Central Interceptor

Main Project Works Detailed Design

WATERCARE SERVICES LIMITED

Combined Settlement Report for the Link Sewers

DSLSC-DEL-REP-GT-J-100262 | 0

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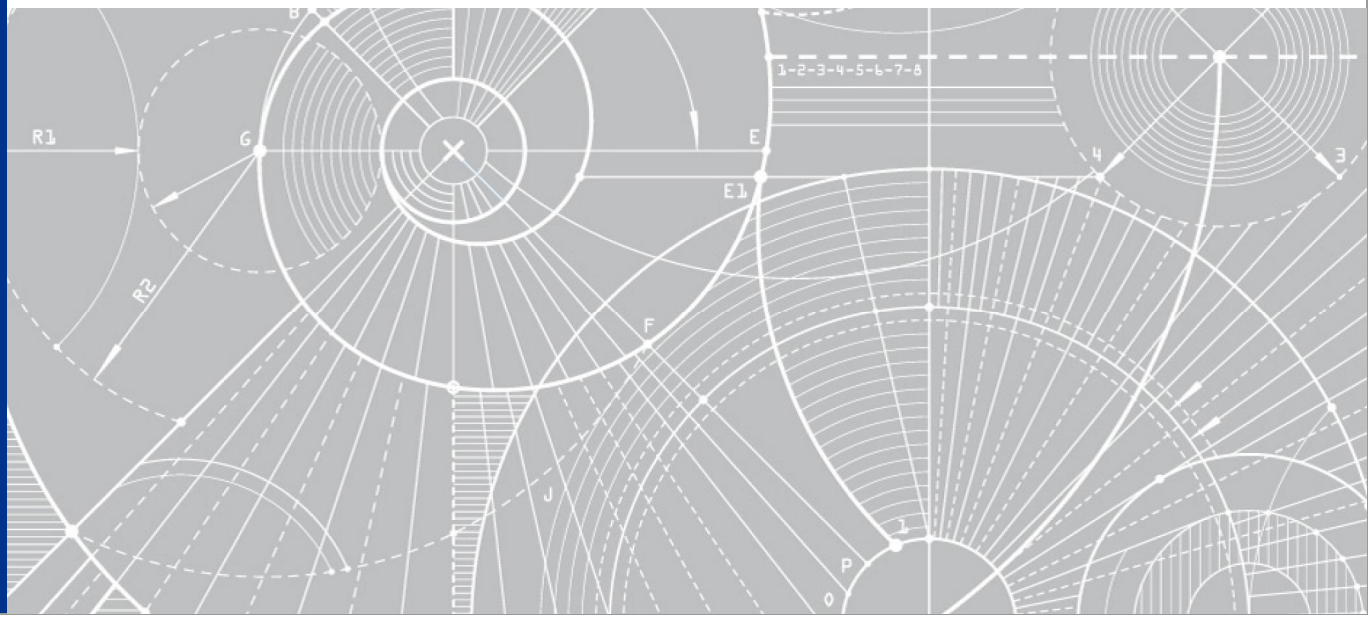

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1. Introduction

1.1 Project overview

The proposed Central Interceptor (CI) project incorporates two link sewers referred to as Link Sewers B and C. The Link Sewers will have nominal diameters of 2.5m and lengths of 1.2km and 3.2km respectively.

The construction of these sewers and their associated shafts will induce settlements on the ground surface due to both mechanical and groundwater drawdown associated effects. The combined settlement is not to exceed limits set in the Consent Conditions which are 50mm total and 1:1,000 differential settlement.

Watercare has engaged Jacobs in association with AECOM (formerly URS) and McMillen Jacobs Associates as Principal Engineering Advisor (PEA) responsible for undertaking various investigations and preparing designs and construction documentation for CI project, including the ground movements due to the construction of the link sewers.

1.2 Scope and objective

The scope of this report is to:

- Assess ground surface settlement due to the mechanical effects of the tunnel excavation process
- Assess ground surface settlement due to the groundwater drawdown from the tunnel excavation process
- Integration of these settlements to assess the differential settlement magnitude and its impact on nearby existing buildings, services and utilities along the link sewer alignments

1.3 Reference documents

Sources of factual information used to produce this report are summarised in Table 1

Table 1 Sources of factual information

Name	Author	Year	Document Number	Use of this report/drawing
Central Interceptor Main Project Works Detailed Design Geotechnical Interpretative Report	PEA	2016	PWCIN-DEL-REP-GT-J-100048 Volumes 1 and 2	Interpreted geological data to be used in analysis
Central Interceptor Main Project Works Detailed Design – Geotechnical Factual Report	PEA	2015	PWCIN-DEL-GT-J-100047 Volumes 1 – 7	Site Investigation data used for geotechnical parameters, risks, geological and geotechnical models.
Drainage Sewer Interceptor Drawings	PEA	2016	DWG No. 2012061.001 (Cover page)	Drawings containing borehole locations, geological sections, shaft locations and general arrangement
Drawdown estimation due to Link Sewer C Tunnel Construction	PEA	2016	XX	Groundwater drawdown data from permeability analysis used on soil models

2. Ground and Groundwater Conditions

Ground and groundwater conditions, geological sections and derivation of geotechnical material parameters expected to be encountered in the project area are discussed in detail in the Geotechnical Interpretive Report (PWCIN-DEL-REP-GT-J-10048 Volumes 1 and 2). Geotechnical drillhole logs, site plans, and other investigation results are presented in the Geotechnical Factual Report (PWCIN-DEL-REP-GT-J-10047 Volumes 1 - 7).

The geological sections and drawings used to develop the ground models for these analyses are presented in Appendix A. Geotechnical units adopted for the project are presented in Table 2. The cross-sections analysed are presented in Appendix B. The geotechnical parameters used in these analyses are attached in Appendix C.

Groundwater varies along the tunnel chainage and is artesian in several areas. Descriptions of groundwater conditions are presented in the GIR. The change in groundwater levels during tunnel construction are documented in a separate report (XXX). Upon completion of the tunnel, it is considered that the groundwater surface over the long term will recharge to the level prior to tunnel construction. The soil is thus expected to rebound to a certain degree.

Table 2 Geotechnical units adopted for design parameters and relationship with geological unit. (after PWCIN-DEL-REP-GET-J-10048)

Stratigraphic/Geological Unit	Geotechnical Units		Lithology	Material Type
Made Ground	Made Ground	Engineered Fill	Clay, silt, sand and gravel	Soil
		Non-Engineered Fill		
Post AVF Tauranga Group alluvium and marine sediments	Recent Alluvium		Silt and sand	
Tauranga Group including Puketoka Fmn., estuarine, undifferentiated, colluvium (TGA)	Undifferentiated Tauranga Group	Cohesive	Clay and silt	
		Granular	Sand	
Kaawa Formation	Kaawa Formation		Shelly with silt and sand	
Auckland Volcanic Field (AVF)	Tuff/Ash/Scoria		Silt, sand and gravel, can be intermixed with clay	
	Basalt		Intact, jointed, vesicular and rubbly	Rock
East Coast Bays Formation (ECBF)	Residually to highly weathered cohesive soils		Silt and clay	Soil
	Residually to highly weathered granular soils		Sand	
	Moderately weathered to unweathered ECBF		Mudstone and muddy sandstone	Rock
Parnell Volcaniclastic Conglomerate (PVC)	Parnell Volcaniclastic Conglomerate		Course sandstone to conglomerate	

3. Methodology

3.1 Settlement criteria

The maximum allowable total settlement and differential settlement for the project has been set at < 50mm and 1 in 1,000 respectively in the Resource Consent.

All structures are therefore, to have a total and differential settlement less than these criteria to prevent damage to existing buildings, services and utilities.

3.2 Expected causes of surface ground movement

The expected causes of surface ground movement due to tunnelling can generally be classified into two broad categories:

- Mechanical Tunnelling induced surface movement
- Indirect surface movement due to dewatering and change in effective stress in the alluvium and residual soil which may extend many meters away from the actual tunnel alignment

3.2.1 Tunelling induced surface movement

Tunnelling induced surface ground movement had been assessed assuming the following main causes:

- Volume loss when actual ground loss caused by the tunnel excavation exceeds the theoretical excavation volume
- Tunnel face stability

The volume loss and can be related to the amount of surface ground movement expected by using analytical and numerical analysis.

Under certain ground conditions and excavation methods, it is also possible for ground heave to occur at the ground surface during the excavation works. This can occur in soft soils when face pressures are too high. However due to the tunnel being predominantly within the unweathered to moderately weathered East Coast Bays Formation (ECBF) Rock, and the depth of the tunnel below ground level, as well as the low stress environment, this is considered to be unlikely for the link sewer excavation.

The magnitude of tunnel induced ground movement is related to the following parameters:

- Depth and diameter of tunnel
- Ground and groundwater conditions at tunnel face and above tunnel crown level; and
- Excavation and tunnel support methods

In this report the surface ground movements due to excavation are assessed using analytical methods proposed by Peck (1969) and Mair (1993). The input parameters for these methods are calibrated by undertaking numerical analyses using Rocscience Finite Element Analysis (FEA) software Phase2 version 9.0.

During tunnel excavation, face stability is not anticipated to be a major issue as the tunnel will predominantly be deeply embedded within unweathered to moderately weathered ECBF, which is homogenous and relatively stiff in nature. Thus there will unlikely be any cavities/weathered zones or mixed face conditions which will cause the face of the excavation to collapse.

3.2.2 Indirect dewatering induced surface movement

Indirect dewatering will occur during the pipejacking of the tunnel before the tunnel route is complete. Depending on the method of tunnelling adopted, there will be some dewatering and depressurisation at the face. There is also potential for seepage into the tunnel until post-jacking grouting has been completed. The dewatering and depressurisation in the excavated area causes changes in effective stress and induces further

surface ground movement. The groundwater drawdown is only expected to affect the Tauranga Group Alluvium (TGA) and residual soil of the ECBF, as the ECBF rock is sufficiently stiff as not to be affected.

Review of the ground conditions along the Link Sewer C and B routes indicates that the majority of the tunnel will be excavated within the ECBF Rock, where the potential for surface ground movement due to groundwater drawdown is considered to be low. However, the increase in effective stress will still affect the overlying TGA which was initially below the water table.

In order to assess the surface ground movement due to dewatering of the alluvium, both numerical and analytical assessments have been undertaken. The numerical analysis was carried out in Rocscience software Phase2 version 9. The analytical analysis was carried out by using the following equation based on theoretical elasticity

$$S = \frac{H\Delta q}{E}$$

Equation 1

Where:

S = Settlement

H = Soil thickness

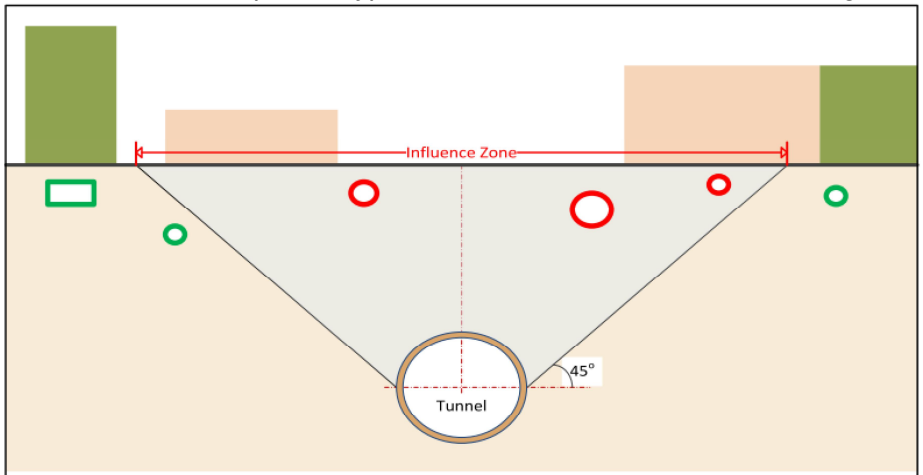
Δq = Change in effective stress

E = Soil modulus

Further details on the drawdown of the groundwater table are available in “Drawdown estimation due to Link Sewer C Tunnel Construction” (XXX).

3.2.3 Zone of influence

The “zone of influence” of tunnelling is the volume of geo-material influenced by the tunnelling work. Any buildings or other structures located within this zone shall be subjected to the damage criteria highlighted in section 3.1. An example of a typical zone of influence can be seen in Figure 1.



Influence Zone of Bored Tunnel Showing Buildings to be considered in pink and Utilities to be considered in red.

Figure 1Typical Zone of Influence observing the Mohr Coulomb failure criterion

3.3 Method of assessment

3.3.1 Overview

The tunnel induced surface ground movements are assessed using analytical methods proposed by Peck (1969) and Mair (1993). Numerical modelling has been undertaken to calibrate appropriate input parameters for the anticipated ground conditions along the tunnel alignment. Presentation of the surface ground movements and plotting of surface ground movement contour plans have been undertaken using Geographical Information System (GIS) software.

A flowchart showing the parameter inputs, settlement calculation formulas and overall process is summarised in the figure below. A description of the assessment methods for tunnels and surface excavations are provided in the following sections.

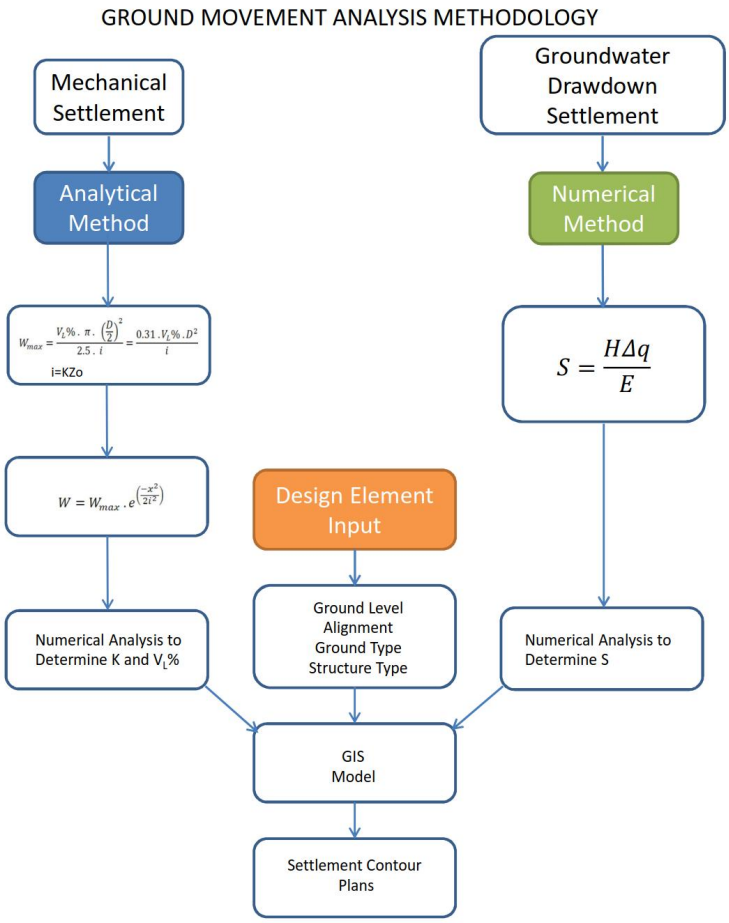


Figure 2 Surface ground movement assessment due to tunnelling flow chart

3.3.2 Analytical method

The analytical method for determining the transverse settlement trough assumes a Gaussian curve and allows for a quick assessment of the surface settlement contours over the tunnel section. This method is based on green field site assessment, though it is widely recognised that the presence of buildings, basement excavations and deep foundations can affect the actual settlements induced by tunnelling.

Settlements are idealised to occur in the shape of an inverted Gaussian trough, with an area equal to the volume loss (V_L) parameter, which is often expressed as a percentage of the tunnel area and represents both material lose at the tunnel face and closure of the tunnel annulus due to relaxation. A trough width parameter (K) determines the steepness and width of the settlement trough and is related to the ground conditions above the tunnel excavation. Figure 3 shows the relationship between the tunnel depth and the shape of the settlement trough.

