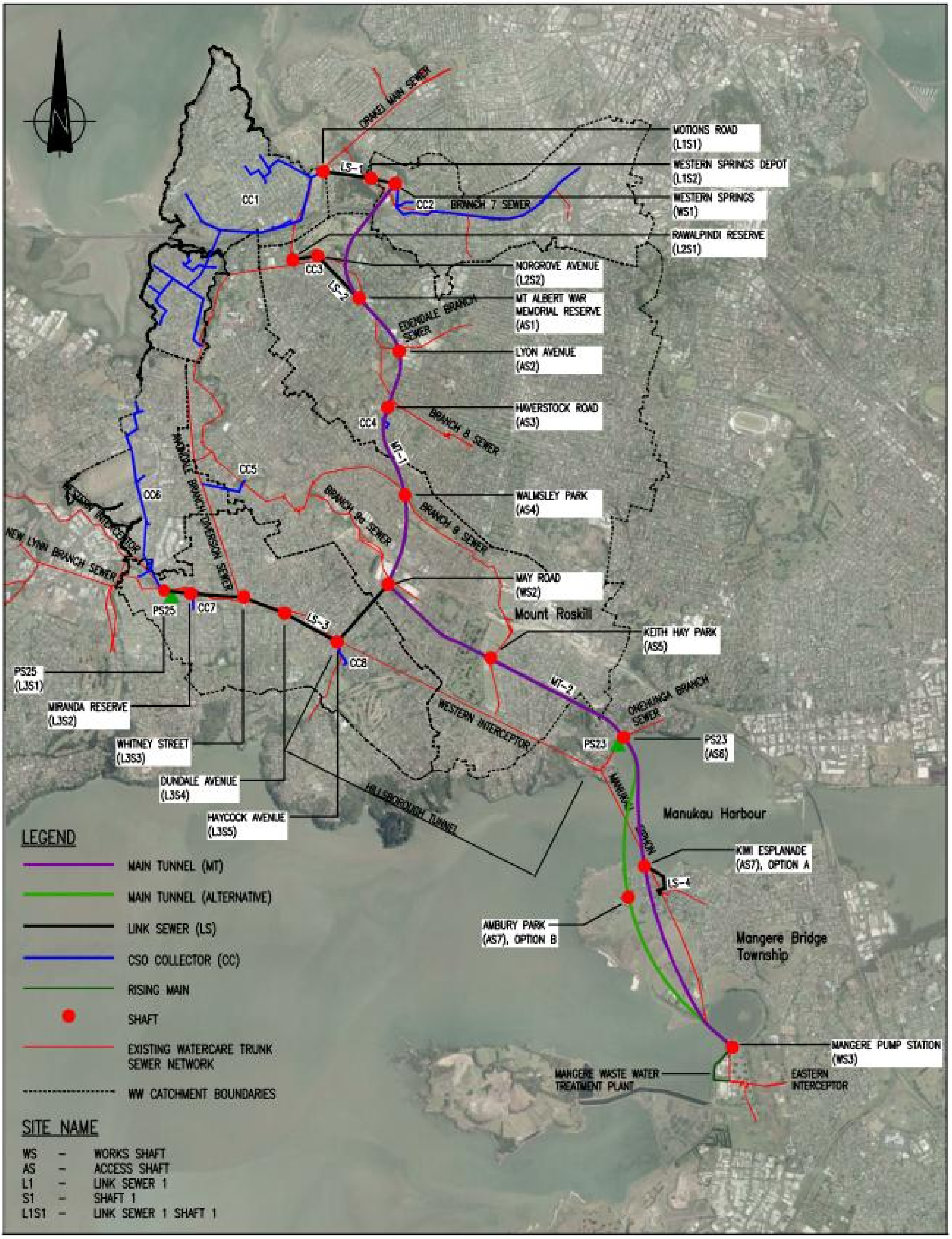

Figure 1- Project Layout

2.2 Land use

The tunnel alignment (main tunnel and link sewers) is primarily beneath densely developed residential and commercial/industrial land in the Auckland isthmus section, and through open land in the Manukau Lowlands in the approach to the MWWTP.

The structures present along the main tunnel and link sewer routes are predominantly low rise (1-3 storey) residential structures. Additionally there are a number of commercial and industrial structures along the length of the tunnel routes. Specific large notable structures that have been identified along the route are tabulated in Appendix F.

Infrastructure, including transport, water, and electricity infrastructure is present above the tunnel throughout various alignments as is the Watercare's Western Interceptor sewer tunnel. Significant infrastructure elements that the tunnel passes underneath are summarised in Appendix F. This includes two motorway crossings, a rail line, as well as buried pipelines, including the NZRC Refinery to Auckland Pipeline.

The main tunnel alignment passes under two major aquifers, the Western Springs Volcanic Aquifer in the basalt, and the Manukau Kaawa Aquifer in the Kaawa Formation. The tunnel also passes a terrestrial watercourse, the Miranda Reserve Stream at the head of the Whau River. These aquifers and watercourses are summarised in Appendix F.

2.3 Construction methodology

The project has been developed to a concept design stage. It is likely that some details may change as the project moves through the detailed design process. Detailed construction method will be determined following appointment of a construction contractor.

2.3.1 Tunnels

While the report provides estimates of settlement effects of tunnel construction, representing a range of potential construction methodologies, Watercare Services Limited has indicated they intend to construct the tunnels using an Earth Pressure Balance (EPB) type TBM. The EPB option has been successfully used in Auckland area for the recently constructed Hobson and Rosedale Outfall tunnels and provides a robust low impact construction methodology.

2.3.2 Shafts

Construction and operation of the tunnel and link tunnels requires a number of shafts. These surface construction sites include:

- 3 "major" construction sites (at Western Springs (WS1), May Road (WS2) and Mangere WWTP (WS3));
- 7 "intermediate" construction sites to provide connections to the main tunnel (AS1-7);
- 10 "small" and "intermediate" sites to provide connections to the link sewers some requiring two shafts ("L_S_").

The major construction sites will be used for launching or retrieving the tunnel boring machine(s) and materials for tunnel construction would be delivered and stored, and tunnel spoil removed. The shafts providing access for tunnel construction ("WS") will typically require excavations in the order of 25 to 35 m in diameter and up to 70 m deep. The shafts constructed for access during tunnel operation ("AS") typically require excavations in the order of 9m diameter, at depths of between 30 to 80 m.

Table 2-1 – Shaft configurations

Type of shaft	Shaft ID/Location		Construction diameter approx. (m)	Construction depth (m)
Construction access	WS1	Western Springs	25 x 15 (oval)	27
	WS2	May Road	25 x 15 (oval)	70
	WS3	Mangere Pump Station	35	32
Operation access	AS1	Mt Albert War Memorial Reserve	13 + 8.5	38
	AS2	Lyon Avenue	9	45
	AS3	Haverstock Road	9	50
	AS4	Walmsley Park	9	67
	AS5	Keith Hay Park	9	81
	AS6	PS23	9	28
	AS7	Kiwi Esplanade	9	29

Details from Central Interceptor Project Design Team - 2011

Construction methodology employed will depend on specific site conditions. 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 draw down to acceptable levels (less than “minor effect”) in surrounding geology:

- Secant piles
- Diaphragm walls
- Open caisson
- Grouting.

Where effects of potential groundwater drawdown are not critical (i.e. “no effect”) for construction shaft management, construction may employ techniques such as:

- Ring beams with shotcrete or timber lagging
- Soldier piles and lagging
- Bored piles with shotcrete infill
- Rock bolts and wire mesh lagging
- Soil nails and shotcrete
- Sheet piles and ring beams.

2.3.3 Link Sewer 4

Link Sewer 4 will be constructed by shallow (1 – 3 m depth) open excavations above groundwater level in basalt. As such there is no potential for groundwater drawdown, or associated surface settlement. Link sewer 4 will not be considered further in this report.

2.4 Tunnel operation

In typical flow situations (tunnel operated flowing partially full), internal pressure will be at or near to local atmospheric pressure, significantly lower than external groundwater pressure, with a tendency for groundwater seepage into the tunnel along its entire length (dependent on the presence and permeability of a tunnel liner).

In an extreme case (10 year storm combined with main pump station failure), estimates are that the tunnel and associated shafts fill to RL 6.85 m at Western Springs, and to 6.07 m at Keith Hay Park (details from Central Interceptor Project Design Team - 2011). A maximum internal tunnel pressure of approximately 30m head results from this event.

External groundwater pressure is expected to exceed this internal pressure in the tunnel from the upstream extent at Western Springs to the Manukau Harbour (resulting in no seepage out of the tunnel during this event). Beneath the Harbour and through Kiwi Esplanade to the Pump Station at WS3, internal pressure can be expected to exceed external groundwater pressure by up to 3 to 5 m in the extreme design event.

2.5 Precedent projects in Auckland

2.5.1 General

Tunnels of similar configuration to that proposed for the Central Interceptor project have been completed successfully in Auckland in recent history. The projects provide a relatively complete picture of the range of potential groundwater and surface settlement effects for a range of possible construction methodologies for the tunnels, (refer Table 2-2). It should be noted that these measured settlements will include a contribution from mechanical as well as groundwater induced settlements. It is expected, however, the contribution from mechanical settlements would be small given excavation was in ECBF rock in each case.

Importantly, the projects traverse much of the range of geological and groundwater environments that are expected during construction of the tunnels. The projects provide a direct correlation of tunnel construction effects where the tunnels are constructed within the Auckland Isthmus.

The surface settlement effects achievable in construction of Central Interceptor tunnels in the Auckland Isthmus are likely to be of similar magnitude to those from these projects, given the similarities in:

- Tunnel size
- Potential construction methodologies
- Geological conditions.

Watercare's intended construction methodology for the tunnel most closely matches those used for Hobson sewer and the Rosedale Outfall.

Table 2-2 – Historic tunnel project surface settlement summary

Project	Tunnel diameter	Mean measured settlement	Maximum measured settlement
Vector Tunnel	3.5m	7 mm	38 mm
Hobson Sewer	3.5 m	<10 mm	30 mm
Rosedale Outfall	2.8 m	4 mm	44 mm

Details of these projects are summarised in Appendix C.

3 Description of geological conditions

The geology interpreted along the tunnel alignments are described in this section. The ground conditions encountered along the tunnel alignment will govern the response of the ground and groundwater to tunnel excavation and operation.

Published data, historic investigations, project specific investigations combined with experience of Auckland Geology have been used to estimate geological conditions along the tunnel alignment.

3.1 Regional geology

The Auckland region is characterised by four major stratigraphic groups:

- Mesozoic "greywacke" basement (not generally of engineering significance in Auckland)
- Miocene Waitemata Group and Waitakere Group marine sedimentary and volcanic rocks
- Late Pliocene to Holocene Tauranga Group alluvial and estuarine sediments, and
- Late Pleistocene basaltic deposits of the Auckland Volcanic Field (AVF).

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 various deposits of the AVF occur over a wide area, but are largely limited to basalt flows or a mantling of tuff and ash. Deep occurrences of igneous material are expected to be limited to the root feeder systems (i.e. diatreme) of the volcanic cones and explosion craters (maar).

Structurally the Auckland region consists of a broadly rectangular patchwork of up-thrown (horst) and down-thrown (graben) blocks, bounded by steep faults. These occur on a number of scales. The Manukau Harbour and adjacent Manukau Lowlands are part of a regional-scale graben down-thrown relative to the Auckland Isthmus. These downthrown areas have subsequently been in filled by successive deposition of alluvial and air fall materials. The dip and dip direction of bedding within the Waitemata Group has been observed to vary significantly over relatively short distances as a result of both folding and faulting.

3.1.1 Project specific investigations

To investigate the distribution of regional geologic units along the tunnel alignment, sub surface investigations were undertaken by the Central Interceptor Project Team from September 2009 until June of 2011 (details from Central Interceptor Project Design Team - 2011). The thirty four boreholes undertaken during this period have, in combination with historic investigation data for the area, been utilised to develop a geology map of the tunnel route, a geological long section of the tunnel alignment, and geological cross sections at key locations along the alignment. The sources of historic information are discussed in detail below. A discussion of the geology specific to the tunnel alignment then follows.

3.1.2 Historic investigations information on surrounding area

Several sources of information have been utilised from other projects in the vicinity of the CSO alignments. Several attributes affected the value of the historic borehole data including:

- Depth of investigations
- Quality/detail of logging
- Whether or not piezometer data is available (piezometers record groundwater level information).

The major sources of data are discussed below:

SH16/SH20 Waterview Connection

The Waterview Connection project is to the west of the main tunnel alignment, from Mt Roskill to Waterview. The project boreholes are relatively close to the tunnel alignment in the Mt Roskill to Mt Albert section, but diverging away further north. The project boreholes include some relatively deep boreholes (up to approximately 40-50 m depth in the area close to the main tunnel) although they are generally not as deep as the tunnel. There is extensive groundwater information and permeability test data available from this project.

SH20 Motorway extension

The SH20 Motorway extension project is approximately parallel with the main tunnel alignment from Mt Roskill through to the Manukau Harbour. The project boreholes are close to the alignment near Mt Roskill, getting further away to the south. The relevant project boreholes (generally up to approximately 30-40 m depth) are not as deep as the main tunnel alignment. There is extensive groundwater information available from this project.

Project Manukau (MWWTP upgrade)

The Project Manukau site is at the southern end of the main tunnel alignment (i.e. at the waste-water treatment plant). The boreholes are close to the main tunnel alignment and particularly to the major shaft WS3. Some of the boreholes are as deep as the tunnel alignment in this area. The records available indicate a number of piezometers were installed for the project; however monitoring records for these piezometers have not been made available to date.

3.2 Alignment geology

3.2.1 General

The project specific investigations, historic investigation data and published geological information allow the geology along the tunnel alignment to be estimated.

The geology of the main tunnel alignment traverses three distinct zones.

- A Northern Zone (Western Springs to Mt Roskill) with ECBF at tunnel level and surface geology dominated by AVF basaltic flows, together with a variable cover of tuff. Depending upon the pre eruptive topography, the AVF deposits either directly overlie the Waitemata Group rocks or Tauranga Group alluvium. Link sewers 1 and 2 are within this zone.
- A Central Zone (Mt Roskill to Hillsborough) with ECBF at tunnel level and outcropping ECBF rocks and minor Tauranga Group cover at the surface. Link sewer 3 is within this zone.
- A Southern Zone (Manukau Harbour and Mangere) with ECBF as well as Kaawa and Puketoka Formation deposits at tunnel level, and surface geology dominated by AVF eruptive centres. Link sewer 4 is also within this zone (it will be constructed by open trenching).

Figure A5 presents a geological section through the isthmus along the alignment of the main tunnel; Figure A7 presents geological sections for Link sewer 1and 2 and Figure A8 for Link sewer 3 in this zone. Figure A6 presents a geological section through the Manukau Harbour – Mangere Bridge area. Figure A9 presents the geological section for Link 4 in this area.

3.2.2 Geological units

The following sections describe in more detail the geological units identified along and above the tunnel alignment (refer to the geological sections identified in Section 3.2.1). Several geological units are present within the vicinity of the project route. For the purposes of this study, eight units are of significance along the route, and form the basis for the geological and hydro-geological model used in this assessment.

3.2.2.1 Auckland Volcanic Field (AVF) Basalt

The AVF basalt is Pleistocene to Holocene in age and consists of hard rock lava flows. Basalt flows were typically along tributaries to the ancestral Waitemata and Manukau Rivers. The basalt is typically well jointed with a relatively high permeability rock mass. Basalt from Mt Albert and Mt Roskill flows is found in the Northern Zone, with Mt Mangere basalt in the Southern Zone. The basalt overlies extensive reaches of the tunnel alignment, up to 30 m in thickness. Basalt has a very low risk of consolidation and settlement due to groundwater drawdown.

3.2.2.2 AVF Tuff

Tuff comprises clayey to sandy silts with some gravels through to silty gravels. It is present in the northern and southern zones but is much less extensive than the basalt. Tuff has been identified along the tunnel route in significant thicknesses around Mt Roskill and at Mangere, most notably forming the tuff ring around the Mangere Lagoon. Thicknesses in the order of 5 m have been identified. Tuff has some risk of consolidation and settlement due to groundwater drawdown, but less than alluvial or estuarine deposits.

3.2.2.3 Estuarine sediments

Estuarine sediments of Pleistocene to Holocene age are found in and around the Manukau Harbour and consist typically of silts and sands with variable shell, gravel and organic content. Estuarine sediments overlie the tunnel alignment as it passes under the Manukau Harbour. Estuarine, together with alluvial deposits are considered to have the highest risk of consolidation and settlement due to groundwater drawdown.

3.2.2.4 Undifferentiated Tauranga Group Alluvium (TGA) and Upper (fine grained) Puketoka Formation (UPF)

For the purposes of this study the TGA deposits have been grouped with the UPF deposits.

The recent TGA deposits are late Pleistocene to Holocene in age, having been deposited within low lying drainage channels and topography. Locally significant thicknesses of alluvium are present, up to approximately 10 m in thickness. On the Auckland Isthmus the alluvium is typically derived directly from the weathering and erosion of ECBF. Within the Manukau Lowlands much of the material is from non-ECBF sources. The alluvium typically consists of silts or clays with variable sand content.

The Puketoka Formation sediments are Late Pliocene to early Pleistocene in age and are generally alluvial to shallow marine in origin. They occur extensively throughout the low-lying areas adjacent to the Waitemata and Manukau Harbours, although also underlie terraces and extensive flat to undulating surfaces at elevation of approximately 10 mRL to 40 mRL. They typically predate and underlie the AVF (basalt, tuff) deposits, and are up to 20 m in thickness. They include a wide variety of material types ranging from clays to gravels, though the upper Puketoka Formation is generally silts and clays with variable sand content.

The TGA and UPF deposits, together with estuarine sediments are considered to have the highest risk of consolidation and settlement due to groundwater drawdown.

3.2.2.5 Lower (coarse grained) Puketoka Formation (LPF)

Lower, coarse grained (predominantly sand) deposits of Puketoka formation have been identified in the Southern Zone at Mangere and have been considered as a separate unit for the purpose of this study. The coarser grained deposits are present in significant thicknesses, up to approximately 20 m. The LPF deposits have some risk of consolidation and settlement due to groundwater drawdown, but less than the UPF, TGA or estuarine deposits.

3.2.2.6 Kaawa Formation

The Kaawa Formation deposits are Pliocene in age and consist of poorly cemented sandstone and sands. They underlie the Tauranga Group and AVF deposits, and are only found at the southern end of the Southern Zone in the vicinity of the Mangere Treatment Plant, in thicknesses of up to 15 m. The Kaawa Formation is considered to have a low risk of consolidation and settlement due to groundwater drawdown.

3.2.2.7 Residually to Highly Weathered East Coast Bays Formation (ECBF)

ECBF rock, formed from Miocene age sediments laid down in the ancestral Waitemata Basin, typically siltstones and sandstones underlie the entire route, with the tunnel expected to be excavated in the ECBF for all but the most southern reaches of the route. The upper surface of the ECBF has a variable weathering profile, with typically less than 5 m of residually to highly weathered ECBF along the route. This material is typically a firm to stiff silt or clay with a variable sand content. The residual ECBF soils have some risk of consolidation and settlement due to groundwater drawdown, but less than the UPF, TGA or estuarine deposits.

3.2.2.8 East Coast Bays Formation Rock (ECBF)

The ECBF rock is typically extremely weak to weak interbedded siltstones and sandstones. It is generally volcanic-poor however it includes mixed volcanic-rich beds as well. Volcanic-poor facies tend to occur in the northern and eastern parts of Auckland, whereas volcanic-rich facies tend to occur in the western and southern parts of the project area. Inter-bedded volcanic-rich and volcanic poor beds occur in the Hillsborough area. ECBF rock is considered to have a low risk of consolidation and settlement due to groundwater drawdown. The SH16/SH20 Waterview Connection investigations (nearby the CI alignment) have encountered zones of Parnell Grit (PG), a subset of ECBF being stronger and having occasionally exhibiting a much higher secondary permeability. Central Interceptor investigations have not encountered Parnell Grit other than at Mangere WWTP). The material encountered was a volcanic rich fine to coarse grained Sandstone, characteristic of the Cornwallis formation with permeability slightly higher than surrounding ECBF material. It is possible that tunnel construction could encounter such high permeability PG material. From a seepage perspective, the PG could have permeability characteristics similar to those adopted for highly fractured ECBF (Discussed in the next section).

3.2.3 ECBF - volcanic disturbance

ECBF adjacent to explosion craters within the Auckland Volcanic Field has been found to be more significantly fractured than material distant from the craters. Specific experience of this relates to investigations along Kepa Road adjacent to the Orakei explosion crater rim. At Three Kings Quarry, groundwater monitoring of dewatering occurring in the Three Kings Volcanic complex indicates disturbance or more intense fracturing extending as much as 800 m to 1,000 m from the crater.

The main tunnel alignment passes close to three eruption features within the Auckland Volcanic Field: the Mangere lagoon, Mt Albert and Mt Roskill. On the basis of the above experience it might be expected that the ECBF country rock within 1,000 m of these features could be more significantly fractured than material distant from eruptive centres. However, route investigations to date close to these features do not support the observations from elsewhere in Auckland. The exception being investigations close to Mangere lagoon where permeability testing indicates a zone of higher permeability ECBF (although visually the core is no different to that from other sites about Auckland).

The potential for more highly fractured, and potentially higher permeability ECBF zones cannot be discounted by the investigations. Sensitivity studies are included in this study of tunnel effects to assess the potential influence of such conditions at key locations.

3.2.4 Rock and soil stiffness parameters

The stiffness of the various rock and soil units is of importance in assessing the settlement effects due to groundwater changes. In order to assign a range of parameters for use in the assessment, both material testing from the Central Interceptor boreholes, as well as experience from previous projects has been consulted. Adopted values for the project analyses are summarised in Table 3-1 below. Table 3-2 compares adopted values with other Auckland projects. Refer to Appendix B for a discussion of laboratory and field test data available for each of the geological units.

Table 3-1 – Summary of estimated material stiffness parameters

Geological unit	Deformation Modulus, "E" (MPa)		
	Assessed minimum	Assessed mean	Assessed maximum
Basalt	Considered practically incompressible in terms of this study		
Tuff	8	12	20
Estuarine sediments	1	2	10
TGA / UPF above 12m depth	2.5	6	20
UPF below 12m depth	10	15	40
Lower Puketoka Formation	10	20	50
Kaawa Sands	50	100	150
Weathered ECBF	4	15	40
ECBF	150	500	1000

Table 3-2 – Comparison of estimated material stiffness parameters with other Auckland projects

Geological unit	Constrained or Deformation Modulus, "M" or "E" (MPa)					
	Central Interceptor (assessed mean)	Waterview Connection[1]	Rosedale tunnel[2]	Hobson Bay Tunnel[3]	Vector Tunnel[3]	Britomart[3]
Tuff	12	N/A	N/A	30[4]	N/A	N/A
Estuarine Sediments	2	N/A	N/A	1	N/A	N/A
Tauranga Group Alluvium / Upper Puketoka Formation	6 above 12m depth, 15 greater than 12m depth	3.3[4] to 9[4] (stress range dependent)	3 (2 – 5) Apollo Drive 1.3 (1.2 – 1.5) Ramsgate Terrace	8-10	N/A	N/A
Lower Puketoka Formation	20	N/A	N/A	N/A	N/A	N/A
Kaawa Sands	100	N/A	N/A	N/A	N/A	N/A
Weathered ECBF	15	3.3[4] to 10[4] (stress range dependent)	10 (4-16)	11[4]	N/A	N/A
ECBF	500	150-1050	N/A	450[4]	560[4]	670[4]

1 - Beca (2010); 2 - Maunsell Limited (2004); 3 - T&T (2004); 4 – Constrained Modulus

4 Description of hydrogeological conditions

The groundwater regime interpreted along the tunnel alignment is described in this section.

The response of the groundwater to tunnelling activities, both construction and operation, will govern the degree to which surface settlement may develop. The engineering properties of each geological unit are discussed from a consideration of engineering test data undertaken for this project, and from projects in similar geological environments in Auckland. The Engineering properties are an essential input in developing models to represent the existing conditions, and for assessing future behaviour.

4.1 Regional groundwater

The Auckland Isthmus is characterised by perched transient groundwater levels within near surface deposits and a deeper more stable groundwater level within the ECBF. Data from piezometers along the route indicate that conditions are broadly hydrostatic in most areas. The ECBF groundwater level is typically a subdued reflection of surface topography, within gradients in 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 water table. In gullies seepage from ECBF rock and basalt aquifers supports stream base flow, or where historic gullies have been in-filled by more recent alluvial or volcanic deposits, groundwater concentrates in directional seepage along the paleo-valleys. Basalt deposits form surface aquifers within ancient gully systems and are typically permanently saturated only in the lower zones near the coast.

From Western Springs through to Mt Albert Road paleo-valleys, in-filled with alluvial and volcanic materials, are recharged from the underlying ECBF with broadly hydrostatic groundwater levels between units having been measured (refer Figure 3 and Figure A5). The tunnel level is relatively shallow below the known paleo-valley bases. Weathered ECBF material is likely to be thin or locally absent in the paleo-valley, giving rise to direct contact between ECBF rock and the alluvium. Therefore the basal alluvial material will be impacted by groundwater level reductions in the ECBF should they occur as a result of tunnelling in these areas.

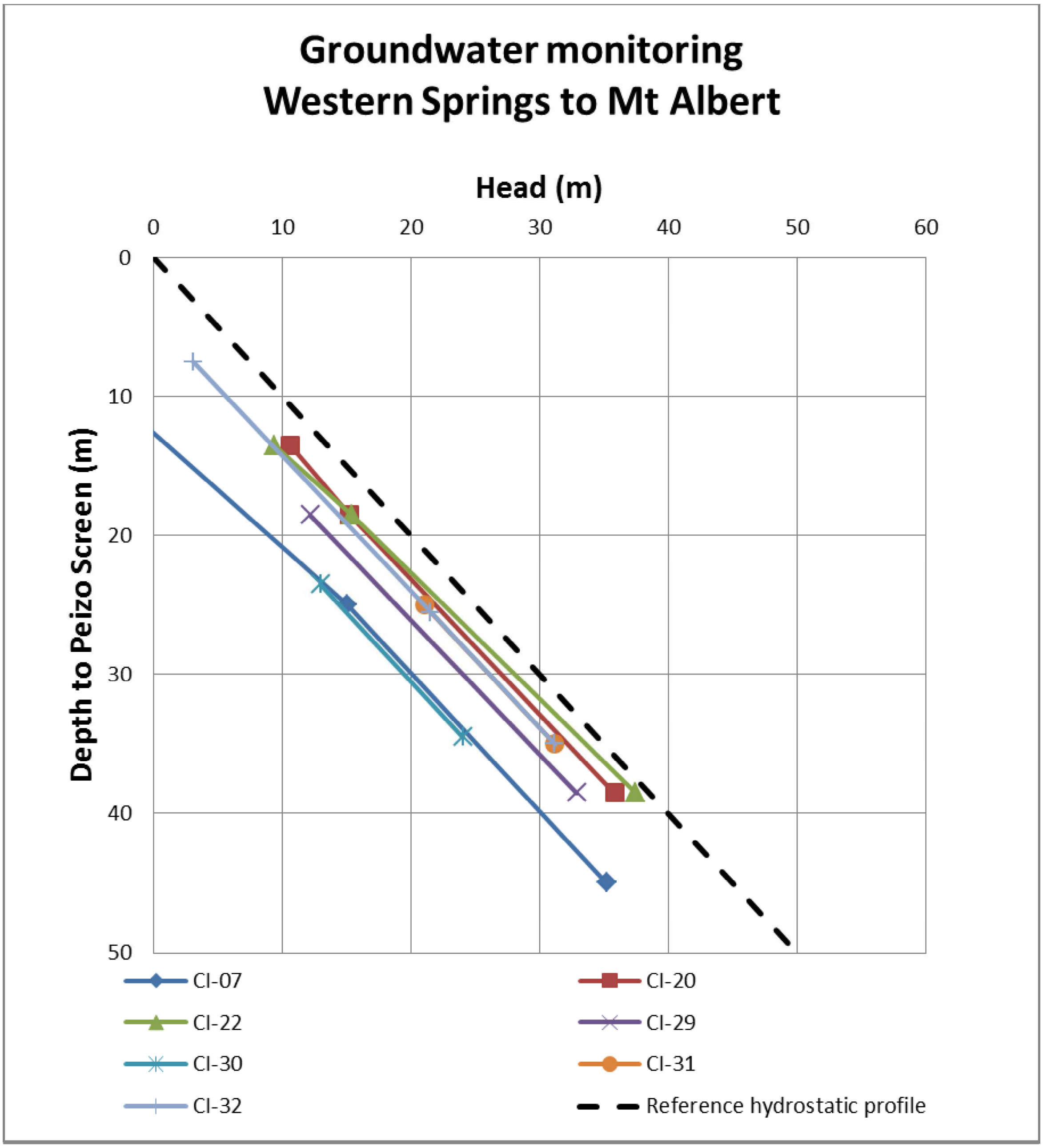

Figure 2 – Groundwater levels from CI borehole piezometers, Western Springs to Mt Albert Road

From Mt Albert Road to Mt Roskill Road, similar hydro-geological conditions are present (refer Figure 4), although the separation between tunnel level and known paleo-valley base is significantly larger (by a factor of about three).

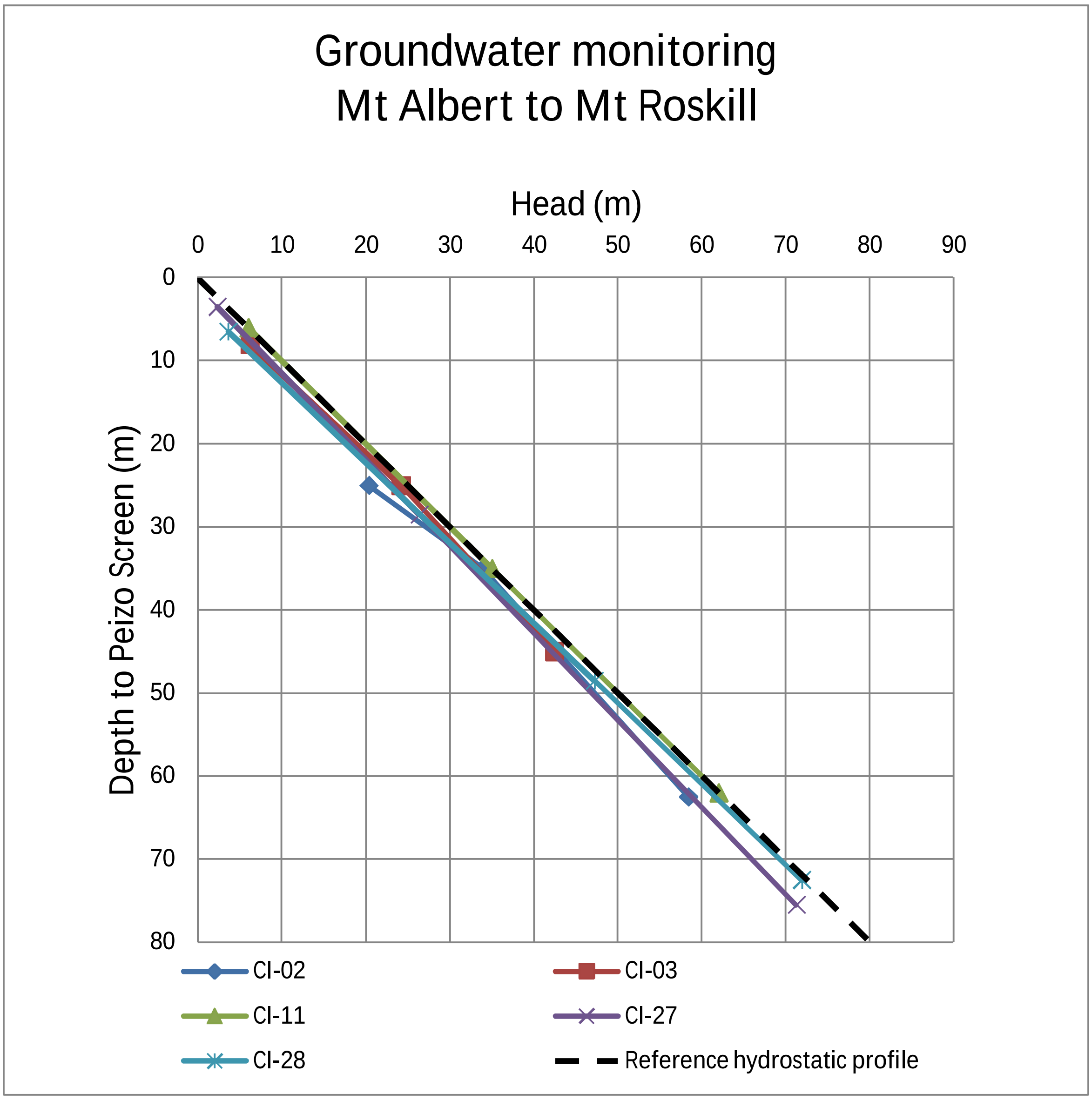

Figure 3 – Groundwater levels from CI borehole piezometers, Mt Albert to Mt Roskill

In the vicinity of Mangere Pump Station, the Manukau Lowlands are characterised by complex inter-bedded sequences of volcanic and alluvial deposits, much of which are below sea level. The Kaawa Formation present in this area forms a locally significant aquifer. The interconnectivity of groundwater in these sequences is expected to be complex reflecting deposition of disparate materials. Monitoring shows that groundwater level in all of the units broadly reflects nearby sea level (i.e. close to RL 0m) (refer to figure 5). Groundwater pressure changes associated with tunnel excavation are likely to propagate rapidly between the connected units. The potential presence of major faults within the ECBF in this area (associated with the change in geological conditions across the Manukau Harbour) raises the possibility of compartmentalised groundwater systems however, although monitoring to date does not identify any significant head differential across the potential fault zone. The presence of these faults is however speculative.

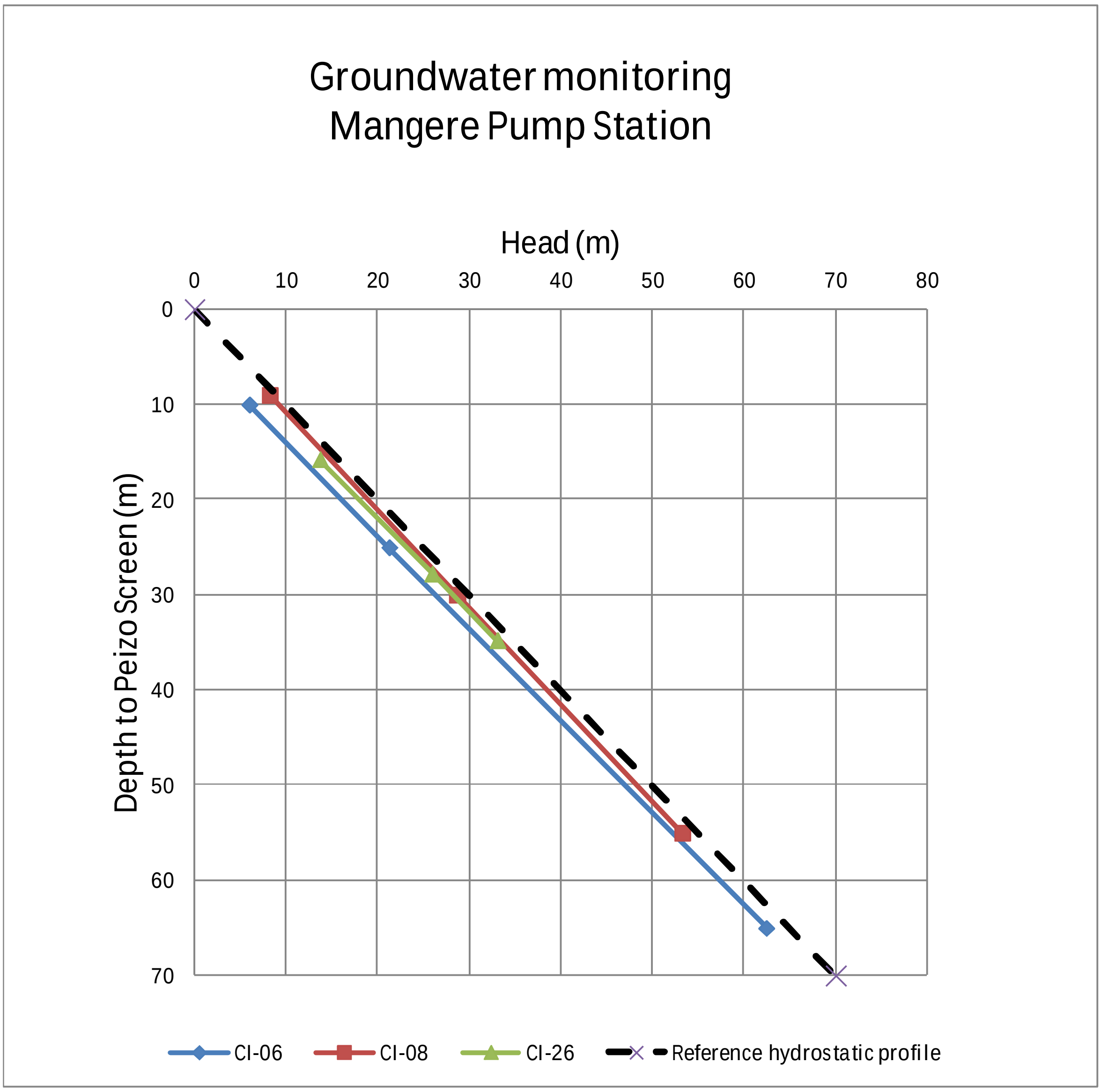

Figure 4 – Groundwater Levels from CI borehole piezometers, Mangere Pump Station, (MWWTP)

4.2 Surface water

Within the Northern Zone, Western Springs Lake is a key surface water feature. The lake is manmade, (constructed in the late 1800's (Russell 1977)) and includes a lining to impound water that naturally flows from basalt aquifers (primarily the Greater Western Springs Aquifer (PDP 2005)). Groundwater levels in the vicinity of the lake identify a downward hydraulic gradient exists between lake and underlying ECBF materials. This indicates that lake is currently under-drained by the ECBF (which is consistent with the need for a liner to impound the lake), and hence the lake would not be expected to be affected by groundwater level changes in the ECBF associated with tunnelling. Russell, 1976 suggests that the spring feeding the lake may be derived primarily from groundwater flow in an upper more permeable basalt flow (Three Kings basalt), separated from the ECBF by lower less permeable flows (Mt Albert basalt).

4.3 Groundwater extraction

A number of consented groundwater takes have been identified from Auckland Council records at various distances from the proposed tunnel and shafts.

In the Auckland isthmus, groundwater extraction wells close to the alignment (within approximately 1 km) have been identified from records as summarised in the table below and on Figure A10 in Appendix A.

Table 4-1 – Consented groundwater takes near alignment

Item	Consent holder	Description	Approx distance from alignment
1	Auckland Council	Groundwater for irrigation of golf course greens - taken from volcanic (surface) aquifer	160m
2	Kings Plant Barn	Groundwater for irrigation of 0.39 hectare of garden centre container plants - taken from volcanic (surface) aquifer	140m
3	Akarana Golf Club	Groundwater for irrigation of golf course greens, tees and fairways - taken from volcanic (surface) aquifer	280m
4	Auckland Council	Groundwater for washing down enclosures, amenities, irrigation and general use in a zoological park - taken from volcanic (surface) aquifer	90m
5	Watercare	Groundwater for dust suppression, odour control, truck washing and construction for works - from Kaawa Aquifer	1100m
6	Watercare	Groundwater for odour and dust control- from Kaawa Aquifer	1100m
7	Watercare	Groundwater to provide process water for sludge dewatering- from Kaawa Aquifer	1100m

4.4 Hydrogeological units

The same as the geological units identified in Section 3.2.2 are adopted as the characteristic hydrogeological units for this study.

4.4.1 Hydro-geological parameters

Numerous in-situ permeability tests have been undertaken as part of the CI investigations. These tests consisted generally of variable head permeability tests undertaken during drilling (mainly rising head) in soils and packer tests in rock. Adopted permeability values for the project are summarised in Table 4-2 below. Table 4-3 compares adopted values with other Auckland projects. Refer to Appendix B for a discussion on test results and derivation of parameters for each of the geological units.