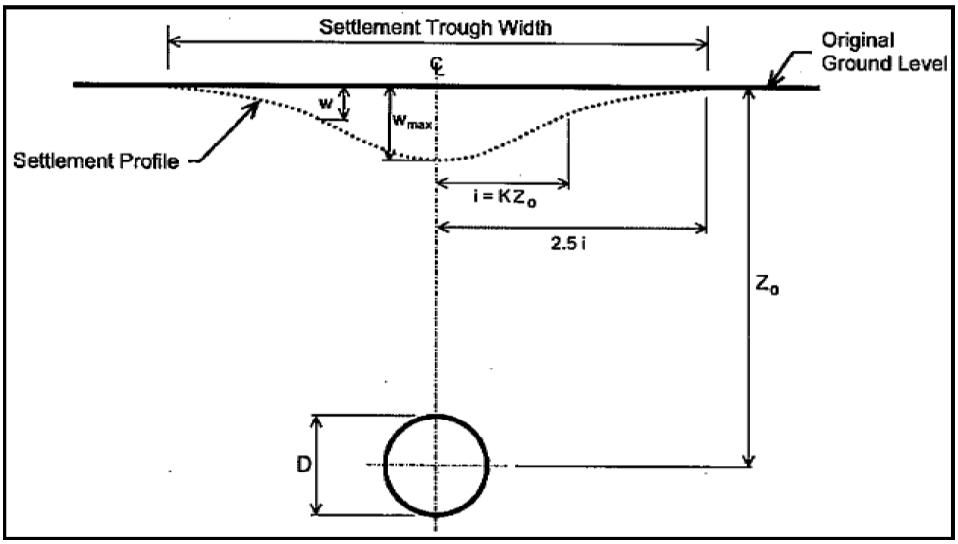


Figure 3 Typical settlement profile due to tunnel excavation

The maximum settlement due to tunnelling is calculated using the following equation:

$$W_{max} = \frac{V_L\% \times \pi \times (\frac{D}{2})^2}{2.5 \times i} = \frac{0.31 \times V_L\% \times D^2}{i}$$

Equation 2

Where:

W_{max} = maximum settlement at the tunnel centreline
V_L = volume of ground loss (ratio of ground loss volume/tunnel volume per meter length in %)
D = equivalent diameter of a tunnel
I = Location of maximum settlement gradient or point of inflection ($i=K.Z_0$)
K = settlement trough parameter (function of ground type)
Z_0 = the depth from ground surface to tunnel springline/tunnel centre

The shape of the curve can be expressed by the following mathematical relationship.

$$W = W_{max} \times e^{\frac{-x^2}{2i^2}}$$

Equation 3

Where:

x is the horizontal distance from the tunnel centre.

A significant amount of research involving field observations and model tests has been devoted to the estimation of W_{max} and the 'i' values for different ground conditions. The recommended 'i' values by various researchers are shown in the table below. However it is not clear how these values related to the ground conditions anticipated along the Link Sewers.

To estimate appropriate K and V_L% values for the anticipated subsurface ground conditions along the Link Sewers, numerical modelling has been undertaken.

Table 3 Recommended 'i' values by various researchers

Researcher Name	i-value	Remarks
Peck (1969)	$\frac{i}{R} = \left(\frac{z_0}{2R}\right)^n$: n = 0.8 to 1.0	Based on field observations
Atkinson & Potts (1979)	$i = 0.25(z_0 + R)$: for loose sand, $i = 0.25(1.5z_0 + 0.5R)$: for dense sand and over consolidated clay	Based on field observations and model tests
O'Reilly & New (1982)	$i = 0.43z_0 + 1.1$: cohesive soil $i = 0.28z_0 - 0.1$: granular soil	Based on field observations of UK tunnels
Mair (1993)	$i = 0.5z_0$	Based on field observations worldwide and centrifuge test
Attewell (1977)	$\frac{i}{R} = \alpha.\left(\frac{z_0}{2R}\right)^n$:α = 1 and n = 1	Based on field observations of UK tunnels
Clough & Schmidt (1981)	$\frac{i}{R} = \alpha.\left(\frac{z_0}{2R}\right)^n$:α = 1 and n = 0.8	Based on field observations of US tunnels

3.3.3 Numerical calibration

The input parameters VL% and K have been calibrated by undertaking numerical analyses using Rocscience software Phase2 version 9.0.

A number of tunnel sections were analysed to represent a few scenarios:

- CH350 of Link Sewer C – where the tunnel was deep within the ECBF Rock
- CH1300 of Link Sewer C – where the tunnel was closer to the surface but still within the ECBF Rock
- CH3200 of Link Sewer C – where the tunnel was within Residual Soil of the ECBF, with the crown of the tunnel surrounded by Tauranga Group Alluvium, and the tunnel being very close to the ground surface

The following assumptions were made in the numerical analyses for calibration of VL% and K:

- Axisymmetric analysis of horizontal tunnel to estimate inward movement of face under differing support conditions
- 2D plane strain elasto-plastic analysis with Mohr-Coulomb failure criteria condition
- Total stress analyses
- Ground conditions as per geotechnical section drawings
- Tunnel diameter with an excavation extrados of approximately 2.4m
- A uniformly distributed surface surcharge of 20kPa across the whole model to represent surface loading
- Full face – unsupported excavation with no face pressure for the tunnel excavation in rock units
- In situ stress ratio, k (horizontal/vertical) as per the GIR and summarised in Table 4
- Geotechnical design parameter inputs as per the GIR and summarised in Table 4

Table 4 Summary of Geotechnical Design Parameters for numerical analysis

Geotechnical Units	Unit Weight (kN/m^3)	Modulus (MPa)	Friction Angle (°)	Effective cohesion (kPa)	Tensile Strength (kPa)	Insitu stress ratio, k
Tauranga Group Alluvium	16	7	28	7	-	0.5
ECBF Residual Soil	18	30	32	6	-	0.47
ECBF Moderately weathered to Unweathered	20	400	34	100	520	1.2

The output of the numerical analysis is the calibration of:

- Volume loss (V_L%) parameter, which is expressed as a percentage of the tunnel area and represents both closure of the tunnel annulus and material lost at the tunnel face due to relaxation; and
- Trough width parameter (K) which determines the steepness and width of the settlement trough.

The outputs of the numerical modelling are given in Appendix E. The summary of the comparison of V_L% and K values of the numerical and analytical methods are given in Table 5.

Table 5 Comparison of VL% and K values for different assessment methods

Assessment Method		Volume Loss (V_L)%	Settlement Trough Parameter, K
Analytical Method(1)		0.5 – 1.0	0.5
Numerical Method(2)	CH350	1.1	~1.0
	CH1300	0.7	~0.3
	CH3200	7	0.7
Notes: 1) Based on the method proposed by Mair (1993) 2) FEA Analysis using Phase^2 3) Settlement at CH350 and CH1300 was <1mm, and highly sensitive to “noise” in the modelling			

Based on the output of the numerical method calibration, the K parameter varied from 0.3 to 1.0. However, the K values at CH350 and CH1300 are approximate as there was a considerable amount of noise and the settlement did not exceed 1mm in the numerical modelling, making it more difficult to ascertain the point of inflection in the trough. Generally, smaller K values will generate a steeper and narrower settlement profile while larger K values will generate a shallower and wider settlement profile. This is shown in Figure 4. Smaller K values thus lead to larger total settlement and steeper differential settlement values. A typical value of 0.5 is considered appropriate for clay or cohesive ground conditions, while 0.35 is approximated for looser sands and gravels. It is known that numerical analyses tend to produce a wider and shallower surface settlement trough compared with the empirical methods and field measurements. Therefore, based on these results and the field and centrifuge experiments of Mair (1993), a **K value set as 0.5** is appropriate and considered slightly conservative.

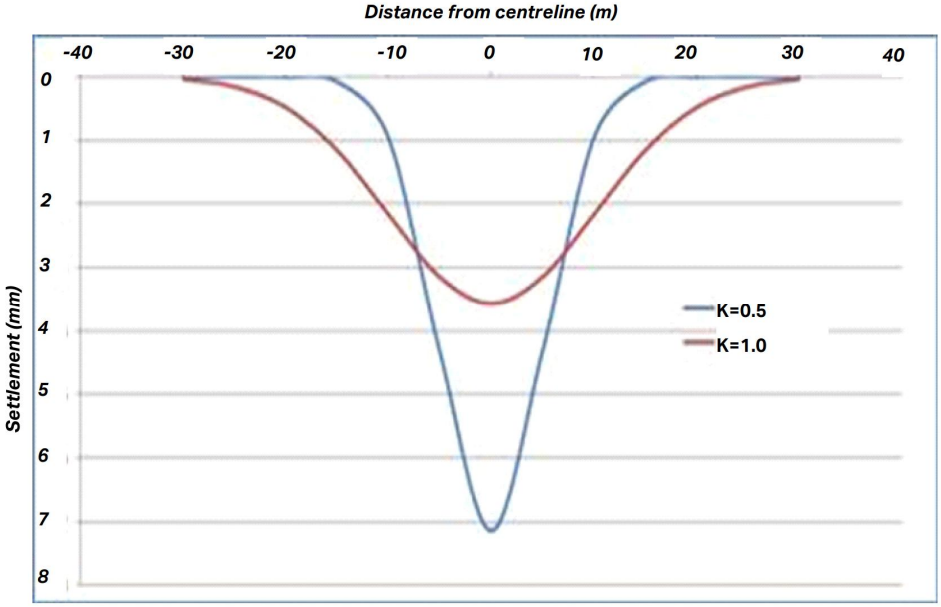


Figure 4 Effect of trough width parameter, K on the shape of the trough

Based on the output of the numerical method calibration, the V_L parameter ranged from 0.7% to 7%. These V_L parameters were obtained via a function in Phase^2 which calculates the volume loss in an excavation, and have been verified by a separate Phase^2 axis-symmetric model which yielded similar values. The higher volume loss of 7% at CH3200 is due to the unsupported excavation in the looser and softer Tauranga Group Alluvium material, as opposed to the 0.7%-1.1% observed at CH350 and CH1300 where the tunnel will be constructed in ECBF rock. This thus indicates that face pressure (FP) will be required at CH3200 to stabilise the ground and bring the V_L closer to 1%. However, with reference to the depth of the tunnel and the tunnel diameter, the magnitude of the surface settlements due to small variances in V_L will be insignificant. This is shown in Figure 5, with an assumed K value of 0.5. As the majority of the Link Sewers will be approximately 20m to 30m below ground level, the total settlement at the surface will be below 5mm, and the variation of total settlement at the surface will only be in the magnitude of a few millimetres. Therefore, based on these results and the site conditions of the pipe, a **V_L parameter value of 1%** is considered appropriate.

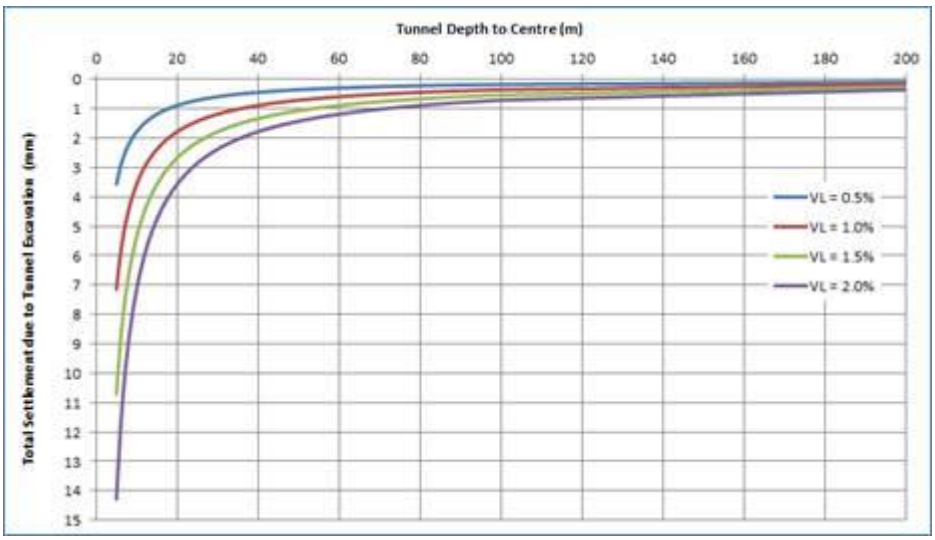


Figure 5 Effect of Volume Change parameter on the surface settlement

The zone of influence obtained from the numerical modelling for the various chainages was also examined and compared with the existing empirical methods. This is summarised in Table 6.

Table 6 Zone of influence of the tunnel excavation

Assessment Method	Width of zone of influence (m)		
	CH350 (69m bgl)	CH1300 (29m bgl)	CH3200 (7m bgl)
Theoretical[1]	138	58	14
Analytical[2]	173	73	18
Numerical[3]	-	-	30

Notes:

1. Based on Mohr coulomb principles and the active wedge failure criterion. Assumed to extend 45° from the tunnel towards the surface
2. Based on the method proposed by Mair (1993) and taken to be 5i, where $i = K.z_0$
3. Based off FEA from Rocscience Phase2
4. The zone of influence was not recorded at CH350 and CH1300 as the magnitude of settlement at the surface was under 1mm.

In the numerical analysis, at CH350 and CH1300 where the tunnel is within ECBF Rock, the amount of vertical deformation of the soil at the surface did not exceed 1mm. The settlement was also highly sensitive to “noise” in the modelling and fluctuated erratically, which made ascertaining the zone of influence difficult. The settlement was thus deemed negligible and there is no zone of influence in the numerical analysis. The width of the influence zone at CH3200 according to the numerical modelling was 30m, but as mentioned earlier, numerical modelling tends to overestimate the width of the trough. Thus the analytical method will be used to obtain the zone of influence when the tunnel is not within the ECBF Rock, and a nominal width of 50m will be adopted for the zone of influence when the tunnel is within the ECBF Rock.

3.3.4 Groundwater table profile indirectly caused from tunnel excavation

The method used to obtain the groundwater table drawdown can be found in the “Drawdown estimation due to Link Sewer C Tunnel Construction” report and is summarised in Figure 6, Figure 7 and Figure 8 for CH 350, CH1300 and CH3200 respectively. The drawdown profiles of the unlined case will be incorporated into the numerical modelling to obtain the settlement due to the groundwater drawdown. It is noted that the upward spike in the groundwater table at the centreline of the model in Figure 6 is due to Whau Stream being located directly above the tunnel at CH350 and providing an infinite source of recharge to the groundwater levels.

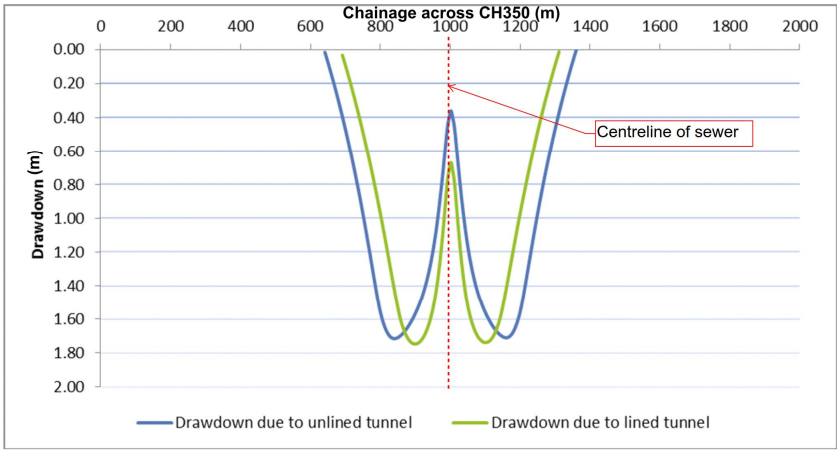


Figure 6 Drawdown through section CH350 due to tunnel construction (in m)

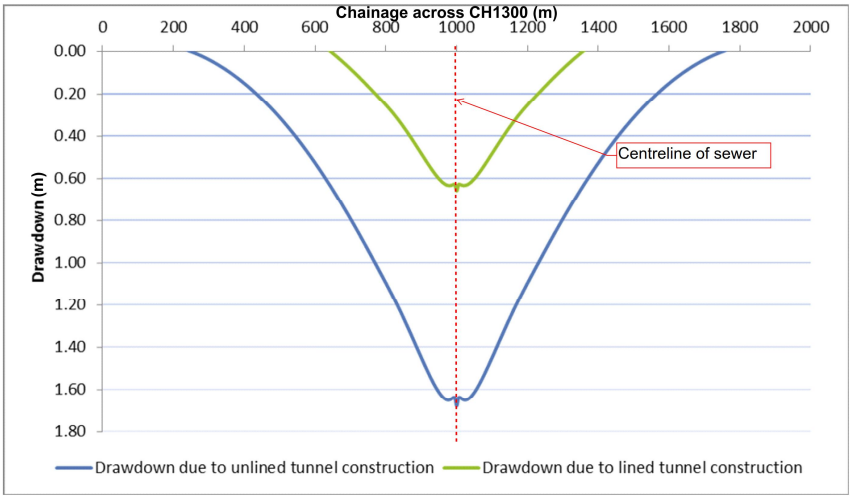


Figure 7 Drawdown through section CH1300 due to tunnel construction (in m)

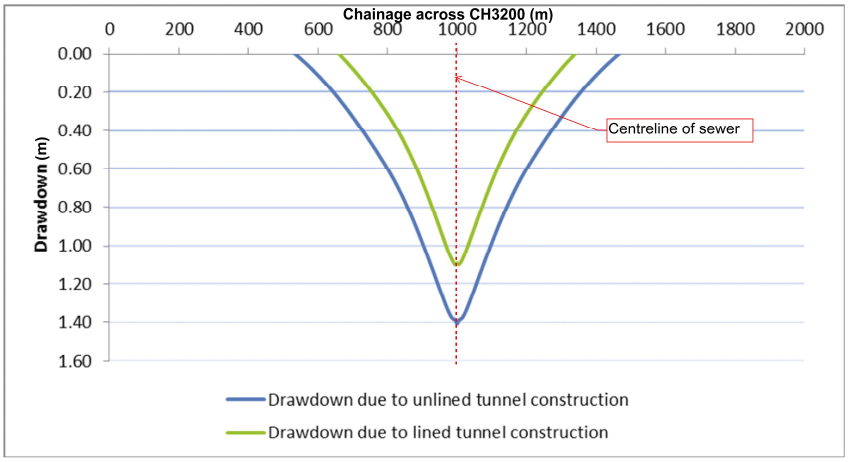


Figure 8 Drawdown through section CH3200 due to tunnel construction (in m)

4. Results

4.1 Tunnelling induced surface movement

Table 7 and Table 8 for Link Sewer C and B respectively are presented below detailing the maximum settlement magnitude, differential settlement and zone of influence anticipated along the chainage due to tunnelling induced surface movement.

Table 7 Settlement magnitudes and zone of influences along Link Sewer C

Chainage (m)	VL	K	i	Depth bgl (m)	Max Settlement (mm)	Zone of influence (m)	Ave Differential Settlement (1 in ...)
100	1	0.5	27.5	55	0.6	50	105,882
150	1	0.5	27.5	55	0.6	50	105,882
200	1	0.5	30	60	0.6	50	126,008
250	1	0.5	32.5	65	0.5	50	147,884
300	1	0.5	35	70	0.5	50	171,511
350	1	0.5	35	70	0.5	50	171,511
400	1	0.5	35	70	0.5	50	171,511
450	1	0.5	32.5	65	0.5	50	147,884
500	1	0.5	30	60	0.6	50	126,008
550	1	0.5	25	50	0.7	50	87,506
600	1	0.5	22.5	45	0.8	50	70,880
650	1	0.5	25	50	0.7	50	87,506
700	1	0.5	22.5	45	0.8	50	70,880
750	1	0.5	20	40	0.9	50	56,004
800	1	0.5	20	40	0.9	50	56,004
850	1	0.5	17.5	35	1.0	50	42,878
900	1	0.5	17.5	35	1.0	50	42,878
950	1	0.5	17.5	35	1.0	50	42,878
1000	1	0.5	15	30	1.2	50	31,502
1050	1	0.5	15	30	1.2	50	31,502
1100	1	0.5	15	30	1.2	50	31,502

Chainage (m)	VL	K	i	Depth bgl (m)	Max Settlement (mm)	Zone of influence (m)	Ave Differential Settlement (1 in ...)
1150	1	0.5	12.5	25	1.4	50	21,876
1200	1	0.5	12.5	25	1.4	50	21,876
1250	1	0.5	15	30	1.2	50	31,502
1300	1	0.5	15	30	1.2	50	31,502
1350	1	0.5	15	30	1.2	50	31,502
1400	1	0.5	17.5	35	1.0	50	42,878
1450	1	0.5	17.5	35	1.0	50	42,878
1500	1	0.5	15	30	1.2	50	31,502
1550	1	0.5	15	30	1.2	50	31,502
1600	1	0.5	15	30	1.2	50	31,502
1650	1	0.5	12.5	25	1.4	50	21,876
1700	1	0.5	12.5	25	1.4	50	21,876
1750	1	0.5	12.5	25	1.4	50	21,876
1800	1	0.5	12.5	25	1.4	50	21,876
1850	1	0.5	12.5	25	1.4	50	21,876
1900	1	0.5	12.5	25	1.4	50	21,876
1950	1	0.5	10	20	1.8	50	14,001
2000	1	0.5	10	20	1.8	50	14,001
2050	1	0.5	12.5	25	1.4	50	21,876
2100	1	0.5	12.5	25	1.4	50	21,876
2150	1	0.5	12.5	25	1.4	50	21,876
2200	1	0.5	17.5	35	1.0	50	42,878
2250	1	0.5	17.5	35	1.0	50	42,878
2300	1	0.5	17.5	35	1.0	50	42,878
2350	1	0.5	15	30	1.2	50	31,502
2400	1	0.5	15	30	1.2	50	31,502

Chainage (m)	VL	K	i	Depth bgl (m)	Max Settlement (mm)	Zone of influence (m)	Ave Differential Settlement (1 in ...)
2450	1	0.5	17.5	35	1.0	50	42,878
2500	1	0.5	12.5	25	1.4	50	21,876
2550	1	0.5	12.5	25	1.4	50	21,876
2600	1	0.5	12.5	25	1.4	50	21,876
2650	1	0.5	10	20	1.8	50	14,001
2700	1	0.5	7.5	15	2.4	50	7,876
2750	1	0.5	7.5	15	2.4	50	7,876
2800	1	0.5	7.5	15	2.4	50	7,876
2850	1	0.5	5	10	3.6	50	3,500
2900	1	0.5	7.5	15	2.4	50	7,876
2950	1	0.5	5	10	3.6	50	3,500
3000	1	0.5	5	10	3.6	50	3,500
3050	1	0.5	2.5	5	7.1	50	875
3100	1	0.5	5	10	3.6	25	3,500
3150	1	0.5	5	10	3.6	25	3,500
3200	1	0.5	5	10	3.6	25	3,500
3250	1	0.5	5	10	3.6	25	3,500

Along the Link Sewer C route, the critical section where the maximum total settlement and differential settlement was predicted to occur was at CH3050 where a maximum total settlement of 7mm was predicted to affect a trough width of 12.5m to yield an average differential settlement of 1 in 875. This was due to this section of the tunnel being relatively near to the surface (~5m). Other than this section on Link Sewer C, the rest of the route yielded settlement values less than the 50mm and 1:1,000 differential settlement values in the Consent conditions. This again was attributed to the consistently thick cover of soil/rock (>20m) over the pipe from CH100 to CH2650, and the tunnel being embedded in the relatively stiff ECBF Rock from CH100 to CH3000.

Table 8 Settlement magnitudes and zone of influences along Link Sewer B

Chainage (m)	VL	K	i	Depth bgl (m)	Max Settlement (mm)	Zone of influence (m)	Ave Differential Settlement (1 in ...)
100	1	0.5	17.5	35	1.0	50	42,878
150	1	0.5	17.5	35	1.0	50	42,878

Chainage (m)	VL	K	i	Depth bgl (m)	Max Settlement (mm)	Zone of influence (m)	Ave Differential Settlement (1 in ...)
200	1	0.5	17.5	35	1.0	50	42,878
250	1	0.5	20	40	0.9	50	56,004
300	1	0.5	20	40	0.9	50	56,004
350	1	0.5	20	40	0.9	50	56,004
400	1	0.5	20	40	0.9	50	56,004
450	1	0.5	20	40	0.9	50	56,004
500	1	0.5	20	40	0.9	50	56,004
550	1	0.5	17.5	35	1.0	50	42,878
600	1	0.5	17.5	35	1.0	50	42,878
650	1	0.5	17.5	35	1.0	50	42,878
700	1	0.5	17.5	35	1.0	50	42,878
750	1	0.5	15	30	1.2	50	31,502
800	1	0.5	12.5	25	1.4	50	21,876
850	1	0.5	15	30	1.2	50	31,502
900	1	0.5	12.5	25	1.4	50	21,876
950	1	0.5	12.5	25	1.4	50	21,876
1000	1	0.5	15	30	1.2	50	31,502
1050	1	0.5	10	20	1.8	50	14,001
1100	1	0.5	12.5	25	1.4	50	21,876
1150	1	0.5	15	30	1.2	50	31,502
1200	1	0.5	12.5	25	1.4	50	21,876

Along the Link Sewer B route, the total settlement due to the mechanical excavation process remained under 2mm, and the differential settlement was predicted to not exceed 1 in 10,000. This was attributed to the tunnel being consistently deeper than 20m below ground level and being embedded in the stiff ECBF rock material. The settlement values experienced at Link Sewer B can thus be considered to be minor compared to that of Link Sewer C.

4.2 Indirect dewatering induced surface movement

The groundwater drawdown profiles from Figure 6, Figure 7 and Figure 8 were applied to the geological sections at CH350, CH1300 and CH3200 respectively in Phase2 v9.0. The settlement profiles output for

CH350, CH1300 and CH3200 are summarised in Figure 9, Figure 10 and Figure 11 respectively. The settlement profiles can be seen to be roughly of the same shape as the groundwater drawdown profiles, after accounting for variances in the geological section. The groundwater drawdown was greatest at CH1300, and this was reflected by having the largest settlement (13mm). CH1300's relatively larger settlement compared to the other two locations was also due to the thicker compressible TGA layer above the tunnel (14m thick compared to 6m at CH3200 and no TGA at CH350), as the TGA is known to have a much lower Young's modulus compared to the ECBF residual soils and rock. Further results of the output are attached in Appendix E.

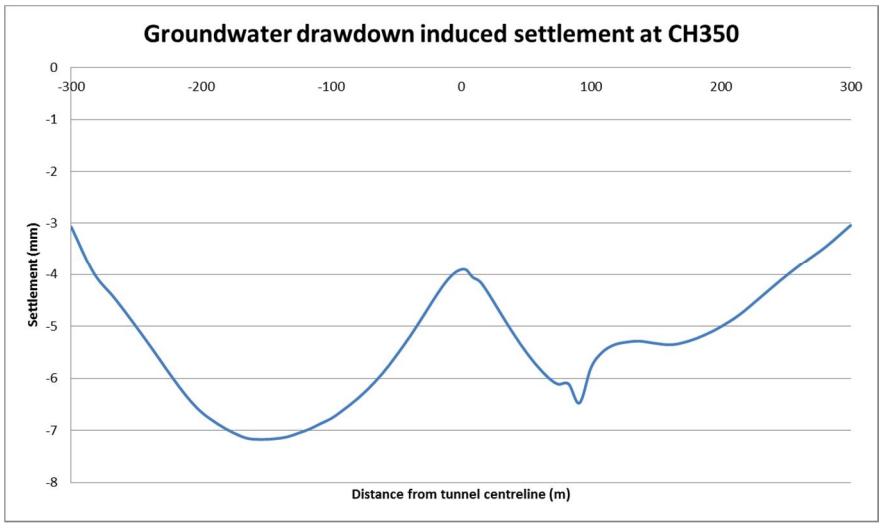


Figure 9 Groundwater drawdown induced settlement at CH350

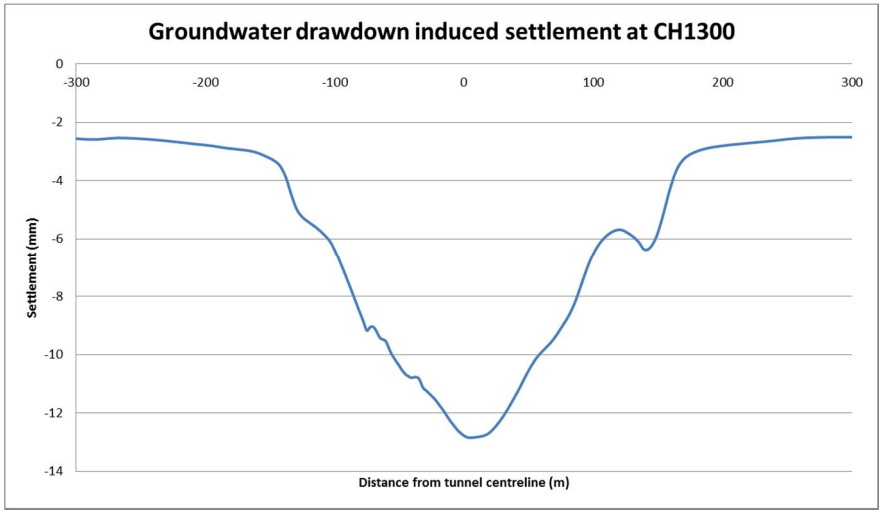


Figure 10 Groundwater drawdown induced settlement at CH1300

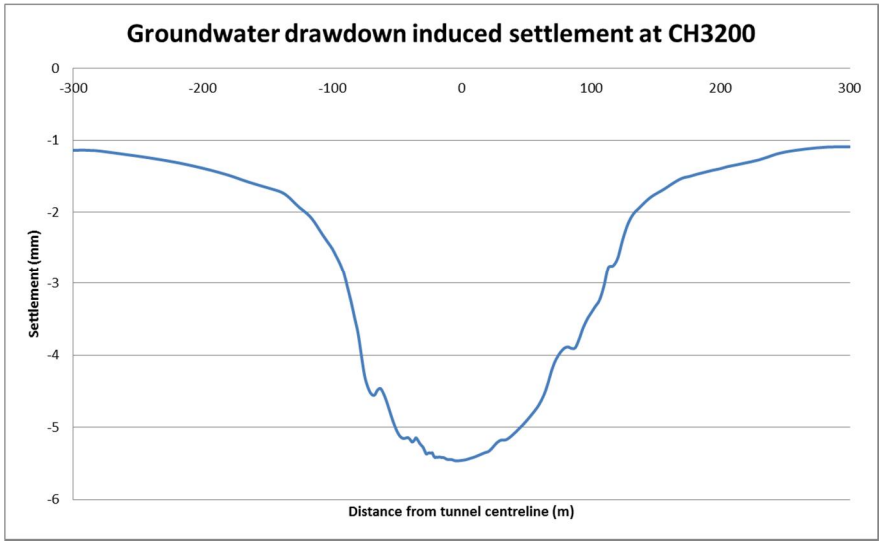


Figure 11 Groundwater drawdown induced settlement at CH3200

4.3 Combined settlement profiles

The analytical results of the mechanical settlement (Table 7 and Table 8) were superimposed with the output from the groundwater drawdown induced settlement (Figure 9, Figure 10 and Figure 11). The settlement profiles for CH350, CH1300 and CH3200 and summarised in Figure 12, Figure 13 and Figure 14.