Table 4-2 – Summary of estimated material permeability parameters

Geological unit	Permeability (m/s) assumed isotropic where not otherwise noted		
	Assessed minimum	Assessed mean	Assessed maximum
Basalt	$1x10^{-6}$	$1x10^{-4}$	$1x10^{-3}$
Tuff	$1x10^{-7}$	$1x10^{-5}$	$1x10^{-3}$
Estuarine Sediments	$1x10^{-9}$	$2x10^{-7}$	$1x10^{-6}$
Tauranga Group Alluvium /Upper Puketoka Formation	$2x10^{-8}$	$2x10^{-7}$	$2x10^{-6}$
Lower Puketoka Formation	$2x10^{-7}$	$2x10^{-6}$	$2x10^{-5}$
Kaawa Sands	$1x10^{-7}$	$1x10^{-6}$	$1x10^{-4}$
Weathered ECBF	$2x10^{-8}$	$2x10^{-7}$	$2x10^{-6}$
ECBF	K_h =$2x10^{-8}$ K_v/K_h=0.1	K_h =$2x10^{-7}$ K_v/K_h=0.1	K_h =$2x10^{-6}$ K_v/K_h=0.1
Fractured ECBF	NA	$5x10^{-4}$	NA

Table 4-3 – Comparison of estimated material permeability parameters (m/s) with other Auckland projects

Geological unit	CI (assessed mean)	Waterview Connection[1]	Vic Park Tunnel[1]	New Lynn Rail[1]	Rosedale tunnel[2]	Hobson Bay Tunnel[3]	Vector Tunnel[4]	Three Kings Quarry[1]
Basalt	$5x10^{-5}$	$K_h = 1.2x10^{-5}$ to $5.0x10^{-5}$ $K_v = 5.0x10^{-5}$	N/A	N/A	N/A	N/A	$2.4x10^{-4}$ to $1.4x10^{-8}$	$K_h = 2.0x10^{-4}$ $K_v = 2.0x10^{-4}$
Tuff	$1x10^{-5}$	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Estuarine Sediments	$2x10^{-7}$	N/A	N/A	N/A	N/A	$1x10^{-9}$	N/A	N/A
Tauranga Group Alluvium / Upper Puketoka Formation	$2x10^{-7}$	$1 x10^{-7}$ to $2.3x10^{-7}$	$K_h = 2.0x10^{-7}$ $K_v = 2.0x10^{-8}$	$K_h = 3.0x10^{-7}$ $K_v = 5.0x10^{-8}$	$2x10^{-7}$ to $4x10^{-9}$	N/A	N/A	N/A
Lower Puketoka Formation	$2x10^{-6}$	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Kaawa Sands	$1x10^{-6}$	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Weathered ECBF	$2x10^{-7}$	$K_h = 2.0x10^{-7}$ $K_v = 2.0x10^{-8}$	$K_h = 2.0x10^{-7}$ $K_v = 2.0x10^{-8}$	$K_h = 3.0x10^{-7}$ $K_v = 5.0x10^{-8}$	N/A	N/A	N/A	N/A
ECBF	$K_h = 2x10^{-7}$ $K_v/K_h = 0.1$	$K_h = 3.5x10^{-7}$ to $5.7x10^{-7}$ $K_v = 5.7x10^{-8}$	$K_h = 1x10^{-7}$ to $5x10^{-7}$ $K_v = 1x10^{-8}$	$K_h = 1x10^{-7}$ $K_v = 1x10^{-8}$	$K_h = 5x10^{-8}$ $K_v / K_h = 0.075$	$K_h = 5x10^{-8}$ (mean), $5x10^{-7}$ (max), $K_v / K_h = 0.075$	$K_h = 1.7x10^{-7}$ to $5.1x10^{-8}$	$K_h = 1.5x10^{-8}$ $K_h = 1.5x10^{-9}$
Fractured ECBF	N/A	$K_h = 2.0x10^{-5}$ $K_v = 6.0x10^{-6}$	N/A	N/A	$5x10^{-4}$	$5x10^{-4}$	N/A	N/A

PART B

5 Potential effects of tunnels and shafts

This section presents a discussion on the potential effects of tunnel construction, and of long term tunnel operation, on the groundwater regime in the area. Surface settlement may arise from groundwater effects associated with both phases of the project. Consideration of potential construction effects provides guidance for refinement of construction methodologies, while long term effects provide guidance on tunnel lining design requirements. The potential effects discussed here provide a basis for the type of analysis and the analysis cases undertaken as part of the groundwater modelling.

Experience from recent projects in Auckland is further discussed, (previously introduced in Section 2), with specific reference to similar geological environments within the project area.

5.1 General

The potential for surface settlement as a result of tunnel construction and operation arises due to:

- Potential changes in the groundwater regime about the tunnel as a result of construction and long term during operation, and
- Through ground loss during excavation.

5.2 Groundwater response to tunnels

If the tunnel is constructed without causing any significant changes in groundwater regime then two long term scenarios are possible:

- The tunnel lining is sufficiently impermeable to limit seepage into the tunnel, resulting in no further significant groundwater effects or surface settlement
- The tunnel lining allows seepage to flow into the tunnel, resulting in long term groundwater effects and surface settlement.

Conversely, if the groundwater is drawn down as a result of construction activities:

- The tunnel lining is sufficiently impermeable to limit seepage into the tunnel, resulting in recovery of the groundwater, and no further significant surface settlement
- The tunnel lining allows seepage to flow into the tunnel, resulting in further long term groundwater effects and surface settlement.

By considering both potential construction and long term effects, this report provides guidance for refinement of construction methodologies and for considerations during the design of the tunnel lining.

The Central Interceptor tunnels will likely be constructed using an EPB TBM with capability to apply a regulated pressure to the excavation face (Watercare have advised this is their intended construction methodology). Face pressure is typically applied to stabilise the excavation face in soft ground, or in cohesionless ground that has the potential to flow due to presence of groundwater. Face pressure can also be applied to balance or partially balance groundwater pressure to prevent or reduce groundwater flows into the excavated face.

When operated without face pressure (in “open mode”) tunnel excavation reduces groundwater pressures in the material immediately surrounding the excavation to atmospheric pressure. This reduction in groundwater pressure leads to groundwater flow towards the excavation and an associated redistribution of groundwater pressures in the surrounding rock-mass. Initially this effect is limited to the rock-mass close to the excavation face. With time this effect will radiate out from the excavation until an equilibrium steady state is achieved. The volume of flow to the tunnel and the extent and rate of drawdown of groundwater level in the rock is a function of hydrogeological conditions, the rock mass properties, and the availability of groundwater to recharge the depressurised rock mass. Where compressible material overlies the rock mass and are affected by depressurisation, there is an increase in effective stress in the material which results in consolidation, and ultimately surface settlement may result.

Construction methodology, and construction activities subsequent to initial excavation control the extent of drawdown and hence the potential for surface settlement. Construction of a tunnel lining (particularly a lining with permeability significantly lower than the material the tunnel is excavated in) will significantly reduce the ability of groundwater to flow into the tunnel void. As the tunnel face advances, groundwater pressures around the lined section of the tunnel will begin to recover and the depressurisation process will reverse. A consideration for tunnel construction is what an acceptable delay between excavation and installation of a liner is, particularly in areas where geology is susceptible to settlement associated with depressurisation. Analyses carried out for this study provide an assessment of this delay (described in Section 6).

When the TBM is operated with face pressure (in 'closed mode"), tunnel excavation reduces groundwater pressure at the face to the pressure which the operator has chosen to apply. This pressure may balance groundwater pressure, in which case there is no re-distribution in groundwater pressure, and no flow to the face. Surface settlement as a result of depressurisation does not result in this instance. Alternatively it may partially balance groundwater pressure, in which case there is a redistribution of pressure and a groundwater flow towards the face similar to that for operation in open mode, but with a reduced magnitude. The reduction in magnitude of redistribution also reduces the degree to which overlying materials may be depressurised, ultimately reducing potential for surface settlement.

A clear understanding of where along the alignment it is important to balance groundwater pressures, to ensure surface settlements are controlled, is key to effective operation of the TBM and tunnel construction. Analyses carried out for this study provide an assessment of the areas where it might be necessary to use closed mode to balance groundwater pressures (described in Section 6).

During operation, long term groundwater changes are possible quite separate from the construction related changes. Groundwater changes associated with steady seepage into the completed tunnel may also impact the groundwater regime and result in surface settlement. Such effects are assessed as part of the analyses carried out (described in Section 6).

In addition to analysis, experience of tunnelling projects in similar geological environments in Auckland provides an indication of likely groundwater and settlement response. A number of projects are considered to be of relevance including Vector Tunnel, Hobson Bay Sewer Tunnel and Rosedale Tunnel. Each of these projects was undertaken in geological environments which will be encountered along the alignment, and monitoring data is available. A brief summary of the key monitoring results from each project is provided in Appendix C.

In addition to tunnelling projects, open excavations in similar geology where groundwater is drawn down are of relevance, as they provide an understanding of surface settlement response due to groundwater drawdown. The Three Kings Quarry in Auckland has monitoring data of relevance to the project and is also summarised in Appendix C.

5.3 Ground loss

Watercare Ltd's Principal Engineering advisor (AECOM) assessed the potential for ground loss settlement as a result of tunnel excavation. For excavation in ECBF rock the assessment of potential surface settlement from ground loss was from 1 – 6 mm.

For excavation in Tauranga Group materials (Manukau Lowlands near the WWTP) the assessment of ground loss settlement was 10 mm based on excavation utilising an EPB TBM and controlling loss to 1%. At 2%, estimated settlement increased to 20 mm.

5.4 Groundwater response to shafts

The Central Interceptor shafts are of varying size and depth, and pass through variable ground conditions. As a result no single construction methodology is proposed for shaft construction, but a number of potential techniques are available as outlined in Section 2.3.2. These construction techniques have varying degrees of water tightness during construction, and hence effects on ground water drawdown.

The construction techniques most effective at controlling groundwater drawdown are secant piling and diaphragm walling. In both techniques the elements (piles or diaphragm wall panels) are installed, from ground level prior to excavation, in an interlocking pattern to form a continuous low permeability barrier. The walls would typically extend to beyond the base of the excavated shaft level in order to provide sufficient cut-off to groundwater flow. Diaphragm walls can generally be extended only to the top of rock level, and will not penetrate rock. Secant piles have the advantage of being able to penetrate rock, depending on the piling equipment and process used. It is also possible to utilise these techniques for partial depth construction. This may be useful if a high permeability layer exists in the upper part of the ground profile that needs to be cut off, but cut off is not required beyond that point.

If these techniques are used for full cut off, groundwater is prevented from migrating directly into the excavation shaft sides, but may enter at the base of the excavation. Initially groundwater pressures will be reduced locally in the vicinity of the excavation base. With time groundwater flow paths will be set up that pass down the outside of the shaft, beneath the cut-off level of the piles/diaphragm walls, and then up to the shaft base. This can potentially cause a lowering of the groundwater table beyond the shaft.

The amount of water entering at the base of the excavation can be controlled by extending the wall toe depth beyond the shaft excavation level, or by ensuring that a sufficiently thick impermeable material has been founded into. If the required cut-off depth is impracticably deep, other techniques may be employed alongside these techniques to further reduce groundwater flow into the excavation base. These may include a grout curtain cut-off through the base of the secant piles/diaphragm walls, or "jet-grouting" at the base of the shaft to provide a low permeability layer. If these techniques are employed the groundwater response can be minimised.

Alternatively, open caissons shafts can be sunk with an open bottom and top during construction, and are usually made of reinforced concrete, or steel. This technique is similar to secant pile/diaphragm walls in so far as groundwater is excluded from the sides of the excavation as excavation proceeds. Because the cut off level of the caisson toe is advanced only as excavation proceeds, there is less of an effective cut-off and hence more potential for groundwater ingress into the base of the excavation than for secant piles diaphragm walls.

Alternatively, the caisson may be excavated "in the wet" (i.e. underwater), in which case there is no effect on groundwater during excavation. A concrete slab may be placed by tremie methods at the base of the shaft once the full excavation depth is reached prior to dewatering. In this way the groundwater drawdown is prevented, except through the caisson itself if it is not watertight.

Where shaft excavation is through basalt, none of the above techniques are likely to be feasible construction techniques. As a result, in basalt, grout curtains are likely to be employed to control groundwater inflows if required.

Other techniques, such as rock bolting, and soldier piles, allow groundwater to flow into the shaft as excavation and dewatering proceeds. Initially this causes drawdown close to the shaft itself, but, as time proceeds, drawdown will extend away from the shaft. This drawdown and depressurisation of adjacent soil may lead to surface settlement, but can potentially be addressed by controlled recharge.

In all cases, a permanent shaft liner may be installed after excavation to prevent groundwater inflow into the shaft in the long term. After this happens, groundwater may be expected to recover to pre-construction levels. The degree to which drawdown and settlement occurs in the intervening period is dependent on the permeability of the materials in which the shaft is excavated, and the time with which the shaft is left open prior to installation of a final lining.

As such, analyses carried out for this study assess these variables (Section 6). Analyses carried out for this study assess the areas where it might be necessary to use techniques to minimise the effects of shaft construction or operation on groundwater and what characteristics those techniques may require.

Shaft construction can introduce an impediment to existing groundwater flow patterns. To overcome this impediment, groundwater is likely to rise on the upstream side of the shaft to drive groundwater flow around it. This is not expected to be significant for the Central Interceptor Project as:

- Shaft diameters proposed are generally in the order of 10m and are small relative to the size of the groundwater flow fields. They are therefore unlikely to present a significant obstruction to groundwater flow, except in the immediate vicinity of the shaft. Pressure heads required to drive groundwater around the structures are expected to be low, such that potential effects such as surface flooding are very unlikely.
- While the major shaft WS3 at Mangere is proposed to be of significantly larger diameter (approximately 40 m in diameter) groundwater levels are broadly governed by sea level, and groundwater gradients are very flat at this site. The heads required to drive groundwater around this structure are not expected to be significant in terms of surface effects.
- Major shafts WS1 and WS2 are in the order of 30 m in diameter, with the main groundwater flow in the higher permeability basalt aquifers. No significant up-gradient or down-gradient groundwater effects are anticipated as a result of this.

5.5 Effects of surface settlement

Settlement of the ground surface is expected as a direct result of dewatering the ground about the tunnel. The magnitude and extent of effects is directly related to the groundwater effects induced and the geology affected by the changes. The nature of the settlement may range from:

- Imperceptible (i.e. settlement is within measurement error for survey methods or are masked by seasonal surface movements due to near surface soil moisture changes)
- Uniform over large areas (where the effects of groundwater changes are spread over a wide area within uniform geology)
- Locally variable (where significant changes in groundwater response occur over short distances, or where locally highly variable geology is affected by groundwater changes).

In all cases the potential for settlement to result in damage to structures depends primarily on the differential settlement, not on the total settlement.

For damage to occur to a structure it must be subject to differential settlement resulting in distortion of the structure. The greatest distortion hazard from dewatering induced settlement is at the centre of the trough and at point of maximum trough curvature. Elsewhere settlements are likely to result in tilting rather than distortion with lower potential for structural damage (although excessive tilting can result in serviceability loss).

Guidance on settlement tolerances for buildings in general is provided in the NZ building code. Appendix B B1/VM4, clause B1.0.2 says:

"Foundation design should limit the probable maximum differential settlement over a horizontal distance of 6m to no more than 25 mm 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 guidance limit of approximately 1:240 differential settlements.

Further guidance on the tolerance of specific building types and/or uses to differential settlement is provided by Bjerrum (1963) as summarised below.

Table 5-1 - Differential settlements and buildings

#	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

The need to protect against aesthetic damage to residential buildings, and to ensure functionality of machinery in an industrial situation is likely to require differential settlements to be limited to 1/500 – 1/750 rather than the more relaxed criteria in the building code.

The level of settlement that is generally accepted in New Zealand as being the upper consentable limit for developed land is a total settlement of 50 mm and a differential settlement of 1:1,000. As an example, the recently consented South-Western Interceptor micro-tunnelling project in Manurewa, Auckland, has an alarm level of 50 mm for total settlements and 1:1000 for differential settlements - AECOM (2011). It is noted, however that the recently consented Waterview Tunnels motorway project was consented with predicted settlements exceeding these limits.

At around 1:1,000 differential settlement, all building types will potentially incur some minor aesthetic damage. At steeper differentials (say 1:500), weatherboard houses with iron roofs would typically incur very little significant damage whereas brick and tile, stucco and concrete block structures would be starting to exhibit surficial damage.

Some buildings may have already been subject to some historical settlement as a result of unexpected ground performance, shrink swell behaviour of foundation soils or due to poor design, construction or maintenance. In fact some buildings may already have suffered minor damage as a result of these situations. In such cases the buildings will have reduced tolerance to further differential compared to a typical structure.

For any services passing above the new route, the actual service will need to be checked during design for its tolerance to the predicted settlement magnitude and shape. Such considerations may include:

- Reduction or reversal of gradient of gravity service pipes.
- Can the service material tolerate settlement and deflection? (E.g. non ductile, cast iron pipes).
- If it is a piped service, can the pipe joints tolerate the deflections that will occur?

6 Groundwater modelling methodology

This section describes how models have been developed to represent the range of geological conditions expected along the tunnel alignment, from which to assess the potential response of the groundwater regime. Models have been used to investigate the potential effects for a range of tunnel construction methodologies ranging from excavation without installation of lining to excavation followed by immediate installation of low permeability liner.

The combination of geological conditions and construction methods provides a basis for assessing the importance of particular methodologies in particular hydro-geological environments.

Sensitivity analyses investigate the effect of potential variability in tunnelling conditions on the estimated response such that contingency plans during design and construction can consider these potential effects.

6.1 Introduction

A number of approaches to the estimation of effects on the groundwater may be adopted. These range from high level conceptual models to complex two or three dimensional numerical modelling and correlation with observations from past experience.

In all cases, a number of assumptions are required to represent existing conditions and to estimate tunnelling and post-tunnelling conditions. The basis for each assumption needs to be carefully considered and it is often more useful to utilise simplified modelling tools tested for a range of conditions rather than complex models, the accuracy of which may not reflect the confidence level in the input parameters.

Numerical modelling of groundwater response has its place in estimating groundwater effects from tunnelling, provided the modelling assumptions are:

- Supported by site investigation data
- Calibrated with measured existing conditions
- Supported by back analyses of groundwater responses from other sites
- Able to be tested by sensitivity analyses.

The approach to modelling groundwater effects due to tunnelling for this study has been to adopt two dimensional seepage models using SEEP/W. The models are calibrated to measured groundwater profiles at selected sections considered of key importance for settlement modelling. That is, the boundary conditions have been set such that initial conditions broadly match measured groundwater levels. Calibration has not been used to verify hydraulic conductivity parameters, as modelled initial conditions tended not to be sensitive to material parameters. Generally this is a result of relatively low hydraulic gradients across the two dimensional models, and the fact that two dimensional analyses are only able to model flow in the plane of the model.

The two dimensional models are not able to represent:

- The effect of the tunnel liner installed some distance behind the current excavation face
- Out of plane groundwater flow towards the excavated face.

The expectation of this is that the two dimensional models are most likely to provide over estimates of:

- The rate at which groundwater responds to tunnel excavation, particularly in cases where a lining installation follows excavation
- In some cases this may also lead to an overestimate of groundwater inflows.

Groundwater effects due to shaft construction have also been modelled using SEEP/W, with axi-symmetric seepage models. Axi-symmetric models have the advantage of being able to simulate three dimensional problems with symmetry about a vertical axis of rotation - which is a reasonable model assumption for shafts. The limitation of an axi-symmetric model is that three dimensional geological variability and groundwater flow cannot be incorporated into the model. For this reason the axi-symmetric models have been kept relatively simple, with assumptions tested during model set up to enable an understanding of the effects of such assumptions.

As well as modelling each of the large shafts (WS1, WS2 and WS3), three other generic models were undertaken to assess the potential range of effects associated with ground conditions for the various smaller access shaft locations along the route.

Models have been investigated to identify the "unmitigated" settlement hazard that is present in each of the representative areas of geology as a result of groundwater drawdown. The unmitigated hazard is assessed as the settlement that is estimated for tunnel or shaft construction where no specific measures are employed to control groundwater flow during or following excavation (i.e. an unlined tunnel or leaky shaft model). Models have then been further investigated to assess the degree to which measures might be required during construction, and long term operation to ensure that settlement that might arise as a result of groundwater drawdown is within what is typically considered consentable (50 mm total settlement and differentials flatter than 1:1,000).

Sections of key importance were selected to represent the range of hydro-geological conditions that tunnel construction can be expected to encounter, particularly in relation to the potential for tunnel construction to result in surface settlement.

6.2 Selection of locations for groundwater and settlement modelling

The approach taken in this assessment is to select locations for modelling such that the range of potential geological environments that tunnel construction is expected to encounter are represented in the modelling. In this way the range of potential surface effects will be estimated.

By identifying environments where dewatering is likely, or very unlikely to result in significant surface settlement, construction methodologies can be targeted at providing a flexible response to ground conditions to optimise tunnel excavation and construction. The specification of an EPB TBM for tunnel excavation and construction provides the opportunity for the equipment to be operated in closed mode (utilising the EPB capability) or in open mode (not utilising EPB capability). Operations are more complex in EPB mode, so understanding where it is critical for this capability to be utilised to control surface effects, as well as those locations where it is not required, will aide construction planning.

Locations selected for groundwater modelling for both tunnels and shafts are shown on drawing CI-GWR-600 in Appendix A.

6.3 Groundwater flow analysis

6.3.1 Finite element mesh

A broadly rectangular mesh has been developed for each analysis, with vertical sides, a horizontal base, and top following the existing ground surface along the section (or a horizontal ground surface for the axi-symmetric models). The mesh extents for each model is at least 300 m from the excavation, giving a total mesh width of greater than 600 m for the 2-Dimensional tunnel models, and 300 m for the axi-symmetric shaft models. The model base is generally 50 m to 100 m below the base of the excavation in each model. Element densities have been increased close to the excavation and reduced at distance from the excavations where flux gradients are lower. Several thousand elements have been incorporated into each model. In all cases the tunnel excavation has been modelled as 5.5 m in diameter (actual finished internal diameter for the tunnel is expected to be 4.5 m which is expected to require excavation of between 5 and 5.5 m depending on specific lining requirements (details from Central Interceptor Project Design Team - 2011).

Figures showing the Model setup for the various analysis sections are attached in Appendix E.

6.3.2 Analysis cases

A number of analysis cases have been undertaken for each seepage model to represent potential short and long term drainage conditions within the shaft or tunnel excavation. Table 6-1 below summarises the main seepage analysis stages/cases undertaken for each seepage model. In all cases, the full excavation is 'wished in place' at the beginning of the analysis.

Table 6-1 - Summary of seepage analysis cases

Reference No.	Analysis stage/ case	Purpose
1	Steady state analysis of existing conditions	Calibrate model against observed piezometric data and use as initial conditions for other analyses, and use as a baseline for pore-water pressure changes for settlement analysis
2	Steady state analysis of excavated unlined tunnel/shaft	Analysis to assess the upper bound of potential groundwater drawdown to develop estimates of unmitigated settlement hazard in each representative geological environment.
3	Steady state analysis of excavated tunnel/shaft with liner of finite permeability	Analysis to assess effect of liner "leakiness" on long term groundwater drawdown to assess the degree of water tightness required by the tunnel liner for design
4	Transient analysis of excavated unlined tunnel/shaft out to 1 year	Analysis to assess effect of the length of time, which may be part of construction methodologies, between excavation and lining/sealing on development of surface effects - particularly in environments with high unmitigated settlement hazard.
5	Sensitivity analyses of material properties	Sensitivity analysis of cases 2-4 to assess material permeability assumptions on groundwater drawdown.

6.3.3 Boundary conditions

A number of boundary conditions types have been included in the various analysis cases. Constant head boundary conditions have been used on the lateral extents of each model (except for the “x=0” boundary in the axi-symmetric models).

Constant flux boundary conditions have been used on the surface of each of the models to represent groundwater recharge. “No-flow” boundary conditions have been used on the base of each model.

To model the excavations, potential seepage face boundary conditions have been employed. Where a liner has been modelled, elements to represent the liner, with a given permeability and thickness, have been included along with the potential seepage face boundary condition. The potential seepage face boundary represents a face exposed to atmospheric conditions, and hence represents the inside of the tunnel when it operates partially full (with free airspace).

Where a lining has been modelled as part of the shaft analysis, the sidewalls only have been assumed to be lined with the base remaining unlined. In the special case of major shaft WS3, the low permeability lining has been extended below shaft level 1 m into the top of the ECBF rock to represent the proposed diaphragm wall construction technique (details from Central Interceptor Project Design Team - 2011) to be used for the shaft, though the base of the excavation remains unlined.

6.3.4 Material constitutive models

The key property used in the material constitutive models is the saturated hydraulic conductivity, with the properties for each material summarised in Table 4-2. Additionally, for unsaturated flow to be modelled in a reasonable way, non-linear hydraulic conductivity functions have been used to represent the variation in hydraulic conductivity with increasing negative pore water pressure. The functions used for each material are shown in Appendix E

For transient analysis, the specific storativity of the materials are of importance in determining the rate of pore water pressure dissipation from initial conditions toward steady state conditions. This is represented by an equivalent M_v value in SEEP/W. The M_v value used for each material is based on the stiffness properties given in Table 3-1 (where it is assumed $M_v \approx 1/E$).

6.4 Surface settlement analysis

Surface settlement analysis has been undertaken based on the changes in pore water pressure estimated in the SEEP/W models. The settlement analysis has been undertaken using the finite element software package, SIGMA/W. SIGMA/W is a general purpose, 2 dimensional geotechnical finite element package. It allows a sequentially coupled consolidation analysis to be undertaken with SEEP/W results based on the change in pore water pressure. This modelling technique has been used to estimate surface settlements.

The stiffness parameters outlined in Table 3-1 have been used directly in the finite element model. Boundary conditions of fixed x-displacement on the lateral boundaries together with fixed x and y displacement on the base have been used. No other loadings are incorporated into the model, and in particular mechanical settlements due to the tunnel excavation process have not been included in the model.

7 Groundwater modelling results - tunnels

This section presents the key results from groundwater models for the 2-Dimensional tunnel analyses. The seven characteristic analysis sections are summarised first, followed by the results from each of these sections. Estimates of inflows to the tunnel during construction and operation are provided, along with estimates of groundwater drawdown in response to tunnelling. These model outputs are key inputs into associated models that assess the potential surface settlement effects as a result of the groundwater effects.

The results presented are derived assuming a range of potential tunnel construction methodologies. The results that are most representative of Watercare's intended construction methodology (EPB TBM) are highlighted in the following Tables.

Appendix E contains selected graphical output of the groundwater modelling for various sections and cases.

7.1 Selected tunnel analysis locations

The range of geological environments for estimation of groundwater effects and surface settlement are summarised below, and in Table 7-1 (Analyses consider the range of expected geological environments to characterise the range of responses to tunnel construction. As such analyses were carried out on representative sections, not all sections. Section 3 was not analysed as it was similar to Section 2):

- Tunnel excavated in ECBF beneath regional groundwater level within the Northern Zone (the Auckland Isthmus). Surface geology consists of a paleo-valley in an ancient ECBF surface in-filled with deep deposits of Puketoka Formation alluvium, further overlain by Auckland Volcanic field Basalt. The Puketoka Formation deposits are known to be sensitive to dewatering and prone to consolidation. This is represented by Analysis Sections 1 and 2.
- Tunnel excavated in ECBF beneath regional ground water level in the Manukau Lowlands area. Overlying geology consisting of Puketoka Formation alluvial deposits, tuff and basalt. This is represented by Analysis Section 4.
- Tunnel excavated in Kaawa sands beneath regional groundwater level in the Manukau Lowlands area. Overlying geology consisting of complex inter-bedding and inter-lensing of Puketoka Formation alluvial deposits, basalt and fill. This is represented by Analysis Section 5.
- Tunnel excavation in ECBF beneath regional groundwater with surface geology consisting of Puketoka Formation deposits. (Northern Zone, Auckland Isthmus). This is represented by Analysis Section 6.
- Tunnel excavated in ECBF beneath regional ground water level with surface geology consisting of residual soils derived from weathering of the ECBF rock (Central Zone Auckland Isthmus). This is represented by generic sections G1 and G2. G1 and G2 vary by the assumed tunnel depth below regional groundwater level, at approximately 40 m and 100 m below groundwater level for G1 and G2 respectively.

Table 7-1 – Tunnel analysis locations

Analysis Section No.	Section Location[1]	Tunnel Geology	Overlying Geology	Assumed Tunnel Invert Level (mRL)	Ground level (mRL)
1	Main tunnel chainage 11,000m	ECBF	Puketoka Fm, Basalt, Undifferentiated Alluvium	-14	12
2	Main tunnel chainage 14,750m	ECBF	Puketoka Fm, Basalt, Undifferentiated Alluvium	-20	52
4	Main tunnel chainage 21,500m	ECBF	Puketoka Fm, Tuff, Basalt	-27	5
5	Main tunnel chainage 23,200m	Kaawa Sands	Puketoka Fm, Marine Sediments, Basalt, Fill	-28	4
6	Link Sewer 3 chainage 100m	ECBF	Puketoka Fm	-20	12
G1	Generic Analysis	ECBF	Residual ECBF	200[2]	160[2]
G2	Generic Analysis	ECBF	Residual ECBF	200[2]	100[2]

1. Chainages from Central Interceptor Project Design Team - 2011
2. Levels are arbitrary for the generic analyses

7.2 Groundwater drawdown

The 2 dimensional models provide an assessment of groundwater drawdown for a range of combinations of liner permeability (including an unlined case) and time periods from excavation to liner installation. Table 7-2 summarises the long term groundwater drawdown that could occur depending on how water tight the tunnel lining is. The unlined case is provided for comparison only, and is unlikely to represent a credible construction case. The highlighted (**) values provide a conservative representation of Watercare's intended construction methodology (the use of gasketed segmental concrete lining is likely to be capable of limiting effects to less than those highlighted).

Table 7-2 – Estimated long term groundwater drawdown for tunnels

Analysis Section	Maximum Estimated Decrease in Phreatic Surface Level (m)			
	Unlined excavation	Liner installed, permeability 10^{-8}m/s	Liner installed, permeability 10^{-9}m/s	**Liner installed, permeability 10^{-10}m/s
Section 1	<0.6	minimal	minimal	minimal
Section 2	5.6	4.8	1.9	0.3
Section 4	1.6 (24)[1]	1.2 (0.5)[1]	0.4 (0.1)[1]	0 (0)[1]
Section 5	19[2]	11	2.1	0.1
Section 6	26	21	0	0
Generic 1	31[2]	28	14	1.5
Generic 2	61	53	24	3.5

1 - Results for high permeability ECBF (highly fractures or disturbed)

2 - Phreatic surface lowered to tunnel level

An important result from Table 7-2 is the relatively small drawdown of analysis sections 1, 2 and 4. This is largely attributed to the presence of basalt at the ground surface. The basalt provides a high volume source of water inflow to mitigate tunnel groundwater effects by providing a reservoir of groundwater to recharge the underlying layers. Sensitivity studies assess the importance of this effect on settlement estimates. In analysis sections where there is no basalt present, the groundwater drawdown in the model is significantly larger, down to tunnel level in some cases.

Table 7-3 summarises the results of groundwater modelling to assess the rate of development of groundwater over time, relevant to construction methodologies where there is a delay between excavation and installation of lining. The highlighted (**) values provide a conservative representation of Watercare's intended construction methodology (the use of an EPB TBM for excavation is likely to allow installation of lining within a shorter period than 7 days after excavation and is therefore likely to be capable of limiting effects to less than those highlighted).

Table 7-3 – Estimated development of groundwater drawdown over time - tunnels

Analysis Section	Maximum Estimated Decrease in Phreatic Surface Level (m)			
	Steady State	**7 Days	30 Days	1 Year
Section 1	<0.6	-	-	-
Section 2	5.6	0	0.1	1
Section 4	1.6 (24)[1]	0.2 (4)	0.3 (16)	1 (24)
Section 5	19 [2]	0.6	3.7	9.5
Section 6	26	0	0	0
Generic 1	31 [2]	0	0.1	1
Generic 2	61	0	0.1	1

1 - Results for high permeability ECBF (highly disturbed or fractured)

2 - Phreatic Line lowered to Tunnel Level

The rate of lowering of the phreatic surface in the models is highly dependent on the permeability of the material in which the tunnel is excavated. In analysis section 5 the tunnel is located in the Kaawa Formation, whereas in all other analysis sections the tunnel is located within less permeable ECBF resulting in slower lowering.

While the phreatic surface level in some models is not significantly affected by drainage into the tunnel, there is depressurisation of the rock mass about the tunnel in the models. The degree and extent of depressurisation depends on the length of time before drainage of groundwater into the tunnel is shut off, (representing the time at which the tunnel liner is installed). It is this depressurisation and its effects that lead to surface settlement being generated in the models (refer later to Table 8-1).

The potential for higher permeability ECBF associated with eruptive centres was assessed in a sensitivity study that was undertaken on analysis Section 4, also showing more rapid lowering.

7.3 Groundwater inflow to tunnels

Groundwater inflows have been assessed for each analysis section as part of groundwater modelling. Models provide estimates for groundwater inflow for a range of tunnel lining options from unlined, to construction of a liner with permeability in the range of 10^{-8}m/s 10^{-10}m/s. [Celestino (2001) investigated a range of tunnel linings and found that permeability typically varied between 10^{-8} m/s and 10^{-11} m/s, with values as low as 10^{-12} m/s possible with use of specific technologies for concrete lining manufacture.]

Table 7-4 summarises the estimated groundwater inflows. The highlighted (**) values provide estimates representative of Watercare's intended construction methodology.

Table 7-4 – Tunnel seepage analysis – estimated groundwater inflows

Analysis Section	Groundwater inflow (m^3/day/m), unlined excavation	Groundwater inflow (m^3/day/m), liner installed, permeability 10^{-08}m/s	Groundwater inflow (m^3/day/m), liner installed, permeability 10^{-09}m/s	**Groundwater inflow (m^3/day/m), liner installed, permeability 10^{-10}m/s
1	0.3	0.3	0.1	0.02
2	0.5	0.4	0.2	0.05
4	0.4 (39)[1]	0.3 (1)[1]	0.08 (0.1)[1]	0.01 (0.01)[1]
5	0.6	0.4	0.1	0.02
6	0.2	0.2	0.1	0.02
G1	0.1	0.1	0.1	0.02
G2	0.4	0.4	0.2	0.04

1 - Results for high permeability ECBF or PG (highly disturbed or fractured)

For comparison with the values in Table 7-4, groundwater inflow into the completed (lined) Vector Tunnel average about 0.03 m^3/day/m. It is noted that visual inspections of the Vector Tunnel lining show that some lengths provide greater levels of water tightness than others. The leakiest sections (where water can be seen flowing in, in isolated locations) average 0.2 m^3/day/m, and the driest 0.001 m^3/day/m.

As intended by Watercare, if an EPB capable TBM is used during construction (with gasketed segmental linings installed) groundwater inflows are expected to be in the order of 5 – 30 m^3/day local to the current construction location for open mode operation.

Where large zones of highly fractured ECBF (including highly fracture Parnell Git) is encountered for significant lengths of excavation, estimated local construction inflows are much higher (by a factor of up to 400). In such situations however the EPB TBM would necessarily be operated in "closed" mode (refer to Section 5.2 for explanation) to limit the groundwater inflow. Careful operation could see the flows restricted to nominal amounts or equivalent to excavation in more typical material (the 5 – 30 m^3 /day local flow from above).

Similarly, if operated in closed mode in more typical material, groundwater inflows could be reduced to less than 5 – 30 m^3/day near the excavation.

The total estimated long term (post construction) groundwater inflow for the 18 km of tunnels is estimated to be approximately 200 to 400 m^3/day (excluding inflows associated with shafts).

Estimated long term groundwater inflows into shafts constructed with low permeability liners are in the order of 10 to 60 m^3/day per shaft depending on shaft size (diameter) and surrounding geology. This is discussed in more detail in Section 9.

7.4 Groundwater users

Consented groundwater users in the vicinity of the project have been identified by a search of the Auckland Council database. Four such groundwater users have been identified close to the proposed tunnel alignment - within 100 m of Link Sewer 1, and within 200 m of the main tunnel. All these users take water from high capacity basalt surface aquifers. Analyses indicate that groundwater drawdown within the ECBF as a result of tunnelling is very unlikely to have a measureable effect on flows in the aquifers, and by inference on the existing groundwater users. Observations from the monitoring of actual draw down that occurred during the Vector tunnel construction support this finding (groundwater drawdown was measured in the ECBF associated with tunnel excavation, but no discernible response was identified in a basalt groundwater levels immediately overlying the ECBF).

Watercare services operate three bores approximately 1 km south of proposed Shaft WS3 at the WWTP, taking water from the Kaawa aquifer. Construction of shaft WS3 would be the most likely element of the project to have an effect on these bores, however the construction methodology proposed for this structure means significant groundwater effects at this distance are expected to be very unlikely.

7.4.1 Potential for sea water intrusion

A recent study of the sea water intrusion risks in New Zealand (Pattle Delamore Partners Ltd, 2011) identified that the primary risk of increased sea water intrusion occurs in unconfined aquifers extending less than 1km inland from the coastline and in confined aquifers where drawdown effects can extend over distances of 5km. The interface between fresh and sea water can be inland of the coast under some normal conditions. It identified that in areas where abnormal sea water intrusion had been identified, groundwater levels inland had typically been drawn down by extraction to below mean sea level (creating a hydraulic gradient for sea water to flow inland).

In the northern and central zone (Isthmus) groundwater models of credible short-term construction and long term operational scenarios for the tunnel indicate that the potential to establish an inland hydraulic gradient is extremely low. For much of the route, even if the excavations were left unlined (not a credible case) establishment of an inland gradient is not expected. In isolated areas where an inland gradient could theoretically be established, models indicate it would only develop if the tunnel excavation was left unlined for significantly in excess of one year (not a credible construction case). Once the tunnel was lined, the gradient would progressively reverse and the sea water /fresh water boundary would migrate back to its normal location. Groundwater users in the northern and central zone typically draw water from the surface aquifers in basalt flows - these aquifers would not be affected even in the event that such temporary sea water intrusion occurred, as they are well above sea level. Where water is extracted from deep ECBF bores, they are significantly inland from the coast and would not be expected to be affected by temporary inland migration of the sea water/fresh water boundary.

In the southern zone, (Manukau Lowlands) ground conditions necessitate the use of EPB TBM tunnelling equipment with rapid lining of the tunnel post excavation. Use of this equipment is expected to minimise the potential for groundwater drawdown through this area, such that the potential for sea water intrusion into the Kaawa Aquifer would be negligible. Extraction of groundwater for use in this area is to the south of the tunnel termination at the WWTP and to the west on Puketutu Island. In the unlikely event that temporary sea water intrusion were to occur as a result of construction activities, it would be in areas distant from these extractions.

7.5 Water-tightness and seepage out of a lined tunnel

In typical flow situations (tunnel operated flowing partially full), internal pressure will be at or near to local atmospheric pressure, significantly lower than external groundwater pressure, with a strong tendency for groundwater seepage into the tunnel along its entire length.

In an extreme case, (10 year storm combined with main pump station failure) estimates are that the tunnel and associated shafts fill to RL 6.85 m at Western Springs, and to 6.07 m at Keith Hay Park (details from Central Interceptor Project Design Team – 2011). A maximum internal tunnel pressure of approximately 30 m head results from this event.

External groundwater pressure is expected to exceed this internal pressure in the tunnel from the upstream extent at Western Springs to the Manukau Harbour. In this case seepage from the tunnel is not expected.

Beneath the Harbour and through Kiwi Esplanade to the Pump Station at WS3, internal pressure can be expected to exceed external groundwater pressure by up to 3 to 5 m in the extreme event. Models have been run to assess the degree to which this surcharge might result in seepage out of the tunnel. The relevant location for analysis is Analysis Section 5.

Table 7-5 provides a summary of estimated tunnel outflows for a range of tunnel liner permeability. The highlighted (**) values are most representative of those that might be expected for Watercare's intended construction methodology.

Table 7-5 – Estimated seepage out of tunnel during extreme 10 year event

Analysis Section	Tunnel water outflow (m^3/day/m), unlined excavation	Tunnel water outflow (m^3/day/m), liner installed, permeability 10^{-08}m/s	Tunnel water outflow (m^3/day/m), liner installed, permeability 10^{-09}m/s	**Tunnel water outflow (m^3/day/m), liner installed, permeability 10^{-10}m/s
5	0.3	0.1	0.02	0.003

8 Surface settlement estimates - tunnels

This section discusses the implications of surface settlement on surface structures, and summarises the settlement estimates prepared for the Central Interceptor Tunnels. Watercare have indicated that they intend to construct the tunnels using an EPB capable TBM. The use of an EPB TBM combined with installation of a sufficiently watertight tunnelling lining is expected to limit surface settlement such that damage is not expected in surface structures for all sections of tunnel. Estimates of the rate of settlement development following initial excavation are discussed for each representative geological environment to provide guidance for detailed construction methodology development.

8.1 Unmitigated settlement hazard

Unmitigated settlement hazard is defined as that settlement that is estimated to occur as a result of the dewatering that might occur if the tunnel were to be constructed without a low permeability liner (a liner with a lower permeability than the material the tunnel is excavated through). This estimate sets the upper bound of potential settlement (refer Table 8-1), and assists in identifying where tunnel construction methodology needs to specifically consider mitigation of settlement effects, and where it does not, providing guidance on ultimate selection of tunnelling methodologies. Estimates have been prepared from groundwater models run to steady state conditions (i.e. long term) with no liner elements installed. The settlements presented are the highest estimated settlement on the analysis section (typically near tunnel centreline, but somewhat dependant on distribution of compressible materials in the analysis section).