It can be seen that the mechanical settlement is relatively minor compared to the groundwater drawdown induced settlement at CH350 and CH1300, but becomes more significant at CH3200. This is due to the depth of the tunnel at CH3200 being much shallower, and will thus result in a steeper and deeper trough according to Equation 2 and Equation 3. The settlement results are summarised in Table 9.

It is apparent that none of the analysed sections exceed the project criteria listed in Section 3.1, but the differential settlement at CH3200 is close to the criteria. This is a result of the mechanical settlement being more pronounced due to the tunnel at CH3200 being at a shallow depth. The presence of TGA around the crown of the tunnel also contributes to this relatively higher differential settlement. Care should thus be taken along this section from CH3200 to CH3250 along Link Sewer C when the tunnel is relatively close to the ground surface and is partially to completely surrounded by TGA.

Table 9 Summary of total and differential settlement of combined settlements

Chainage section	Max settlement (mm)	Max differential settlement
CH350	7	1 in 12,500
CH1300	14	1 in 6,000
CH3200	10	1 in 1,100

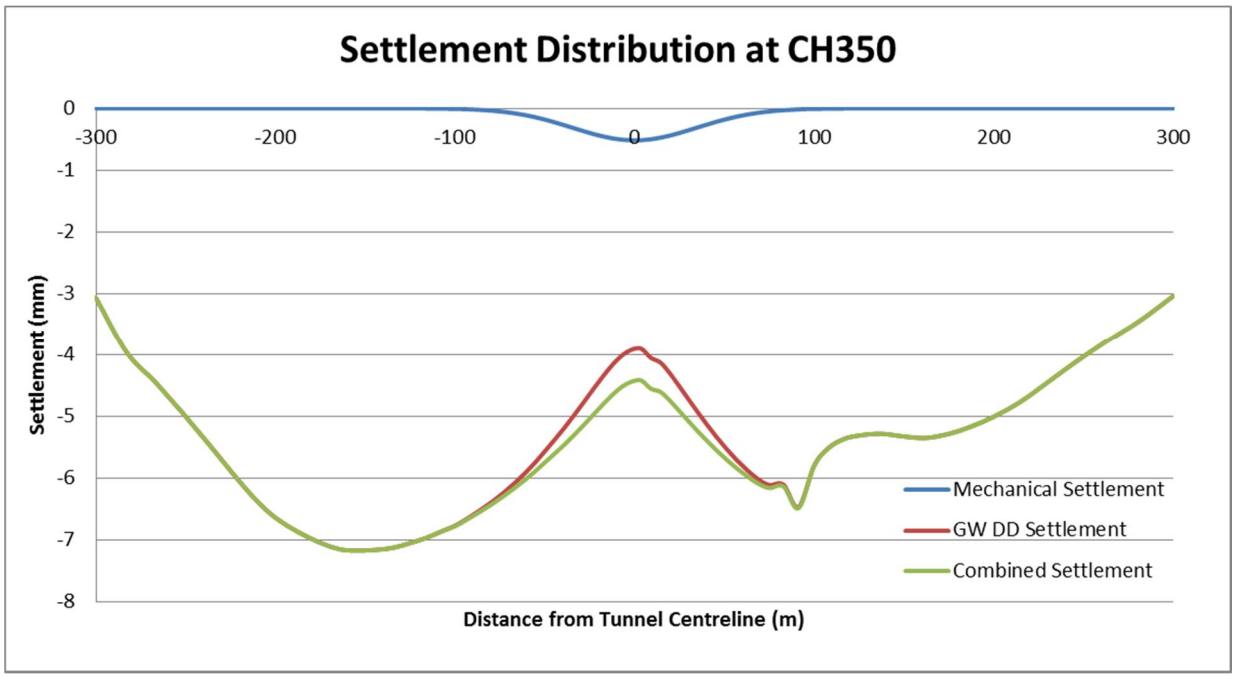


Figure 12 Combined settlement profile at CH350

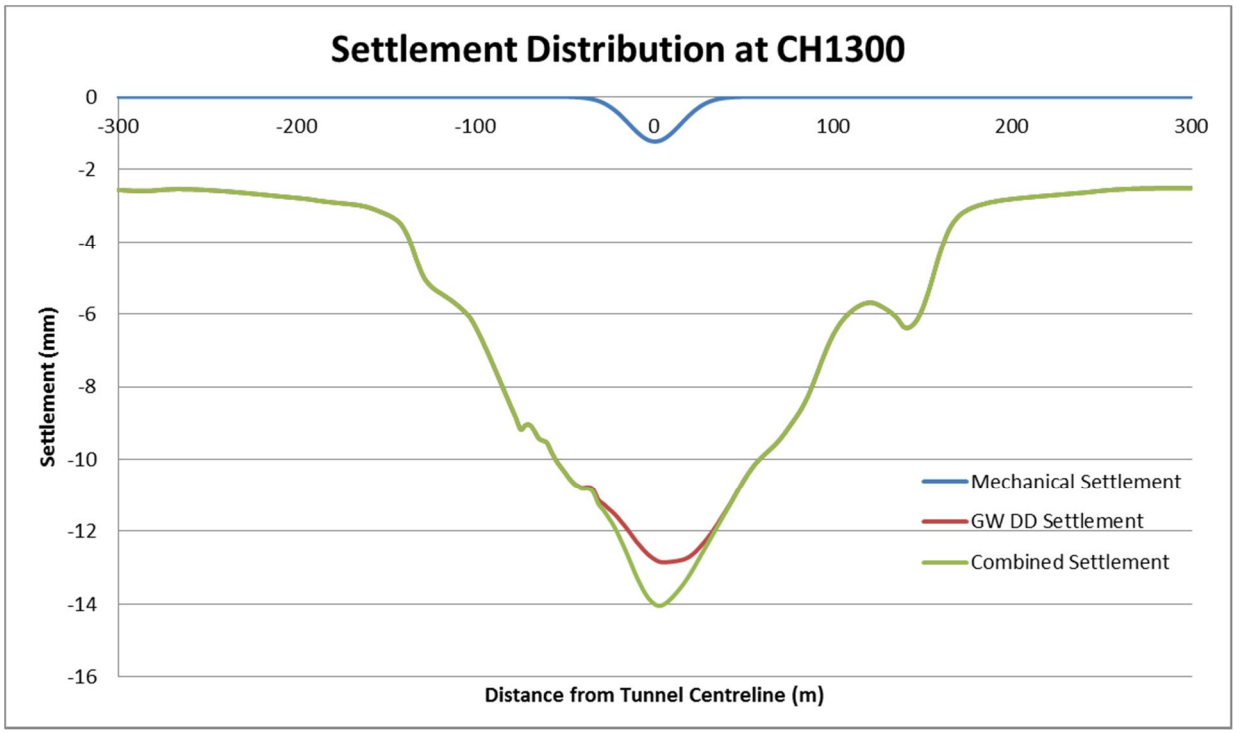


Figure 13 Combined settlement profile at CH1300

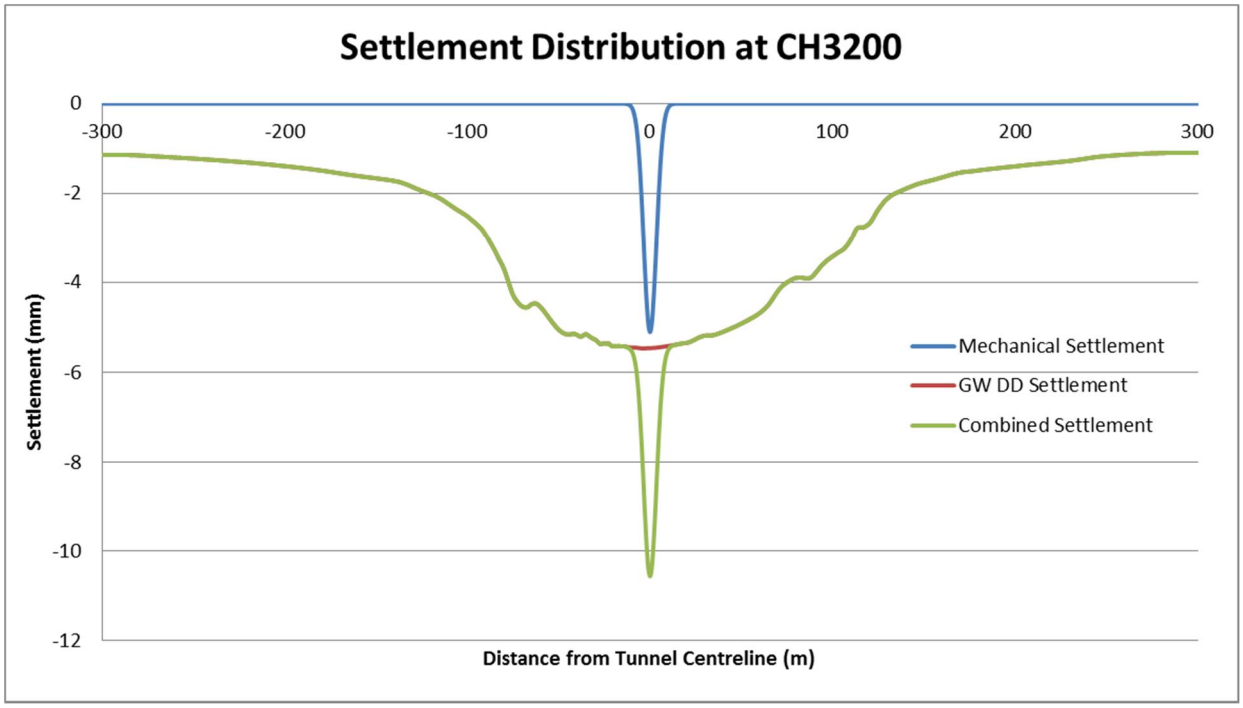


Figure 14 Combined settlement profile at CH3200

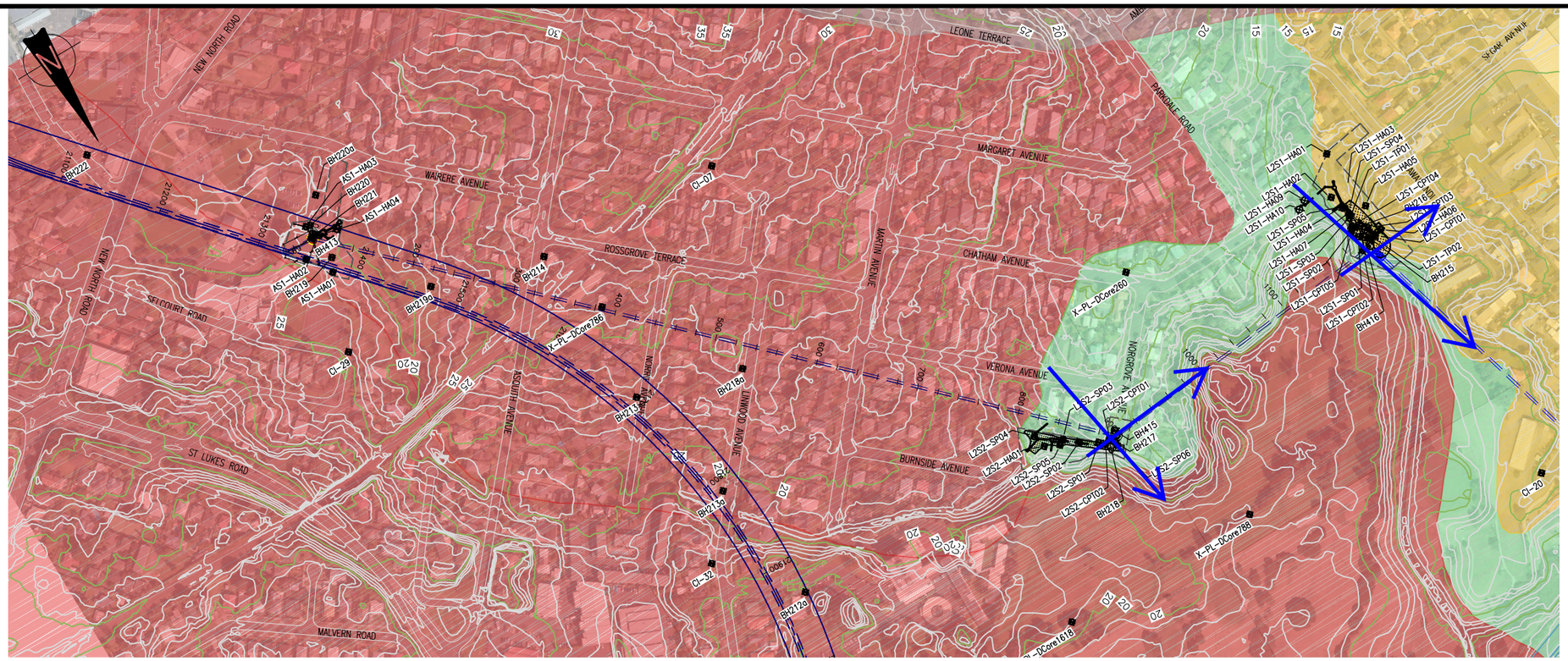
5. Conclusion

Based on the results of the analysis of the tunnel induced ground movements and indirect dewatering induced surface movement, we make the following conclusions and recommendations:

- This report has highlighted the anticipated ground movements due to both the mechanical effects and groundwater drawdown induced settlement.
- Total and differential settlements are generally significantly less than the Consent limits of 50mm total and 1:1000 differential settlement, with an exception circa Ch 3050 where differential settlement of 1:875 is estimated.
- The settlement data from Table 7 and Table 8 should be superimposed with the groundwater drawdown induced settlement and projected onto aerial photographs and plan sections of the Link Sewers to see if any structures of importance will be affected.
- The main influence factors for settlement include the depth of the TGA layer above the Link Sewers, depth of the Link Sewers and groundwater drawdown profiles.
- The majority of the Link Sewer route should not experience critical total or differential settlement, but extra care should be taken from CH3200 to CH3250 where the tunnel is much closer to the surface and is partially to completely surrounded by Tauranga Group Alluvium.

6. Appendices

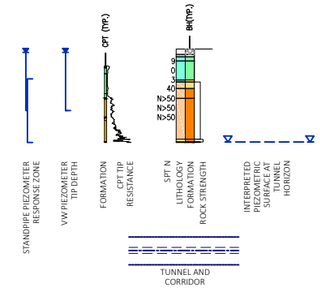
6.1 Geotechnical Longitudinal Section



LEGEND

BH215 −6m Borehole ID and offset from section. (Negative numbers are into the page.)

	BH LEFT	PLAN AND SECTION	BH RIGHT
No core			Made Ground (FILL)
Fill and topsoil			Recent Alluvium (Q1a)
Basalt		TAURANGA GROUP	
Other volcanics			Undifferentiated Tauranga Group (TG)
Clay and clay dominated mixtures			Kaawa Formation (KAAWA)
Organic clay and organic clay dominated mixtures		AUCKLAND VOLCANIC FIELD (PLEISTOCENE)	
Peat			Basalt (AVL)
Silt and silt dominated mixtures			Tuff / Ash / Scoria (AVT)
Organic silt and organic silt dominated mixtures		WAITEMATA GROUP (MIOCENE)	
Sand and sand dominated mixtures (clean)			Residually to Highly Weathered - Cohesive Soils (RS)
Sand and sand dominated mixtures (with fines)			Residually to Highly Weathered - Granular Soils (RS)
Organic sand and organic sand dominated mixtures			Parnell Volcaniclastic Conglomerate (PVC)
Sandstone			Moderately Weathered to Unweathered ECBF (ECBF-ROCK)
Mudstone			
Interbedded mudstone and sandstone			
Conglomerate			
Coal			



NOTES

1. HORIZONTAL AND VERTICAL ALIGNMENT BASED ON X-DSCIN-T-001 FOR MAINLINE, AND RELEVANT LINK SEWER MODELS X-DSLSA(-D)-C-001. DESIGNS CURRENT TO MAY 2016.
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3. BACKGROUND AERIAL PHOTOGRAPHY & CADASTRAL DATA FROM AUCKLAND COUNCIL GIS DATA. 21/08/2015
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5. COORDINATE SYSTEM NZTM.
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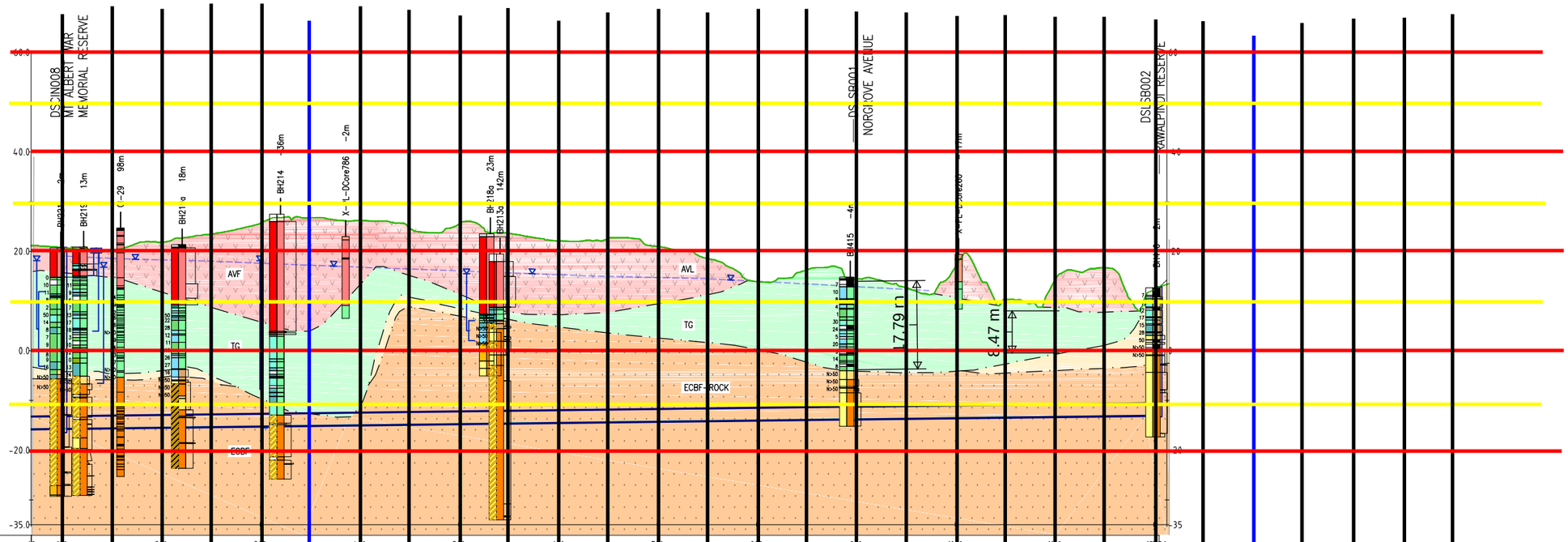

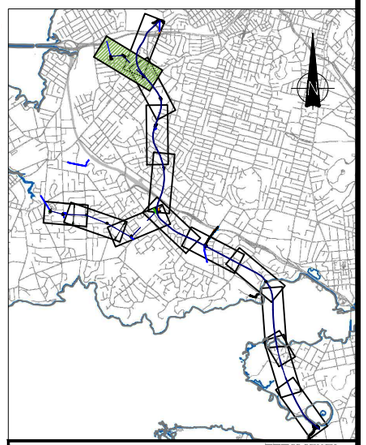

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ISSUE	DATE	AMENDMENT	BY	APPD.
3	07.06.16	ISSUED FOR USE	AP	ADC
2	18.12.15	ISSUED FOR CLIENT REVIEW	KL	ADC
1		PRELIMINARY	KL	

DESIGNED	RR	04.15
DES. CHECKED	HM	05.16
DRAWN	KL	04.15
DWG. CHECKED	ADC	05.16
PROJECT LEADER	DK	05.16
INFRAST'R APP'D		
	BY	DATE

OPERATIONS

INFRASTRUCTURE

DSLSB – DRAINAGE SEWER LINK SEWER B
SITE GENERAL
LINK SEWER B LONG SECTION

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REF. No. CI-GEOTECH	ISSUE
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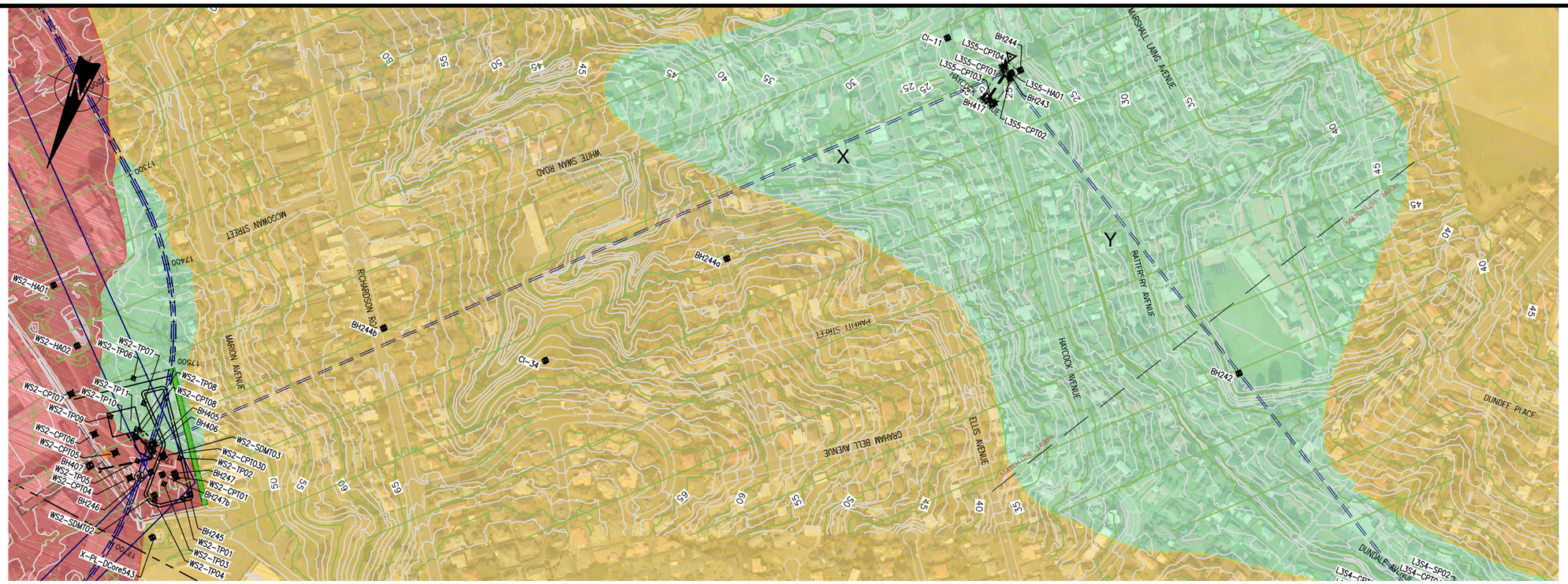
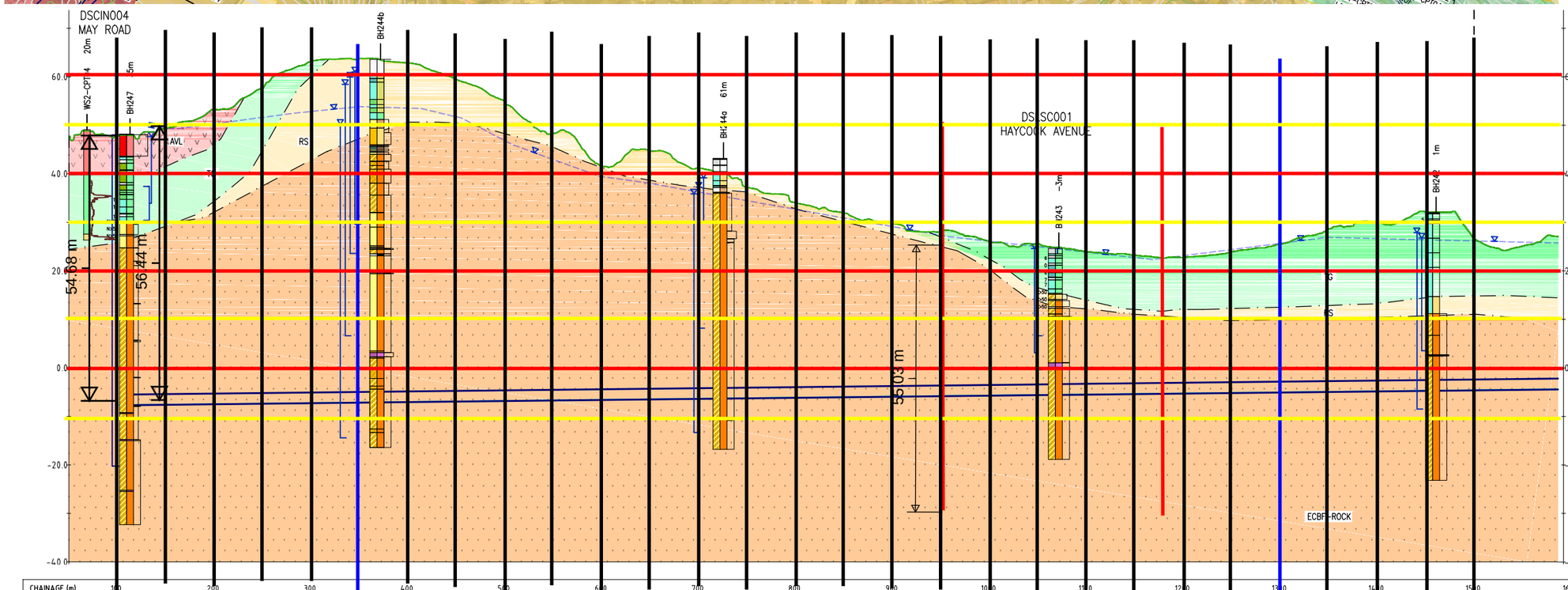




CHAINAGE (m)	100	200	300	400	500	600	700	800	900	1000	1100	1200	1300	1400	1500	1600
GROUND LEVEL (mRL)	47.81	53.28	63.16	62.84	54.80	41.55	41.13	34.02	29.83	26.06	23.92	22.75	25.54	30.06	26.57	25.33
CROWN LEVEL (mRL)	-5.57	-5.35	-5.13	-4.90	-4.68	-4.46	-4.24	-4.01	-3.79	-3.57	-3.35	-3.13	-2.91	-2.69	-2.46	-2.24
INVERT LEVEL (mRL)	-7.67	-7.45	-7.23	-7.01	-6.78	-6.56	-6.34	-6.12	-5.90	-5.67	-5.45	-5.23	-5.01	-4.78	-4.56	-4.34

LEGEND

BH215 −6m Borehole ID and offset from section. (Negative numbers are into the page.)

BH LEFT | PLAN AND SECTION | BH RIGHT

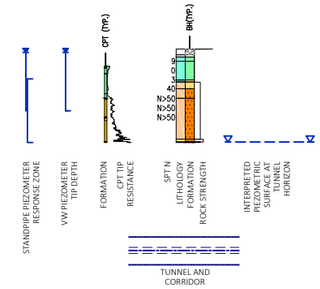
- No core
- Fill and topsoil
- Basalt
- Other volcanics
- Clay and clay dominated mixtures
- Organic clay and organic clay dominated mixtures
- Peat
- Silt and silt dominated mixtures
- Organic silt and organic silt dominated mixtures
- Sand and sand dominated mixtures (clean)
- Sand and sand dominated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

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

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

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

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



NOTES
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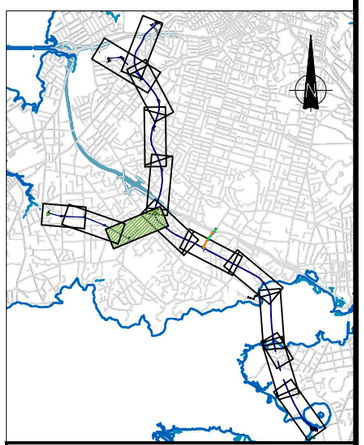

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1		PRELIMINARY	KL	

	BY	DATE
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DRAWN	KL	04.15
DWG. CHECKED	ADC	05.16
PROJECT LEADER	DK	05.16
INFRAST'R APP'D		

OPERATIONS

INFRASTRUCTURE

Watercare

DSLSC – DRAINAGE SEWER LINK SEWER C
SITE GENERAL
LINK SEWER C LONG SECTION – SHEET 1 OF 3

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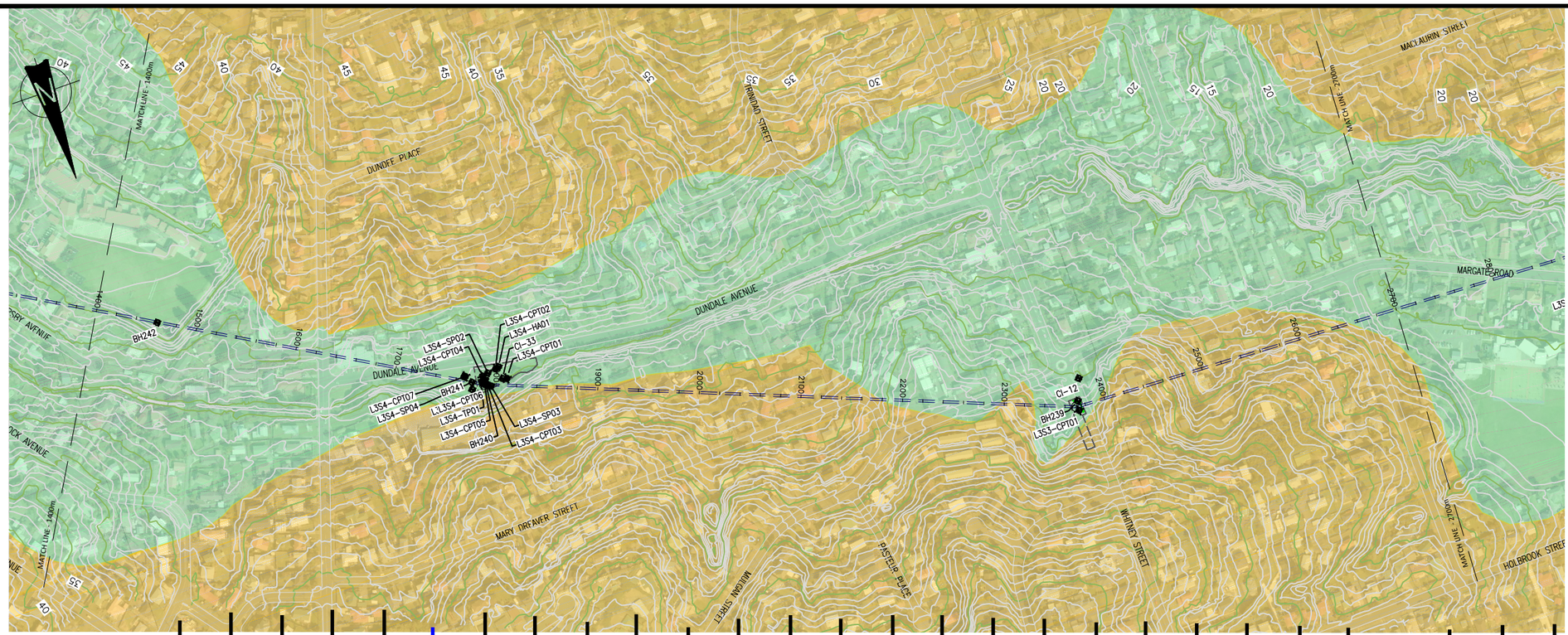

LEGEND

BH215 −6m Borehole ID and offset from section. (Negative numbers are into the page.)