Table 8-1 - Unmitigated settlement hazard - tunnels

Analysis Section	Representative geology	Estimated maximum unmitigated settlement hazard (mm)	Estimated maximum differential settlement hazard
1	Tunnel excavated in ECBF. Surface geology consists of paleo-valley in ancient ECBF surface in filled with deep deposits of Puketoka Formation alluvium further overlain by Auckland Volcanic field Basalt.	20 (90)[1]	Flatter than 1:2,000
2	Tunnel excavated in ECBF. Surface geology consists of paleo valley in ancient ECBF surface in filled with deep deposits of Puketoka Formation alluvium further overlain by Auckland Volcanic field Basalt.	100 (350)[1]	Flatter than 1:2,000
4	Tunnel excavated in ECBF with surface geology consisting of residual soils derived from weathering of the ECBF rock, further overlain by Auckland Volcanic field Basalt.	20 (100)[1]	Flatter than 1:2,000
5	Tunnel excavated in ECBF in the Manukau Lowlands area. Overlying geology consisting of Puketoka Formation alluvial deposits, tuff and basalt.	200	1:600

Analysis Section	Representative geology	Estimated maximum unmitigated settlement hazard (mm)	Estimated maximum differential settlement hazard
6	Tunnel excavated in ECBF. Surface geology consists of in-filled valley in ECBF with deposits of Puketoka Formation alluvium.	180	1:1000
G1	Generic analysis – ECBF Rock with mantle of residual ECBF.	60 [30][2]	Flatter than 1:2,000
G2	Generic analysis – ECBF Rock with mantle of residual ECBF.	170 [60][2]	Flatter than 1:2,000

1. Sensitivity studies assess the ability of the basalt aquifer to buffer groundwater drawdown in underlying compressible materials. The unmitigated settlement hazard increases significantly (shown in brackets) if the basalt is assumed to be less permeable by one order of magnitude. The permeability adopted here is supported by PDP (2005), however it is noted that prior to that study, lower permeability have been adopted for modelling in other Auckland projects.
2. Experience from other projects indicates that these models may overestimate potential settlement by a factor of two or more. Monitoring at Three Kings Quarry (refer Appendix E) indicates an upper bound for G1 and G2 settlement based on this experience [shown in square brackets].

Differential settlement associated with the unmitigated settlement is typically 1:2000 or flatter as shown in Table 8-1. The exceptions to this are:

- Section of 5, where higher differentials are indicated for a small area to the north of the tunnel alignment near the WWTP, associated with the geological boundary between volcanic surface deposits and Tauranga Group deposits. At this boundary differentials are estimated at approximately 1:600. These higher differentials are not expected to have any significant effect as there are no surface structures in this area.
- Section 6, where a maximum differential of 1:1000 is estimated. When models include a low permeability tunnel liner installed soon after construction, estimated differentials are flatter than 1:2000.

The rate at which this settlement could develop has been estimated from the same models, but run as transient analyses (i.e. not steady state). From these models the time-dependent development of settlement towards the unmitigated settlement hazard has been estimated, as presented in Table 8-2 below. The maximum surface settlement developed by specific timeframes (time from when excavation carried out) are reported.

The estimate rate of development in combination with the magnitude estimate provides guidance on whether construction planning needs to consider the time delay between excavation and lining as a means of restricting settlement development. The highlighted (**) values provide a conservative representation of Watercare's intended construction methodology (the use of an EPB TBM for excavation is likely to allow installation of lining within a shorter period than 7 days after excavation and is therefore likely to be capable of limiting effects to less than those highlighted).

Table 8-2 – Estimated development of settlement over time – unlined tunnels

Analysis Section	Estimated maximum unmitigated settlement hazard (mm)	**Estimated maximum settlement at 7 days (mm)	Estimated maximum settlement at 30 days (mm)	Estimated maximum settlement at 1 year (mm)
1	20	10	15	20
2	100 (40) [1]	20 (20) [1]	30 (25) [1]	50 (30) [1]
4	20 (100) [1]	5 (40) [1]	10 (50) [1]	15 (70) [1]
5	200	50	70	110
6	180	10	20	40
G1	60 [30] [2]	10 [5] [2]	15 [10] [2]	20 [10] [2]
G2	170 [60] [2]	15 [10] [2]	40 [20] [2]	80 [30] [2]

1. The effect of potentially much higher permeability ECBF material, associated with disturbance and fracturing about eruptive centres or highly permeable PG lenses, has been assessed in sensitivity analyses (results showing in brackets). The analyses have been undertaken on Analysis Section 2 near Mt Roskill, and analysis Section 4 near the Mangere Lagoon and Mangere Mountain.
2. Experience from other projects indicates that these models may overestimate potential settlement by a factor of two or more. Monitoring at Three Kings Quarry (refer Appendix E) indicates an upper bound for G1 and G2 settlement based on this experience [shown in square brackets].

The differing results for the high-permeability ECBF sensitivity cases for analysis Section 2 and analysis Section 4 reflect the varying extents of high-permeability ECBF in the models. In analysis Section 2 the high-permeability ECBF is assumed to extend up to approximately 100 m from the tunnel – the tunnel itself is modelled in unmodified ECBF. For analysis Section 4, the ECBF is assumed to be present throughout the model. The differing assumption is due to the distance of the cross-section from the eruptive centre.

8.2 Liner water-tightness and long term settlement development

The degree of water-tightness required of the tunnel liner to control long term settlement development has been assessed. Models have been run with a range of liner permeabilities, ranging from a very leaky liner (permeability very similar to surrounding rock) to one with permeability significantly lower than surrounding rock.

The long term settlements that might develop for the level of water-tightness (i.e. liner permeability) are reported below in Table 8-3.

Table 8-3In all cases, the liner has been assumed to be 300 mm thick, although it is recognised that lining thickness will be determined as part of final design. The highlighted (**) values are most representative of those that might be expected for Watercare's intended construction methodology.

Table 8-3 – Estimated long term settlement and liner permeability – tunnels

Analysis Section – Refer to Figure A1	Unmitigated maximum settlement hazard (mm)	Mitigated maximum settlement as a result of liner installation (mm)		
		Liner installed, permeability $1x10^{-08}$m/s	Liner installed, permeability $1x10^{-09}$m/s	**Liner installed, permeability $1x10^{-10}$m/s
1	20	20	5	<5
2	100	90	40	5
4	20	15	5	<5
5	200	150	30	5
6	180	170	110	20
G1	60 [30][1]	60 [30] [1]	40 [20] [1]	10 [5] [1]
G2	170 [60][1]	160 [60] [1]	90 [40] [1]	20 [10] [1]

1. Experience from other projects indicates that these models may overestimate potential settlement by a factor of two or more. Monitoring at Three Kings Quarry (refer Appendix E) indicates an upper bound for G1 and G2 settlement [shown in square brackets].

8.3 Discussion on implications of estimated settlement

8.3.1 Tunnel construction

Analyses indicate the potential for relatively large settlement (i.e. greater than 50 mm) to develop rapidly when:

- The tunnel is excavated in the Kaawa and Puketoka Formation, (Analysis section 5)
- The tunnel is excavated in ECBF and extensive and significantly more highly permeable ECBF (or PG) material is encountered (such as might be expected close to eruptive centres). Sensitivity studies identify the potential for large settlement to develop rapidly where Puketoka formation sediments are also present.

In such areas, groundwater control measures may need to be installed soon after excavation (typically within less than 7 days) to satisfactorily control groundwater drawdown and to limit resulting surface settlement. Construction by EPB TBM and installation of a precast segmental concrete liner with gaskets would be one methodology that could be expected to provide the level of control on groundwater effects required. This methodology has been successfully utilised in recent projects in Auckland (refer Section 2.5 and Appendix C) and is Watercare's intended construction methodology for all tunnels (including link sewers). Its ability to actively control groundwater drawdown between excavation and lining (through application of a temporary face pressure) further enhances the capability of this construction methodology to control surface settlement effects, potentially to less than those predicted here.

Where the tunnel is excavated in ECBF overlain by Puketoka Formation (analysis Sections 1, 2 and 6) sensitivity studies identify the potential for large settlement to develop, but over a longer time period. In these areas installation of a water-tight liner a short time after excavation (circa 7-30 days) is expected to be a sufficient mitigation measure.

Where the tunnel is excavated through more typical ECBF, analysis indicates that settlement is likely to develop more slowly, potentially allowing a more significant delay between excavation and tunnel lining.

8.3.2 Tunnel operation

The unmitigated settlement hazard estimated for analysis Sections 2, 5, G1 and G2 (Table 8-1) are in excess of or close to what is has historically been consented for similar projects and environments. Sensitivity studies indicate that at analysis Sections 1, 2 and 4, estimated settlements exceed these limits if the overlying basalt is assumed to have a lower capacity to buffer dewatering effects in underlying compressible materials. In areas represented by these analysis sections the permanent works design of the tunnel will need to include measures to control long-term groundwater drawdown such that surface settlement is also controlled.

The proposed tunnel lining is expected to be an effective groundwater control measure provided it is engineered to have a sufficiently low permeability.

A liner (nominally 300 mm thick) of permeability 10^{-10} m/s or equivalent combination is estimated to control drawdown such that long term settlements of 20 mm or less are estimated. This is Watercare's intended construction detail.

Measurements of groundwater inflow to the completed Vector tunnel (Refer Appendix F) support that this degree of water tightness is achievable, as does Celistino (2001). Estimates indicate that for a liner with permeability 10^{-9} m/s could ultimately result in groundwater drawdown sufficient to cause settlement to approach and potentially exceed consentable limits.

8.3.3 Sea floor settlement

In southern section of the alignment where it passes beneath the Manukau Harbour and the Mangere Lagoon construction of the tunnel has the potential to result in settlement of the sea bed, in much the same way that is does on the dry land sections. The Western Interceptor and NZRC Refinery to Auckland pipeline both lie upon the seafloor. It is understood that the former was constructed by blasting a trench in the basalt to lay the pipe and the later was constructed by excavating excess sediment and laying the pipe directly on the Basalt. Near Hillsborough it is likely that both pipeline are sitting upon residual ECBF and soft sediments. Final design should consider the potential impacts of tunnel construction on these pipelines. Watercare's intended construction method of an EPB capable TBM would be expected to be capable of satisfactorily controlling settlement of these services (with careful operation in closed mode - refer Section 5.2 for explanation).

The magnitude of sea floor settlement would be expected to be similar or less than that predicted on dry land (for equivalent construction methodologies) as the body of water above the sea bed is expected to provide recharge that is likely to buffer groundwater depressurisation in the compressible marine deposits on the sea floor.

Settlement that does occur is likely to be of an order that is not locally noticeable on mud flats, and there is not expected to be any impact to the natural processes within the intertidal or sub-tidal areas.

9 Groundwater modelling – shafts

This section presents the key results from groundwater models for the axi-symmetric shaft analyses. The seven characteristic analysis sections are summarised first, followed by the results from each of these sections. Estimates of inflows to the shafts during construction and operation are provided, along with estimates of groundwater drawdown in response to construction. These model outputs are key inputs into associated models that assess the potential surface settlement effects as a result of the groundwater effects.

Appendix E contains selected graphical output of the groundwater modelling of the shafts for various analysis sections and cases.

9.1 Selected shaft analysis locations

Shaft analyses represent the two types of shafts; large construction access shafts (denoted "WS") and smaller operation access shafts (denoted "AS"). Specific analyses have been undertaken at each of the three large shaft locations. Additionally four analyses have been undertaken using simplified ground conditions to represent the range of ground conditions anticipated for the 16 smaller operational access shafts. Three of these analyses are based on specific shaft sites that are considered representative (AS3, AS4 and AS7), while the "Generic" analysis represents a typical ECBF only model. Table 9-1 summarises the profiles analysed, and the project shafts they are representative of.

Table 9-1 – Shaft analysis locations

Shaft No.	Shaft Location[2]	Shaft Geology	Assumed Excavation Diameter (m)	Assumed Shaft Base Level (mRL)	Ground level (mRL)	Representative of other shafts
WS1 (Western Springs)	Chainage 10,000m	Undifferentiated Alluvium, Basalt, ECBF	28	-20	12	WS1
WS2-a[1] (May Rd)	Chainage 15.450m	Basalt, Tuff, Puketoka Fm, ECBF	28	-25	48	WS2
WS2-b[2] (May Rd)	As above	As above	As above	As above	As above	WS2
WS3 (Mangere Pump Station	Chainage 23,200m	Puketoka Fm, Kaawa Sands, ECBF	38	-34	2	WS3
AS3 (Haverstock Rd)	Chainage 13,200m	Puketoka Fm, ECBF	10	-20	35	AS3, L1S2, L2S2, L3S1 - L3S4
AS4 (Walmsley Park)	Chainage 14,300m	Basalt, Puketoka Fm, ECBF	10	-20	50	AS4, L1S1,
AS7 (Kiwi Esplanade)	Chainage 20,600m	Marine sediments, Basalt, Puketoka Fm, ECBF	10	-26	3	AS7
Generic	-	Weathered ECBF, ECBF Rock	10	-20	35	AS5, AS6, L2S1, L2S5

1. WS2-a and WS2-b have different initial groundwater level assumptions. For WS2-a it is assumed that the basalt is under the water-table and hence provides a high-permeability reservoir of groundwater. WS2-b assumes the basalt does not influence groundwater at the shaft site. These two different assumptions are made as groundwater levels are not well understood at the shaft site, which is expected to be sited at the edge of the basalt flow.
2. Chainages from Central Interceptor Project Design Team - 2011).

9.2 Groundwater inflow to shafts

Groundwater inflows in each shaft model have also been output as part of the groundwater analysis. Flows have been estimated for both an open, leaky excavation, and for a lined excavation (i.e. with secant piles, diaphragm walls, or a caisson). As for the tunnel analysis, a range of values for the liner permeability have been assumed. The permeabilitiy given assumes a 0.5 m thick liner. The liners are modelled as extending over the sides of the shaft only, not across the shaft base (i.e. it is assumed the shaft will have a permanent drained base). If the base is not drained, but is instead fully tanked, long term groundwater inflows can be expected to be significantly lower. Table 9-2 summarises the estimates of shaft inflows. The unlined case is provided for reference only, and does not represent a credible long term shaft condition. Shaft lining constructed to a competent standard of workmanship would typically be expected to achieve permeability equivalent to 10^{-9} m/s.

Table 9-2 - Shaft seepage analysis – Estimated groundwater inflows

Shaft Analysis	Groundwater inflow (m^3/day), unlined excavation	Groundwater inflow (m^3/day), liner installed, permeability 10^{-08}m/s	Groundwater inflow (m^3/day), liner installed, permeability 10^{-09}m/s	Groundwater inflow (m^3/day), liner installed, permeability 10^{-10}m/s	Groundwater inflow (m^3/day), liner installed, impermeable
WS1 (Western Springs)	2000	40	30	20	20
WS2-a (May Rd)	400[100] [1]	100[100] [1]	60[60] [1]	30[30] [1]	30[30] [1]
WS3 (Mangere Pump Station)	200	70	20	20	20
AS3 (Haverstock Rd)	50	40	20	10	10
AS4 (Walmsley Park)	200	50	20	10	10
AS7 (Kiwi Esplanade)	1000	20	10	10	10
Generic	50	30	10	10	10

1 - WS2-a and WS2-b have different initial groundwater level assumptions. For WS2-a, it is assumed that the basalt at the model is under the water-table and hence provides a high-permeability reservoir of groundwater. WS2-b assumes the basalt aquifer does not buffer groundwater drawdown at the shaft site. These two different assumptions are made as groundwater levels are not well understood at the shaft site, which is expected to be sited at the edge of the basalt flow. WS2-a is given, with WS2-b in square brackets.

Based on these analyses, estimates of maximum potential short term construction inflows, and maximum long term operational inflows have been made for the shaft sites. Some sites have multiple shafts associated with them. The inflows have been factored to allow for this. Long term operational groundwater inflows are presented in Table 9.3.

Table 9.3 – Estimated long term groundwater inflows at shaft sites

Shaft site	Representative analysis	Estimated long term groundwater inflows, (m^3/day) for lined shafts (permeability 10^{-09} m/s)
WS1	WS1	25
WS2	WS2	65
WS3	WS3	25
AS1	AS4	20
AS2	AS4	20
AS3	AS3	15
AS4	AS4	20
AS5	Generic	15
AS6	Generic	15
AS7	AS7	10
L1S1	AS4	20
L1S2	AS3	15
L2S1	Generic	15
L2S2	AS3	15
L2S3	AS4	20
L3S1	AS3	15
L3S2	AS3	15
L3S3	AS3	15
L3S4	AS3	15
L3S5	Generic	15

For the shafts sites associated with the main tunnel (WS and AS shafts), maximum construction groundwater inflows are expected to be in the order of 50 to 220 m^3/day on the assumption the shafts are unlined during excavation. If the lining is installed in concert with excavation, flows are likely to lower - in the order of 10 to 65 m^3/day per site.

At shaft sites WS1, WS2, and AS7 very high flows would be expected if they were excavated unlined (particularly through the surface basalt material. Shafts at these sites will necessarily require lining in concert with excavation, or some other methodology, to control inflows 10-60 m^3/day.

For the shafts sites associated with the link sewers (L_S_ shafts), maximum construction groundwater inflows are expected to be in the order of 50 to 150 m^3/day on the assumption the shafts are unlined during excavation. If the lining is installed in concert with excavation, flows are likely to be reduced to 10 to 20 m^3/day per site.

10 Surface settlement estimates - shafts

This section discusses the implications of surface settlement on surface structures, and summarises the settlement estimates made for the Central Interceptor shafts. The use of typical construction methodologies such as those outlined in Section 2.3.2 are expected to limit surface settlement such that damage is not expected in surface structures. Estimates of the rate of settlement development following initial excavation are discussed to provide guidance for detailed construction methodology development.

10.1 Unmitigated settlement hazard

The unmitigated settlement hazard for shafts is based on an unlined, drained shaft excavation, where groundwater is free to flow into, and is pumped from, the excavation. This estimate sets the upper bound of potential settlement (refer Table 10-1), and assists in identifying where shaft construction methodology needs to specifically consider mitigation of settlement effects, and where it does not.

Table 10-1 - Unmitigated settlement hazard - shafts

Shaft Analysis	Representative geology	Estimated maximum unmitigated settlement hazard (mm)
WS1	Undifferentiated Alluvium, Basalt, ECBF	20
WS2-a [b] [1]	Basalt, Tuff, Puketoka Fm, ECBF	70 [100][1]
WS3	Puketoka Fm, Kaawa Sands, ECBF	300
AS3	Puketoka Fm, ECBF	100
AS4	Basalt, Puketoka Fm, ECBF	90
AS7	Marine sediments, Basalt, Puketoka Fm, ECBF	100
Generic	Weathered ECBF, ECBF Rock	50

1. WS2-a and WS2-b have different initial groundwater level assumptions. For WS2-a, it is assumed that the basalt at the model is under the water-table and hence provides a high-permeability reservoir of groundwater. WS2-b assumes the basalt aquifer does not buffer groundwater drawdown at the shaft site. These two different assumptions are made as groundwater levels are not well understood at the shaft site, which is expected to be sited at the edge of the basalt flow. WS2-a is given, with WS2-b in square brackets.

As for the tunnel analysis, the rate at which this settlement may develop has been estimated from the same models using transient analyses (refer Table 10-2). The transient analyses assume full excavation at time t=0, and reported settlement estimates are based on this. In reality excavation will occur over a period of time dependant on a number of factors. The times given can therefore be considered to be the time since pumping from the excavation first commences.

Table 10-2 – Estimated development of settlement over time - shafts

Analysis Section	Estimated maximum unmitigated settlement hazard (mm)	Estimated maximum settlement at 7 days (mm)	Estimated maximum settlement at 30 days (mm)	Estimated maximum settlement at 1 year (mm)
WS1	20	10	10	20
WS2-a [b] [1]	70 [100] [1]	60 [30] [1]	70 [40] [1]	70 [50] [1]
WS3	300	100	200	200
AS3	100	30	40	60
AS4	90	70	90	90
AS7	100	80	80	100
Generic	50	10	20	30

1. WS2-a and WS2-b have different initial groundwater level assumptions. For WS2-a, it is assumed that the basalt at the model is under the water-table and hence provides a high-permeability reservoir of groundwater. WS2-b assumes the basalt aquifer does not buffer groundwater drawdown at the shaft site.

The estimate rate of development in combination with the magnitude estimate provides guidance on whether construction planning needs to consider the time delay between excavation and lining as a means of restricting settlement development.

10.2 Shaft liner water-tightness and long term settlement development

The long term settlements that might develop for the level of water-tightness of the shaft liner are reported in Table 10-3 below along with the permeability of the tunnel liner. In all cases (except WS3), the liner has been assumed to be 500 mm thick, although it is recognised that lining thickness will be determined individually for each shaft as part of final design. The liner is assumed to cover the sides of the shafts, but not the base of the shaft, in all cases (except WS3).

For the special case of WS3, where a diaphragm wall construction is proposed, a shaft lining thickness of 1 m is assumed, and it is assumed to penetrate below the base of the shaft excavation 1 m into the top of ECBF rock.

Table 10-3 – Estimated long term settlement and liner permeability - shafts

Shaft Analysis	Unmitigated maximum settlement hazard (mm)	Estimated maximum mitigated settlement as a result of liner installation (mm)		
		Liner installed, permeability 10^{-08}m/s	Liner installed, permeability 10^{-09}m/s	installed, permeability 10^{-10}m/s
WS1	20	10	10	10
WS2-a [b]	70[100]1	30[70] [1]	10[20] [1]	10[10] [1]

Shaft Analysis	Unmitigated maximum settlement hazard (mm)	Estimated maximum mitigated settlement as a result of liner installation (mm)		
		Liner installed, permeability 10^{-08}m/s	Liner installed, permeability 10^{-09}m/s	installed, permeability 10^{-10}m/s
WS3	300	100	40	30
AS3	100	80	30	10
AS4	90	30	10	10
AS7	100	10	10	10
Generic	50	40	10	10

1. WS2-a and WS2-b have different initial groundwater level assumptions. For WS2-a, it is assumed that the basalt at the model is under the water-table and hence provides a high-permeability reservoir of groundwater. WS2-b assumes the basalt aquifer does not buffer groundwater drawdown at the shaft site. These two different assumptions are made as groundwater levels are not well understood at the shaft site, which is expected to be sited at the edge of the basalt flow. WS2-a is given, with WS2-b in square brackets.

10.3 Discussion on implications of estimated settlement

Analyses indicate (Table 10-2) the potential for relatively large settlement to develop rapidly when:

- A shaft is excavated in the Kaawa and Puketoka Formation, (WS3)
- A shaft is excavated in ECBF overlain by Puketoka Formation (WS2, AS3, AS4 and AS7).

In these areas, where shafts are in the vicinity (within approximately 200-300 m) of settlement sensitive structures, construction methodologies that allow control of groundwater effects are likely to be required. With appropriate design, one of the construction methodologies discussed in Section 2.3.2, or a combination of such methods (e.g. Secant piling, diaphragm walling, open caisson and/or basalt grouting) can be expected to provide the level of control on groundwater effects required.

Alternatively locating the shafts such that they are sufficiently offset from any structures may also be possible in some cases such that the settlements have no effect on structures.

Analyses indicate (Table 10-3) that a nominally 500 mm thick lining of permeability 10^{-10} m/s (or equivalent combination) is expected to sufficiently control groundwater long term in all cases, which is considered readily achievable.

Where shafts are excavated through more typical competent ECBF (generic analysis), analysis indicates that groundwater drawdown settlement is likely to develop more slowly, potentially allowing for a permanent liner to be delayed for some time (circa 1 month to 1 year).

11 Monitoring and effects mitigation

A monitoring programme should be implemented to monitor tunnel and shaft construction. It would measure the effect that construction has on the groundwater system and confirm that any associated surface settlement is within acceptable limits.

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

While both groundwater and surface levels should be subjects of the monitoring programme, it is surface levels that are of importance for protecting public and private property from potential adverse effects associated with construction.

Groundwater response may vary significantly from those estimated by pre construction models but provided the settlement resulting from the groundwater responses are within tolerable (and consented) limits then this variance in itself should not be considered a reason to interrupt construction. The exception to this would be where groundwater response to construction has a more significant impact on consented groundwater users.

11.1 Approach to monitoring

The recommended approach for monitoring and responding to groundwater and surface level changes is to set in place a programme for monitoring construction effects relative to the actual construction programme and the estimated settlement hazard associated with construction. In areas where the settlement hazard is estimated to be low (such as in low compressibility geology) monitoring locations would be more widely spaced than in areas where settlement hazard is estimated to be higher, (such as where tunnelling passes beneath compressible Pleistocene deposits). Similarly, monitoring networks would be denser in built up areas, and more dispersed in undeveloped areas.

Triggers would be set such that additional monitoring (frequency and/or locations) is required in the event that alert levels (levels set to reflect expected behaviour) are approached. In the event such alert levels were reached, additional modelling/estimates would be prepared to confirm that alarm levels (levels set to reflect consent conditions) are not expected to be threatened despite variance from expected behaviour. Mitigation measures would be provided that could be implemented in the event that alarm levels were threatened.

11.2 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 repeat surveys prior to commencing construction.

Experience from long term settlement monitoring at Three Kings Quarry is that while stated survey accuracy within any survey round was plus or minus 2-3 mm, survey repeatability between survey rounds bounced within a broader band of plus or minus 5 mm.

11.3 Monitoring installations

Monitoring of tunnel construction could include:

- Multilevel piezometers installed in close proximity to the tunnel as well remotely to monitor groundwater level response to construction within the material the tunnel is excavated. Piezometer installations should be installed to capture the groundwater response in a representative range of geological units, particularly those with potential to consolidate
- A network of surface level monitoring marks installed on a number of representative cross sections to the tunnel alignment
- Additional surface level monitoring marks located on or near settlement sensitive structures.

Piezometers installed during investigations for tunnel design are likely to form part of the monitoring network.

11.4 Consent conditions

It is recommended that Consent Conditions for the project consider the need for monitoring plans to control the potential for adverse effects.

The conditions should include provisions such that, where required:

- The tunnel and shafts are designed with a low permeability liner such that long term groundwater drawdown, and associated surface settlement is controlled to an acceptable level.
- The tunnel construction methodology includes the capability to pressurise the excavated face and unlined annulus where excavations pass under settlement sensitive geology. At such locations, the construction methodology should also allow for the lining to be installed within seven days of excavation.
- The effects of tunnel construction on groundwater and surface settlement be monitored as the tunnel and shafts are excavated and for a period of no less than two years following lining completion.
- Prior to construction a monitoring plan is prepared detailing the extent and frequency of monitoring. The plan should be targeted to respond to the actual excavation programme, and be specific to the potential settlement hazard at any location.
- Surface settlement associated with tunnel construction and operation is limited to a maximum of 50 mm total settlement and a differential no steeper than 1:1000 in developed areas.

PART C

12 Summary and conclusions

Analyses have been undertaken to estimate potential groundwater and surface settlement effects of the tunnels and shafts based on a range of geological and hydro-geological conditions and for a range of potential construction techniques. The following key findings of these analyses are summarised below:

- The analysis results presented can be considered as relatively conservative (over) estimates of settlement, by comparison with observed performance of tunnelling and other relevant projects in Auckland. The assessment results presented have been made for the shallowest tunnel alignment being sought for consent. Assessments for the shallowest alignment would be expected to provide higher surface effects than deeper alignments. The assessments made here are not expected to be sensitive to the final location of the tunnel within the 40 m wide corridor, or to the two alternative alignments through Kiwi Esplanade.
- The upper bound for surface settlement along the route has been estimated by considering construction methodologies (for shafts and tunnels) that allow no control on groundwater drawdown. This unmitigated settlement hazard varies along the route, depending on the local hydro-geological conditions, and is estimated to be up to approximately 50 mm-300 mm in places. In these cases, the design and construction methodology will need to specifically consider this hazard, and ensure that measures provide for control of the settlement to acceptable levels. Experience from past projects (Hobson Bay Sewer and Rosedale Tunnel) indicates that there are practical construction methodologies available that have been used successfully to address similar issues. Watercare intend to construct all the tunnels with an EPB capable TBM as (successfully used on the recent Hobson Bay Sewer project).
- The rate at which such settlement would develop is generally expected to be slow relative to potential construction methodologies for both shafts and tunnels. The settlements are generally expected to take a time to develop, over a period of greater than one month to one year after excavation. The exception to this is where excavation is within Kaawa Sands or Lower Puketoka Formation materials near the Mangere WWTP, or in high permeability ECBF (potentially associated with historic volcanic eruptive centres or with PG lenses). At these locations settlements are expected to develop more rapidly, and in these locations it may be necessary to utilise an EPB capable TBM (in closed mode) or similar construction methodology to control settlement, and/or ensure a suitably water tight liner is installed rapidly following excavation.
- Controlling surface settlements to acceptable levels will require consideration of the combined effects of the construction methodologies for both shaft and tunnel construction. Mechanical settlements for tunnel construction (settlement associated with ground loss) have been assessed by Watercare's Principal Engineering Advisor and are anticipated to typically be less than less than 10 mm. Experience from recent tunnelling projects in Auckland indicates that the construction methodologies proposed are readily able to limit the combined effects to within consentable limits. Mechanical settlements due to shaft construction are dependent on the design and construction methodology adopted for each shaft and will need to be considered in during detailed design. The range of construction methodologies proposed here are expected to be able to control the combined effects of shaft construction to within consentable limits.
- Under normal operating conditions, groundwater seepage will tend to be into the tunnel, and no seepage outflows are expected. Total long term groundwater inflow to the tunnel

system is estimated to be 600 - 800 m^3/day for the 18 km of tunnels and associated shafts (for construction with a low permeability liner). Reversal of groundwater flow and leakage from the tunnel is possible during an extreme event where a prolonged pump station failure coincides with a large storm event. In this instance, a leakage rate of 0.02 m^3/day/m is estimated for a 300mm concrete tunnel lining (for permeability less than 10^{-9} m/s).

Watercare's intended tunnel construction methodology (excavation using an EPB capable TBM and lining the tunnels using gasketed segmental concrete lining) in conjunction with appropriate shaft construction methodologies is expected to be capable of adequately controlling groundwater and surface settlement. It is concluded that it is possible to design and construct the tunnels and shafts for the Central Interceptor Project such that:

- There is negligible risk of tunnel and shaft construction having an effect on groundwater users in the vicinity.
- Surface settlements due to dewatering above the tunnel alignment and in the vicinity of shafts are limited to less than 50 mm with a low risk of exceedance.
- Differential settlements above the tunnel and in the vicinity of shafts are limited to less than 1:1000 with a low risk of exceedance.
- There is negligible risk of structural damage to buildings and service because of tunnel and shaft excavation and long term operation.
- There is a low risk of measurable changes in groundwater quality immediately about the tunnel and negligible risk of any adverse effect on regional groundwater quality.

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:

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Senior Geotechnical Engineer

Robert Hillier
Group Manager, Geotechnical

GRT

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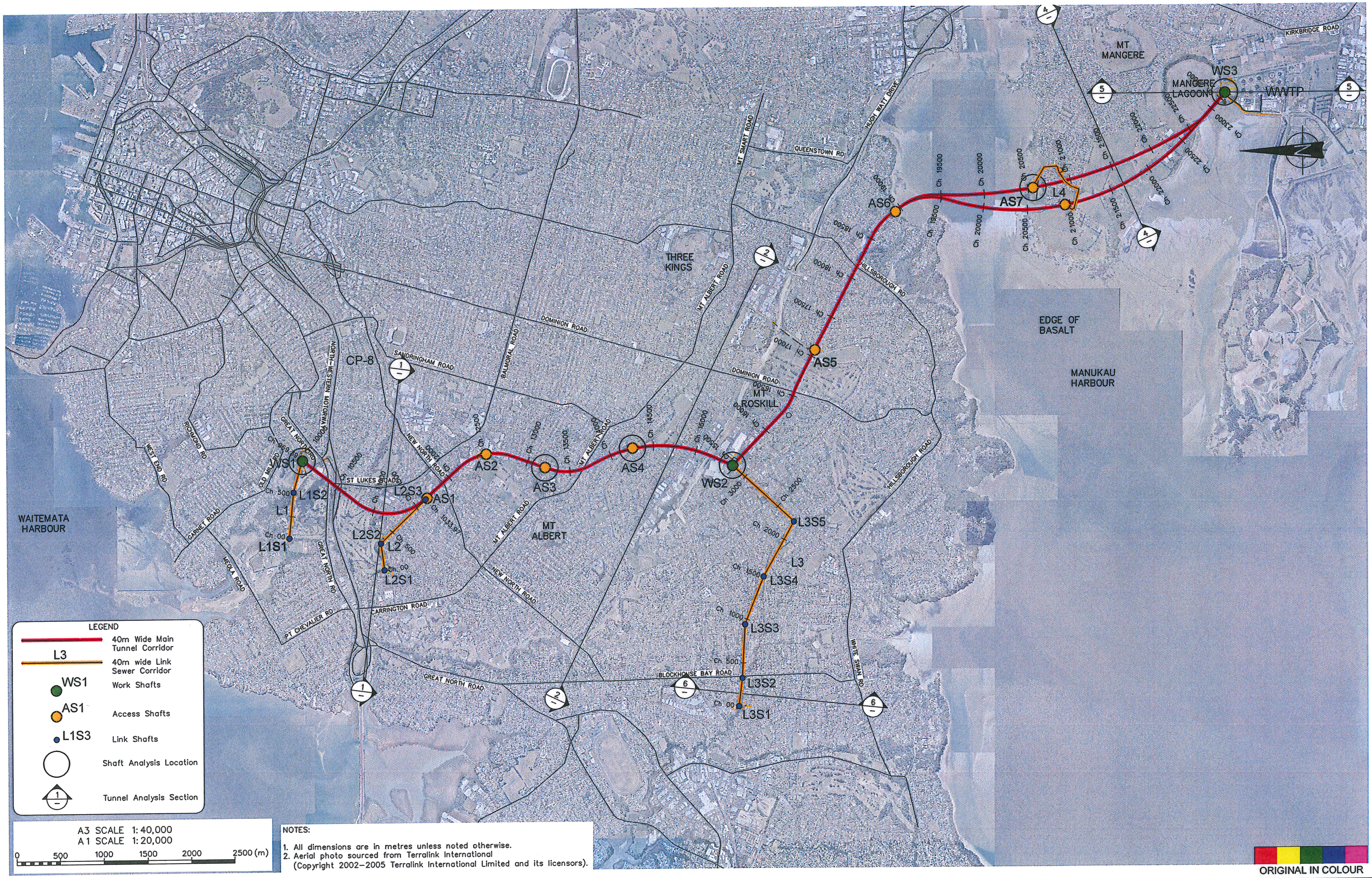
Appendix A: Drawings

- Figures A1 to A9



NOTES:

1. All dimensions are in metres unless noted otherwise.
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APP'D P.DIR		24/7/12	ASSET MANAGER
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water Care services limited

CENTRAL INTERCEPTOR
GROUNDWATER REGIME AND SURFACE SETTLEMENT
TUNNEL SECTION & SHAFT ANALYSIS LOCATIONS

CAD FILE 26145.300-CI-FA1_FA4 DATE 1-Jun-12

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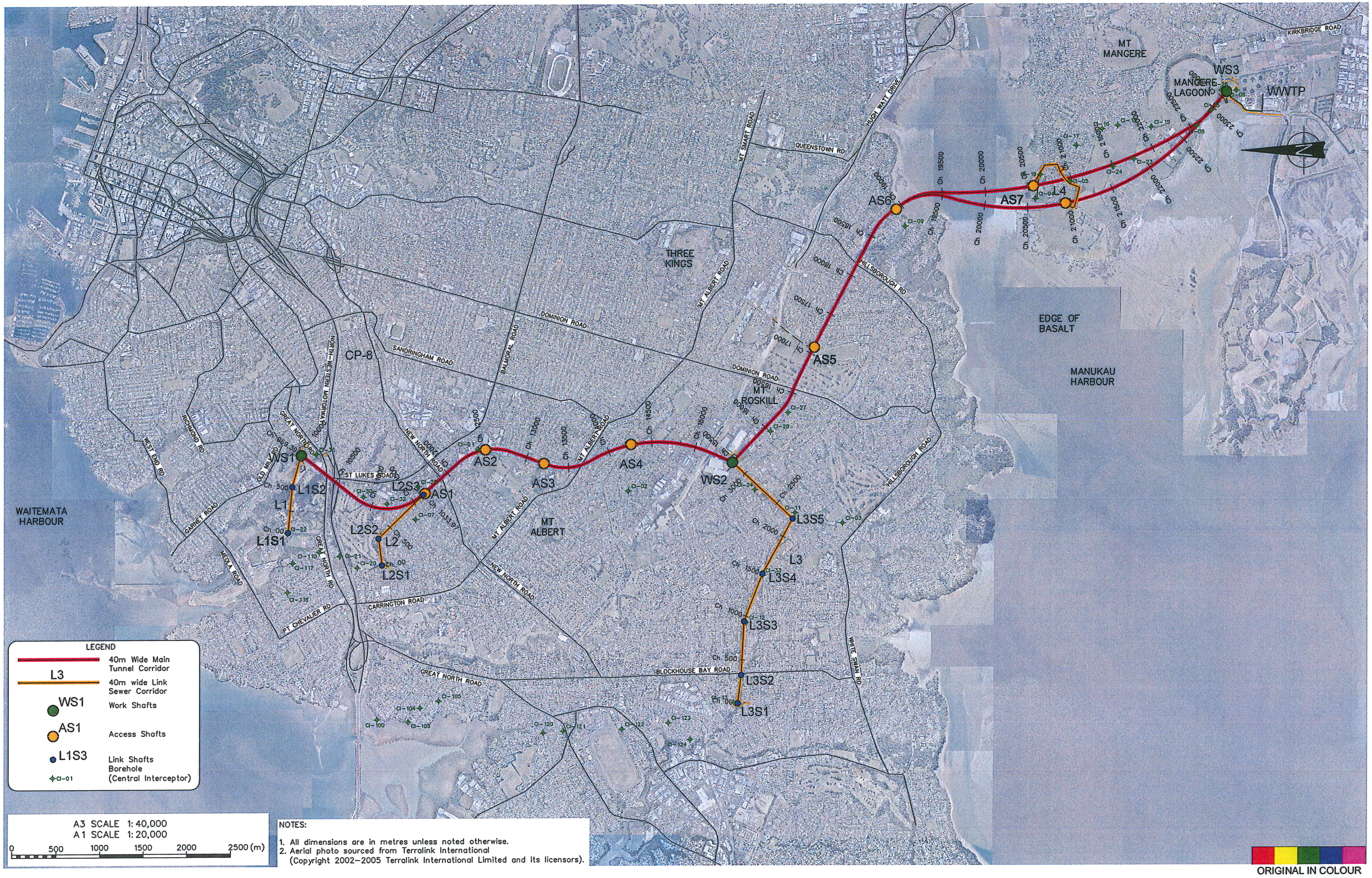
FIGURE A1	



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OPERATIONS
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CENTRAL INTERCEPTOR AND ASSOCIATED WORKS
GEOTECHNICAL CONDITIONS
PROJECT SPECIFIC INVESTIGATIONS

CAD FILE 26145.300-CI-FA1_FA4 DATE 1-Jun-12

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FIGURE A2	

LEGEND

- 40m Wide Main Tunnel Corridor
- L3 — 40m wide Link Sewer Corridor
- WS1 — Work Shafts
- AS1 — Access Shafts
- L1S3 — Link Shafts
- CI–01 — Borehole (Central Interceptor)
- Relevant Historical Borehole
- BH1 — Relevant Historical Borehole with Ground Water Monitoring
- Existing ground profile
- Tunnel crown
- Tunnel invert
- ? — Inferred geological boundary
- Envelope 5m above tunnel invert level
- Envelope 15m below tunnel invert level
- Fault (inferred)
- Borehole
- Piezometer screen and water level
- Vibrating wire piezometer and water level
- Artesian water

- Fill

TAURANGA GROUP (HOLOCENE)
- Estuarine muds and sand
- Undifferentiated Tauranga Group alluvium plus miscellaneous fill

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

TAURANGA GROUP (PLIOCENE – PLEISTOCENE)
- Puketoka Formation Upper (fine–grained) facies
- Puketoka Formation Lower (coarse grained) facies
- Kaawa Formation

WAITEMATA GROUP (MIOCENE)
- Undifferentiated
- Volcanic–poor flysch facies (East Coast Bays Formation)
- Muddy flysch facies
- Mixed volcanic–rich and volcanic–poor flysch facies
- Volcanic–rich to debris flow facies Including Parnell Grit

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CENTRAL INTERCEPTOR AND ASSOCIATED WORKS
GEOTECHNICAL CONDITIONS
GEOLOGY LEGEND

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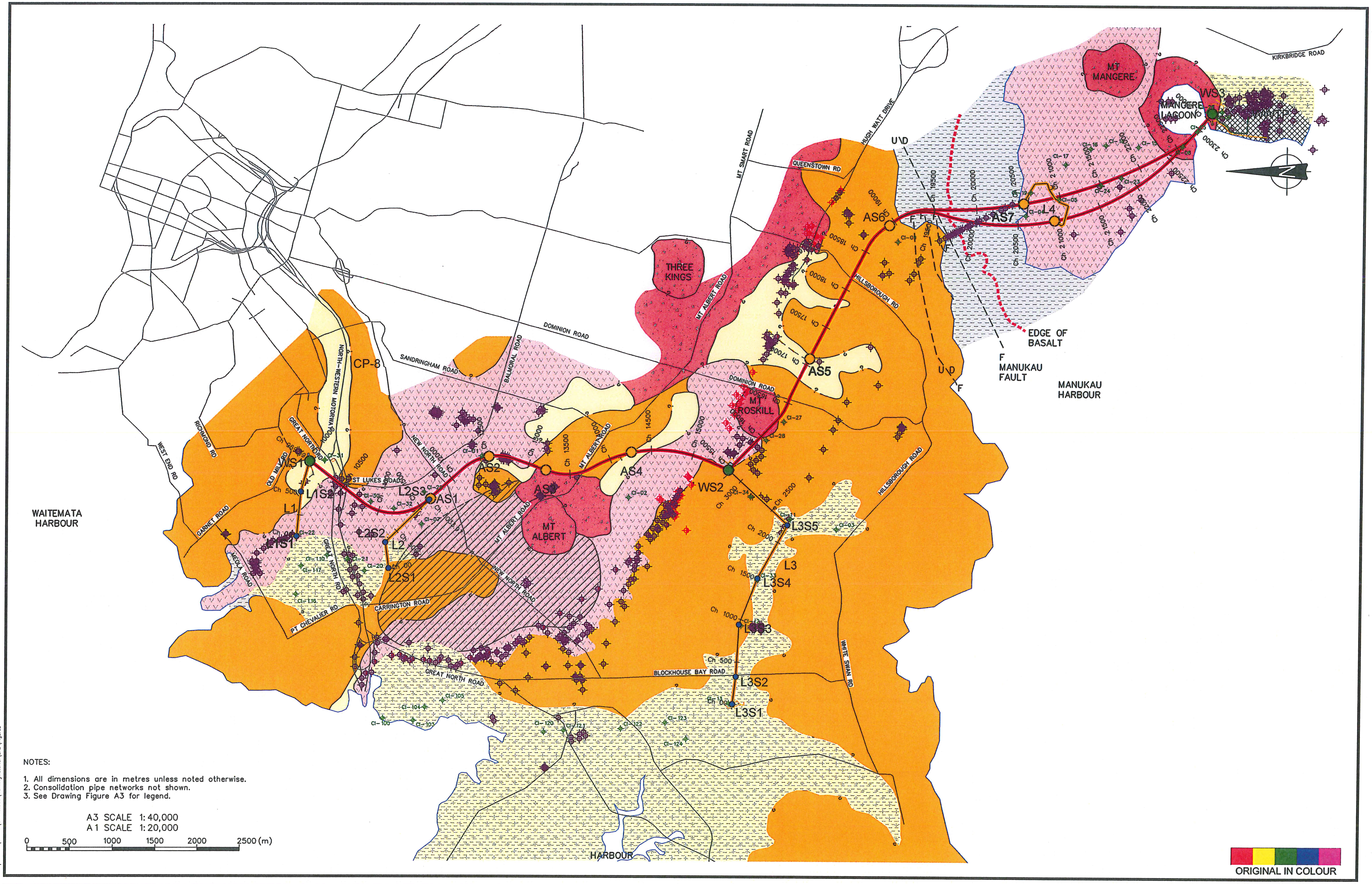

NOTES:

1. All dimensions are in metres unless noted otherwise.
2. Consolidation pipe networks not shown.
3. See Drawing Figure A3 for legend.