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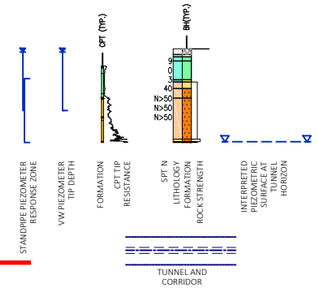
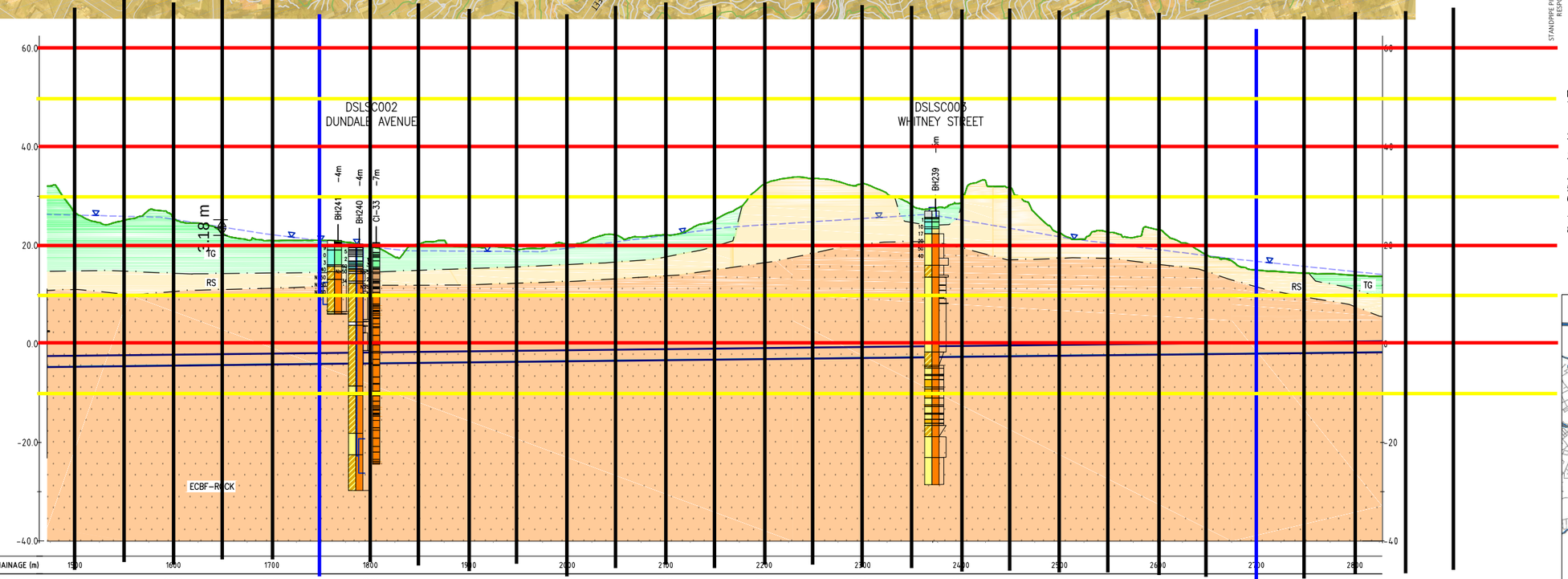
No core			Made Ground (FILL)
Fill and topsoil			Recent Alluvium (Q1a)
Basalt		TAURANGA GROUP	
Other volcanics			Undifferentiated Tauranga Group (TG)
Clay and clay dominated mixtures			Kaawa Formation (KAAWA)
Organic clay and organic clay dominated mixtures		AUCKLAND VOLCANIC FIELD (PLEISTOCENE)	
Peat			Basalt (AVL)
Silt and silt dominated mixtures			Tuff / Ash / Scoria (AVT)
Organic silt and organic silt dominated mixtures		WAITEMATA GROUP (MIOCENE)	
Sand and sand dominated mixtures (clean)			Residually to Highly Weathered - Cohesive Soils (RS)
Sand and sand dominated mixtures (with fines)			Residually to Highly Weathered - Granular Soils (RS)
Organic sand and organic sand dominated mixtures			Parnell Volcaniclastic Conglomerate (PVC)
Sandstone			Moderately Weathered to Unweathered ECBF (ECBF-ROCK)
Mudstone			
Interbedded mudstone and sandstone			
Conglomerate			
Coal			





CHAINAGE (m)	1500	1600	1700	1800	1900	2000	2100	2200	2300	2400	2500	2600	2700	2800
GROUND LEVEL	26.57	25.33	20.99	19.81	19.70	20.25	22.16	32.46	32.56	28.67	22.53	23.17	14.99	13.94
CROWN LEVEL	-2.46	-2.24	-2.02	-1.80	-1.58	-1.36	-1.14	-0.91	-0.69	-0.47	-0.25	-0.02	0.20	0.42
INVERT LEVEL	-4.56	-4.34	-4.12	-3.90	-3.67	-3.45	-3.23	-3.01	-2.78	-2.56	-2.34	-2.12	-1.90	-1.67

NOTES
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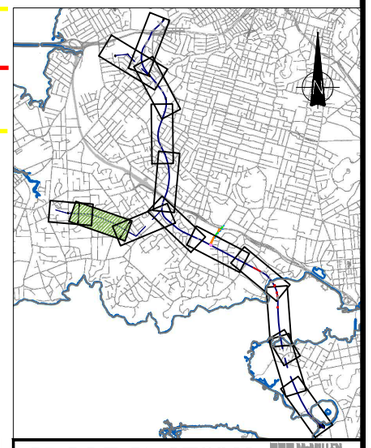

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INFRAST'R APP'D		
	BY	DATE

OPERATIONS

INFRASTRUCTURE

Watercare

DSLSC – DRAINAGE SEWER LINK SEWER C
SITE GENERAL
LINK SEWER C LONG SECTION – SHEET 2 OF 3

CAD FILE 2012004.002	DATE 07.06.16
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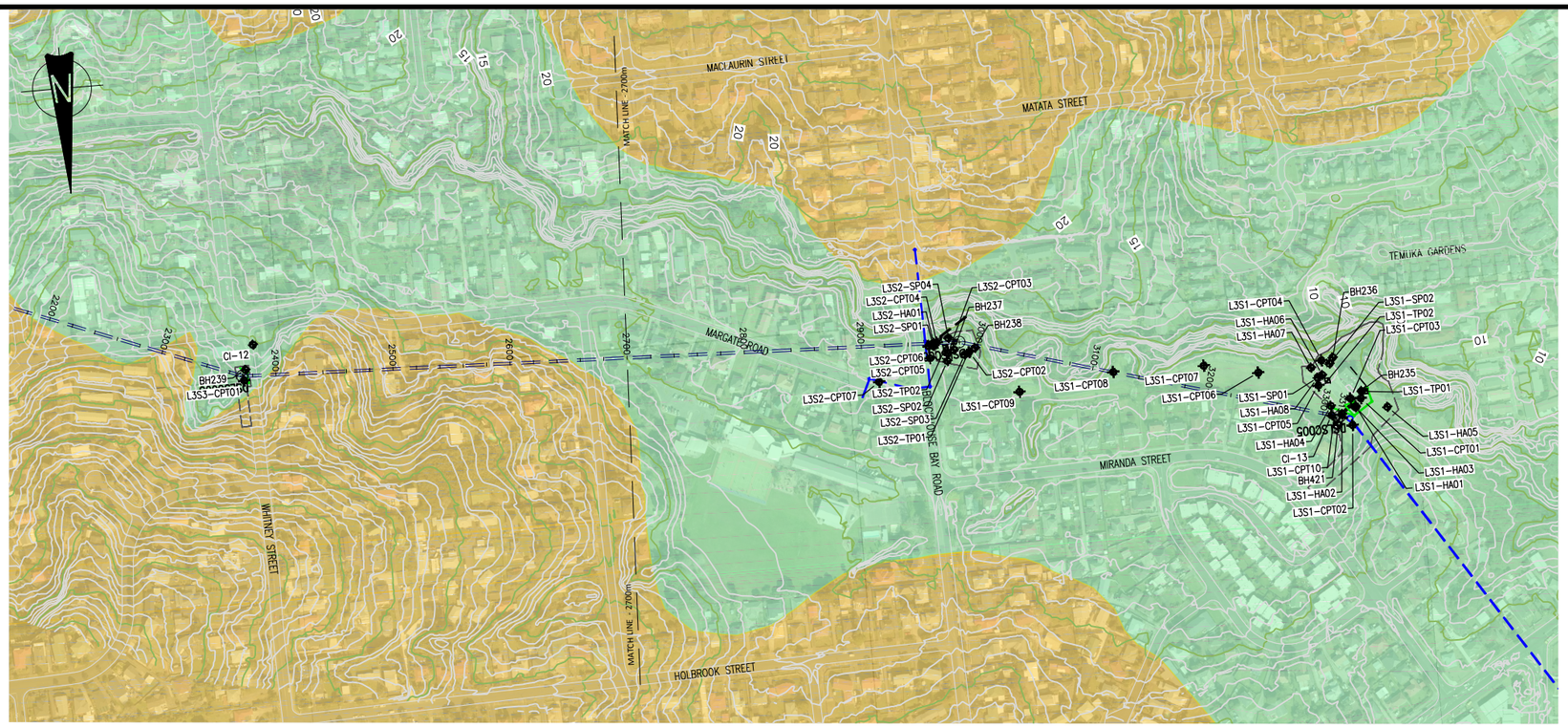
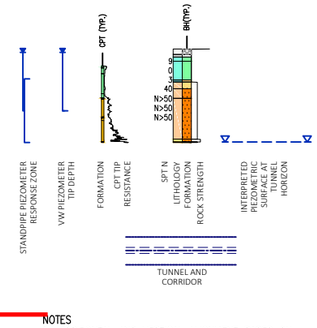
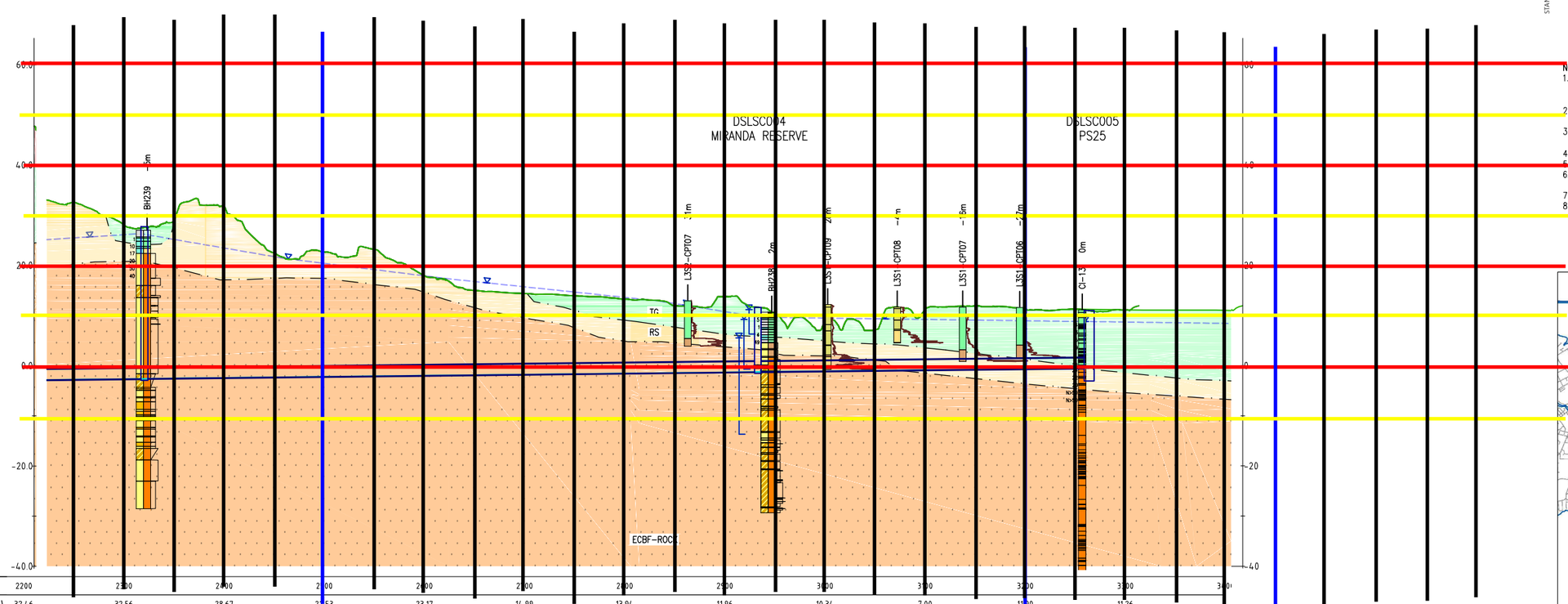


LEGEND

BH215 −6m Borehole ID and offset from section. (Negative numbers are into the page.)

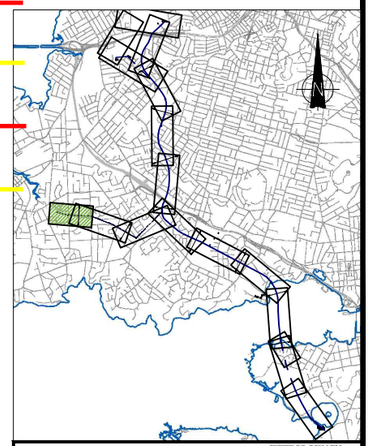
	BH LEFT	PLAN AND SECTION	BH RIGHT
No core			Made Ground (FILL)
Fill and topsoil			Recent Alluvium (Q1a)
Basalt		TAURANGA GROUP	
Other volcanics			
Clay and clay dominated mixtures			Undifferentiated Tauranga Group (TG)
Organic clay and organic clay dominated mixtures			Kaawa Formation (KAAWA)
Peat		AUCKLAND VOLCANIC FIELD (PLEISTOCENE)	
Silt and silt dominated mixtures			Basalt (AVL)
Organic silt and organic silt dominated mixtures			Tuff / Ash / Scoria (AVT)
Sand and sand dominated mixtures (clean)		WAITEMATA GROUP (MIOCENE)	
Sand and sand dominated mixtures (with fines)			Residually to Highly Weathered - Cohesive Soils (RS)
Organic sand and organic sand dominated mixtures			Residually to Highly Weathered - Granular Soils (RS)
Sandstone			Parnell Volcaniclastic Conglomerate (PVC)
Mudstone			Moderately Weathered to Unweathered ECBF (ECBF-ROCK)
Interbedded mudstone and sandstone			
Conglomerate			
Coal			





CHAINAGE (m)	2200	2300	2400	2500	2600	2700	2800	2900	3000	3100	3200	3300	3400
GROUND LEVEL (mRL)	32.46	32.56	28.67	22.53	23.17	14.99	13.94	11.96	10.34	7.00	11.90	11.26	
CROWN LEVEL (mRL)	-0.91	-0.69	-0.47	-0.25	-0.02	0.20	0.42	0.64	0.86	1.09	1.31	1.53	
INVERT LEVEL (mRL)	-3.01	-2.78	-2.56	-2.34	-2.12	-1.90	-1.67	-1.45	-1.23	-1.01	-0.78	-0.56	