A3 SCALE 1:40,000
A1 SCALE 1:20,000

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					REV'D P.MGR		7/12
					APP'D P.DIR		26/7/12

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CENTRAL INTERCEPTOR AND ASSOCIATED WORKS
GEOTECHNICAL CONDITIONS
GEOLOGICAL MAP - RELEVANT BOREHOLE INVESTIGATIONS

CAD FILE 26145.300-CI-FA1_FA4 DATE 1-Jun-12

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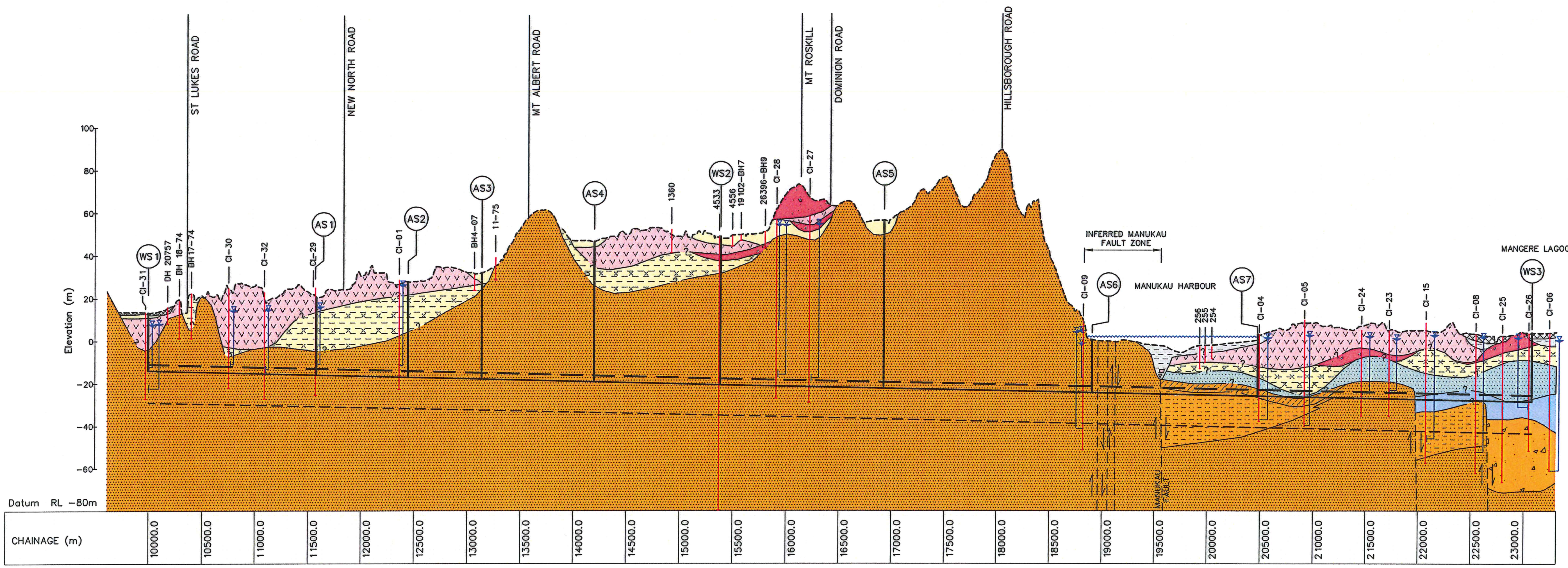
DRAWING No. FIGURE A4

ISSUE



A3 SCALE 1: 40000 (H)
A1 SCALE 1: 20000

0 0.5 1.0 1.5 2.0 2.5 (km)

A3 SCALE 1: 2000 (V)
A1 SCALE 1: 1000

0 10 20 30 40 60 80 100 (m)

NOT TO BE SCALED FROM AT ANY OTHER PAPER SIZE OTHER THAN THE ONES STATED.

NOTES:

1. All dimensions are in metres unless noted otherwise.
2. The ground surface topography is based on Lidar data supplied by ALGGi.
3. Only these piezometers located near to tunnel level are shown on this drawing.
3. See Figure A3 for legend

ORIGINAL IN COLOUR

ISSUE	DATE	AMENDMENT	BY	APPD.

	BY	DATE
DESIGNED	RPG	24/7/12
DES. CHECKED		
DRAWN	JATG	
DWG. CHECKED		7/12
REV'D P.MGR		
APP'D P.DIR		24/7/12

OPERATIONS

ASSET MANAGER

water Care
services limited

CENTRAL INTERCEPTOR AND ASSOCIATED WORKS
GEOTECHNICAL CONDITIONS
GEOLOGICAL SECTION, MAIN TUNNEL

CAD FILE 26145.300-CI-FA5_FA10 DATE 23-Jun-11

ORIGINAL SCALE A1	CONTRACT No.
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FIGURE A5	

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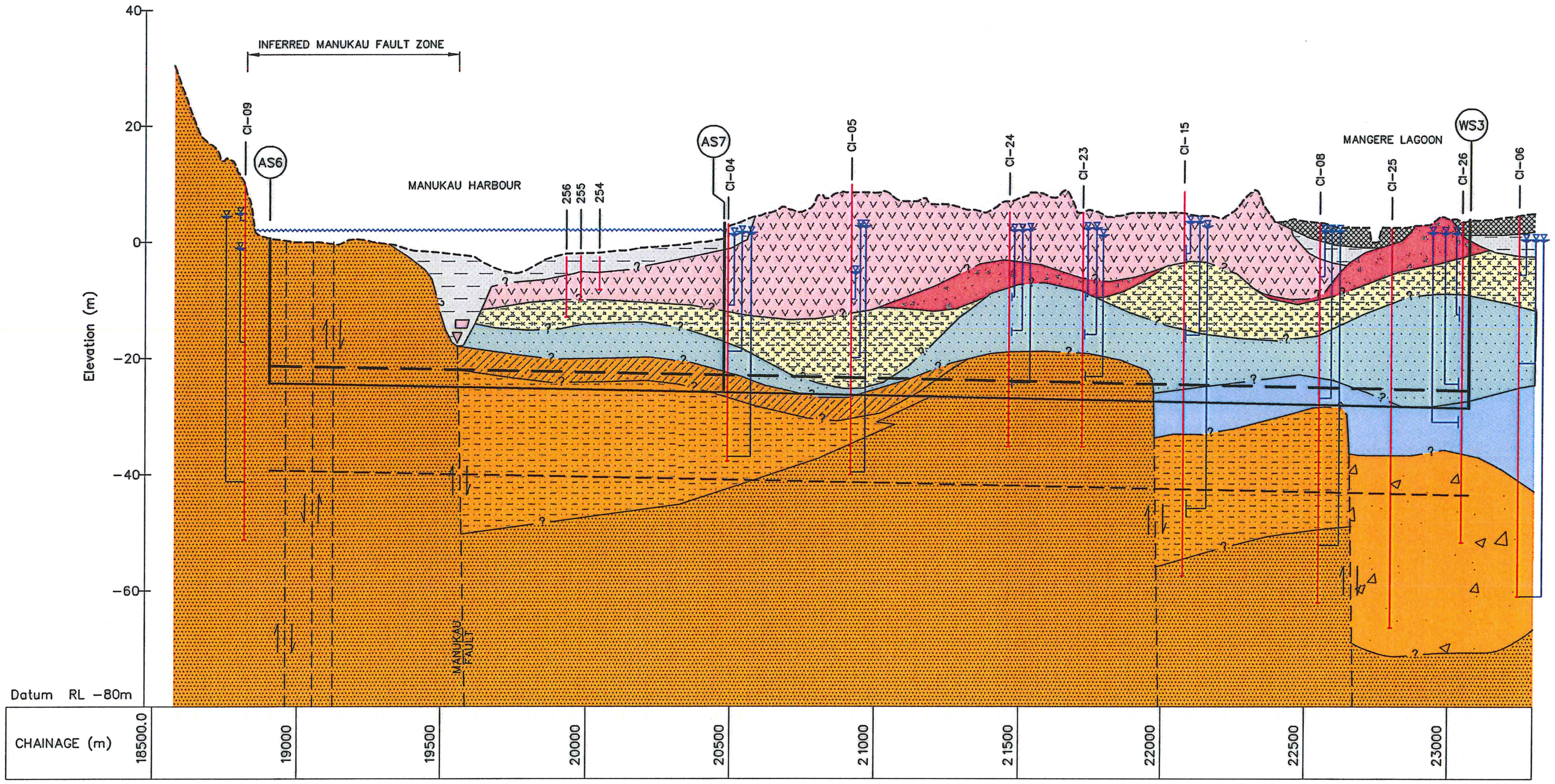

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A1 SCALE 1: 10000
0 0.2 0.4 0.6 0.8 1.0 (km)

A3 SCALE 1: 1000 (V)
A1 SCALE 1: 500
0 10 20 30 40 50 (m)

NOTES:

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3. Only these piezometers located near to tunnel level are shown on this drawing.
3. See Figure A3 for legend

ORIGINAL IN COLOUR

ISSUE	DATE	AMENDMENT	BY	APPD.

DESIGNED	RRG	24/7/12
DES. CHECKED	KJF	24/7/12
DRAWN	JATG	
DWG. CHECKED	KAS	7/12
REV'D P.MGR	GRT	7/12
APP'D P.DIR		24/7/12
	BY	DATE

OPERATIONS

ASSET MANAGER

water Care services limited

CENTRAL INTERCEPTOR AND ASSOCIATED WORKS
GEOTECHNICAL CONDITIONS
GEOLOGICAL SECTION, MANGERE BRIDGE

CAD FILE 26145.300-CI-FA5_FA10 DATE 23-Jun-11

ORIGINAL SCALE A1: AS SHOWN

CONTRACT No.: –

DRAWING No.: FIGURE A6

ISSUE

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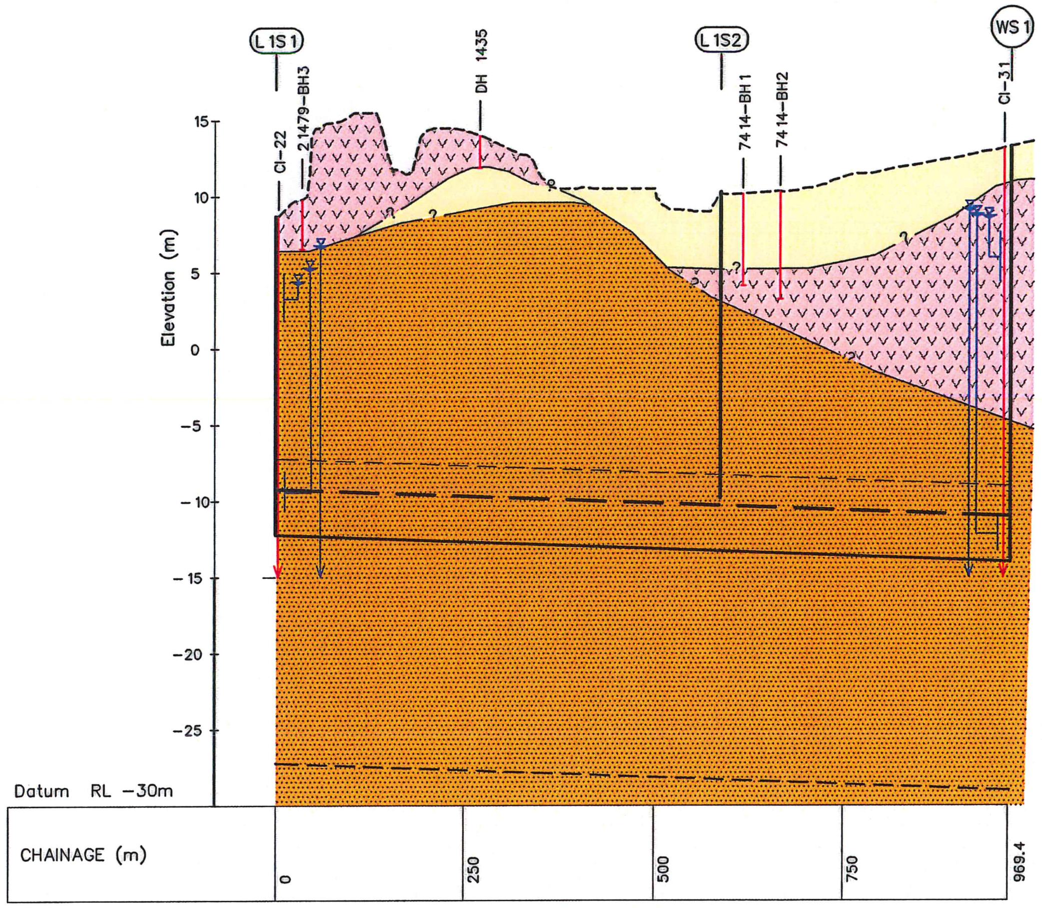
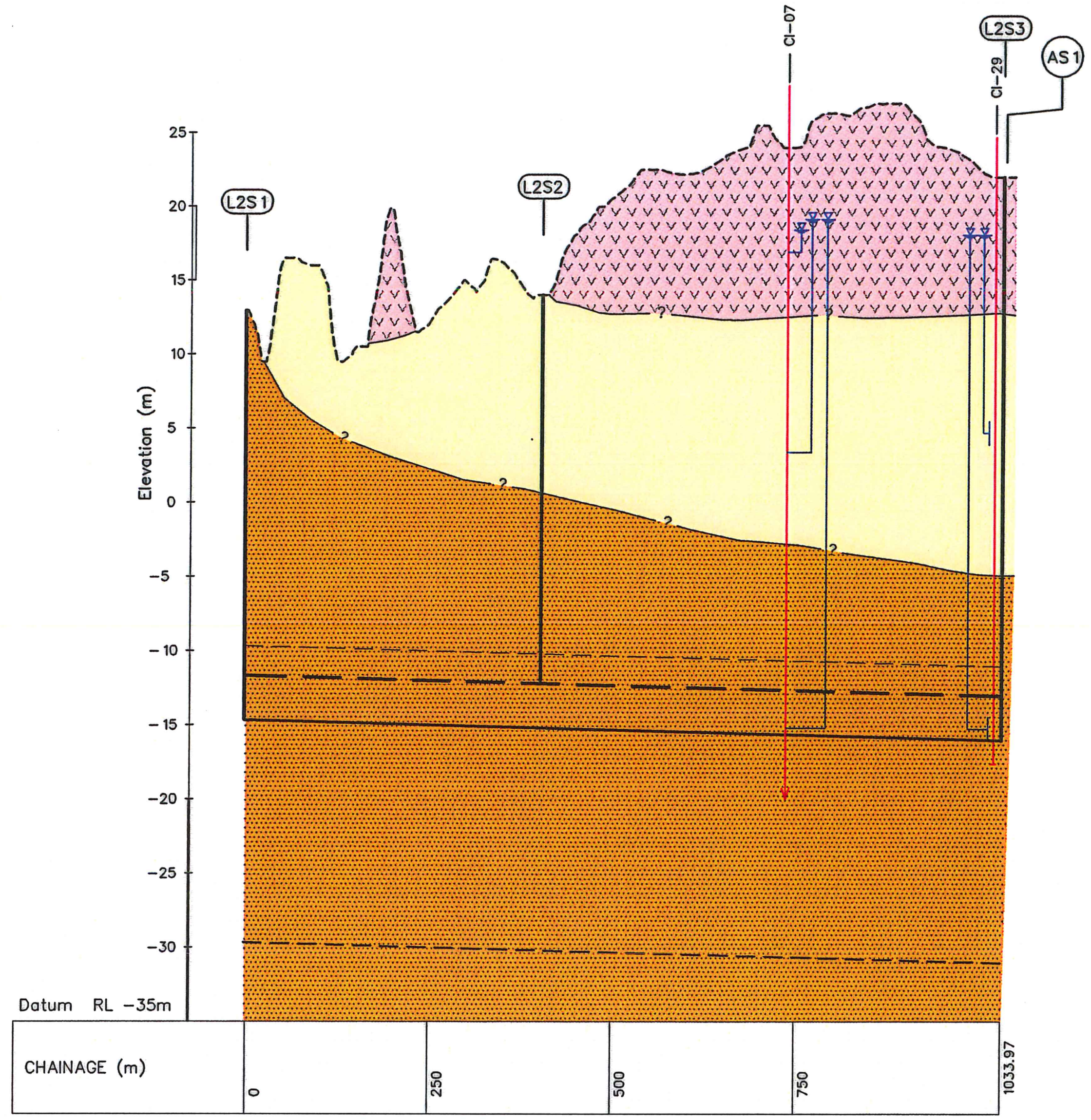


LINK SEWER LS1



LINK SEWER LS2

NOTES:

1. All dimensions are in metres unless noted otherwise.
2. The ground surface topography is based on Lidar data supplied by ALGGi.
3. Only these piezometers located near to tunnel level are shown on this drawing.
3. See Figure A3 for legend

A3 SCALE 1:500 (V)
A1 SCALE 1:250
0 10 20 30 (m)

A3 SCALE 1:10000 (H)
A1 SCALE 1:5000
0 100 200 300 400 500 (m)

ORIGINAL IN COLOUR

ISSUE	DATE	AMENDMENT	BY	APPD.

	BY	DATE
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DES. CHECKED	KJH	24/7/12
DRAWN	JATG	
DWG. CHECKED	RGS	7/12
REV'D P.MGR	GRT	27/7/12
APP'D P.DIR	RDJ	24/7/12

OPERATIONS

ASSET MANAGER

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CENTRAL INTERCEPTOR AND ASSOCIATED WORKS
GEOTECHNICAL CONDITIONS
GEOLOGICAL SECTIONS, LINK SEWERS LS1 AND LS2

CAD FILE 26145.300–CI–FA5_FA10 DATE 23–Jun–11

ORIGINAL SCALE A1	CONTRACT No.
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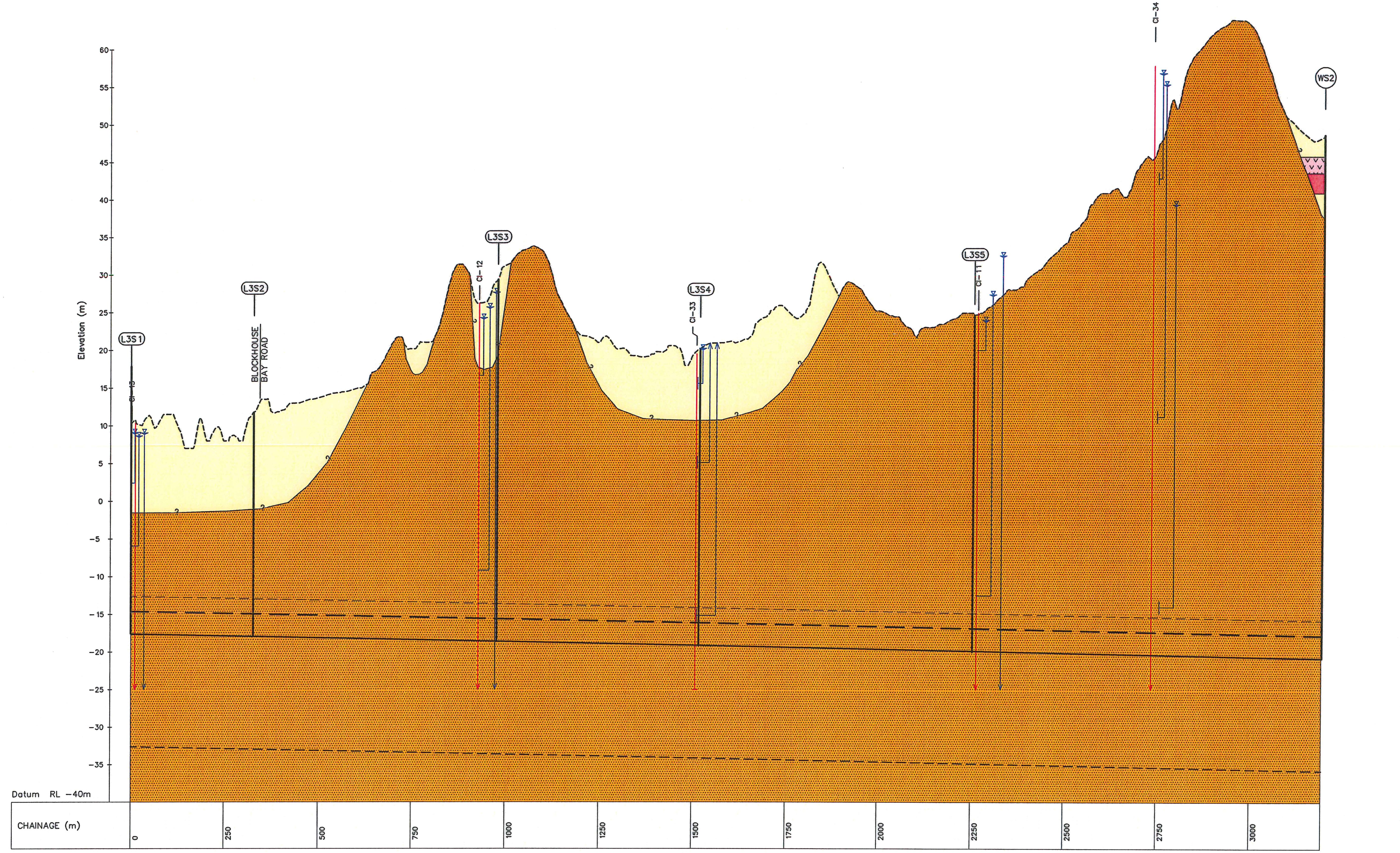
DRAWING No.	ISSUE
FIGURE A7	



LINK SEWER LS3

A3 SCALE 1: 10000 (H)
A1 SCALE 1: 5000
0 100 200 300 400 500 (m)

A3 SCALE 1: 500 (V)
A1 SCALE 1: 250
0 10 20 30 (m)

NOTES:

1. All dimensions are in metres unless noted otherwise.
2. The ground surface topography is based on Lidar data supplied by ALGGI.
3. Only these piezometers located near to tunnel level are shown on this drawing.
3. See Figrue A3 for legend

ORIGINAL IN COLOUR

ISSUE	DATE	AMENDMENT	BY	APPD.

DESIGNED	RRG	24/7/12
DES. CHECKED	KJH	24/7/12
DRAWN	JATG	
DWG. CHECKED	KSS	7/12
REV'D P.MGR	SPFT	07/12
APP'D P.DIR	TDH	24/7/11
	BY	DATE

OPERATIONS

ASSET MANAGER

water Care services limited

CENTRAL INTERCEPTOR AND ASSOCIATED WORKS
GEOTECHNICAL CONDITIONS
GEOLOGICAL SECTION, LINK SEWER LS3

CAD FILE 26145.300-CI-FA5_FA10 DATE 23-Jun-11

ORIGINAL SCALE A1	CONTRACT No.
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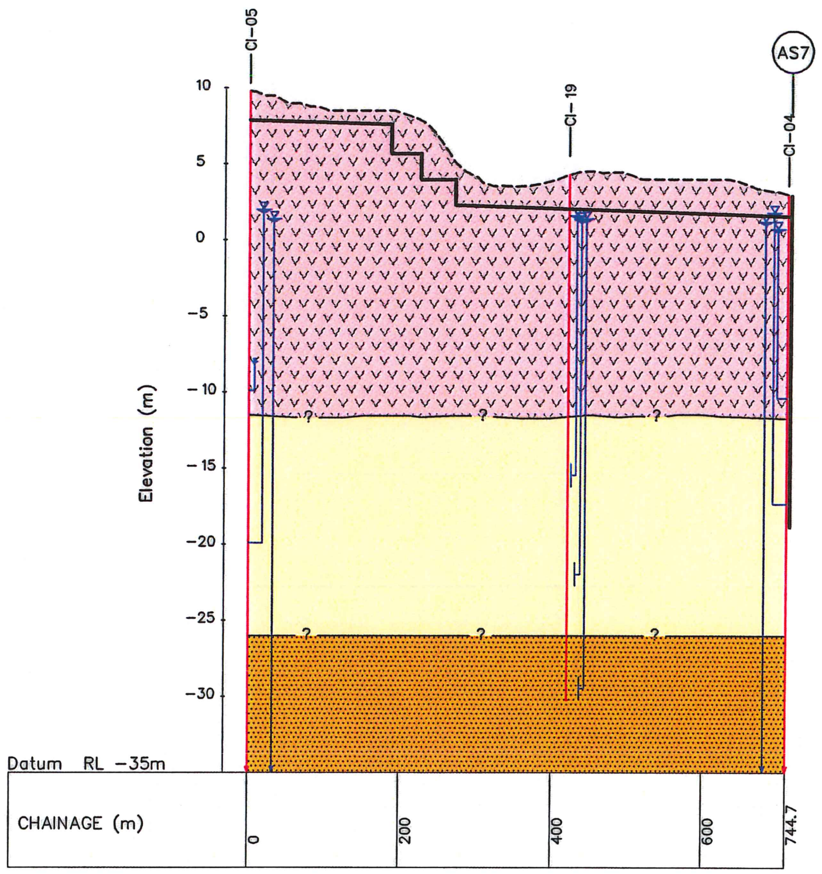
DRAWING No.	ISSUE
FIGURE A8	



LINK SEWER LS4

A3 SCALE 1:500 (V)
A1 SCALE 1:250
0 10 20 30 (m)

A3 SCALE 1: 10000 (H)
A1 SCALE 1:5000
0 100 200 300 400 500 (m)

NOTES:

1. All dimensions are in metres unless noted otherwise.
2. The ground surface topography is based on Lidar data supplied by ALGGI.
3. Only these piezometers located near to tunnel level are shown on this drawing.
3. See Figure A3 for legend

ORIGINAL IN COLOUR

Plot Date: 24-Jul-12 12:58 p.m.
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ISSUE	DATE	AMENDMENT	BY	APPD.

	BY	DATE
DESIGNED	RRG	24/7/12
DES. CHECKED	RJH	24/7/12
DRAWN	JATG	
DWG. CHECKED	KBS	7/12
REV'D P.MGR	GRT	07/12
APP'D P.DIR		24/7/12

OPERATIONS

ASSET MANAGER

water Care services limited

CENTRAL INTERCEPTOR AND ASSOCIATED WORKS
GEOTECHNICAL CONDITIONS
GEOLOGICAL SECTION, LINK SEWER LS4

CAD FILE 26145.300-CI-FA5_FA10 DATE 23-Jun-11

ORIGINAL SCALE A1	CONTRACT No.
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FIGURE A9	

Appendix B: Stiffness and Permeability Data Discussion

Stiffness Data

As part of the various stages of ground investigation for the CI project, numerous laboratory and field tests have been undertaken on the various geological units. This testing, together with experience on other Auckland projects form the basis for the stiffness parameters adopted in the settlement analysis. A discussion of the available testing data is given below.

Upper Puketoka Formation

A total of 27 Oedometer tests have been undertaken on UPF samples from the CI investigations. Figure 5 Mv versus applied vertical stress – all data below plots M_v against stress range, and Figure 6 Mv for selected stress range (initial effective stress plus 50kPa)plots M_v vs. depth. The M_v vs. depth plot contains a single data point from each Oedometer test, chosen to represent the likely stress range that would result from groundwater drawdown for each sample. This plot clearly indicates an increase in stiffness) with depth. Above 12m depth a mean M ($1/M_v$) of 6MPa, with a mean M of 16MPa below 12m depth.

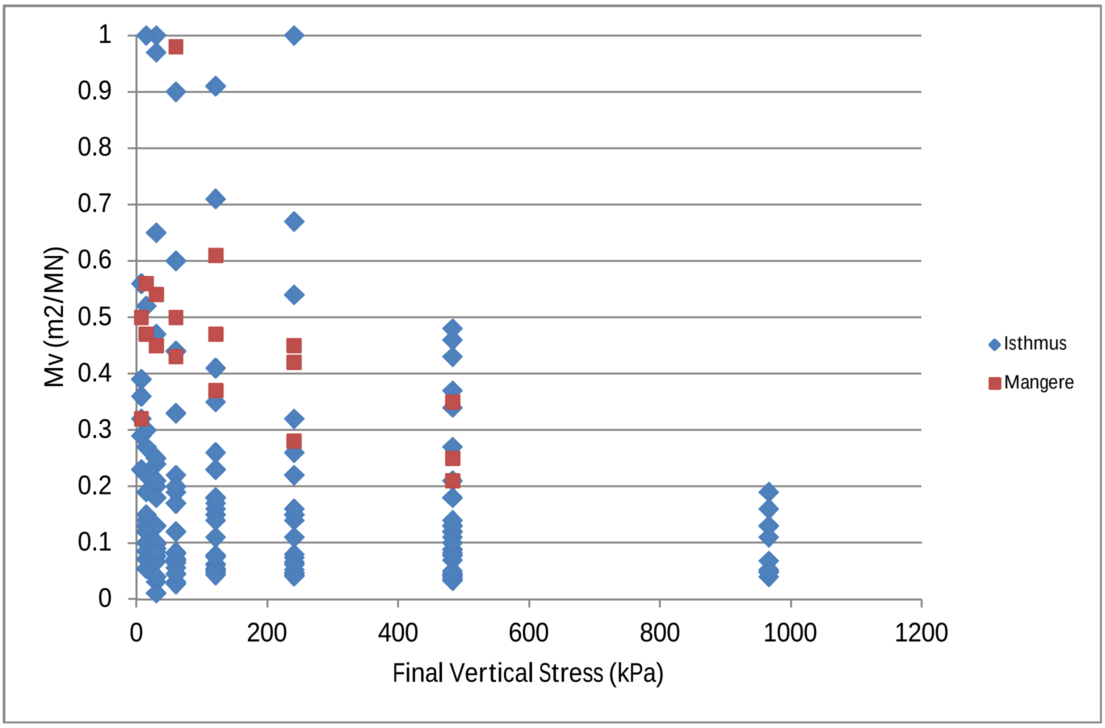


Figure 5 Mv versus applied vertical stress – all data

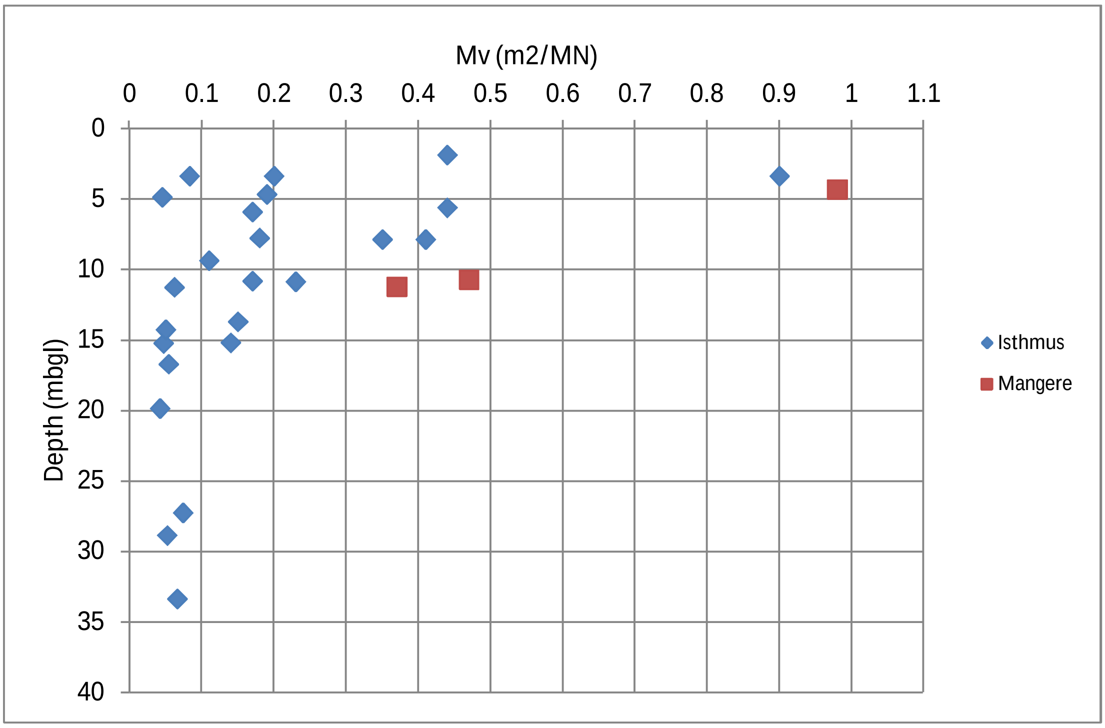


Figure 6 Mv for selected stress range (initial effective stress plus 50kPa)

One pressuremeter test was undertaken on a UPF sample at 15m depth in CI 15A. This gave an initial elastic modulus of 32MPa.

Two unconsolidated undrained triaxial tests have been undertaken with a mean initial elastic moduli of 9MPa.

Eight consolidated undrained triaxial tests have been undertaken with a mean initial elastic modulus of approximately 20MPa below 100kPa initial confining stress and approximately 60MPa between 100kPa and 200kPa confining stress (plot shows consistent increase in stiffness with increasing confining stress.)

Lower Puketoka Formation

Three pressuremeter tests have been undertaken on the LPF with results of 20MPa, 38MPa and 106MPa. No laboratory testing has been undertaken

Marine sediments

A single consolidated undrained triaxial test has been carried out on Marine sediments, with an initial elastic undrained modulus of 4MPa to 20MPa for confining stresses of up to 100kPa.

Auckland Volcanic Field (AVF) Basalt

Two dilatometer tests have been undertaken on basalt, with initial elastic moduli of approximately 2GPa to 3GPa.

UCS Testing was undertaken without strain measurement on 13 samples with compressive strengths ranging from approximately 20MPa to 200MPa.

AVF Tuff

No testing has specifically been undertaken on the Tuff for CI

Kaawa Formation

No testing has been specifically undertaken on the Kaawa Sands. SPT 'N' values are generally 50+

Residually to Highly Weathered East Coast Bays Formation (ECBF)

A single oedometer test was undertaken on weathered ECBF, with constrained moduli of approximately 10MPa for the relevant stress range.

Two undrained unconsolidated triaxial tests have been undertaken with initial elastic moduli of 21MPa and 115MPa.

One consolidated undrained triaxial test has been undertaken with initial elastic moduli of approximately 20MPa to 40MPa for confining stresses up to 100kPa.

Three pressuremeter tests were undertaken on weathered ECBF with initial elastic moduli of approximately 40MPa to 90MPa

ECBF rock

9 dilatometer tests have been undertaken on ECBF rock with an initial elastic moduli ranging from approximately 250 to 800MPa averaging 600MPa.

Over a hundred UCS tests were undertaken on the ECBF with strain measured. Modulus values have a mean value of 500MPa and a median value of 365MPa. Compressive strengths have a mean approximately 3MPa. Figure 7 below gives a plot of deformation moduli of ECBF Rock from various sources including UCS tests, dilatometer tests and UU Triaxial tests. Figure 8 plots the UCS test results themselves.

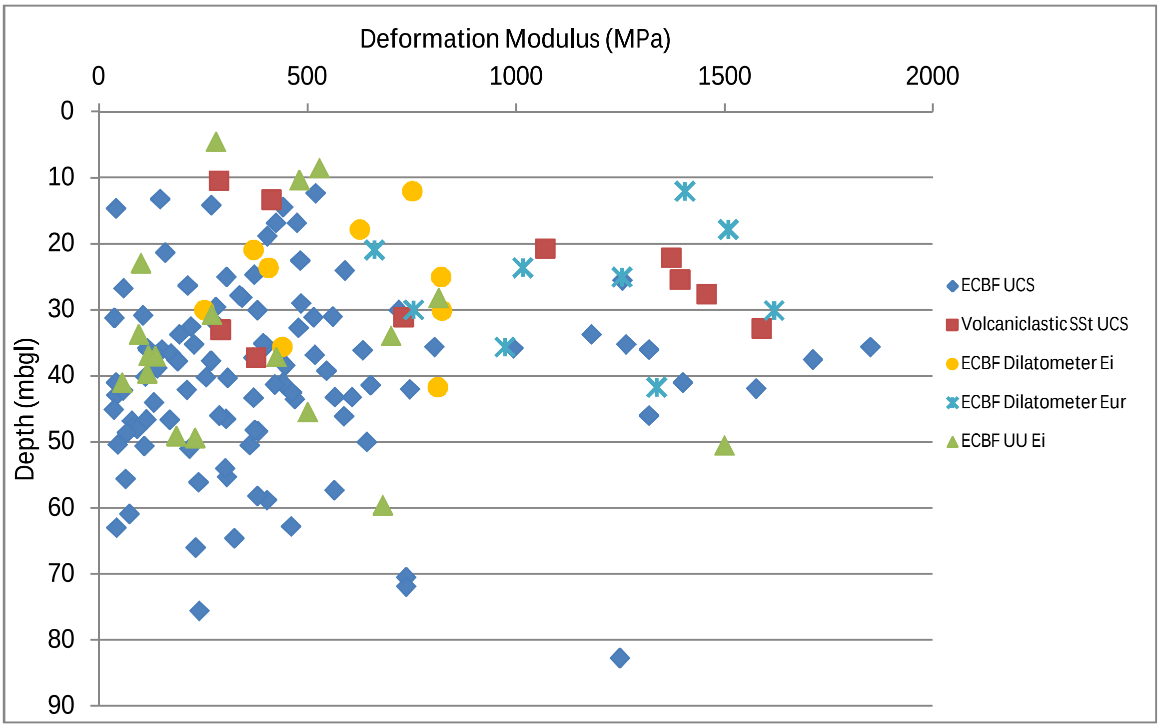


Figure 7 Deformation Moduli, Various Sources - ECBF Rock

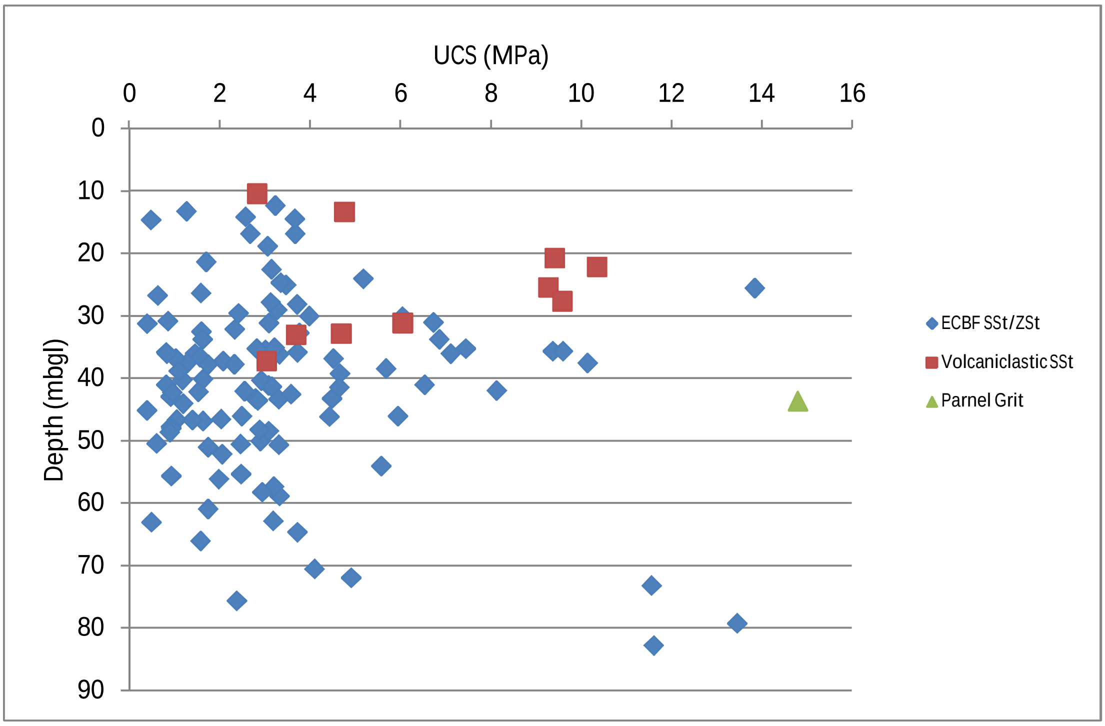


Figure 8 UCS Test results - ECBF Rock

Table B1 Summary of material stiffness parameters

Geological unit	Deformation Modulus (MPa)			Comment
	Assessed minimum	Assessed mean	Assessed maximum	
Basalt				Considered incompressible in terms of this study
Tuff	8	12	20	
Estuarine sediments	1	2	10	
TGA / UPF above 12m depth	2.5	6	20	
UPF below 12m depth	10	15	40	
Lower Puketoka Formation	10	20	50	
Kaawa Sands	50	100	150	
Weathered ECBF	4	15	40	
ECBF	150	500	1000	

Hydraulic Conductivity Data

As part of the various stages of ground investigation for the CI project, numerous field permeability tests (including packer tests and variable head tests) have been undertaken on the various geological units. This testing, together with experience on other Auckland projects form the basis for the hydraulic conductivity parameters adopted for the seepage analysis. A discussion of the available testing data is given below. Figure 9 shows permeability tests results for all the materials tested.

Tauranga Group Alluvium (TGA) and Upper (fine grained) Puketoka Formation (PF)

24 variable head permeability tests have been undertaken in UPF and TGA materials. The test results have a geometric mean of $4x10^{-7}$m/s and a median of $3x10^{-7}$m/s.

Lower (coarse grained) Puketoka Formation (LPF)

12 variable head permeability tests have been undertaken in LPF materials. The test results have a geometric mean of $2x10^{-6}$m/s and a median of $8x10^{-7}$m/s.

Marine sediments

No insitu test have been undertaken on marine sediments

Auckland Volcanic Field (AVF) Basalt

A single packer test has been undertaken in Basalt with a result of $2x10^{-6}$m/s. As well as experience from other civil engineering projects in Auckland, use has been made of the Global Aquifer Study (GAS) undertaken for the then Auckland City Council and Metrowater (Pattle Delamore Partners Ltd, 2005). In this study typical basalt permeabilities of $1x10^{-3}$m/s to $1x10^{-4}$m/s were adopted, based on the calibration of a regional 3-D groundwater model, and results from a number of pumping tests undertaken in the basalt.

AVF Tuff

No insitu tests have been undertaken on tuff deposits.

Kaawa Formation

No insitu tests have been undertaken on Kaawa Formation deposits

Residually to Highly Weathered East Coast Bays Formation (ECBF)

Three variable permeability tests have been carried out on weathered ECBF, with values ranging from $2x10^{-6}$m/s to $3x10^{-4}$m/s

ECBF rock

49 packer tests and 2 variable head permeability tests have been undertaken on ECBF rock. The test results have a geometric mean of $5x10^{-8}$m/s and a median of $8x10^{-8}$m/s.

The vertical permeability of the ECBF rock has not been directly measured, but is generally taken as one to two orders of magnitude lower than the horizontal permeability. The anisotropy is caused by the bedding and the contract in permeability between sandier and siltier beds.

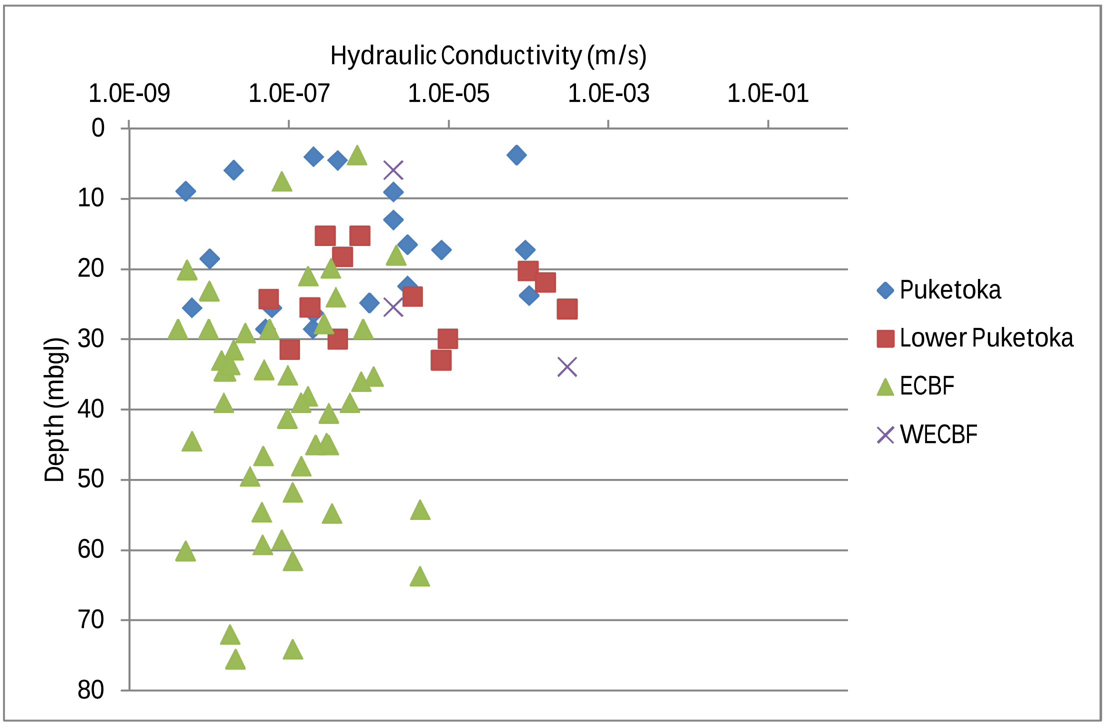


Figure 9 Permeability test data – all materials

Table B2 Summary of material permeability parameters

Geological unit	Permeability			Comment
	Assessed minimum	Assessed mean	Assessed maximum	
Basalt	1E-6	5E-5	1E-3	
Tuff	1E-7	1E-5	1E-3	
Estuarine Sediments	1E-9	2E-7	1E-6	
Upper Puketoka Formation/ Tauranga Group Alluvium	2E-8	2E-7	2E-6	
Lower Puketoka Formation	2E-7	2E-6	2E-5	
Kaawa Sands	1E-7	1E-6	1E-4	
Weathered ECBF	2E-8	2E-7	2E-6	
ECBF	K_h =2E-8 K_v/K_h=0.1	K_h =2E-7 K_v/K_h=0.1	K_h =2E-6 K_v/K_h=0.1	
Fractured ECBF	NA	$5x10^{-4}$	NA	

Unsaturated hydraulic conductivity parameters

Unsaturated hydraulic conductivity and volumetric water content functions have been estimated using in-built estimation methods in the groundwater modelling package Seep/W. Unsaturated volumetric water content functions have been estimated using the sample functions most appropriate for the material being modelled. Saturated moisture content values have been estimated based on typical values for the various materials. Residual water content values have been assumed as 0.05 m^3/m^3 for all materials. Unsaturated hydraulic conductivity functions have been estimated using the Fredlund & Xing method. Table B3 below summarises the main input parameters used for each material (note the moisture content values are defined in terms of volume-per-volume).

Table B3 Summary of input values used for unsaturated hydraulic conductivity parameter estimation

Geological unit	Saturated moisture content (m^3/m^3)	Residual moisture content (m^3/m^3)	Volumetric water content function type	Hydraulic conductivity function estimation method
Basalt	0.12	0.05	Gravel	Fredlund & Xing
Tuff	0.45	0.05	Clayey silt	Fredlund & Xing
Estuarine Sediments	0.7	0.05	Silt	Fredlund & Xing
Upper Puketoka Formation/ Tauranga Group Alluvium	0.45	0.05	Silt	Fredlund & Xing
Lower Puketoka Formation	0.45	0.05	Silty sand	Fredlund & Xing
Kaawa Sands	0.5	0.05	Sand	Fredlund & Xing
Weathered ECBF	0.45	0.05	Clayey silt	Fredlund & Xing
ECBF	0.3	0.05	Silty sand	Fredlund & Xing
Fractured ECBF	0.3	0.05	Silty sand	Fredlund & Xing

Appendix C: Groundwater response to tunnels – experience in Auckland

Vector Tunnel Construction

The Vector Tunnel was constructed between 1997 and 2002 in Auckland City to provide access to the CBD for additional reticulated electricity supply. The tunnel connects supply cables at Hobson Street in the city, to a substation at Penrose. The tunnel is approximately 9km in length, and typically around 3.5m in diameter.

The Vector Tunnel was constructed within broadly similar geology to the proposed Central Interceptor tunnels. The tunnel size, and proposed construction methodology are also broadly similar and as such, the Vector Tunnel provides a good reference model for the main CI tunnel, and the link tunnels to a lesser extent.

Both TBM and road header excavation techniques were used in the construction of the vector tunnel. TBM was used where the alignment allowed for it, whereas a road-header was used where alignment radii were too tight for TBM.

The vector tunnel was excavated by road header and TBM between 20m and 100m below ground level, and substantially below sea level. The tunnel was excavated in ECBF overlain variously by compressible Tauranga Group deposits and more recent volcanic deposits of ash, tuff and basalt.

The road header excavated section of the tunnel was not lined until some 12 months after excavation. In the TBM section, lining installation immediately followed excavation however post grouting behind the segments (to provide for water tightness) was undertaken some time later - in most areas up to 18 months later.

The maximum groundwater drawdown during the construction period was 33m, with a mean drawdown across all piezometers of 12 m. The measured groundwater effects extended laterally at tunnel level to approximately 150 m from the tunnel, but measureable surface settlement did not occur in all areas. Close to tunnel level, groundwater levels measured in the ECBF were close to tunnel invert level, however where basalt overlay the alignment, there was no discernible response in the basalt. Measured settlements were in the order of a maximum of 20 mm, with a mean of 7 mm, with the exception of a single location where settlement of 38 mm was observed.

Time histories available for the location of maximum recorded settlement (above a paleo-valley in-filled with Tauranga Group material overlain with basalt), surface settlement reached 50% of its final value in approximately 50 days, and 75% in 160 days. Groundwater level data in the basalt indicate that groundwater levels were not affected by depressurisation in the ECBF.

The rate of groundwater inflow has historically been measured at intervals along the Vector Tunnel, and compared to groundwater level monitoring information. A study of this information provides real, verifiable data to assist in the modelling of potential effects of the tunnel on groundwater for this project, and is included in detail in Appendix D.

In summary, the following was concluded from the study.

There is a greater risk of groundwater effect on strata overlying the ECBF where;

- ECBF cover to the tunnel crown is reduced
- There is a thin, or no weathering surface on the ECBF
- ECBF strata are inclined and coarse sandstone beds may connect with surface deposits.
- The ECBF is coarse and thickly bedded.

Hobson Bay Sewer Tunnel

Watercare completed construction of the Hobson Bay Sewer Tunnel in late 2009 to replace the aging pipeline that crossed Hobson Bay.

The nominally 3.5 m diameter tunnel was excavated using an EPB capable TBM from Parnell to Orakei, beneath Hobson Bay and the residential development of Orakei ridge.

Excavations were primarily within ECBF overlain variously by Tauranga Group deposits and more recent volcanic deposits of ash, tuff and basalt. Beneath the bay, the tunnel passed beneath soft marine deposits.

Construction monitoring included piezometers to measure groundwater effects from tunnel excavation and surface markers to measure associated surface deformation. The maximum measured groundwater drawdown from construction was 17 m, and mean 2.5 m. The 17 m maximum related to a major shaft construction at Orakei, otherwise general tunnel construction resulted in a maximum of 6 m drawdown and a mean of less than 2 m. Groundwater drawdown effects were detected at a maximum of 250 m from the tunnel centreline in the ECBF.

Maximum recorded surface deformation was approximately 30 mm at two discrete locations. These isolated settlement measurements are out of context with surrounding marks which record significantly lower settlements. It is considered that these readings represent measurement error rather than actual behaviour in response to tunnelling. Otherwise settlements had a mean of less than 10 mm.

Prior to consenting, engineering reports (Tonkin & Taylor 2004) predicted surface settlement of typically less than 25mm with potential (albeit low) for isolated settlement of up to 50 mm for the construction methodology that was ultimately utilised.

Rosedale tunnel

Constructed from 2009 to 2010, the Rosedale tunnel provides the Rosedale Waste Water Treatment Plant with an additional offshore outfall for treated effluent.

Like the Hobson Bay project, the nominally 2.8 m diameter tunnel was excavated using an EPB capable tunnel boring machine. The tunnel route traverses from Rosedale treatment plant to Mairangi Bay, beneath commercial and residential areas. The EPBM was operated both in earth pressure balancing (closed) mode, and in open mode.

The tunnel was excavated primarily in ECBF, and extended beneath paleo-valleys filled with deep compressible Tauranga Group deposits. Groundwater conditions included up to 70 m of head at the tunnel level.

Construction monitoring was undertaken including piezometers and surface settlement markers. The maximum measured groundwater drawdown was 18 m, with a mean value of approximately 6 m. The maximum recorded surface settlement was 44 mm (which was out of context with surrounding marks, suggesting that it may reflect movement other than in response to tunnelling). The mean recorded settlement for the monitoring points across the project was approximately 4 mm, with a 95th percentile of approximately 10 mm.

Prior to consenting, engineering reports predicted the potential for settlement of up to 70-120 mm could occur in areas underlain with Tauranga Group deposits if excavation methodologies allowed groundwater drawdown for long periods of time.

Three Kings Quarry

While not a tunnelling project, the Three Kings Quarry Dewatering project is of relevance to this study in that it involved dewatering of Auckland Isthmus geology in a dense urban area.

To allow extraction of resource from below regional groundwater level, pumping has lowered groundwater some 23 m at the Three Kings Quarry. The open pit quarry extracts scoria (and a small quantity of basalt) from beneath two of the three main volcanic cones after which the area is named. During the eruptive event that formed the cones, volcanic rock intruded through a ridge in the Waitemata Group geology (ECBF). In lowering the groundwater level within the quarry, groundwater in the ECBF has also been lowered.

A network of groundwater bores and surface level marks surrounding the quarry monitor the response to the pumping. Surrounding the quarry, basement geology consists of ECBF overlain variously by Tauranga Group deposits and more recent volcanic deposits of ash, tuff and basalt.

Piezometers installed in the ECBF indicate that dewatering beneath the quarry influenced groundwater levels to a distance of approximately 750 m to 1,100 m from the quarry, with the maximum influence of 23 m at the quarry boundary.

The maximum recorded settlement was 36 mm, the median settlement 9 mm. Of the 222 marks that recorded settlement, 90% recorded 21 mm or less and 1% recorded settlement in excess of 28 mm.

Correlation of measured surface settlement to measured groundwater drawdown (in locations where geology was known to be residual ECBF overlying ECBF rock), identified as an upper bound the relationship of 1 mm settlement at the surface for each metre of groundwater drawdown measured at depth in ECBF.

Appendix D: Vector tunnel groundwater inflow study

Introduction

As part of the investigations and analyses carried out for the North Shore Sewer Tunnel (NSST), the groundwater inflows into the Vector Tunnel were investigated. The data collected from that study is also of relevance to the CI study.

The Vector Tunnel was constructed within broadly similar geology to the proposed NSST and the CI tunnels. The tunnel size, and proposed construction methodology are also broadly similar as such, the Vector Tunnel provides a working model of the NSST and CI tunnel to some extent.

The rate of groundwater inflow was measured at intervals along the Vector Tunnel, and compared to groundwater level monitoring information already available.

The study was carried out to provide real, verifiable data to assist in the modelling of potential effects of the tunnel on groundwater.

Background

The Vector Tunnel was constructed between 1997 and 2002 in Auckland City to provide access to the CBD for additional reticulated electricity supply. The tunnel connects supply cables at Hobson Street in the city, to a substation at Penrose. The tunnel is approximately 9km in length, and typically around 3.5 m in diameter.

The nominally 3m diameter tunnel was excavated by road header and tunnel boring machine (TBM) between 20 and 100 m below ground level, and substantially below sea level. The tunnel was excavated in East Coast Bays formation inter bedded sandstone and siltstone, overlain variously by compressible Pleistocene alluvial deposits and more recent volcanic deposits of ash, tuff and basalt.

The road header excavated section of the tunnel was not lined until some 12 months after excavation. In the excavated TBM section, lining followed excavation however post grouting behind the segments (to provide for water tightness) was undertaken some time later.

A concrete liner was installed over the entire length of the tunnel. The liner was typically 150mm thick, and constructed from pre-cast units. A small portion of the tunnel length was lined with a cast insitu liner. In general the liner construction was completed some 12 to 18 months following excavation of the tunnel.

The maximum groundwater draw down during construction was 33 m, with average across all piezometers of 12 m. Typically the groundwater effects extended laterally at tunnel level to approximately 150 m from the tunnel. The maximum surface settlement recorded was 38 mm (a single location). Typically however settlements were in the order of a maximum of 20 mm and averaged 7 mm.

A small channel formed in the base of the tunnel collects groundwater, condensation and other water. The channel flows into three sumps (at chainage 21,000, 24,000 and 29,000 approximately), where the water is collected and pumped out to stormwater systems or ground soakage. (Note the tunnel chainage is marked from the Hobson Street end to Penrose, commencing at chainage 20,000 at Hobson Street.)

Methodology for Groundwater Inflow Measurements

Weirs were used to measure the flow within the channel at coarse centres (500 m intervals) along the tunnel. The flow in the channel is assumed to have originated solely from groundwater inflows in these measurements.

It is expected that condensation and other sources of flow contribute a very small component of the flow in the channel. No attempt has been made to verify this assumption.

Sections of high inflow flow identified by these measurements were further investigated at closer centres in later measurements. The later measurements provide additional detail, in an attempt to isolate the locations of the high inflows.

At the three sumps, additional weir measurements were taken immediately upstream of the discharge to the sump along with sump inflow measurements (depth of water in sump verses time) taken between pump out cycles. The measured flow into the sumps was compared to the theoretical flow from the weir measurements to provide a specific calibration for the weirs.

Results

The results of the groundwater inflow measurements are shown on Figures E1 - E4.

A summary of key results and interpretation is given below. These results are discussed further later in this Appendix.

- The total groundwater inflow collected in the three sumps over the 9km of the tunnel was measured as approximately 4 l/s.
- The average inflow was approximately 0.5 litres/second/lineal kilometre of tunnel (l/s/km).
- 25% of the total inflow to the tunnel occurs in two sections approximately each 300m long.
- With the two isolated peak inflows removed, the remaining sections of tunnel have an average inflow of approximately 0.3 l/s/km.
- Expressed in terms of the original head above the tunnel, the average inflow was 7x10-03 l/s/km, per metre of peizometric head originally existing above the tunnel
- The peak inflow measured on any section of the tunnel was approximately 2/l/s/km.
- The minimum inflow measured was approximately 0.1l/s/km.
- The maximum peizometric drawdown associated with the construction of the lined tunnel, was 24 m, (BH 15, approximately 25 m off the centreline of the tunnel)

Discussion of Results

Measured Inflows

The peak inflow rate of approximately 2l/s/km occurs in the section of tunnel from chainage 27,700 to 27,900, which collectively has a high inflow rate.

This location of high inflow (CH 27,700-27,900) coincides with:

- A syncline in the ECBF geology, (opposite dips of up to 40 degrees)
- An area of limited or no weathering rind on the ECBF
- Thickly bedded, sandstone dominated ECBF.
- Total cover of approximately 40 m (compared to the average of approximately 70 m in the 19 borehole locations)
- ECBF cover of approximately 15 m (compared to an average of approximately 30 m in the 19 borehole locations)

- A pre-tunnelling peizometric head above tunnel level of approximately 50 m, (the average for all the locations measured along the tunnel is 50 m)
- A peizometric drawdown associated with the installation of the lined tunnel of approximately 20 m (BH 15, 25 m off tunnel centreline)
- A maximum peizometric response occurring approximately 30 days after initial response
- A peizometric head recovery on lining of the tunnel of only 13%, compared with the average value of approximately 30%.

The high inflow at chainage 24,000 was isolated to a single spring in the wall of the tunnel.

The lowest inflow rates of approximately 0.1 l/s/km were recorded at approximate chainage 22,500 and 23,500.

These locations of low inflow coincide with:

- An area of thick weathering rind on the ECBF
- Thinly bedded siltstone and sandstone dominated ECBF
- Total cover of approximately 65 – 70 m (compared to the average of approximately 70 m in the 19 boreholes)
- ECBF cover of approximately 35 – 40 m (compared with the average value of approximately 30 m in the 19 boreholes)
- A pre-tunnelling peizometric head above tunnel level of approximately 45 – 60 m, (the average for all the locations measured along the tunnel is 50 m)
- A peizometric drawdown associated with the installation of the lined tunnel of approximately 6 – 13 m (BH 4 and BH6, approximately 50 m off tunnel centreline)
- A maximum peizometric response occurring approximately 80 - 110 days after initial response
- A peizometric head recovery on lining of the tunnel of 30 - 60%, compared with the average value of approximately 30%.

Calibration of inflow rates to peizometric head

Measured tunnel inflow rates have been compared with peizometric measurements along the tunnel to investigate the possibility of a correlation. Both pre-existing peizometric head and post-tunnel construction head have been compared to the measured tunnel inflows. When assessing this data, it must be appreciated that the piezometers are at a varying distance from the centreline of the tunnel, and hence have been affected to a varying degree by the tunnel.

A poor correlation was found between tunnel inflows and both pre-existing, and post-tunnelling peizometric head.

Relevance of Results to the proposed CI Sewer Tunnel

As briefly discussed earlier, the CI tunnel is proposed in similar geology to that which the Vector Tunnel was constructed in. The following interpretations may be used for guidance in the design of the CI.

There is a greater risk of groundwater effect on strata overlying the ECBF where

- ECBF cover is reduced
- There is a thin, or no weathering surface on the ECBF

- ECBF strata are inclined and coarse sandstone beds may connect with surface deposits.
- The ECBF is coarse and thickly bedded.

Conversely, there is reduced risk of groundwater effects on strata overlying the ECBF where

- There is significant ECBF cover
- There is a thick weathering surface on the ECBF
- ECBF strata are flat lying
- The ECBF is thinly bedded.

Appendix E: Groundwater Modelling

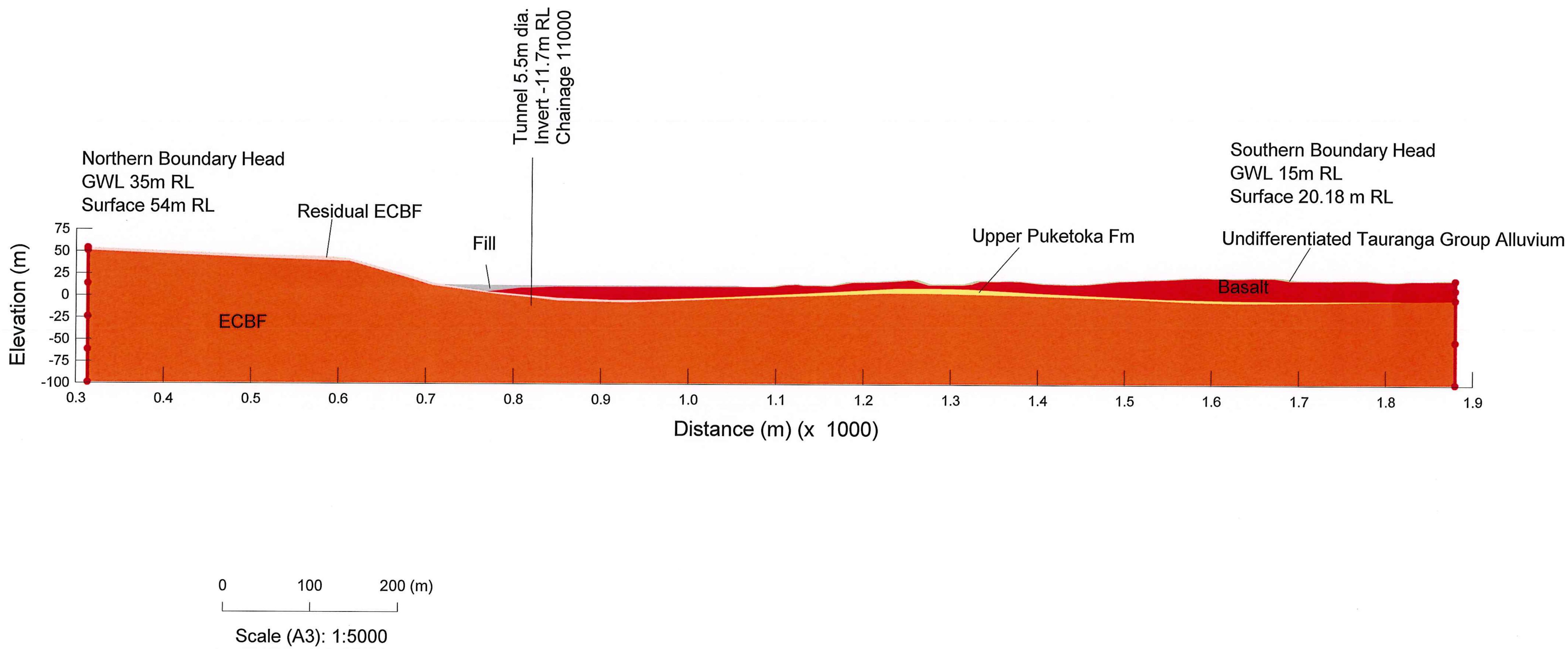
- Hydraulic Conductivity and Water Content Functions
- Groundwater Model Setup
- Groundwater Model Selected Results
    - Pressure head contours
    - Settlement Contours

Notes on seepage and settlement analysis figures

The contents of this appendix are ordered in the following way:

- Tunnel analysis sections for sections 1, 2, 4, 5, 6 and Generic;
- Tunnel seepage and settlement modelling output for each of the sections in the same order;
- Shaft analysis sections for WS1, WS2, WS3, AS3, AS4 and AS7;
- Shaft seepage and settlement modelling output for each of the sections in the same order.

The seepage output shows contours of pressure head (in metres) together with flow vectors. The settlement output (denoted "stresses") shows contours of vertical displacement (in metres).



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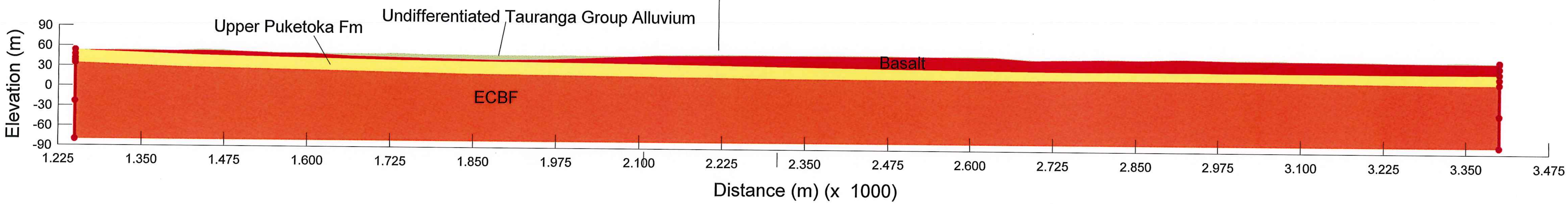
Central Interceptor - Groundwater and Surface Settlement Study
Geological Section
Section 1

Designed: RRG
Checked:
Date: Nov 2011
Job Number: 26145.300

SE Boundary Head
GWL 51.8m RL
Surface 53.4m RL

Tunnel 5.5m dia.
Invert -16m RL
Chainage 14750

NW Boundary Head
GWL 42.5m RL
Surface 46.5m RL



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Tonkin & Taylor

Central Interceptor - Groundwater and Surface Settlement Study
Geological Section
Section 2

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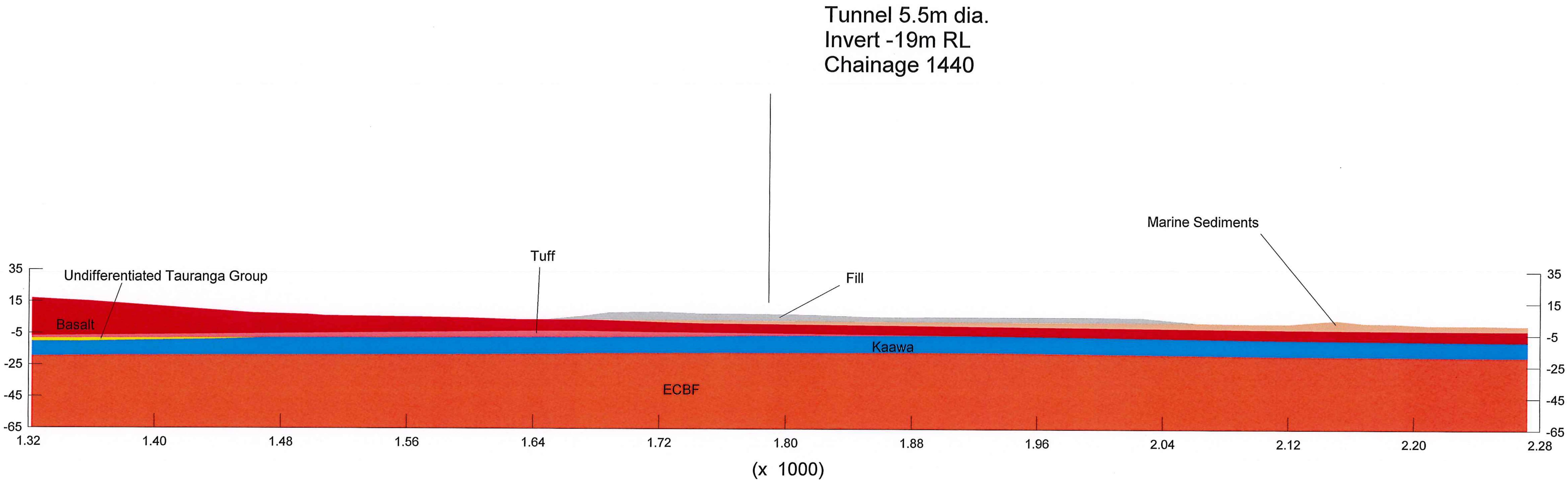
Date: Nov 2011
Job Number: 26145.300



A3 Scale: 1:2500

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Tonkin & Taylor

Central Interceptor - Groundwater and Settlement Study
Gelogical Section
Section 4

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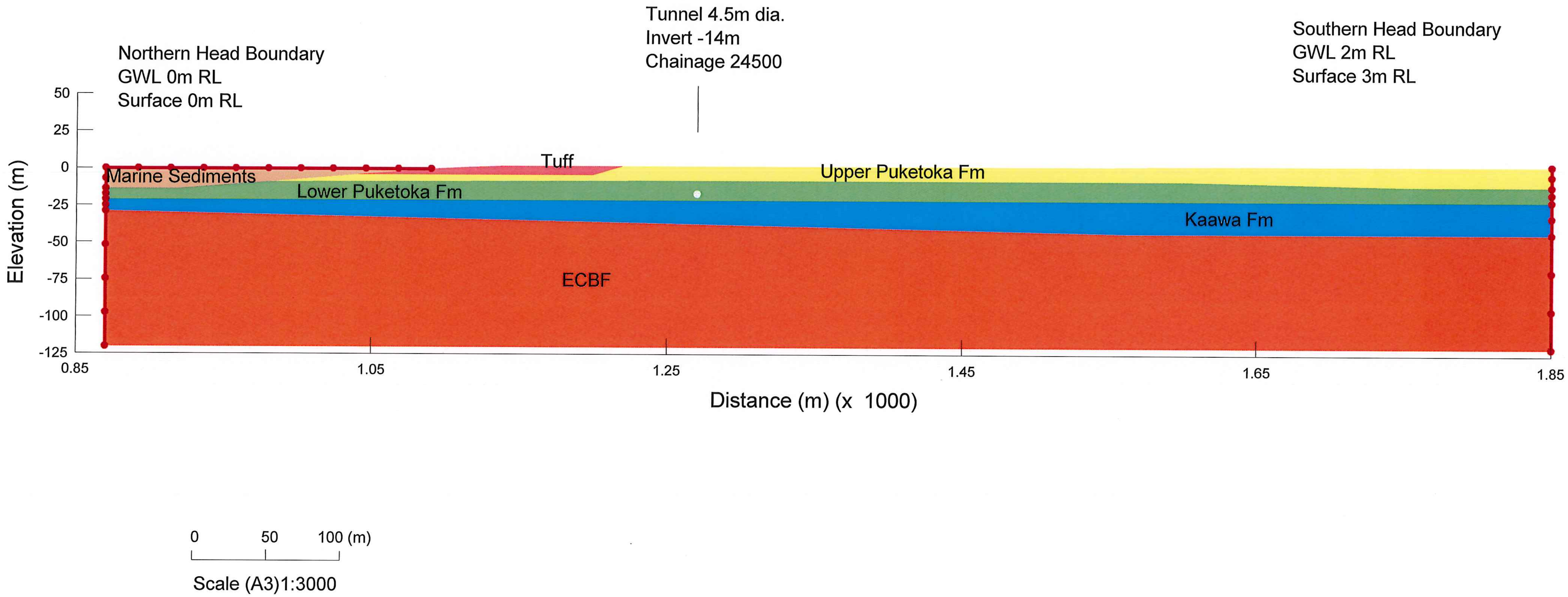
Job Number: 26145.300



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Central Interceptor - Groundwater and Surface Settlement Study
Geolgical Section
Section 5

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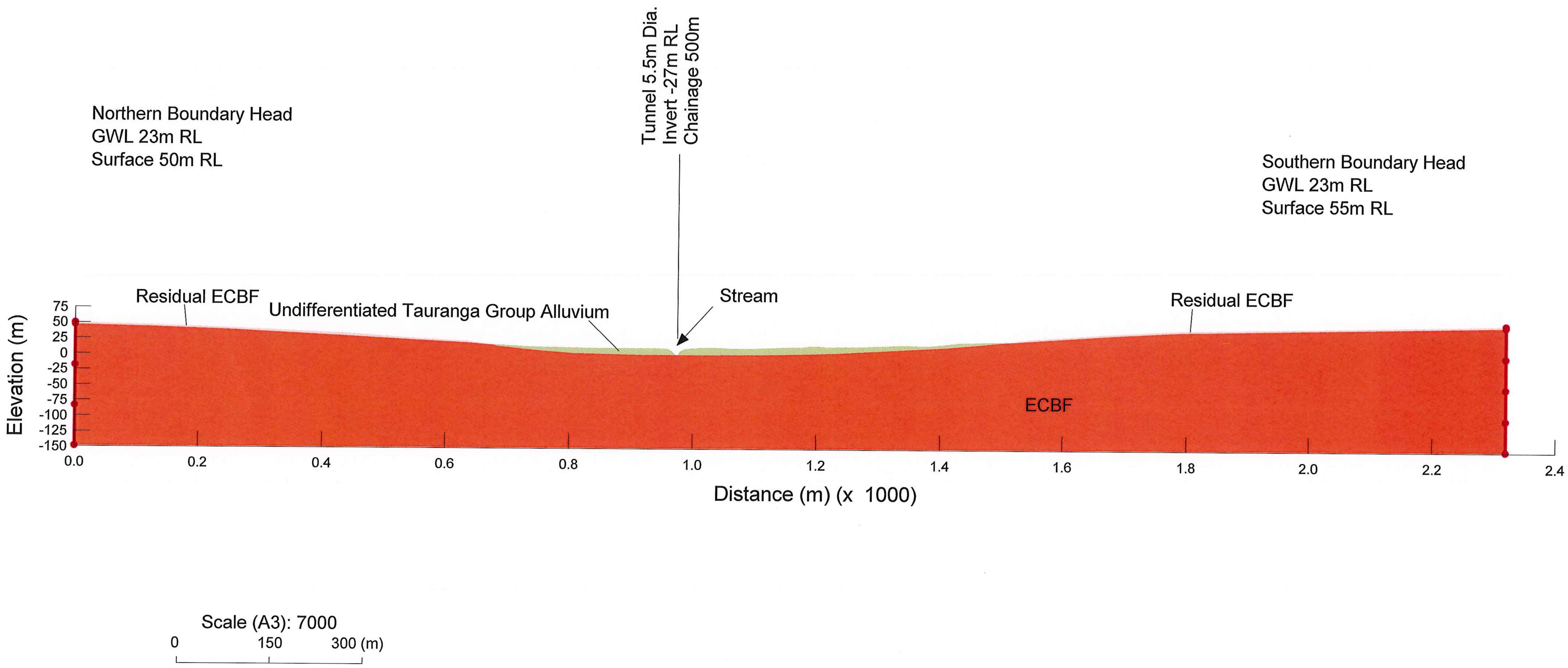
Job Number:26145.300



Tonkin & Taylor

Central Interceptor - Groundwater and Surface Settlement Study
Geological Section
Section 6