NOTES
1. HORIZONTAL AND VERTICAL ALIGNMENT BASED ON X-DSCIN-T-001 FOR MAINLINE, AND RELEVANT LINK SEWER MODELS X-DSLSA(-D)-C-001. DESIGNS CURRENT TO MAY 2016.
2. EXISTING GROUND SURFACE MODELLED FROM AUCKLAND COUNCIL GIS DATA AND LANDSURVEY DATA PROVIDED BY BECA.
3. BACKGROUND AERIAL PHOTOGRAPHY & CADASTRAL DATA FROM AUCKLAND COUNCIL GIS DATA. 21/08/2015
4. VERTICAL HEIGHT DATUM AUCKLAND 1946.
5. COORDINATE SYSTEM NZTM.
6. ALL DIMENSIONS IN METRES AND ELEVATIONS IN mRL UNLESS OTHERWISE NOTED.
7. NOT ALL BOREHOLE LOGS SHOWN TO AID CLARITY.
8. THE GEOTECHNICAL INFORMATION SHOWN IS FOR GUIDANCE ONLY. FOR DETAILED GEOTECHNICAL INFORMATION REFER TO THE GEOTECHNICAL FACTUAL REPORT. (PWCIN-DEL-REP-GT-J-100047)



ISSUED FOR USE

JACOBS AECOM McMILLEN JACOBS ASSOCIATES

ISSUE	DATE	AMENDMENT	BY	APPD.
3	07.06.16	ISSUED FOR USE	AP	ADC
2	18.12.15	ISSUED FOR CLIENT REIEW	KL	ADC
1		PRELIMINARY		

	BY	DATE
DESIGNED	RR	04.15
DES. CHECKED	HM	05.16
DRAWN	KL	04.15
DWG. CHECKED	ADC	05.16
PROJECT LEADER	DK	05.16
INFRAST'R APP'D		

OPERATIONS

INFRASTRUCTURE

Watercare

DSLSC – DRAINAGE SEWER LINK SEWER C
SITE GENERAL
LINK SEWER C LONG SECTION – SHEET 3 OF 3

CAD FILE 2012004.003 DATE 07.06.16
ORIGINAL SCALE A1 1:2500 CONTRACT No. 5747
REF. No. CI-GEOTECH ISSUE
DWG. No. 2012004.003 3

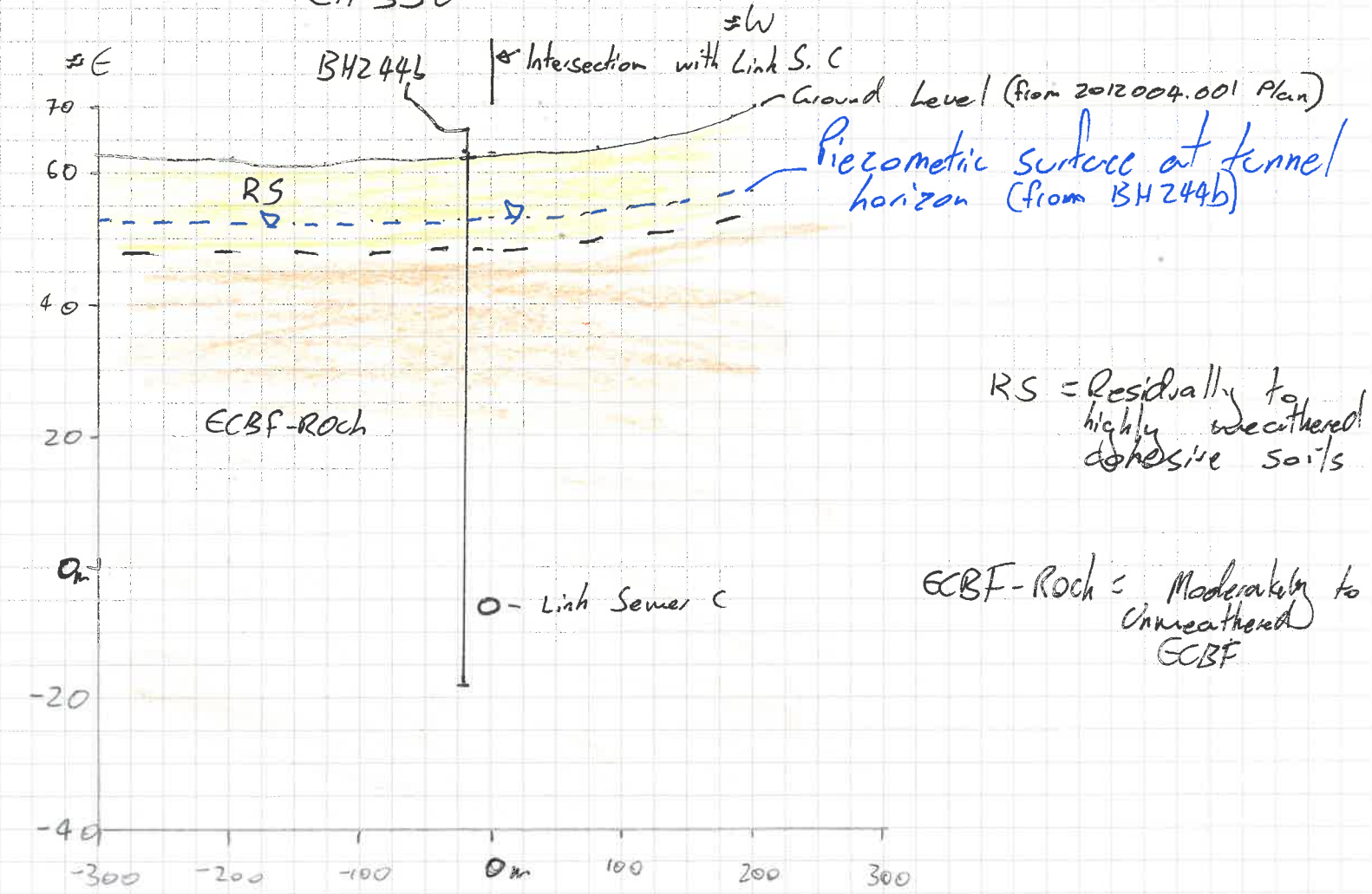
C:\users\apatel2\dms52193\ 2012004.003.dwg
Plot Date Jun 13, 2016 - 1:19pm

Appendix B. Cross Section Sketches

Link Sewer C

Cross section at CH 350

12/10/16



RS = Residually to highly weathered cohesive soils

ECBF-Roch = Moderately to Unweathered ECBF

Scale V 1:1000
Scale H 1:5000

Page

Date

Project

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By

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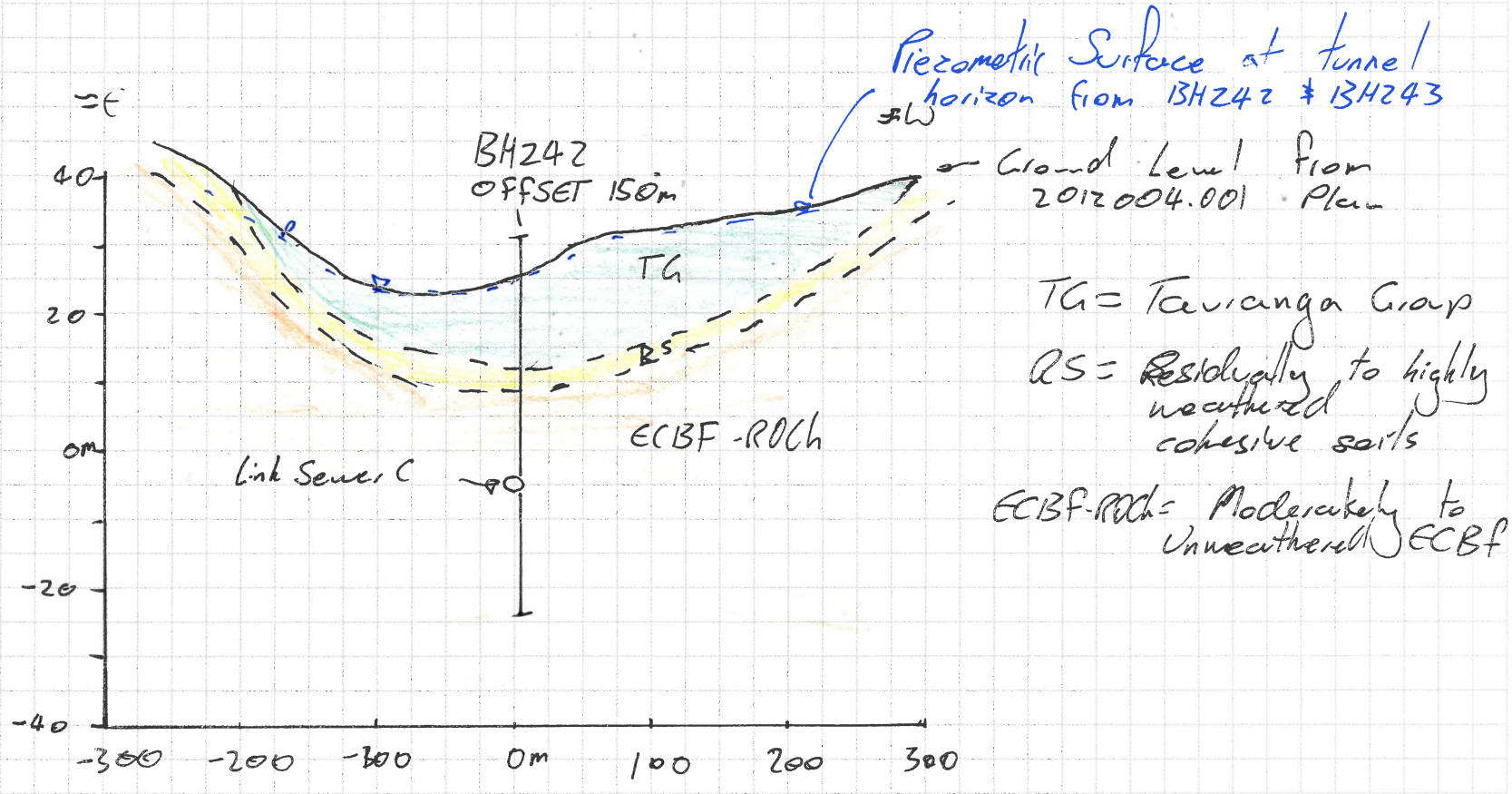
Link Sewer C

Cross Section at CH 1300

12/10/16



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Page

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Project

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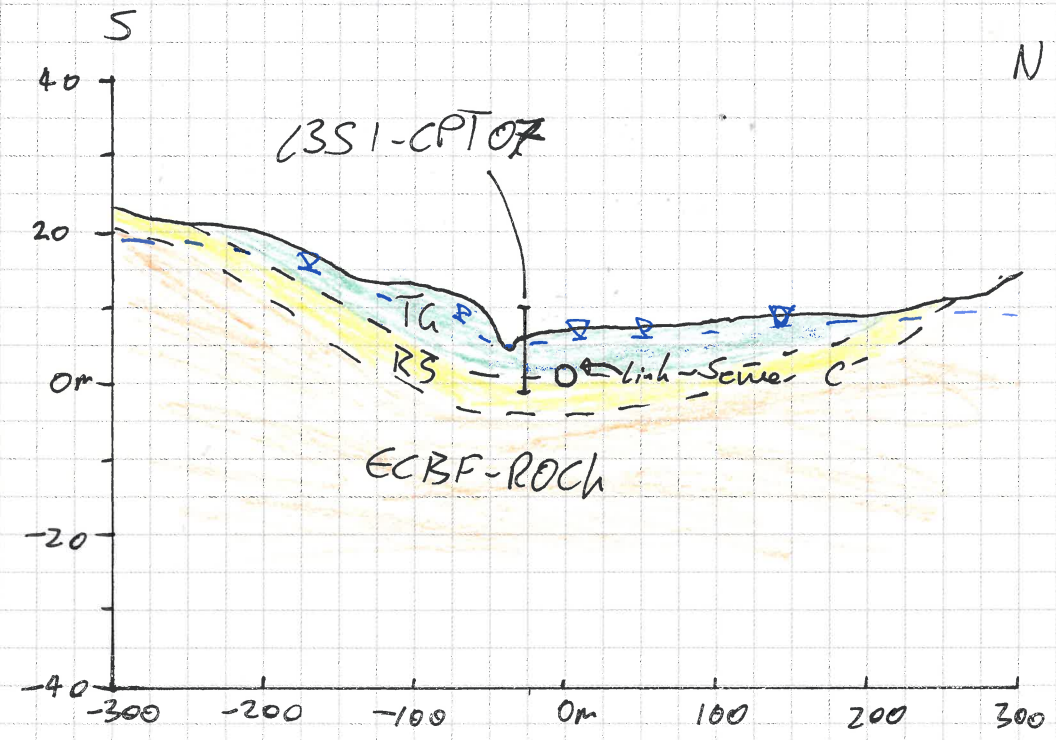
By

AECOM

Link Sewer C

Cross Section at CH 3200

12/10/16



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Date
Project
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By