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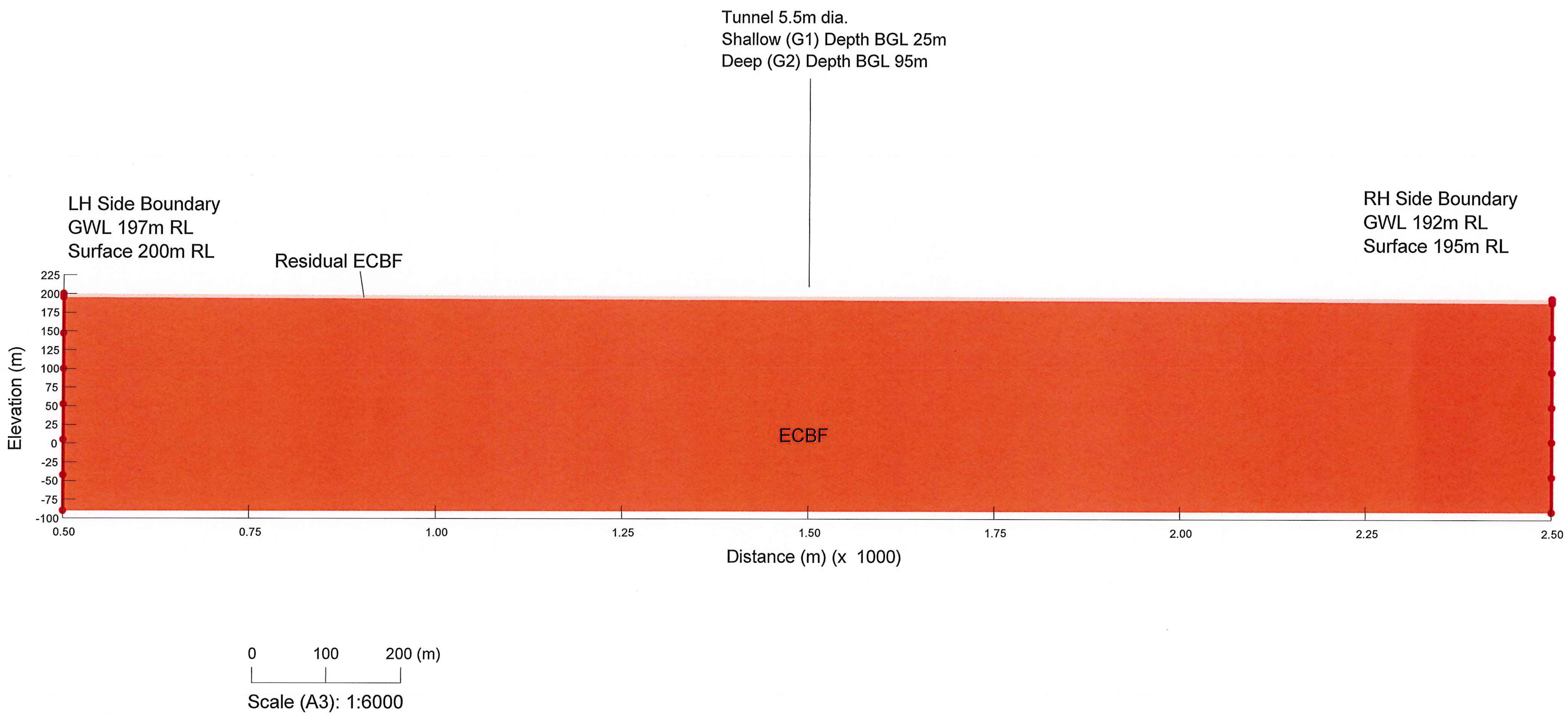
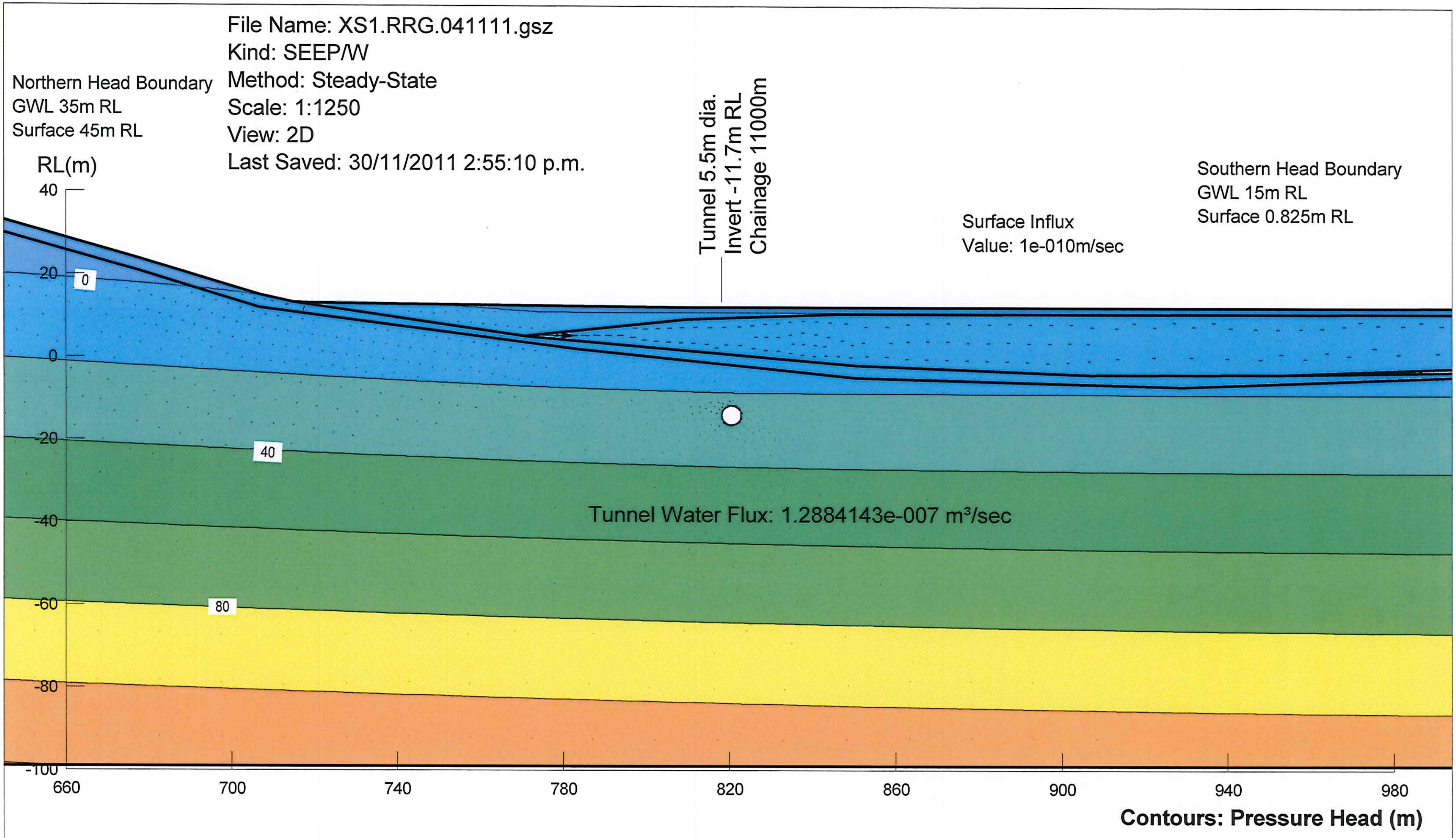
Date: Nov 2011
Job Number: 26145.300



Tonkin & Taylor

Central Interceptor - Groundwater and Surface Settlement Study
Geological Section
Generic Section

Designed: RRG
Checked:
Date: Nov 2011
Job Number: 26145.300



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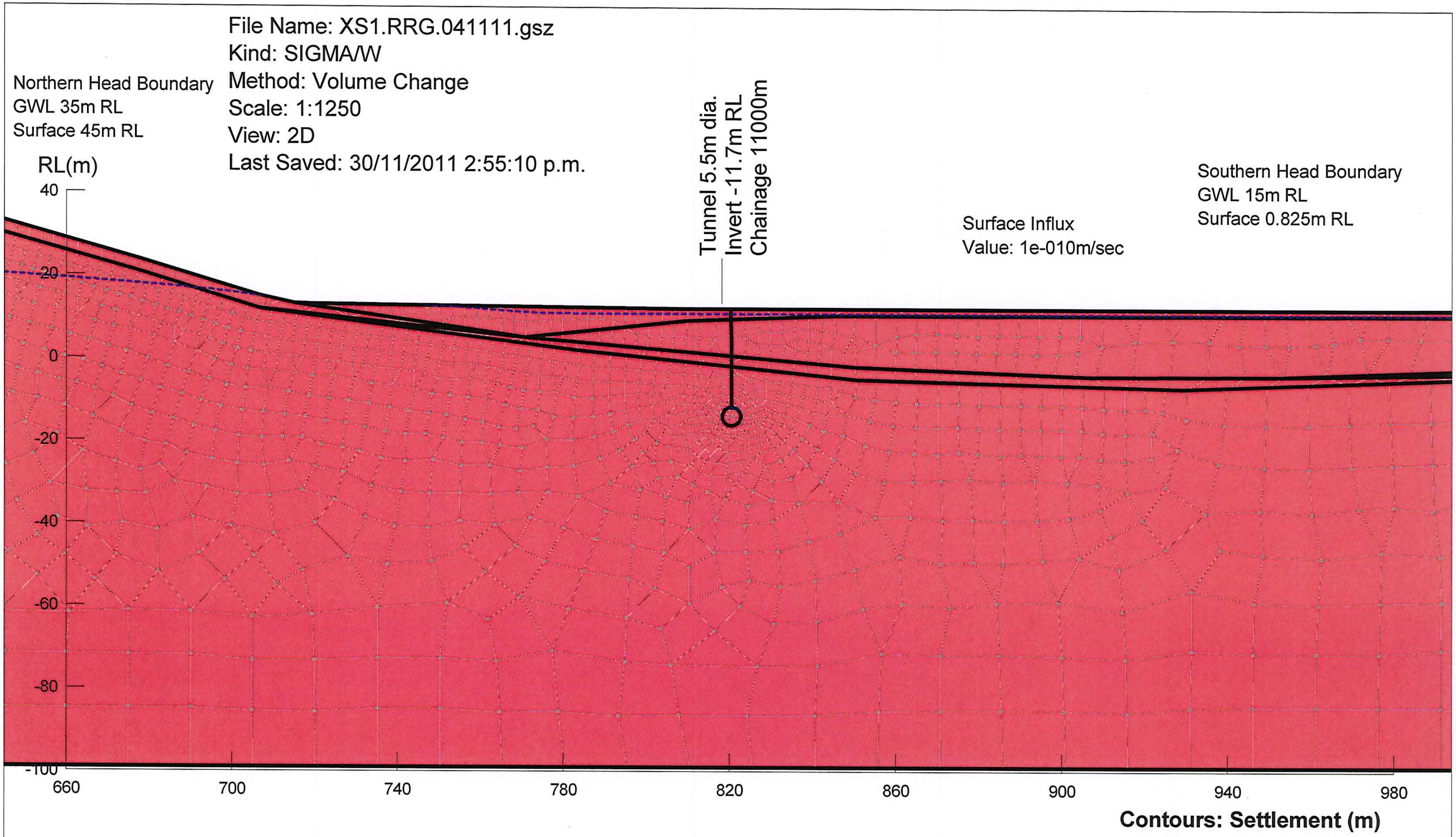
Central Interceptor - Groundwater and Surface Settlement Study

Section 1

1.1.5.1 Lined Tunnel <1e-10>- Steady State

Analysed: RRG
Checked:
Date: Nov 2011
Job Number: 26145.3



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Central Interceptor - Groundwater and Surface Settlement Study

Section 1

1.3.5.1 Lined Tunnel<1e-10> - Stresses

Analysed: RRG
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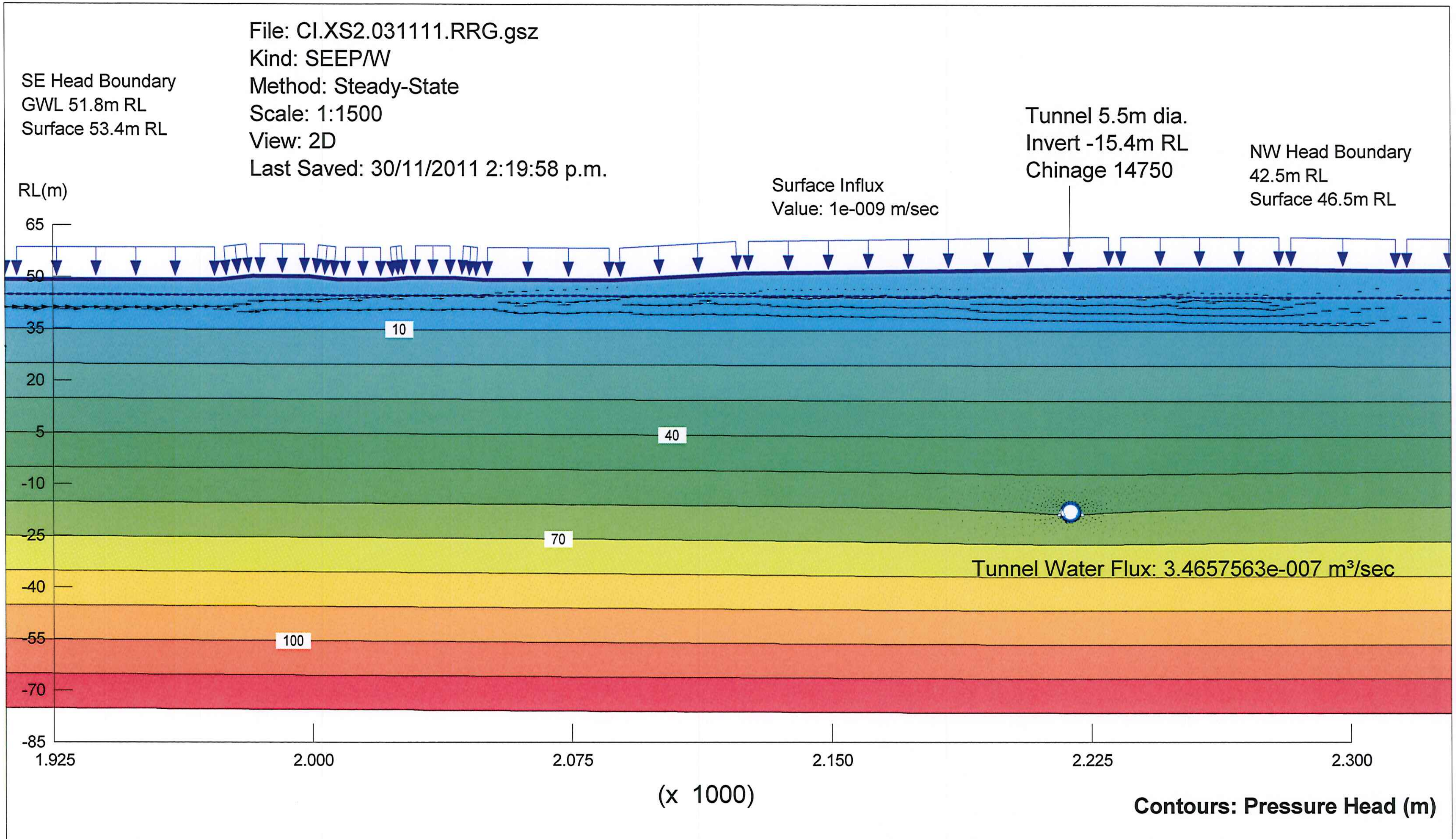
Job Number: 26145.3



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Central Interceptor - Groundwater and Surface Settlement Study

Section 2

2.1.5.1: Lined Tunnel <1e-10> - Steady State

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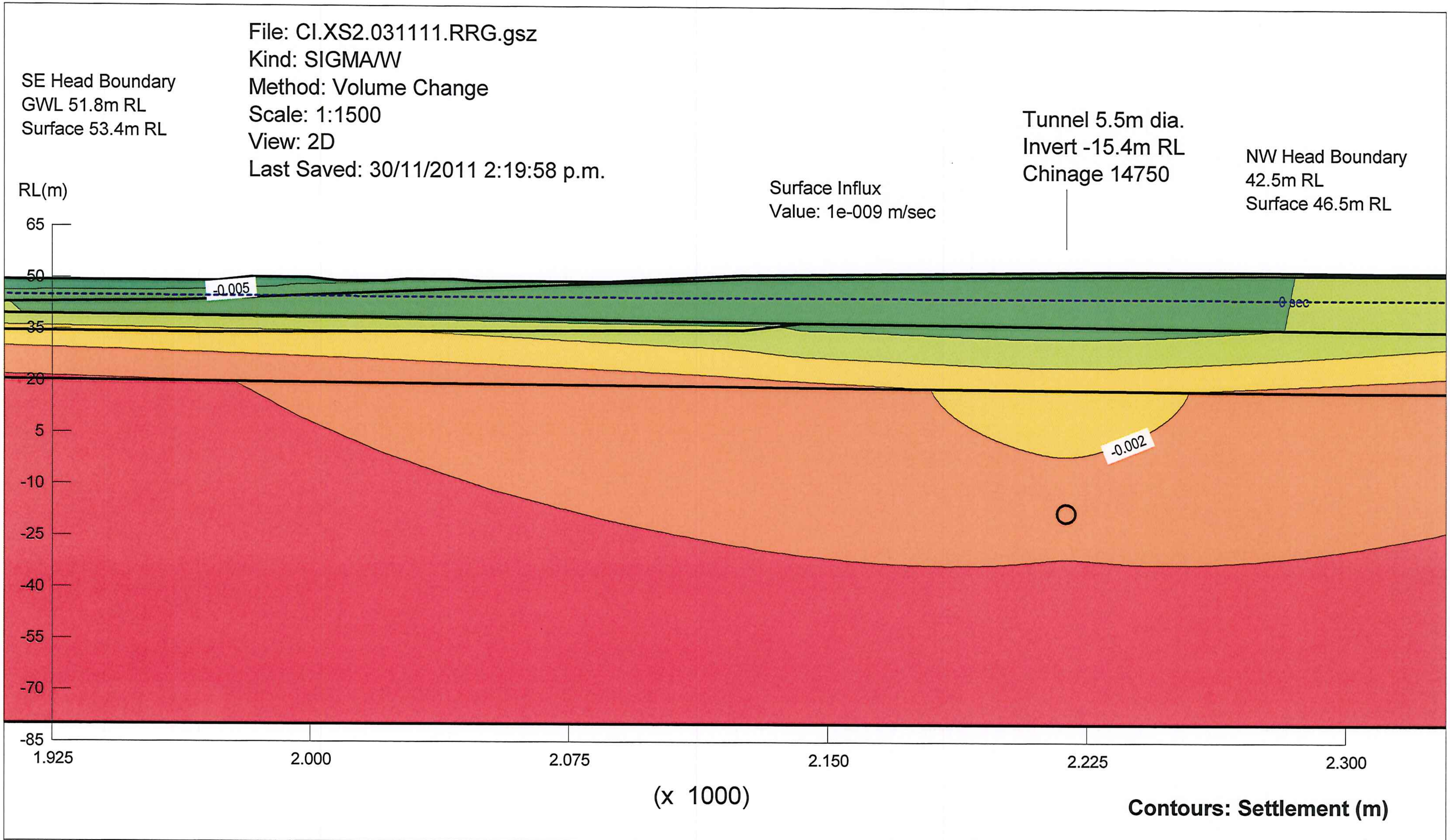
Job Number: 26145.3



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Central Interceptor - Groundwater and Surface Settlement Study

Section 2

2.3.5.1: Lined Tunnel <1e-10> - Stresses

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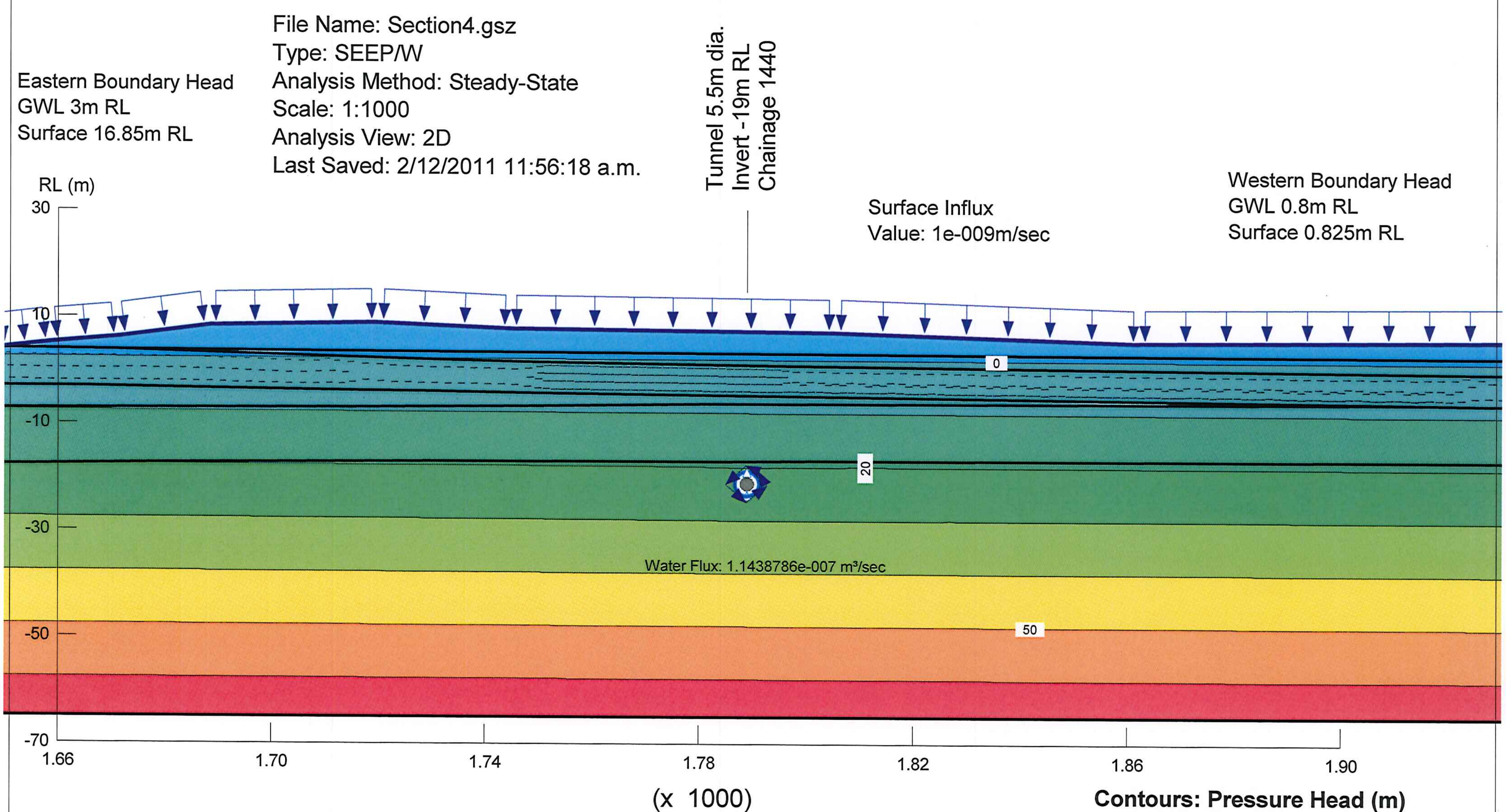
Job Number: 26145.3



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Central Interceptor - Groundwater and Surface Settlement Study

Section 4

4.1.5.1 Lined Tunnel <1e-10> - Steady State

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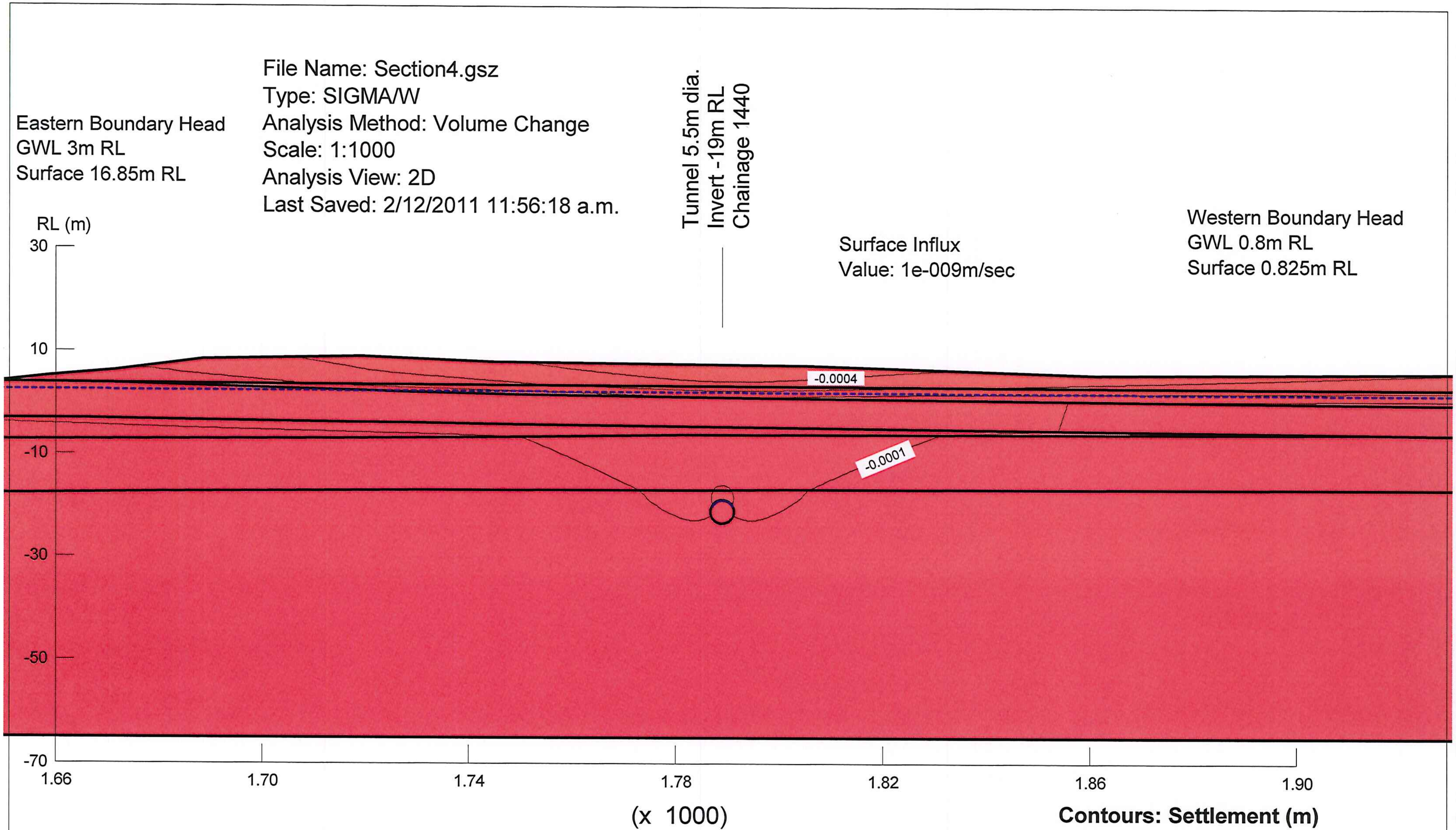
Job Number: 26145.3



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Central Interceptor - Groundwater and Surface Settlement Study

Section 4

4.3.5.1 Lined Tunnel <1e-10> - Stresses

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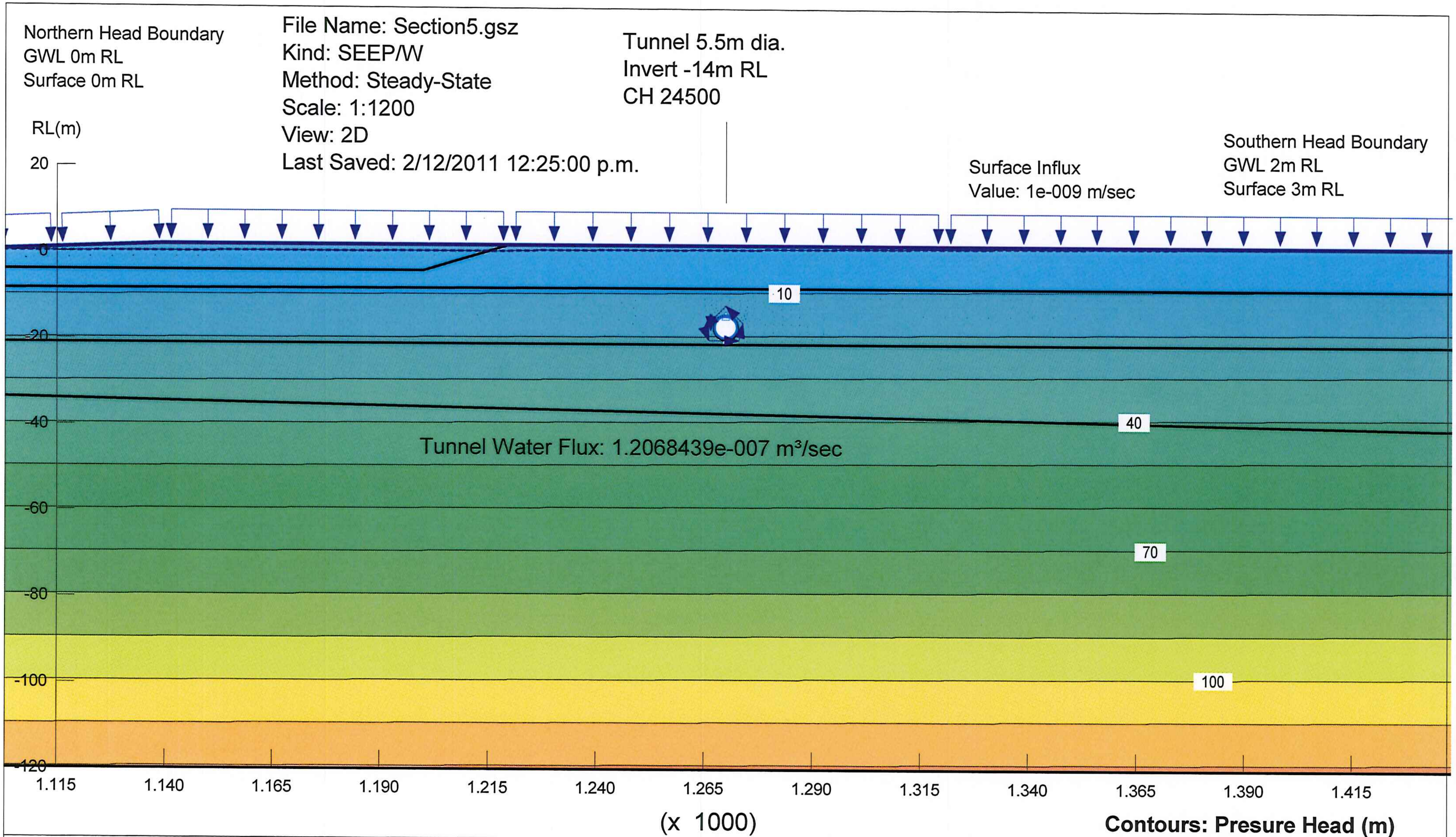
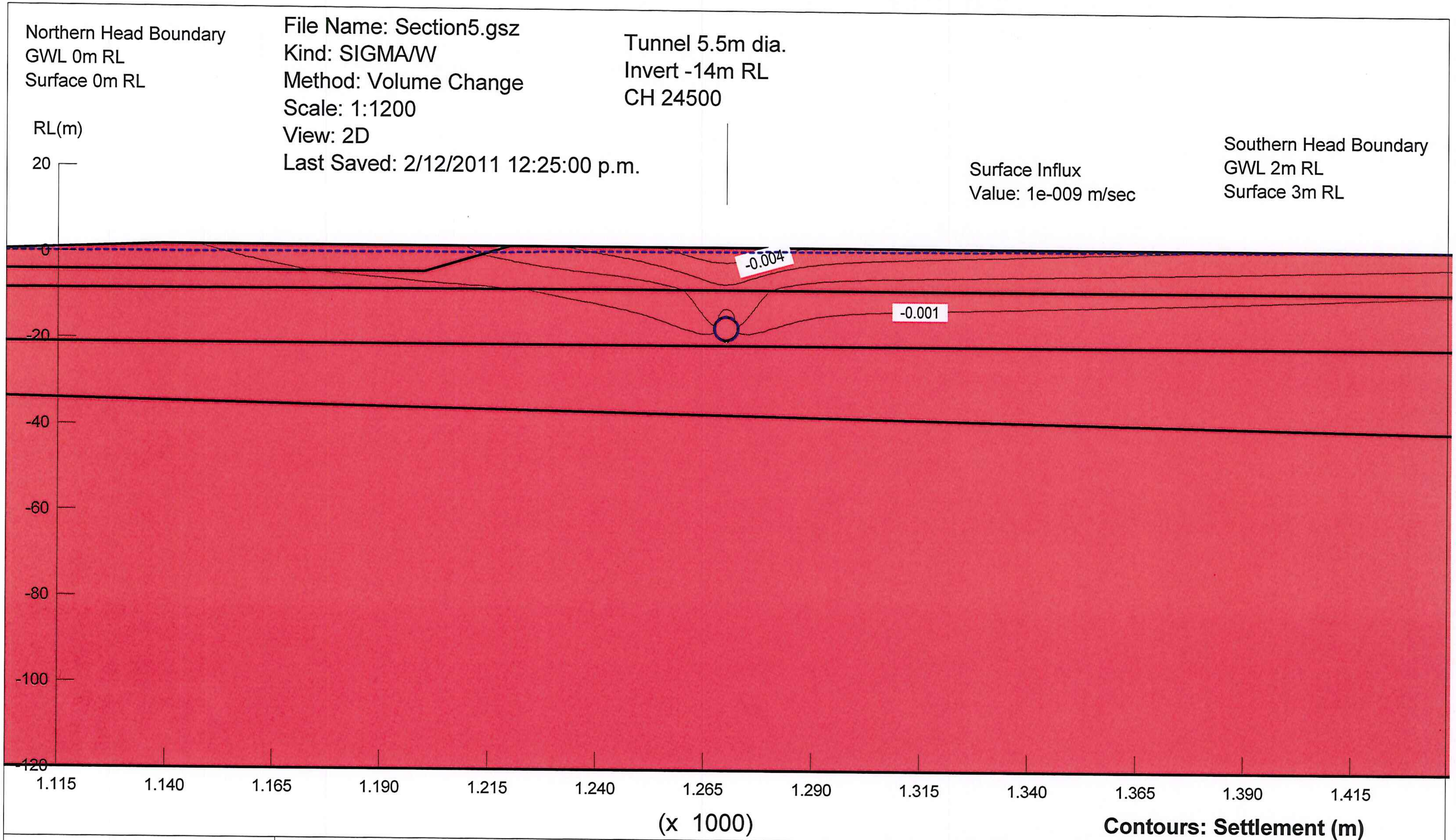


Tonkin & Taylor	**Central Interceptor - Groundwater and Surface Settlement Study** Section 5 *5.1.5.1 Lined Tunnel <1e-10> - Steady State*	Analysed: RRG Checked: Date: Nov 2011 Job Number: 26145.3



Tonkin & Taylor

Central Interceptor - Groundwater and Surface Settlement Study

Section 5

5.3.5.1 Lined Tunnel <1e-10> - Stresses

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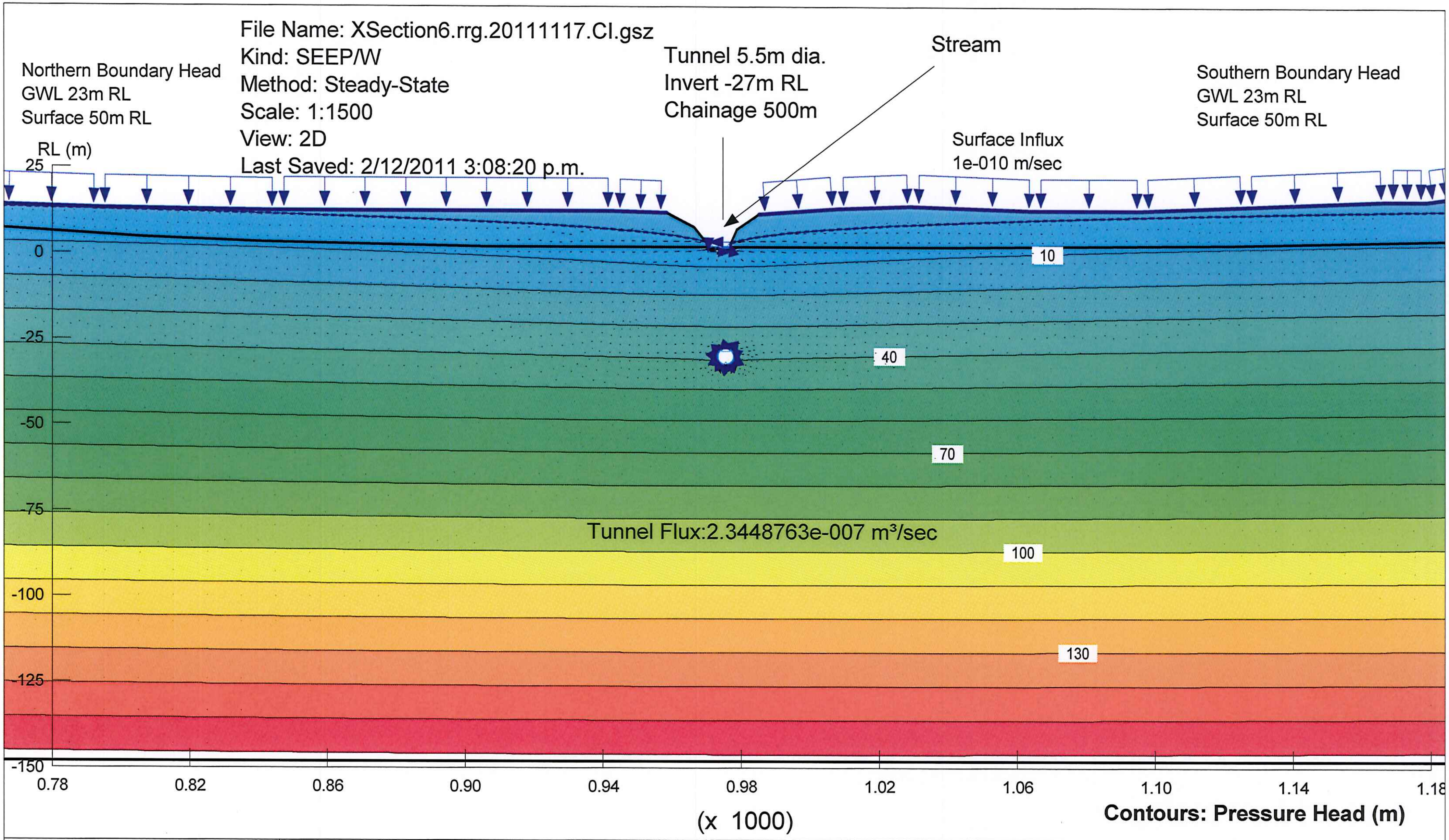
Job Number: 26145.3



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Central Interceptor - Groundwater and Surface Settlement Study

Section 6

6.1.5.1 Lined Tunnel <1e-10> - Steady State

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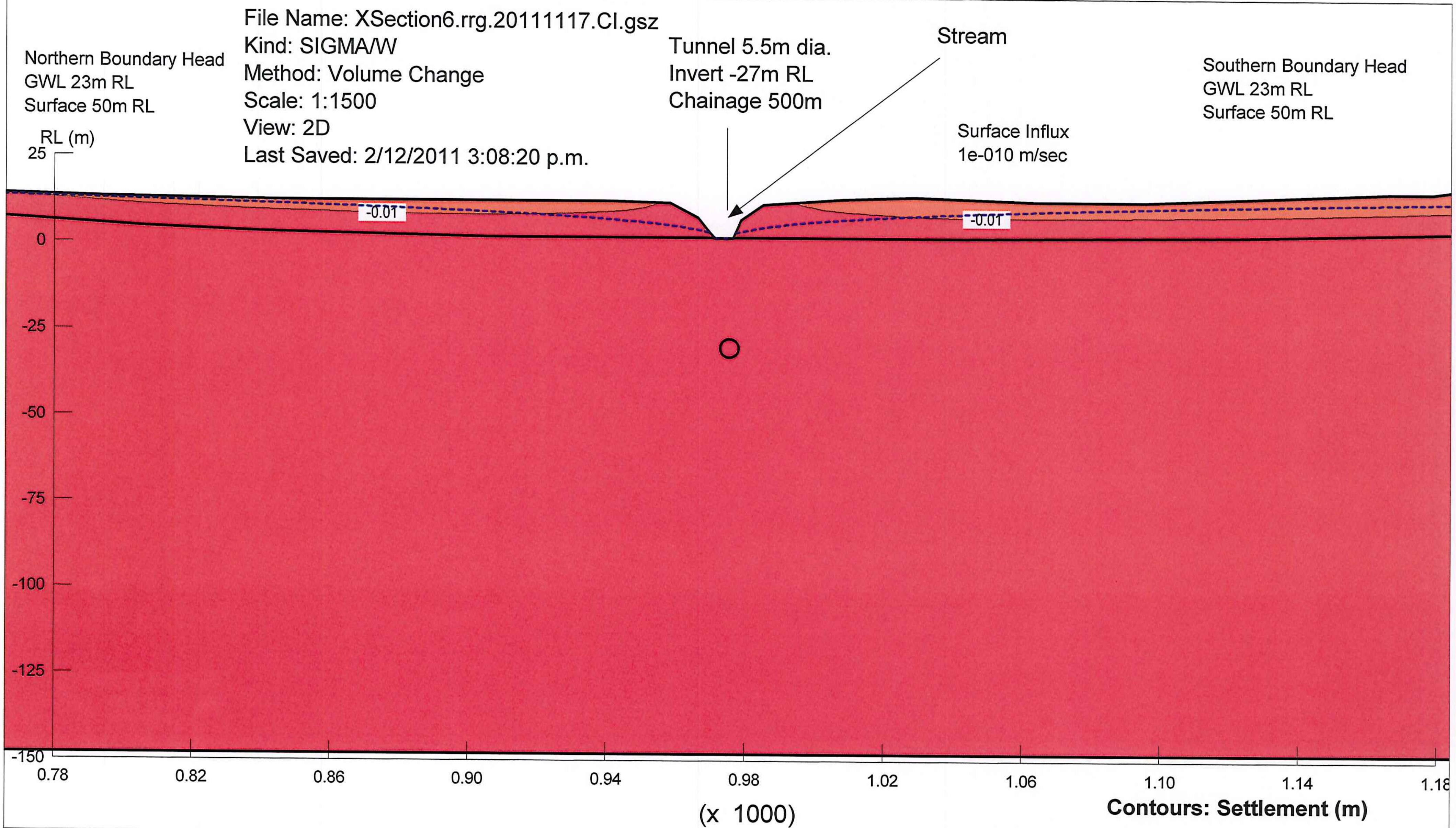
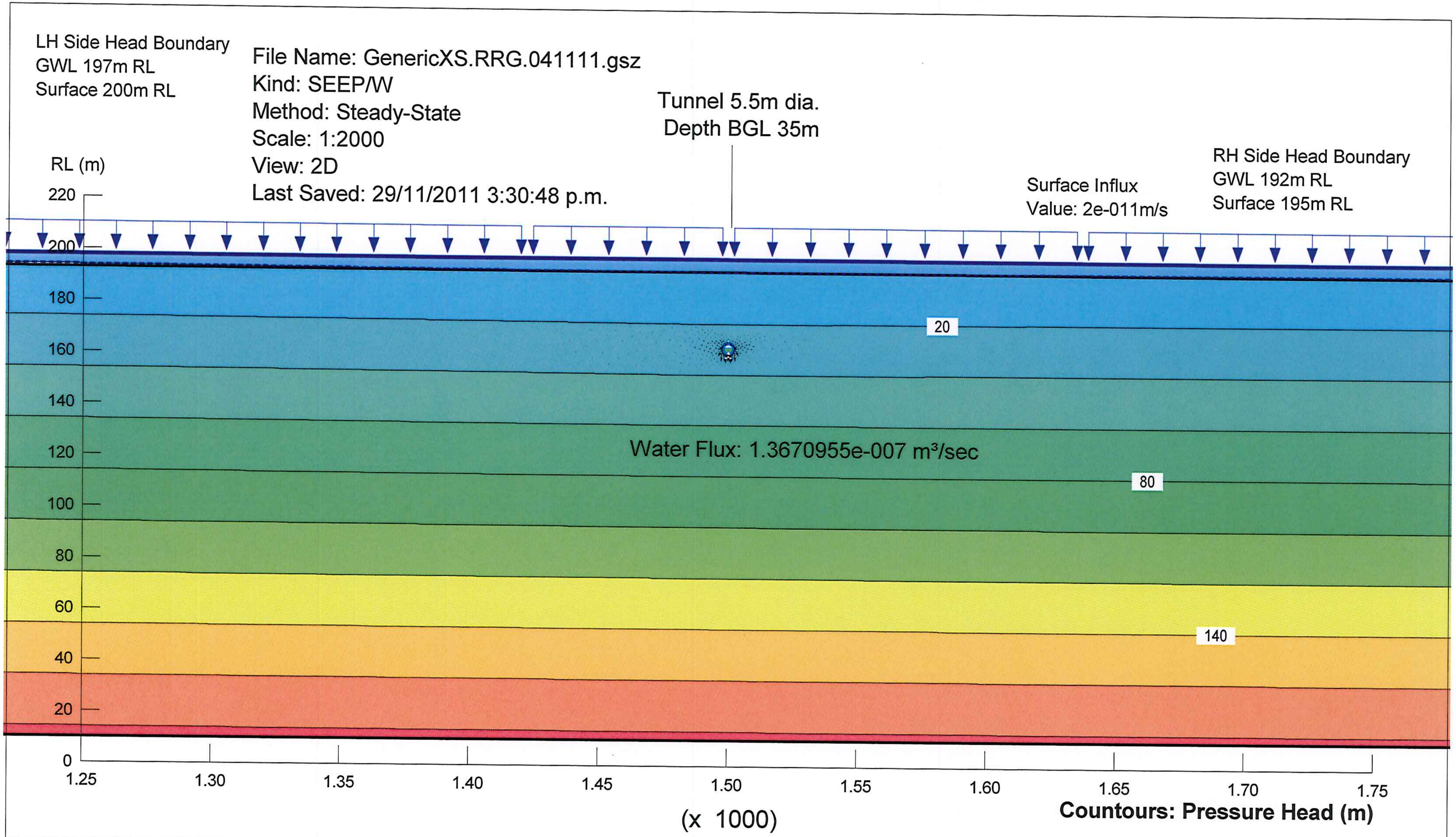
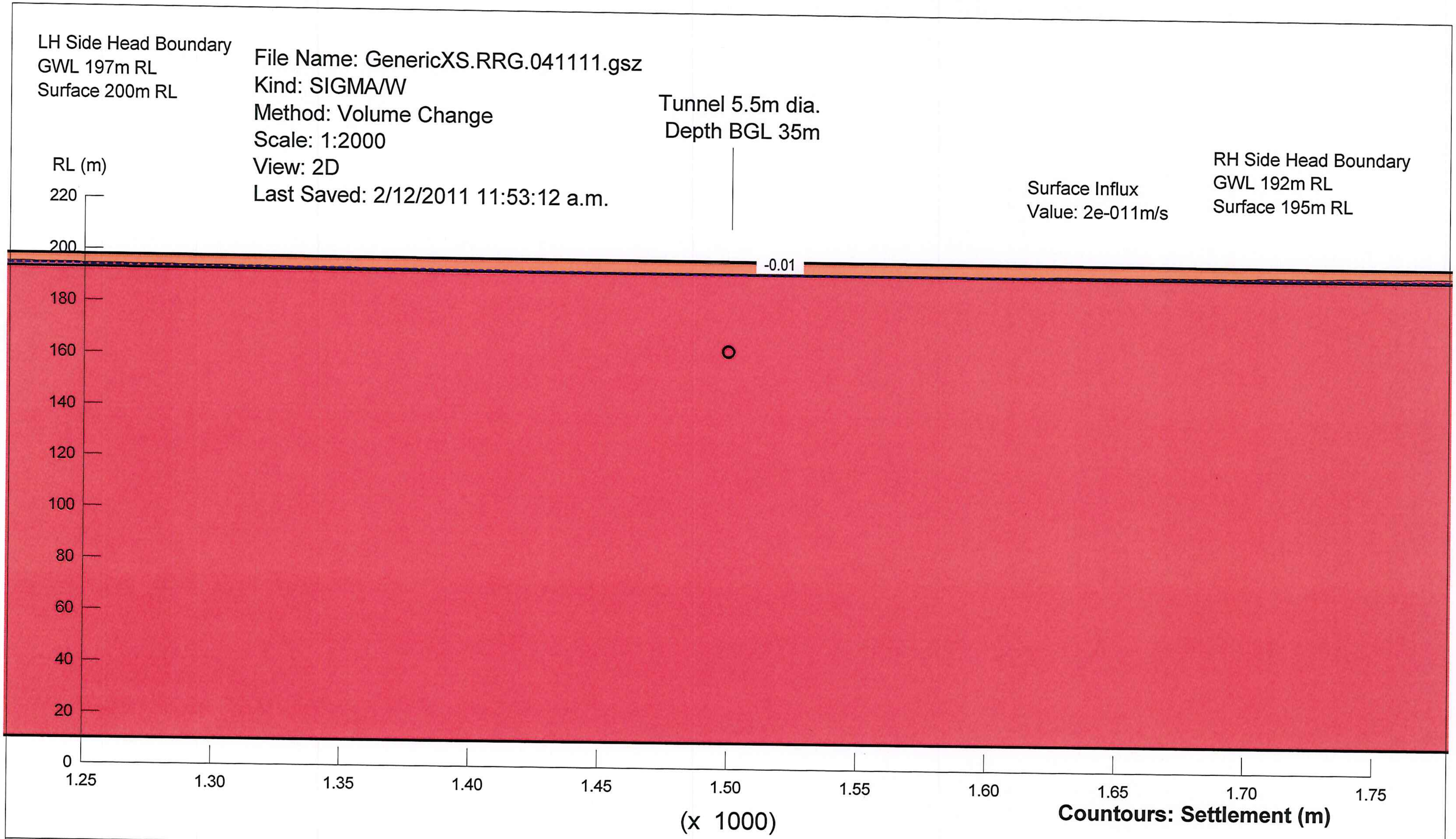
Job Number: 26145.3



Tonkin & Taylor	**Central Interceptor - Groundwater and Surface Settlement Study** Section 6 *6.3.5.1 Lined Tunnel <1e-10> - Stresses*	Analysed: RRG Checked: Date: Nov 2011 Job Number: 26145.3



Tonkin & Taylor	**Central Interceptor - Groundwater and Surface Settlement Study** Generic 1 *G1.1.5.1 Lined Tunnel <1e-10> - Steady State*	Analysed: RRG Checked: Date: Nov 2011 Job Number: 26145.3



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Central Interceptor - Groundwater and Surface Settlement Study

Generic 1

G1.3.5.1 Lined Tunnel <1e-10> - Stresses

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Job Number: 26145.3

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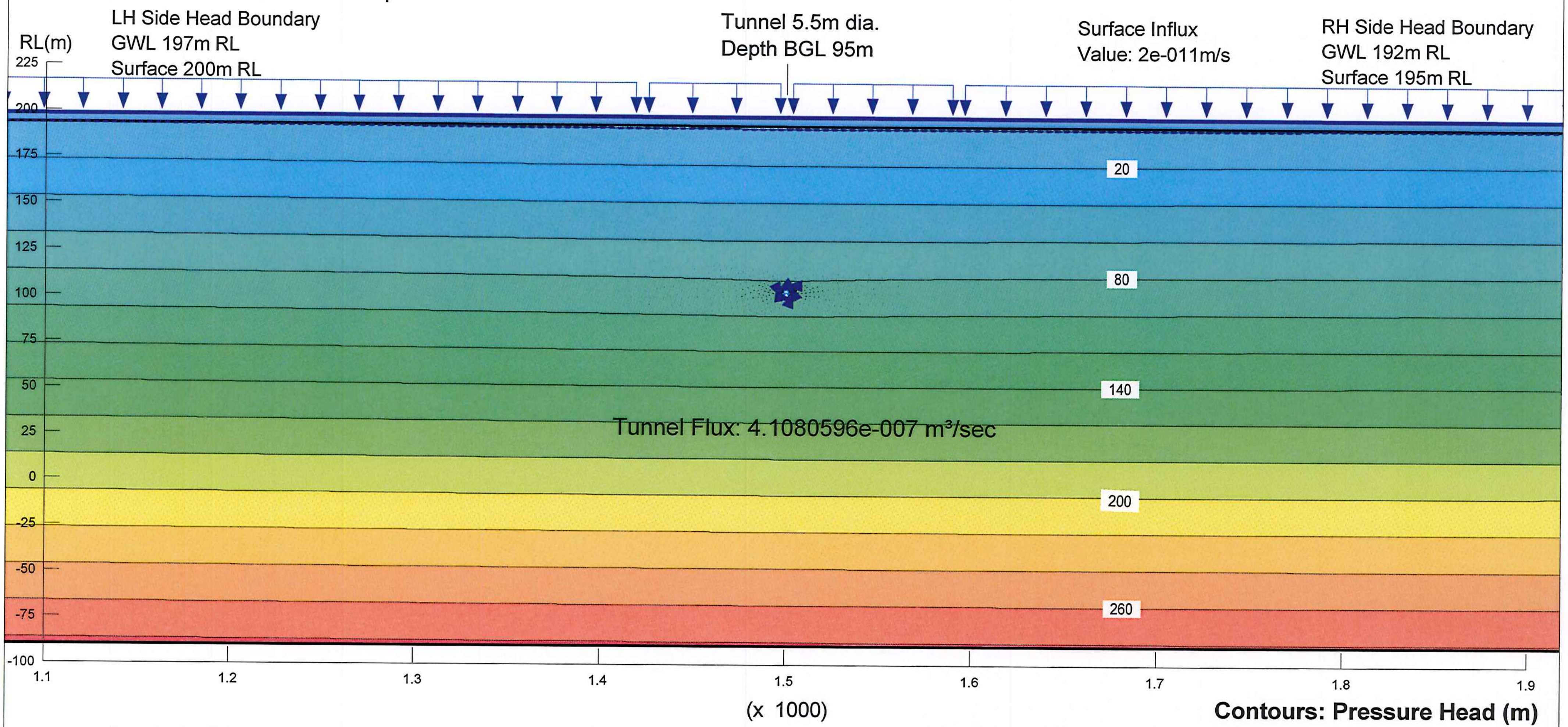

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Central Interceptor - Groundwater and Surface Settlement Study

Generic 2

G2.1.5.1 Lined Tunnel <1e-10> - Steady State

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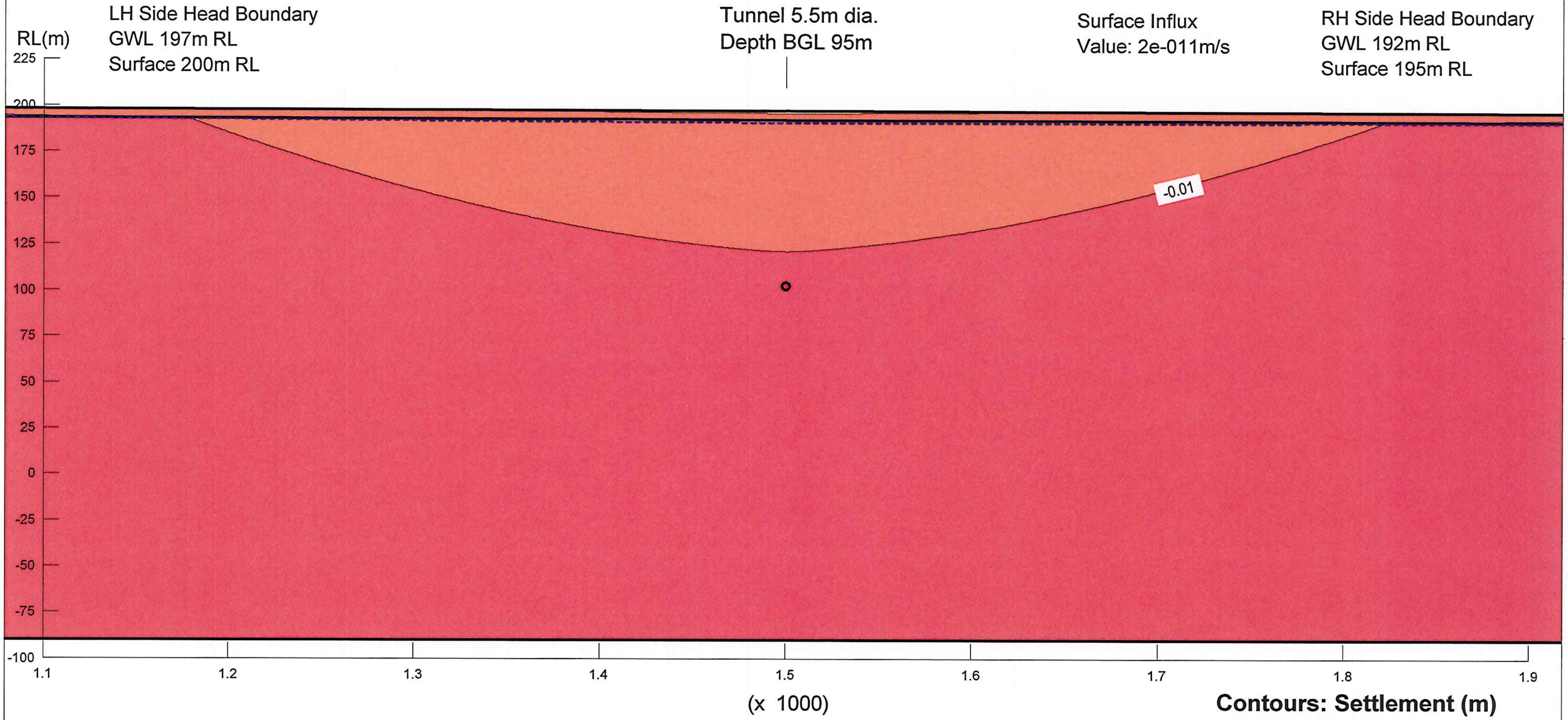

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Central Interceptor - Groundwater and Surface Settlement Study

Generic 2

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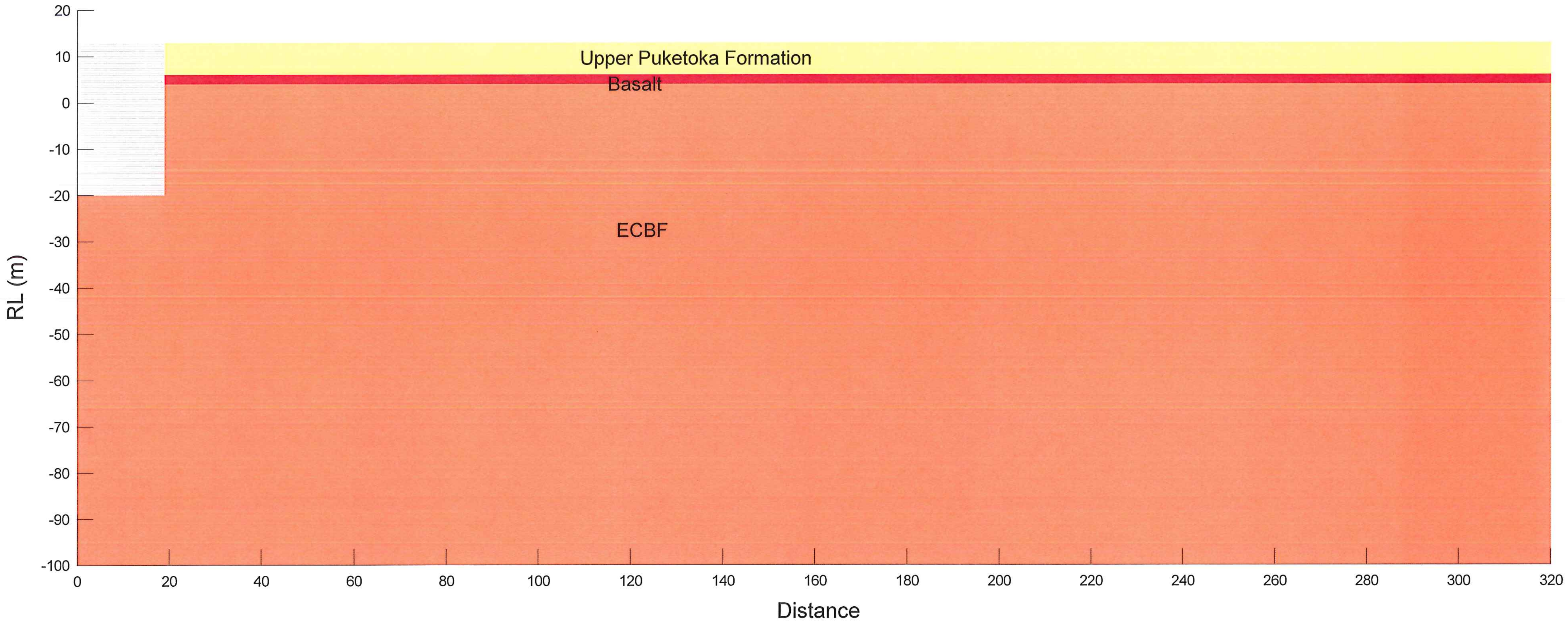
Job Number: 26145.3



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Central Interceptor - Groundwater and Surface Settlement Study
Working Shaft 1
Geological Section

Analysed: EYST
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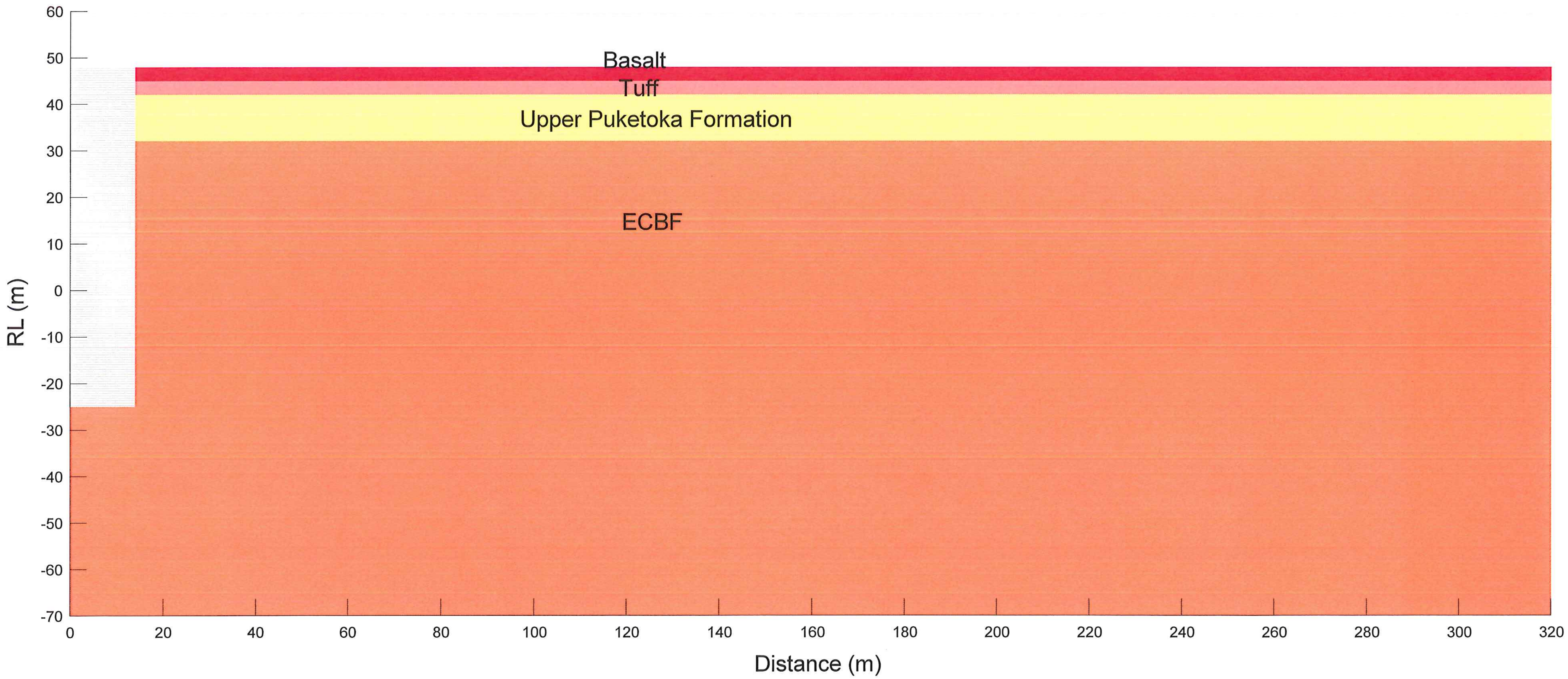
Job No: 26145.300



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Central Interceptor - Groundwater and Surface Settlment Study
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Geological Section

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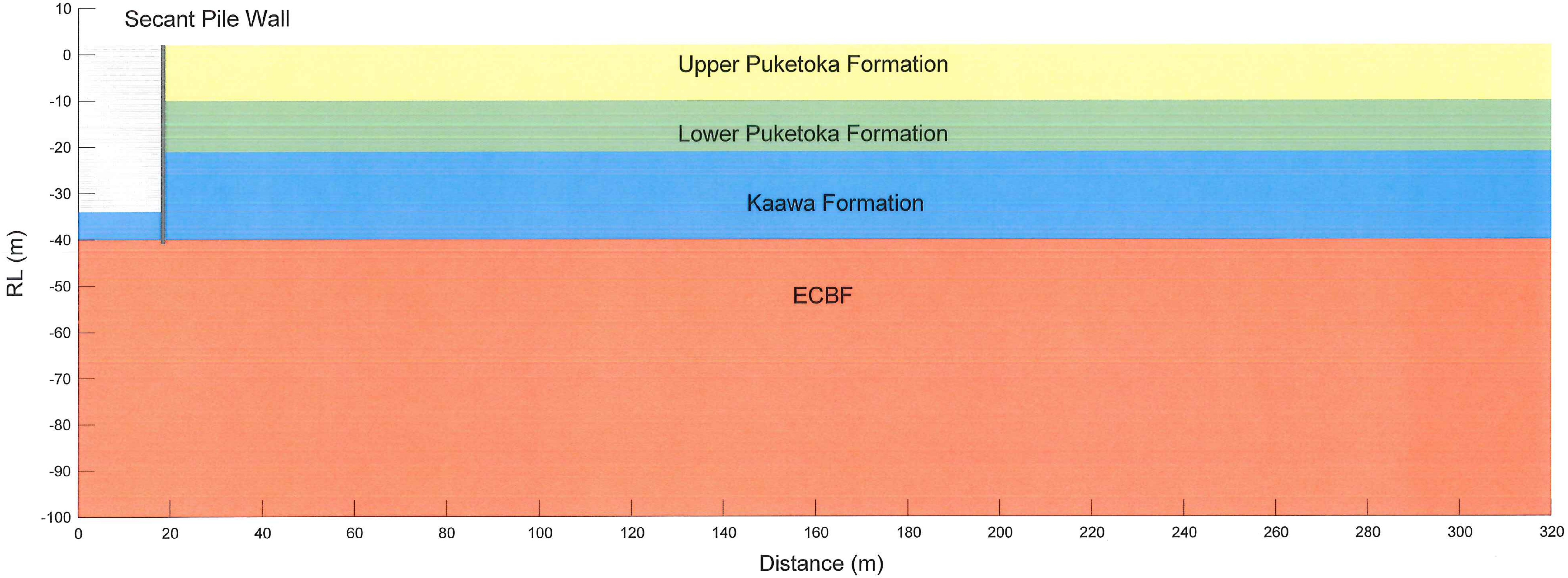

Tonkin & Taylor

Central Interceptor - Groundwater and Surface Settlement Study
Working Shaft 3
Geological Section

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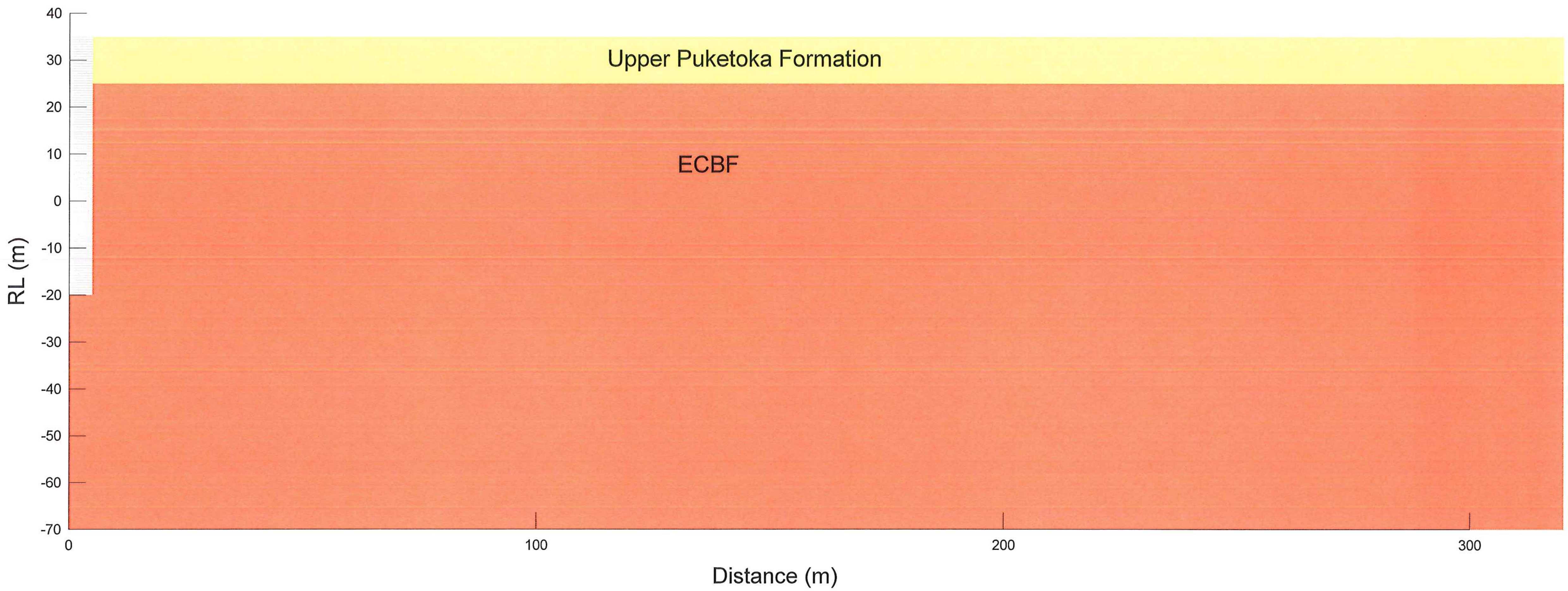

Tonkin & Taylor

Central Interceptor - Groundwater and Surface Settlement Study
Access Shaft 3
Geological Section

Analysed : EYST
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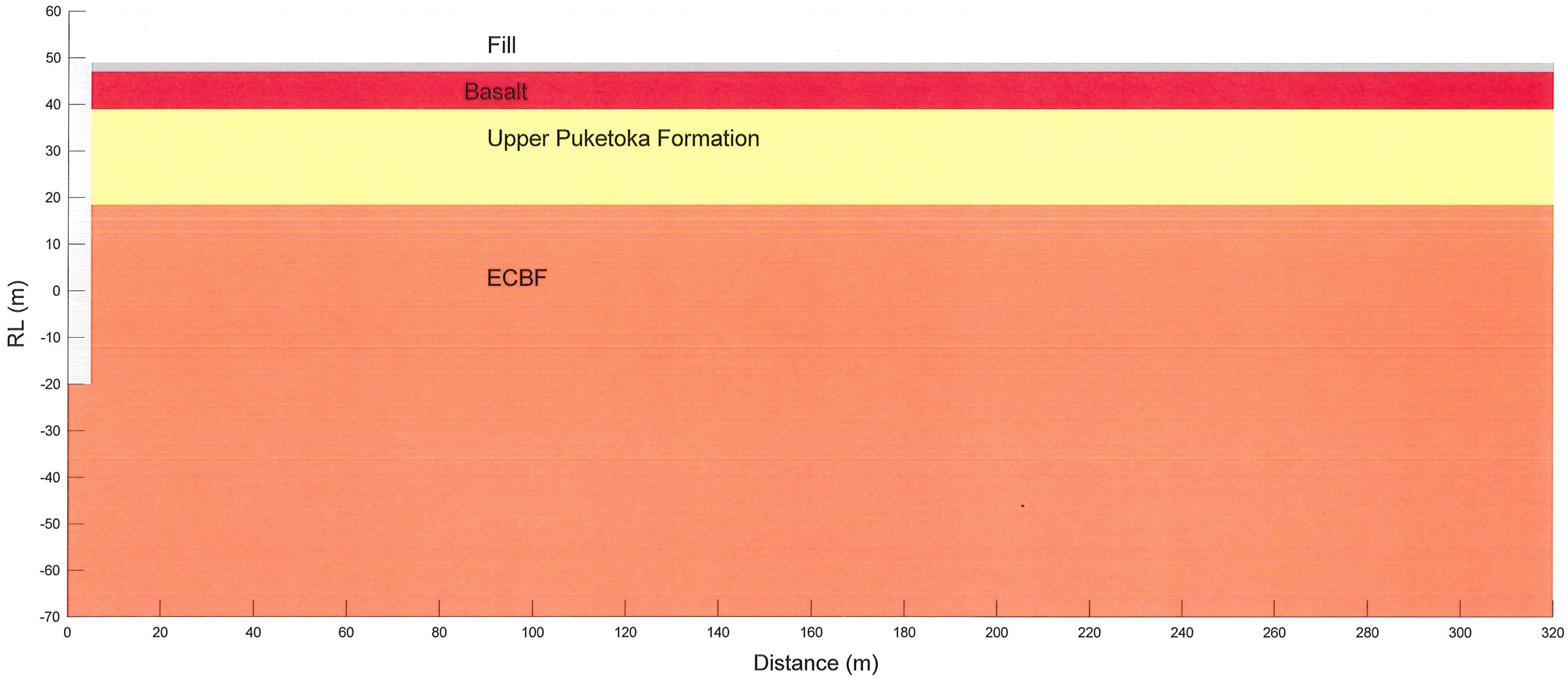

Tonkin & Taylor

Central Interceptor - Groundwater and Surface Settlement Study
Access Shaft 4
Geological Section

Analysed : EYST
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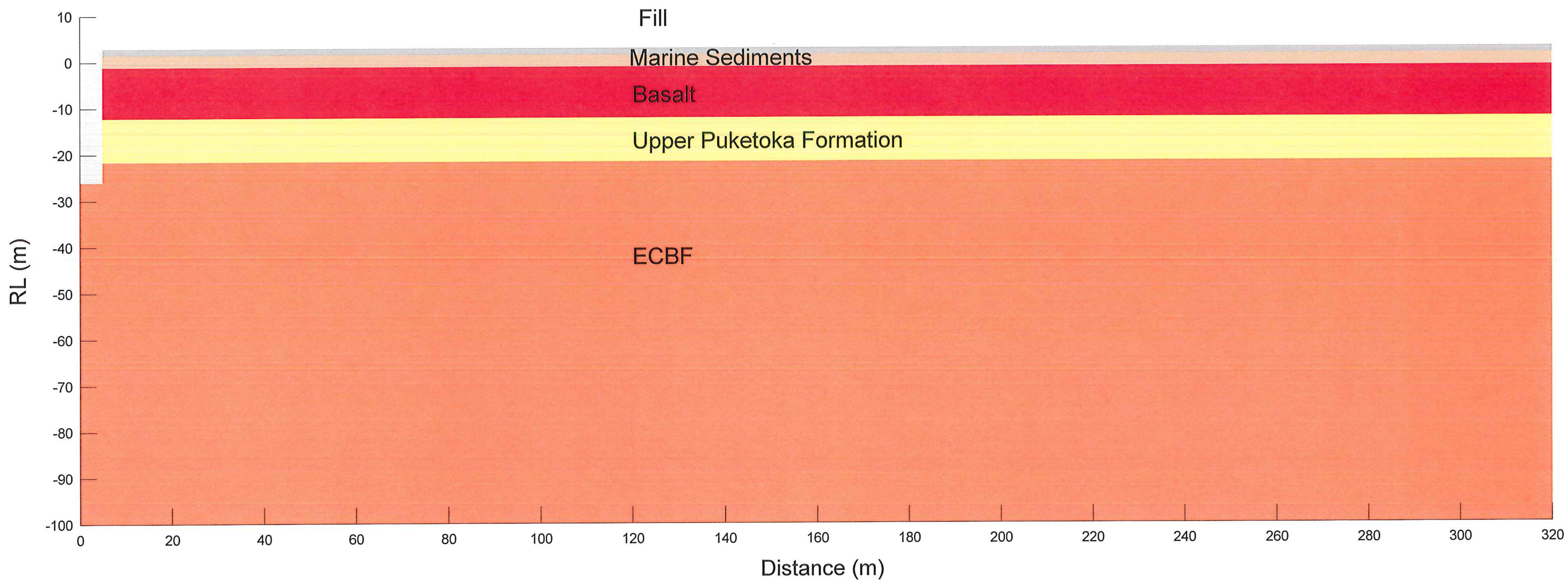
Date: Nov 2011
Job No: 26145.300



Tonkin & Taylor

Central Interceptor - Groundwater and Surface Settlement Study
Access Shaft 7
Geological Section

Analysed : EYST
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Date: Nov 2011
Job No: 26145.300

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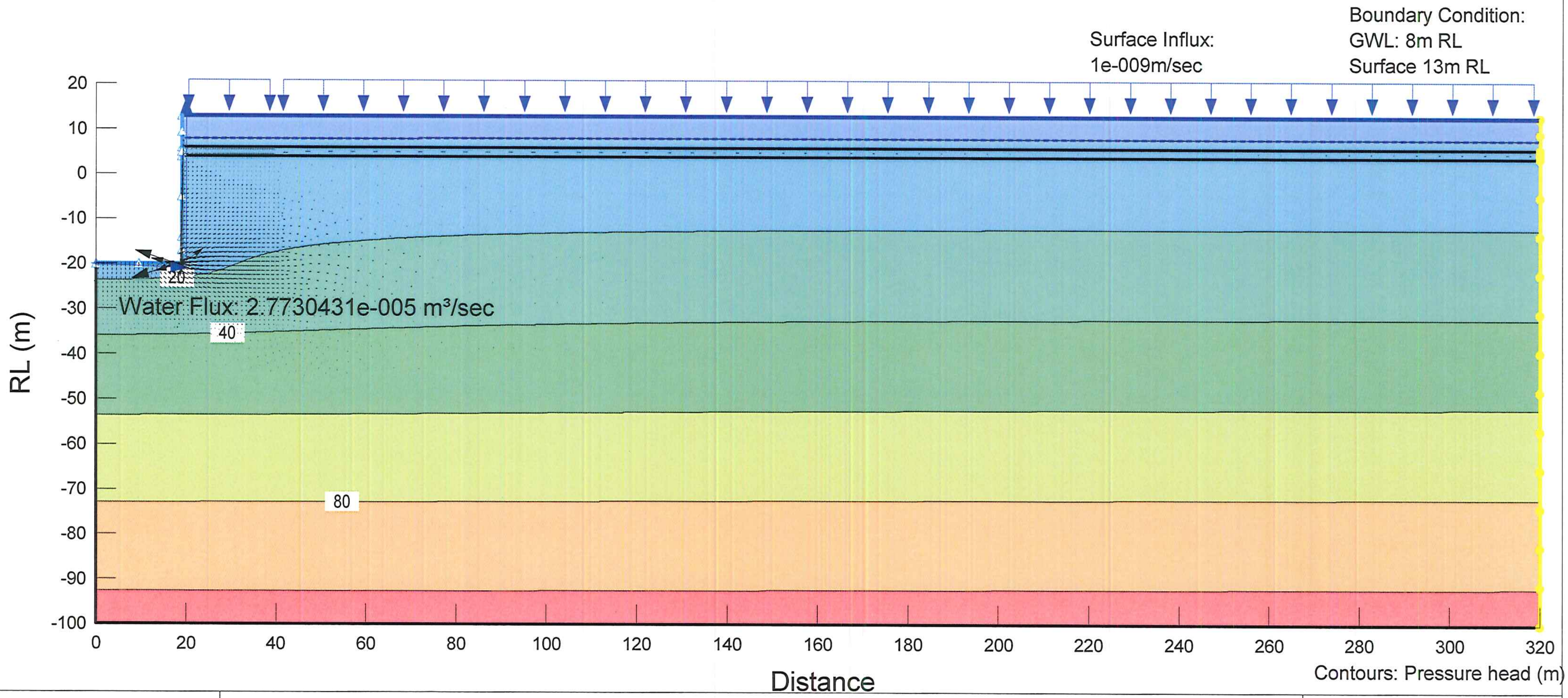

Tonkin & Taylor

Central Interceptor - Groundwater and Surface Settlement Study
Working Shaft 1
WS1.1.4.1 Lined Shaft <1E-9> - Steady State

Analysed: EYST
Checked:
Date: Nov 2011
Job No: 26145.300

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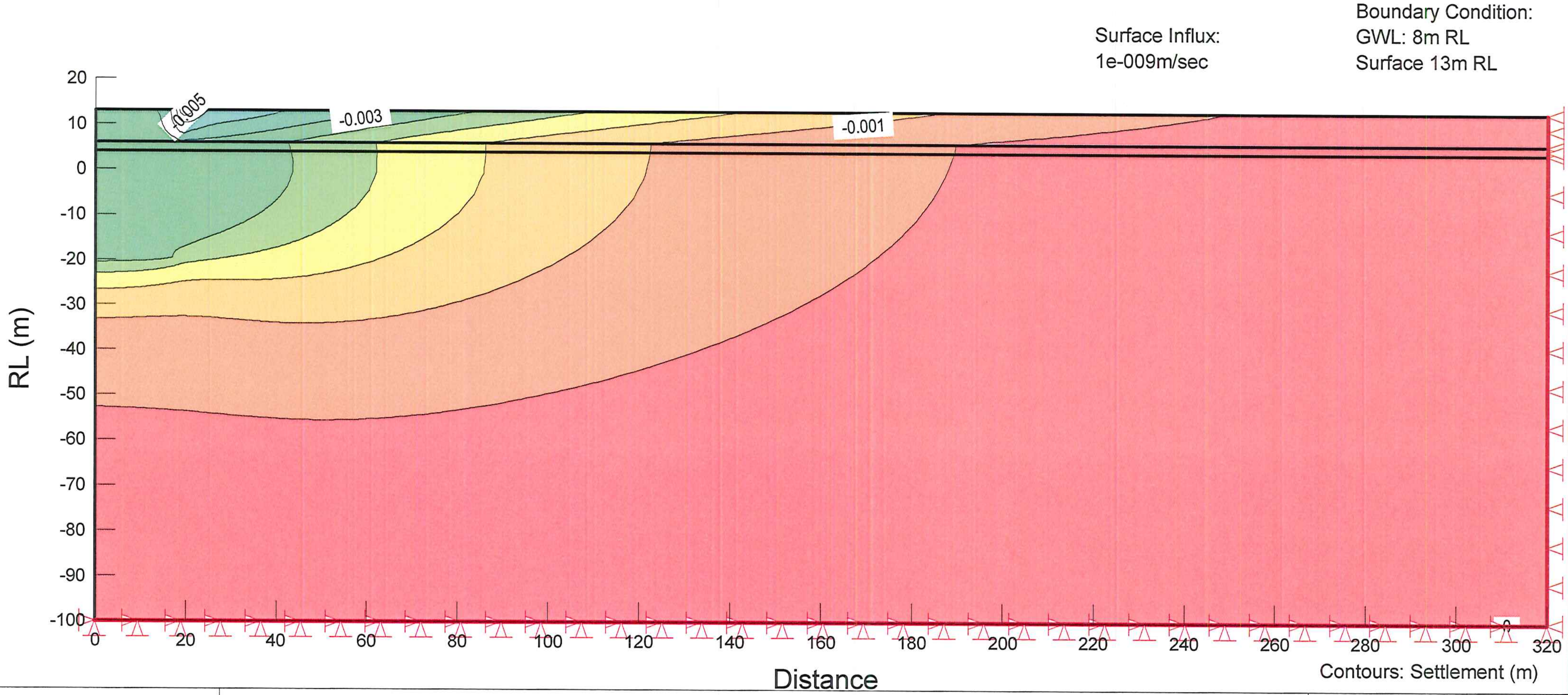

Tonkin & Taylor

Central Interceptor - Groundwater and Surface Settlement Study
Working Shaft 1
WS1.3.4.1 Lined Shaft <1e-9> - Steady State - Stresses

Analysed: EYST
Checked:
Date: Nov 2011
Job No: 26145.300

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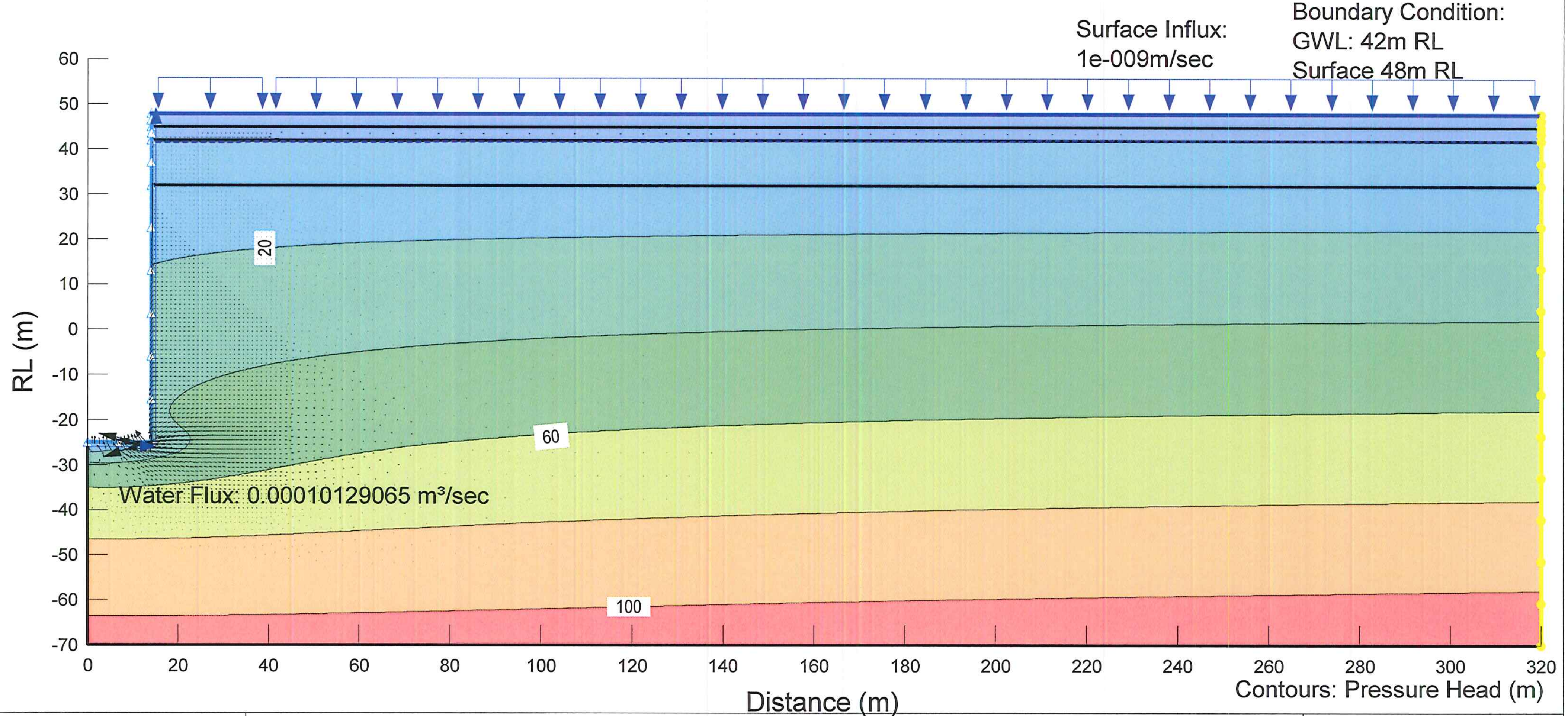

Tonkin & Taylor

Central Interceptor - Groundwater and Surface Settlment Study
Working Shaft 2
WS2.1.4.1 Lined Shaft <1e-9> - Steady State - GWL 42mRL

Analysed: EYST
Checked:
Date: Nov 2011
Job No: 26145.300

File Name: CI.GWS.WS2 axi.gsz
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Analysis Method: Volume Change
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Analysis View: Axisymmetric
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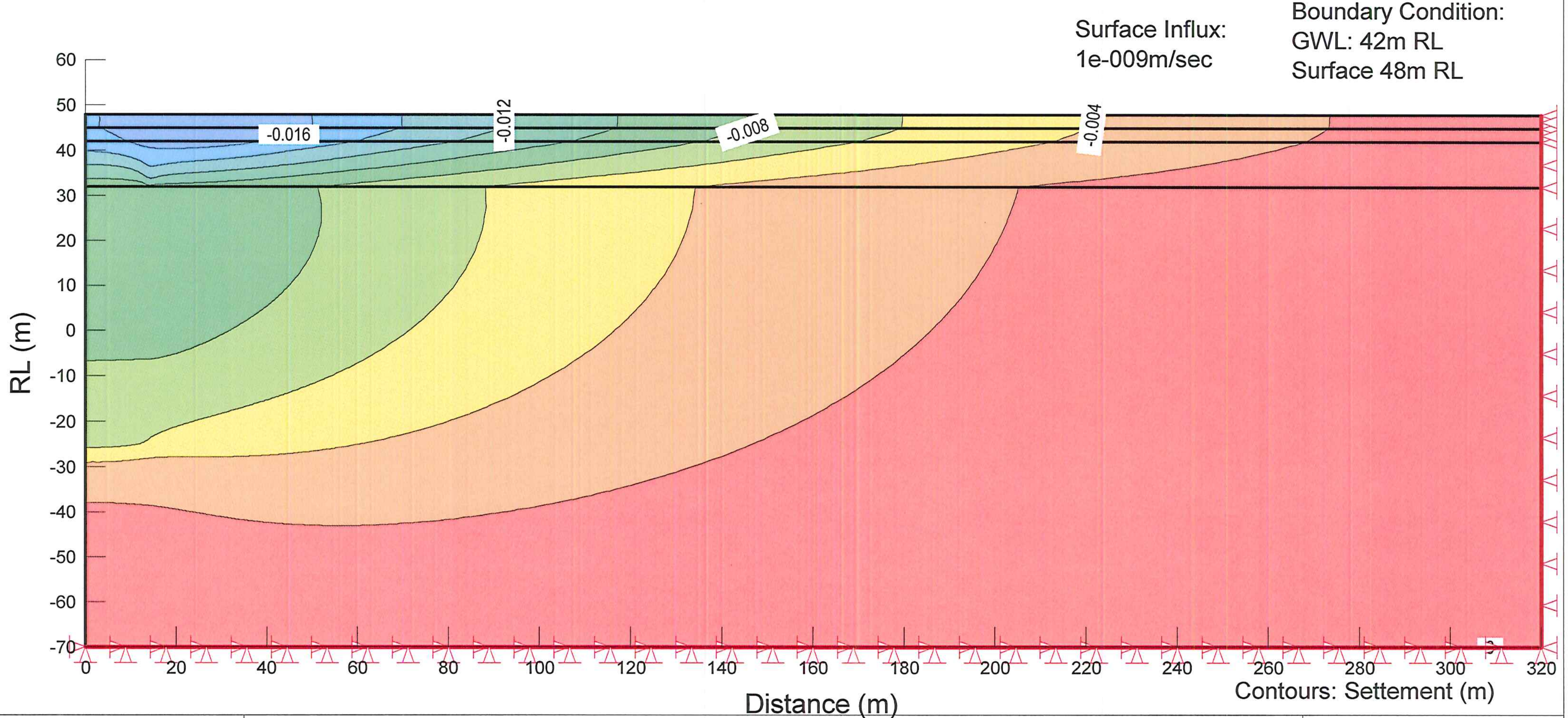

Central Interceptor - Groundwater and Surface Settlment Study
Working Shaft 2
WS2.3.4.1 Lined Shaft <1e-9> - Steady State - GWL 42mRL

Tonkin & Taylor

Analysed: EYST
Checked:
Date: Nov 2011
Job No: 26145.300

File Name: MangerePumpStation_MGW.gsz
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Analysis Method: Steady-State
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Analysis View: Axisymmetric
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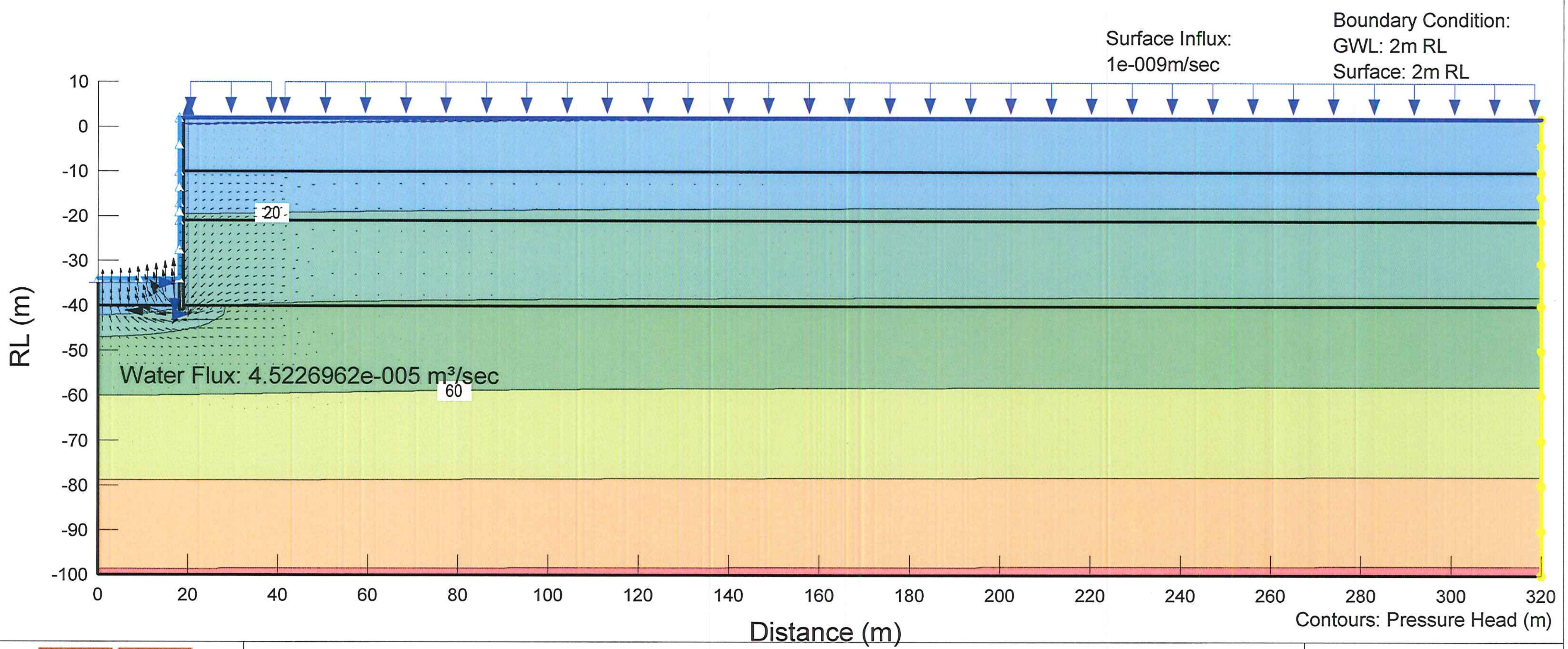

Tonkin & Taylor

Central Interceptor - Groundwater and Surface Settlement Study
Working Shaft 3
WS3.1.4.1 Lined Shaft <1e-9> - Steady State

Analysed: EYST
Checked:
Date: Nov 2011
Job No: 26145.300

File Name: MangerePumpStation_MGW.gsz
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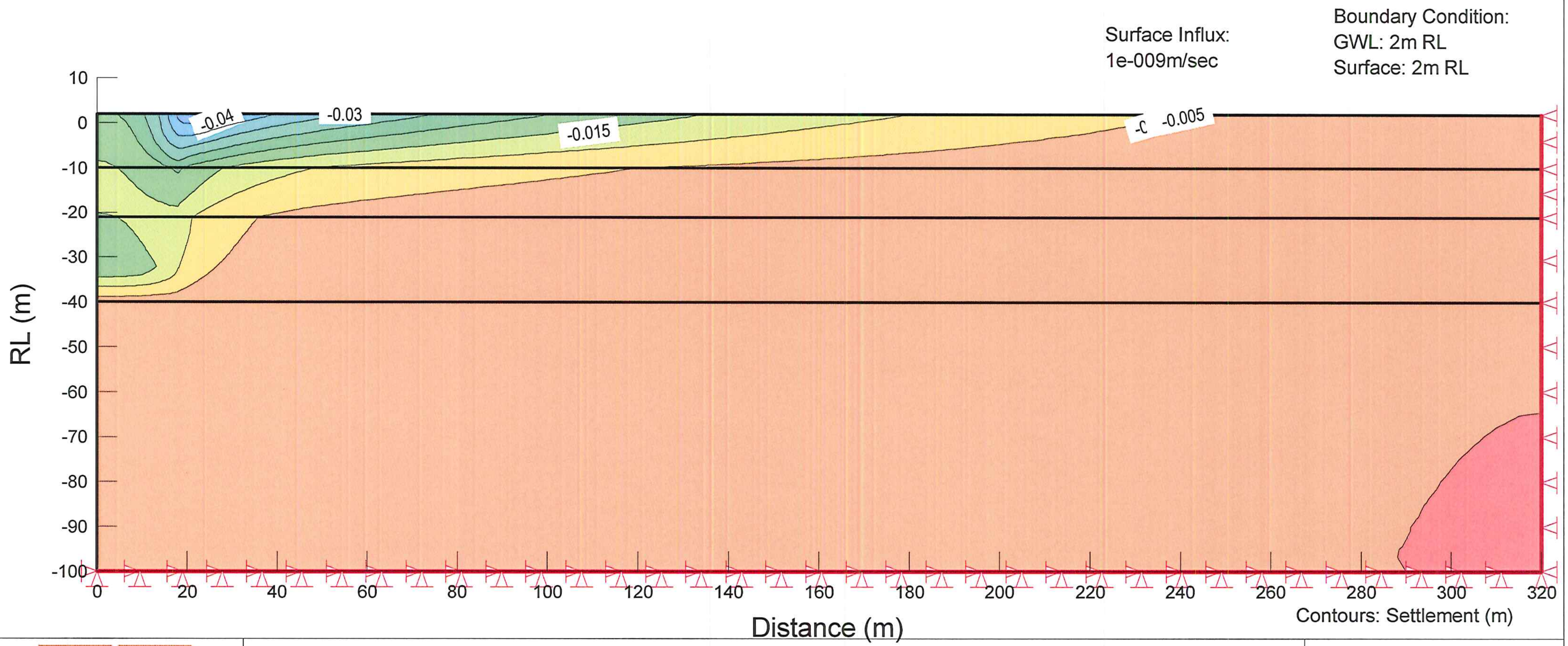

Tonkin & Taylor

Central Interceptor - Groundwater and Surface Settlement Study
Working Shaft 3
WS3.3.4.1 Lined Shaft <1e-9> - Steady State - Stresses

Analysed: EYST
Checked:
Date: Nov 2011
Job No: 26145.300

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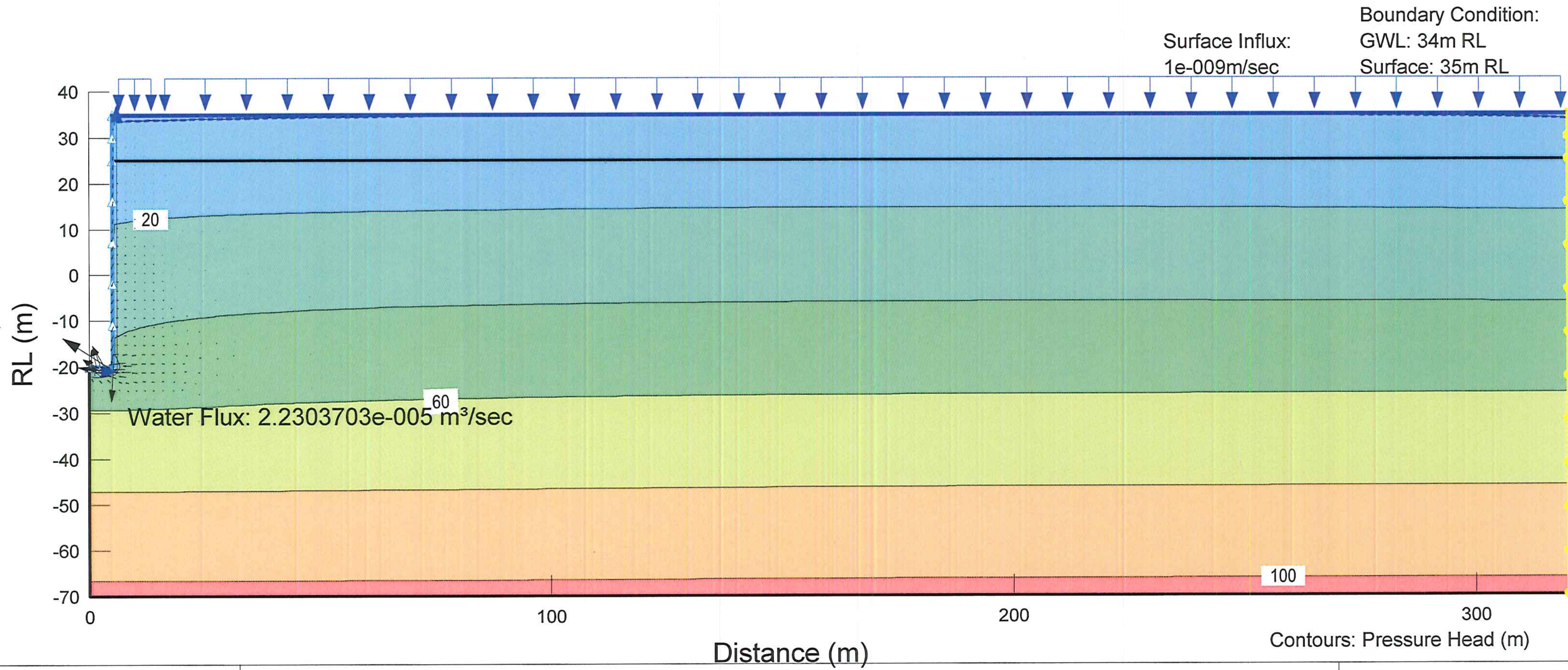

Tonkin & Taylor

Central Interceptor - Groundwater and Surface Settlement Study
Access Shaft 3
AS3.1.4.1 Lined Shaft <1e-9> - Steady State

Analysed : EYST
Checked:
Date: Nov 2011
Job No: 26145.300

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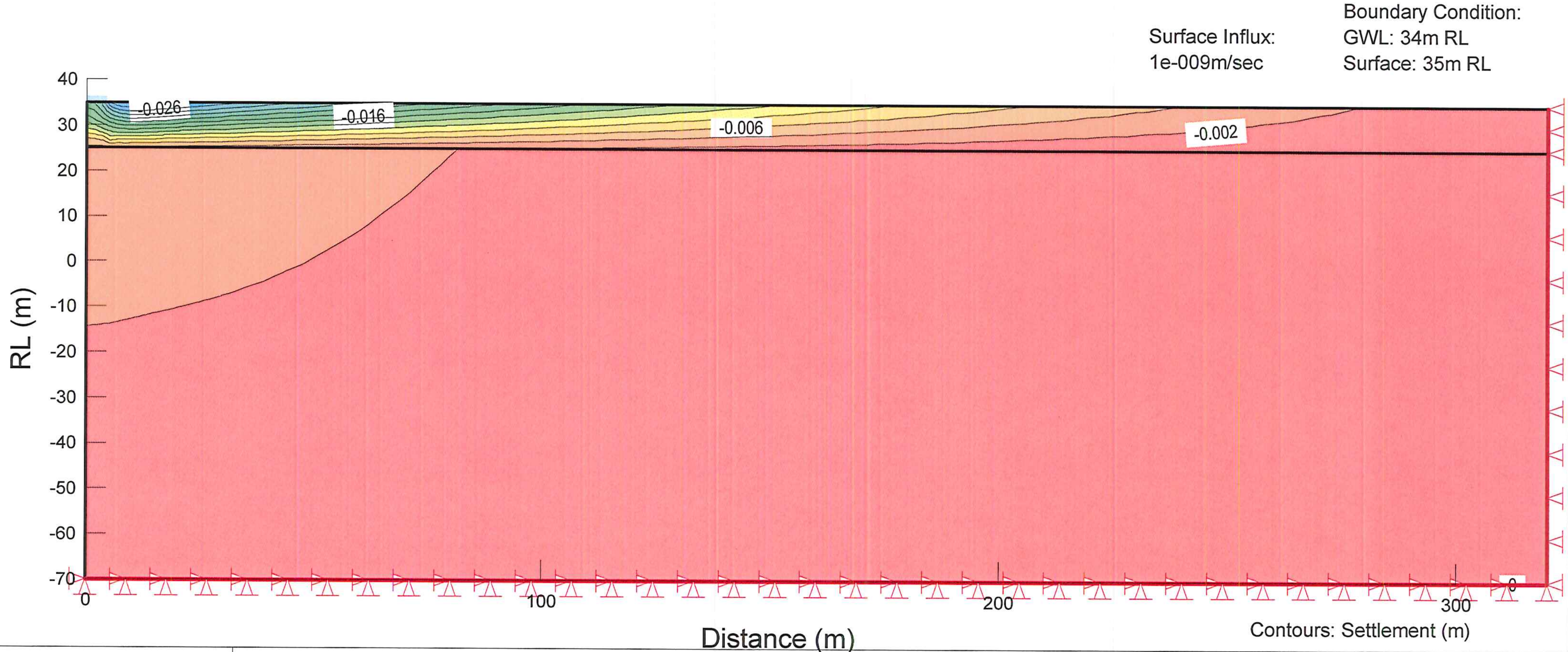

Tonkin & Taylor

Central Interceptor - Groundwater and Surface Settlement Study
Access Shaft 3
AS3.3.4.1 Lined Shaft <1e-9> - Steady State - Stresses

Analysed : EYST
Checked:
Date: Nov 2011
Job No: 26145.300

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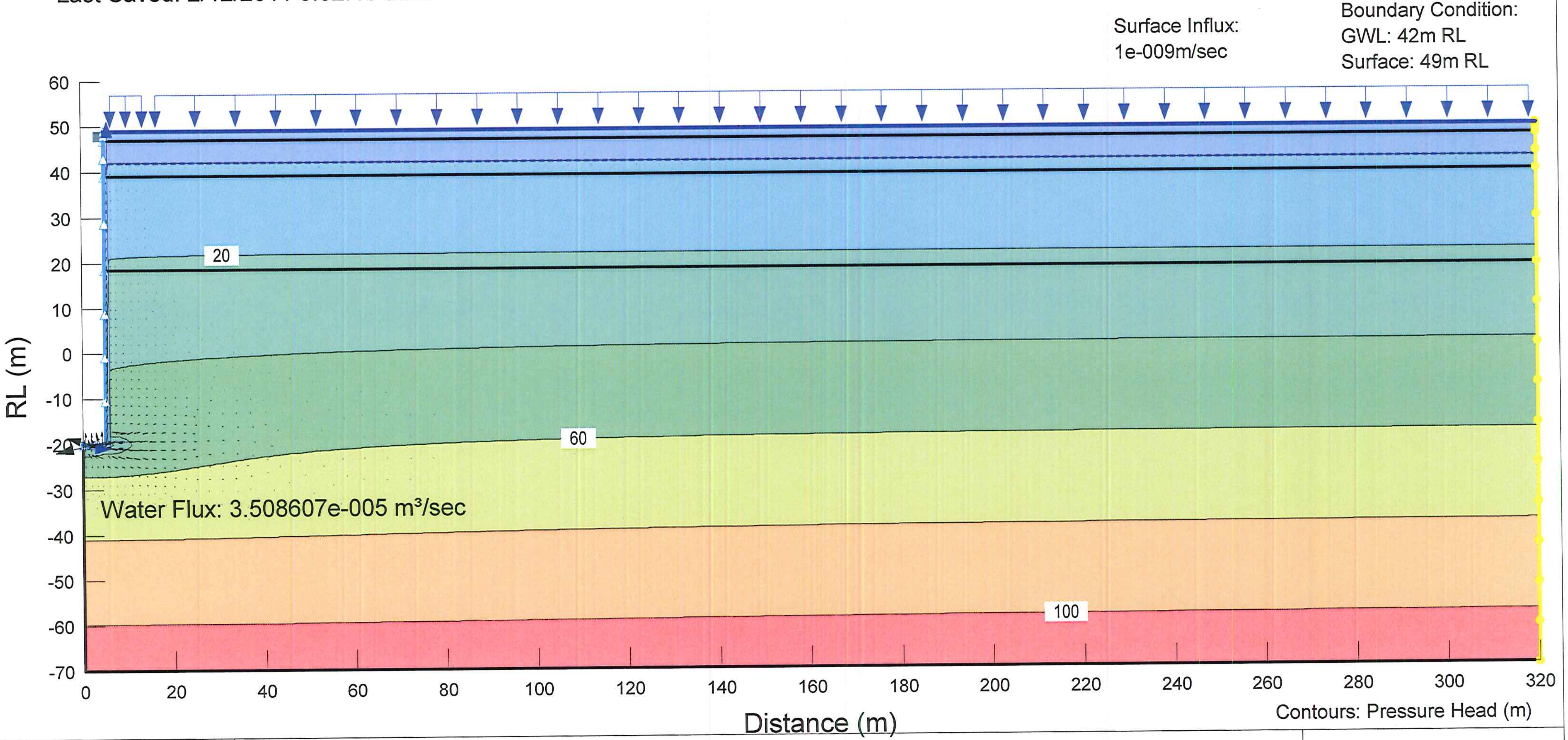

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Central Interceptor - Groundwater and Surface Settlement Study
Access Shaft 4
AS4.1.4.1 Lined Shaft <1e-9> - Steady State

Analysed : EYST
Checked:
Date: Nov 2011
Job No: 26145.300

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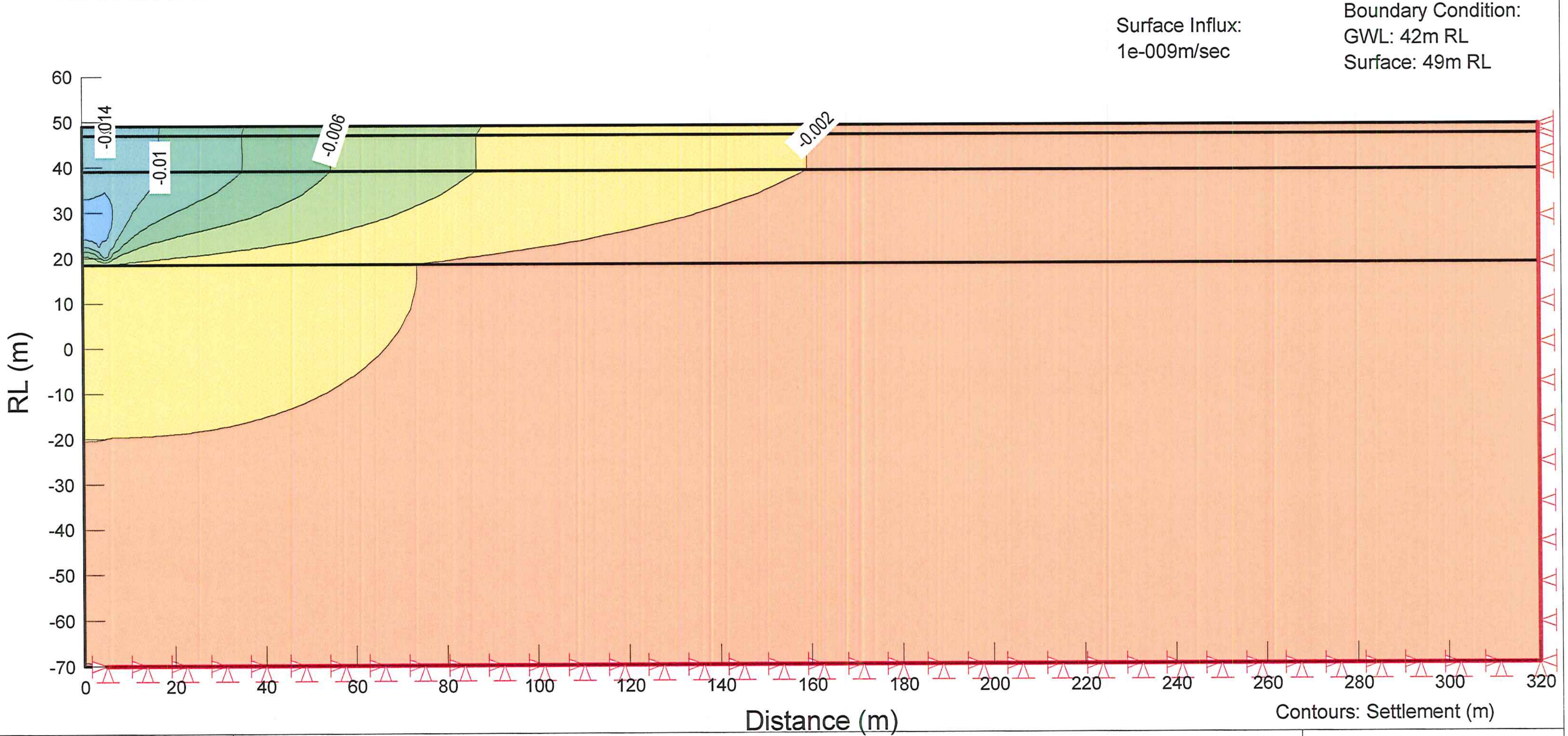

Tonkin & Taylor

Central Interceptor - Groundwater and Surface Settlement Study
Access Shaft 4
AS4.3.4.1 Lined Shaft <1e-9> - Steady State - Stresses

Analysed : EYST
Checked:
Date: Nov 2011
Job No: 26145.300

File Name: CI.GWS.AS7 axi.gsz
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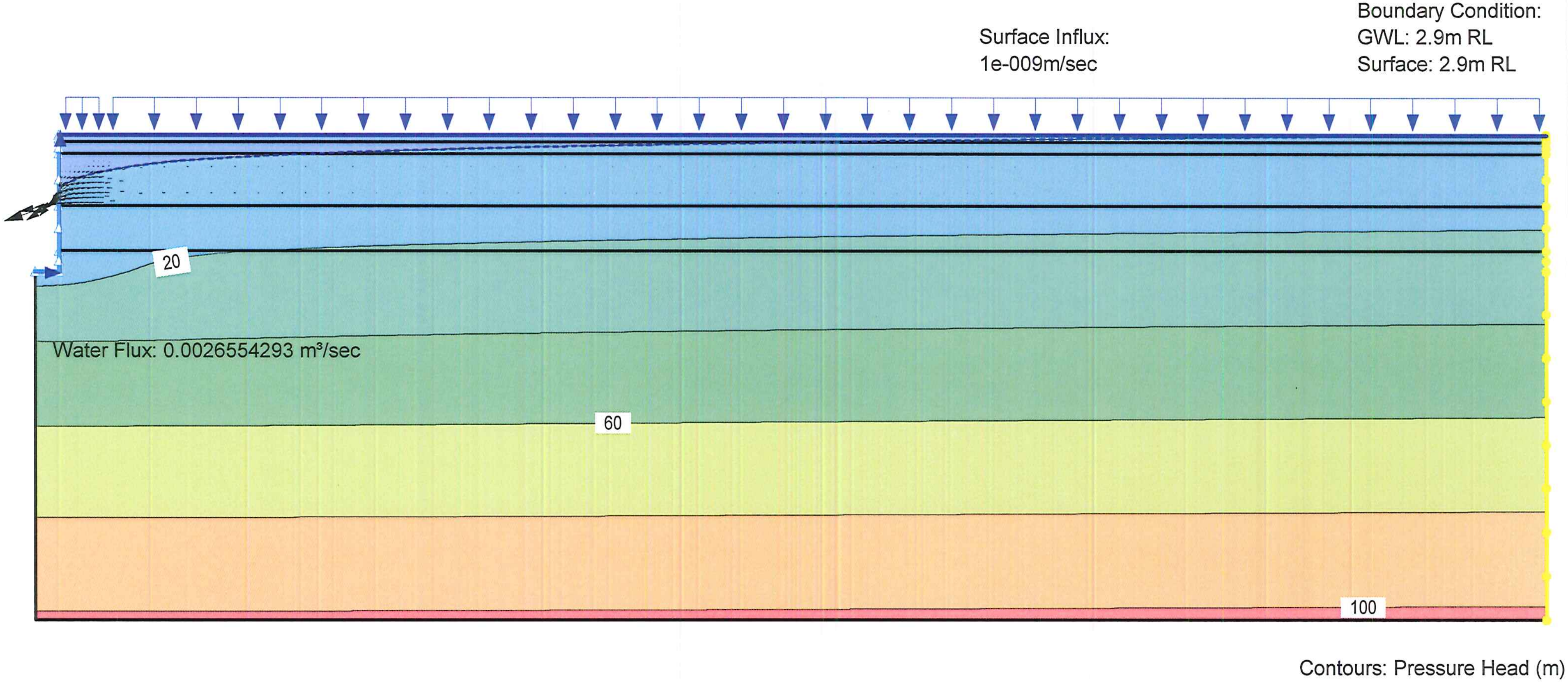

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Central Interceptor - Groundwater and Surface Settlement Study
Access Shaft 7
AS7.1.2.1 Unlined Shaft - Steady State

Analysed : EYST
Checked:
Date: Nov 2011
Job No: 26145.300

File Name: CI.GWS.AS7 axi.gsz
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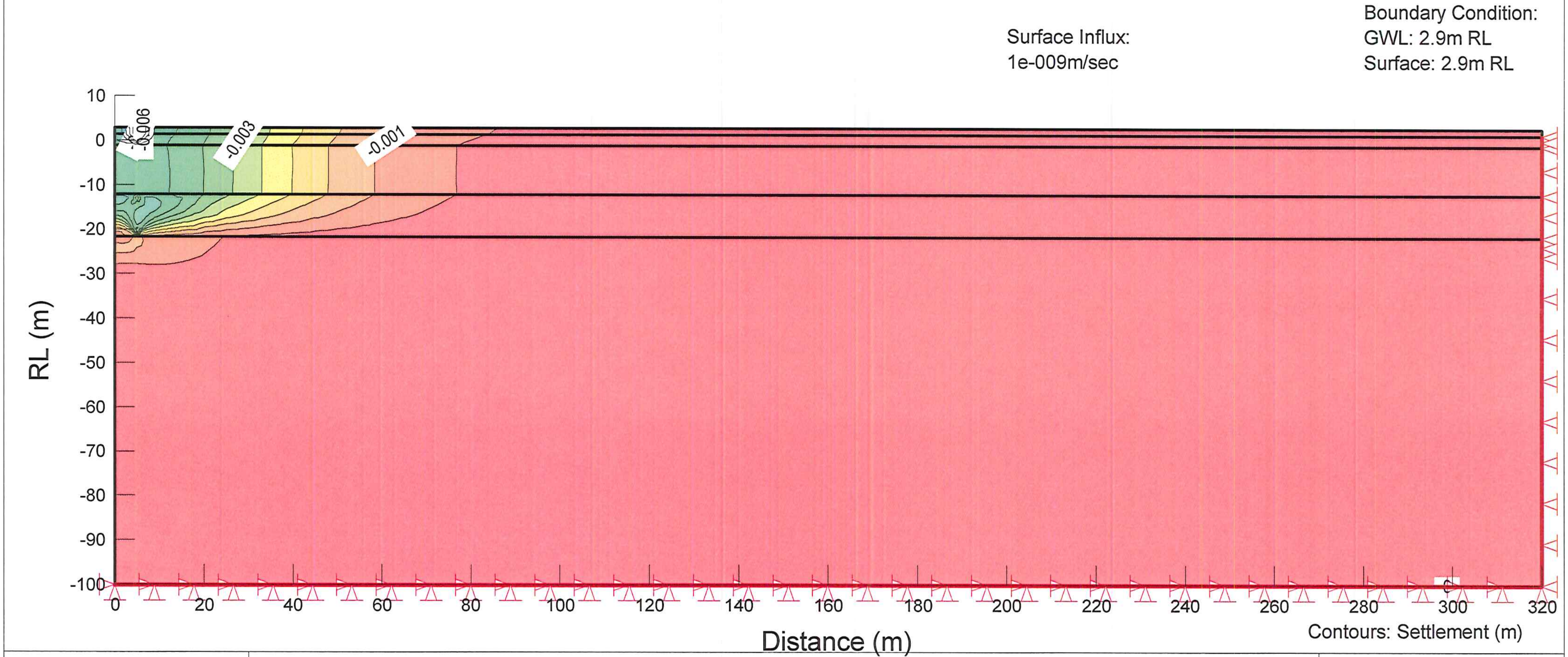

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Central Interceptor - Groundwater and Surface Settlement Study
Access Shaft 7
AS7.3.4.1 Lined Shaft <1e-9> - Steady State - Stresses

Analysed : EYST
Checked:
Date: Nov 2011
Job No: 26145.300

Appendix F: Selected structures and other features potentially sensitive to groundwater changes and settlement along route

- Table F1: Structures
- Table F2: Infrastructure
- Table F3: Aquifers and Watercourses

Table F1 – Selected structures in vicinity of tunnel alignment

Feature Chainage	Location	Offset from tunnel alignment	Description
10,200 to 10,300 Main tunnel alignment	West of Gt North Road, Western Springs	Directly above alignment to 80m offset	Motat Complex, including modern low rise structures (largest approx. 70m x 50m) and two storey masonry brick structure (approx. 25m x 20m)
11,100 Main tunnel alignment	South of Linwood Ave, St Lukes	100m offset	Auckland Institute of Studies, multi-storey office block (approx. 50m x 30m)
12,200 Main tunnel alignment	Off Lyon Ave, St Lukes	Directly above alignment	Large warehousing facility (approx. 80m x 60m)
12,400 Main tunnel alignment	Morning Star Place, St Lukes	Directly above alignment	Multi-storey apartment blocks (up to 5-storeys) (circa 10 structures up to approx. 50m x 20m in size).
12,500 Main tunnel alignment	Wagener Place, St Lukes	Directly adjacent to alignment	2-3 storey large office/retail block (Two structures - approx. 140m x 50m, 100m x 40m)
14,700 Main tunnel alignment	William Blofield Ave, Mt Roskill	Direct adjacent to alignment	Wesley Intermediate School Multi storey brick structure (approx. 100m x 15m)
15,000 Main tunnel alignment	Stoddard Road, Mt Roskill	Directly above alignment	Large warehousing facility (approx. 220m x 70m)
15,500 to 15,500 Main tunnel alignment	May Road, Mt Roskill	70m - 100m offset from alignment	Large warehousing facility (approx. 240m x 160m)
23,200 Main tunnel alignment	Mangere WWTP	100m - 200m offset from alignment	Waste Water Treatment Plant structures (Size of total facility is approximately 500m x 400m)
2,700 Link Sewer 3 (LS3) Alignment	White Swan Road Mt Roskill	30m - 40m offset from alignment	Multi storey structure and large electrical substation (Size of total facility is approximately 100m x 60m)

Table F2 –Infrastructure in vicinity of tunnel alignment

Feature Chainage	Location	Offset from tunnel alignment	Description
10,500 Main tunnel alignment	Western Springs	Tunnel alignment passes under motorway alignment. Interchange structures are directly above alignment	North-Western Motorway, St Lukes Road interchange. Two span overbridge and associated retaining walls.
11,500 Main tunnel alignment	Near Asquith Ave, Mt Albert	Tunnel alignment passes under rail alignment	Western Rail Line, approximately at-grade section of railway. Asquith Ave level crossing adjacent.
15,100 to 15,200 Main tunnel alignment	Mt Roskill	Tunnel alignment passes under motorway alignment	South-Western Motorway, approximately at-grade section of motorway.
18,150 Main tunnel alignment	Hillsborough Road	Tunnel alignment passes under pipeline alignment	"Refinery to Auckland" Oil Pipeline ~275mm OD steel pipeline in shallow trench, together with high pressure gas pipeline.
20,100 Main tunnel alignment	Manukau Harbour	Tunnel alignment passes under sewer alignment	Western Interceptor precast concrete sewer line in dredged harbour trench.
21,150 Main tunnel alignment	Ambury Regional Park, Mangere	Tunnel alignment passes under pipeline alignment	"Refinery to Auckland" Oil Pipeline ~275mm OD steel pipeline in shallow trench.

Table F3 –Aquifers in vicinity of tunnel alignment

Feature Chainage	Location	Offset from tunnel alignment	Description
10,000 to 13,000 Main tunnel alignment	Western Springs to Mt Albert	Tunnel alignment passes under regional aquifer	Western Springs Volcanic Aquifer. Uses include groundwater for potable supply, groundwater for industrial use, disposal of stormwater and springs for recreational use. (Pattle Delamore Partners Limited, 2005). The aquifer had a groundwater allocation of 2.84M m3/year as of 2002 (Crowcroft & Bowden, 2002), which is low compared with availability. Aquifer is main source of water to Western Springs Lake, and groundwater take is carefully managed to ensure sufficient supply for lake flushing (Crowcroft & Bowden, 2002)
23,200 Main Tunnel Alignment	Mangere	Tunnel alignment terminates at edge of mapped regional aquifer	Manukau Kaawa Aquifer. Important local source of groundwater used for Irrigation and Commercial uses at Mangere. Bores are generally screened across coarse shell and/or sand beds. The Mangere aquifer had a groundwater allocation of circa 630,000 m3/year as of 2002 (Crowcroft & Bowden, 2002)