AECOM

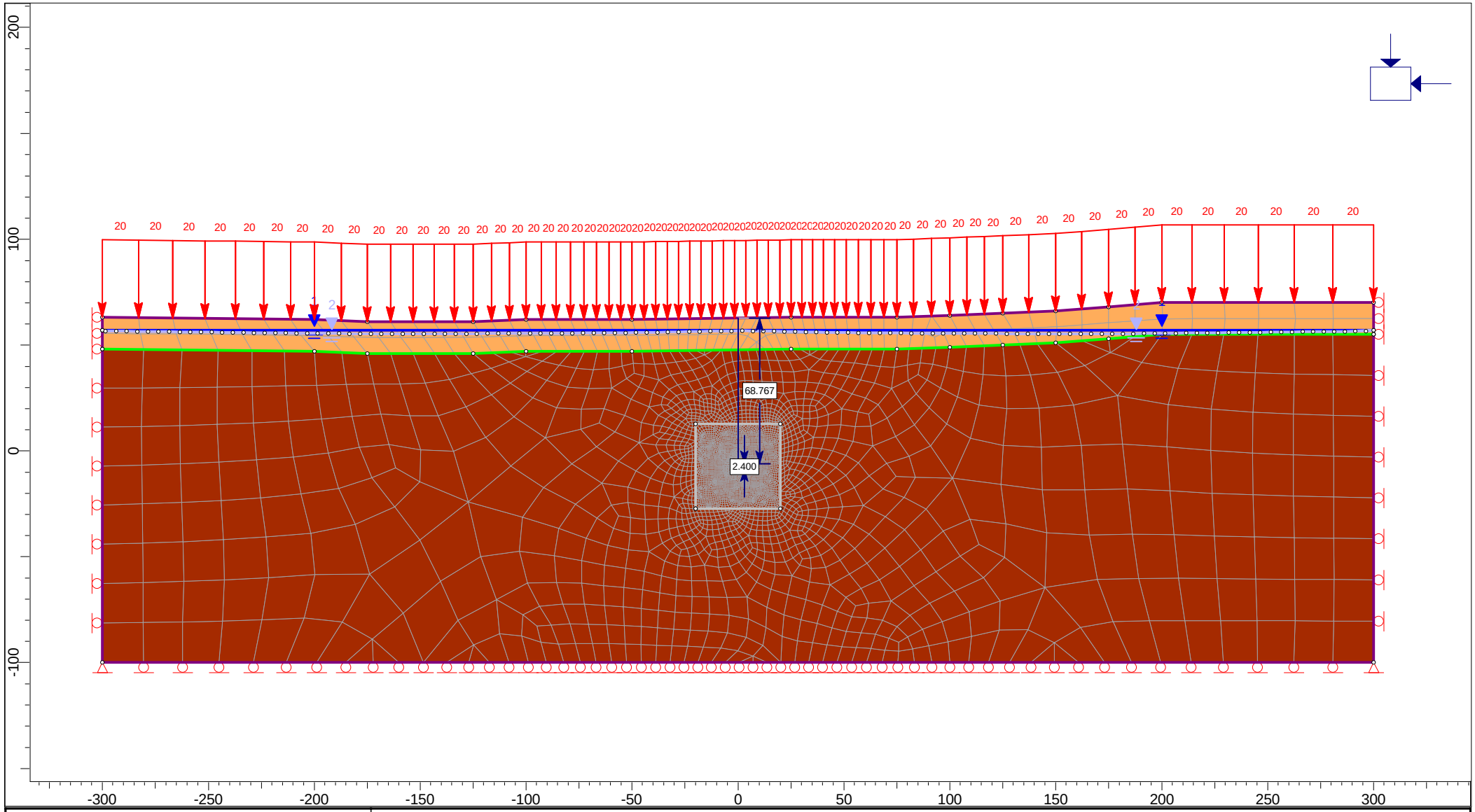
Appendix C. Material Parameters

Table 9.1 : Recommended Geotechnical Parameters

Geotechnical Design Parameters Table for Central Interceptor												Date: 3 June 2016 Revision: 4
Formation/Geological Units	Made Ground		Post AVF Tauranga Group alluvium and marine sediments	Tauranga group including Puketoka Fmn., estuarine, undifferentiated, colluvium		Kaawa Formation	Auckland Volcanic Field		East Coast Bays Formation			Parnell Volcaniclastic Conglomerate
Geotechnical Units	Engineered Fill	Non-Engineered Fill	Recent Alluvium	Undifferentiated Tauranga Group – Cohesive	Undifferentiated Tauranga Group – Granular	Kaawa Formation	Tuff/Ash/Scoria	Basalt	Residually to highly weathered cohesive soils	Residually to highly weathered granular soils	Moderately weathered to unweathered ECBF	
Lithology/Material Description	Clay, silt, sand and gravel		Silt and sand	Clay and silt	Sand	Shelly with silt and sand	Silt, sand and gravel, can be intermixed with clay	Intact, jointed, vesicular and rubbly	Silt and clay	Sand	Mudstone and muddy sandstone	Coarse sandstone to conglomerate
Soil Consistency/ Rock weathering	Dense – Very Dense	Loose – Medium Dense	Soft / Loose	Soft – Firm	Loose – Medium Dense	Loose – Dense	Stiff / Dense	MW – SW	Very Stiff – Hard RS – HW	Dense – Very Dense RS – HW	MW – UW	MW – UW
Material Type	Soil							Rock	Soil		Rock	
Bulk Density (unit weight) (kN/m^3) (Note 1, 2)	18 – 23 (20)	14 – 19 (15)	12 – 16 (12)	13 – 20 (16)	15 – 20 (17)	18, 19	16 -20 (17)	26 -29 (27)	18, 19	16 – 20 (20)	19 – 21 (20)	18 – 20 (20)
Moisture Content (%)	45 – 65 (50)	24 – 45 (26)	90 – 220 (90)	25 – 82 (32)	22 – 78 (22)	26 – 44 (27)	48 – 75 (53)	0.8 – 6.1 (5.0)	17 – 42 (25)	22 – 27 (22)	9 – 25 (15)	12 – 33 (15)
Liquid Limit (%) (Note 3)	65 – 91	45 – 65	102 – 198	37 – 100	-	-	41, 46, 120	36	50 – 95	-	-	-
Plastic Limit (%) (Note 3)	28 – 37	19 – 28	44 – 65	18 – 39	-	-	24, 27, 43	15	20 – 36	-	-	-
Plasticity Index (%) (Note 3)	38 – 52	23 – 38	21 – 58	18 – 66	-	-	17, 19, 74	21	27- 61	-	-	-
Unconfined Compressive Strength, UCS (MPa) (Note 4)	-	-	-	-	-	-	-	40 – 230 (120)	-	-	1.0 – 9 (2)	1.5 – 11 (10)
Tensile (Intact) Strength (kPa) (Note 5)	-	-	-	-	-	-	-	9,000 – 18,000 (15,000)	-	-	240 – 1,300 (520)	300 – 1,300 (525)
Geological Strength Index, GSI (Note 6)	-	-	-	-	-	-	-	40 – 80 (60)	-	-	35 – 80 (70)	50 – 85 (80)
Material Constant, mi (Note 7)	-	-	-	-	-	-	-	20 – 30 (25)	-	-	7 – 17 (10)	15 -24 (15)
Young's Modulus (Rock Substance), E_i (MPa) (Note 8)	-	-	-	-	-	-	-	14,000 – 60,000 (24,000)	-	-	70 – 1,350 (540)	280 – 1,400 (800)
Modulus Ratio (MR) E_i/UCS (Note 4)	-	-	-	-	-	-	-	140 – 335 (240)	-	-	80 – 220 (125)	130 – 225 (175)
Possion's Ratio (Note 9)	0.2 – 0.3 (0.3)	0.2 – 0.3 (0.3)	0.2 – 0.3 (0.3)	0.3 – 0.5 (0.4)	0.2 – 0.3 (0.3)	0.2 – 0.3 (0.3)	0.2 – 0.4 (0.35)	0.33 – 0.37 (0.35)	0.3 – 0.5 (0.4)	0.2 – 0.3 (0.3)	0.21 – 0.33 (0.25)	0.08 – 0.13 (0.10)
Undrained Shear Strength (kPa) (Note 10)	-	28 – 166 (64)	16 – 78 (18)	18 – 144 (34)	-	–-	31 – 66 (34)	-	33 – 158 (53)	1130, 1250	-	-
Effective Friction Angle ϕ' (°) (Note 11)	35 – 50 (40)	25 – 35 (32)	35, 58 (28)	22 – 36 (28)	28 – 40 (30)	28, 35 (32)	32 – 36 (35)	45 – 65 (50)	32 – 39 (32)	35 – 45 (40)	30 – 38 (34)	36 – 44 (40)
Effective cohesion, c' (kPa) (Note 11)	0 – 5 (2)	0 – 2 (1)	0, 6 (0)	3 – 34 (7)	(0)	24, 219 (25)	0 – 5 (2)	125-670 (200)(Note 11)	3 – 24 (6)	(0)	75 – 135 (100)	100 – 180 (140)
Soil / Rock Mass Modulus, E (MPa) (Note 12)	50 – 200 (100)	25 – 70 (25)	5 – 10 (5)	3 – 38 (7)	3 – 30 (10)	6 – 89 (20)	10 – 50 (12)	500 – 16,000 (3,000)	15 – 80 (30)	25 – 100 (50)	100 – 1,200 (400)	100 – 1,300 (700)
Coefficient of consolidation (m^2/year) (Note 4)	-	-	2.6 – 10 (5.0)	5.1 – 8.8 (7.2)	-	-	-	-	8.6 – 48 (19)	-	-	-
Coefficient of compressibility, mv (1/MPa) (Note 4)	-	-	0.4 – 1.1 (0.7)	0.04 – 0.6 (0.15)	-	-	-	-	0.03 – 0.14 (0.07)	-	-	-
Coefficient of secondary compression (%) (Note 4)	-	-	0.02 – 1.6 (1.5)	0.02 – 0.07 (0.01)	-	-	-	-	0.02 – 0.06 (0.04)	-	-	-
Hydraulic conductivity, k(m/sec) (Note 13)	$1x10^{-8}$ – $1x10^{-6}$	$1x10^{-8}$ – $1x10^{-6}$	$1x10^{-7}$	$4x10^{-5}$ – $2x10^{-4}$–		$4x10^{-5}$ – $1x10^{-4}$	$1x10^{-7}$ – $1x10^{-3}$	$1x10^{-7}$ – $1x10^{-3}$	$1x10^{-6}$ - $1x10^{-7}$	–N/A	$2x10^{-8}$ – $2x10^{-5}$	$5x10^{-7}$ – $1x10^{-3}$
Insitu Stress Ratio, Soil (K_0) (Note 14)	0.23 – 0.43 (0.36)	0.43 – 0.58 (0.47)	0.15, 0.43 (0.53)	0.41 – 0.63 (0.50)	0.36 – 0.53 (0.47)	0.43 – 0.53 (0.47)	0.41	-	0.37 – 0.47 (0.47)	0.29 – 0.43 (0.36)	-	-
Insitu Stress Ratio, Rock (K) (Note 15)	-	-	-	-	-	-	-	0.8 – 1.5 (1.2)	-	-	0.8 – 1.5 (1.2)	0.8 – 1.5 (1.2)
Post Excavation Stress Ratio	-	-	-	-	-	-	-	0.06 – 0.10	-	-	0.06 – 0.10	0.06 – 0.10

Explanatory Notes	
Note 1	Range is typically from 10th percentile to 90th percentile. The values given in brackets (25th percentile if not stated otherwise) are recommended design values but should not be taken as mandatory. Where there is no testing data available for particular geotechnical units, design parameters are established/estimated based on best engineering practice and experience. The design strength values (as given in brackets) are recommended for design and stability assessments whereas upper bound strength values should be considered for equipment performance and excavatability assessments. Where only a reduced number of tests were performed (less than four), the individual numbers are given.
Note 2	Lower bound value shall be used when estimating resistance/passive force/pressure, whereas upper bound value shall be used when estimating driving/active force/pressure.
Note 3	Atterberg Limits are based on laboratory test results of soil sample and rock residue from abrasivity testing.
Note 4	The values given in brackets represent values based on testing data from Central Interceptor and best engineering practise and experience.
Note 5	Rock intact tensile strength is measured indirectly in laboratory by conducting Brazilian test on rock core samples. The preliminary design strength values (as given in brackets) are recommended for design and stability assessments whereas upper bound strength values should be considered for equipment performance and excavatability assessments
Note 6	Geological Strength Index (GSI) of rock units is estimated considering rock composition and structure to be sandstone with thin inter-layers of siltstone to thick bedded very blocky sandstone with fair to very good condition (ECBF and Parnell Grit), and blocky disturbed to blocky with fair to good condition (Basalt) (Ref: E Hoek 2007).
Note 7	Material Constant mi of rock units is estimated from published values using RocLab software 1.033.
Note 8	Young's modulus values are obtained from UCS testing, Pressuremeter tests and Dilatometer tests. 68ites68ents value in brackets represents mean values.
Note 9	Where no laboratory or insitu data is available soil values is derived using soil consistency from published data (Look, 2007). Rock parameters are derived from laboratory testing.
Note 10	Results are from either insitu hand held shear vane testing (peak values) or laboratory UU triaxial results. For undifferentiated Tauranga and residually to highly weathered ECBF – cohesive soils, results are a summary of both.
Note 11	Friction angle and cohesion for rock is estimated using RocLab with the following assumptions: 'Tunnels' for ECBF and Parnell Grid, disturbance factor = 0.0 for ECBF as TBM, depth = 30m and 80m. 'Slopes' for basalt as open excavations, Disturbance factor = 0.7 for basalt as open excavation with rock breaker or careful blasting, depth = 15m. For basalt, mechanical analysis for global stability should be considered and screening is likely to be required to control falling material.
Note 12	Soil values are derived using soil consistency from published data (Look, 2007). Rock mass modulus I is estimated using Hoek and Diederichs (2006) simplified and generalised equations (whichever is lesser), which uses GSI, disturbance factor, D, and Young's Modulus, E_i, as input. $E_{rm}(\text{MPa}) = 100,000\left(\frac{1-D/2}{1+e^{((75+25D-\text{GSI})/11)}}\right)$ Simplified Eq. $E_{rm} = E_i\left(0.02+\frac{1-D/2}{1+e^{((60+15D-\text{GSI})/11)}}\right)$ Generalised Eq. Where available data from Pressuremeter tests were incorporated. Values in brackets represent mean values.
Note 13	Permeability values are from slug, Lugeon and pumping tests. Assumed values have been adopted for Recent Alluvium, Tauranga Group Granular, Tuff/Ash/Scoria, and Residual Soils
Note 14	The earth pressure at-rest (K_0) for soil is estimated using Jaky's (1944) method, K_0 = 1-s'nφ'
Note 15	Insitu stress ratio, k (p_h/p_v) for the rock is estimated based on geological origin/stress history of the material and Pressuremeter test. It is recommended that the tunnel lining will be checked for a stress ration in the range of 0–5 - 2.0.

Appendix D. Phase 2 Model Inputs



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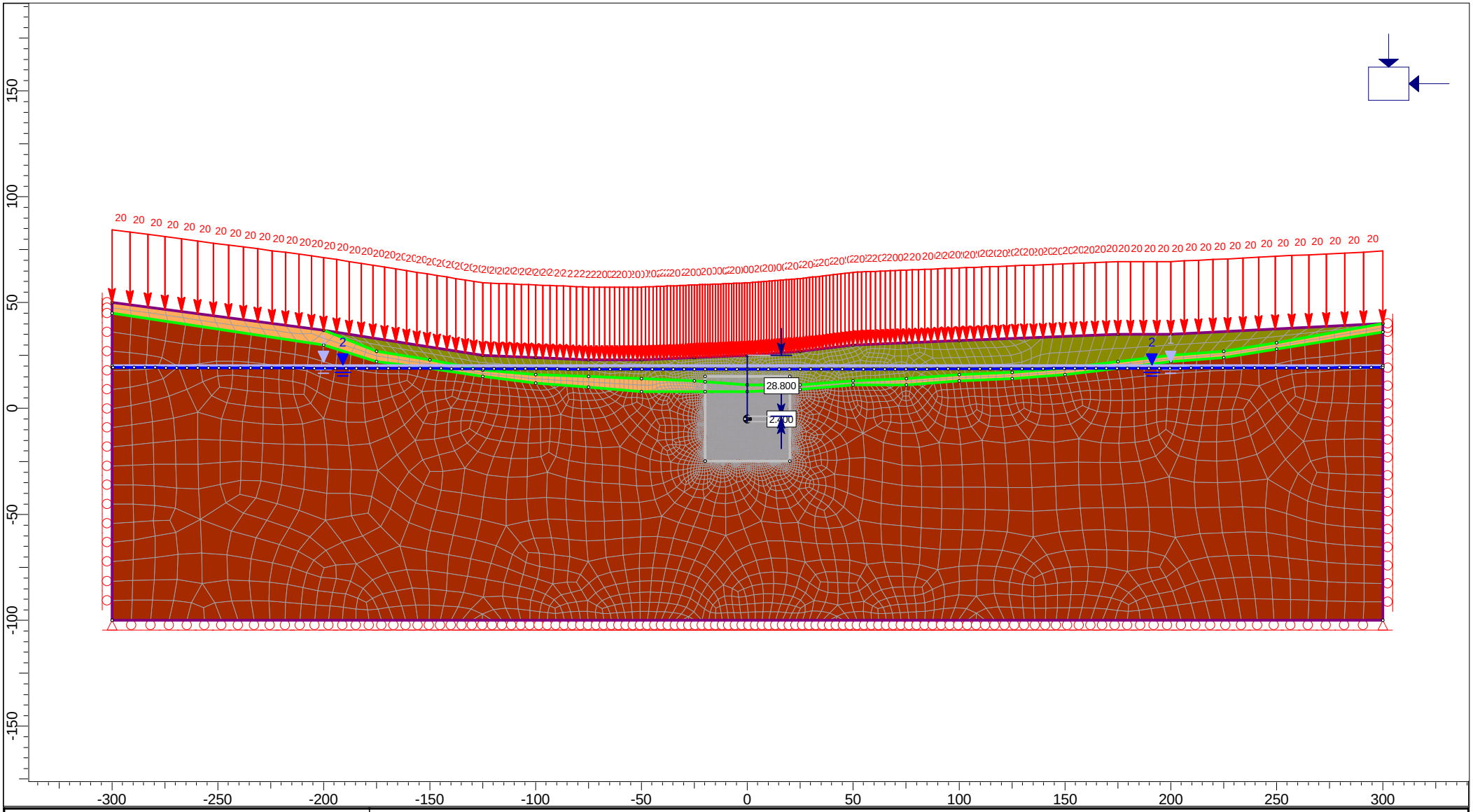

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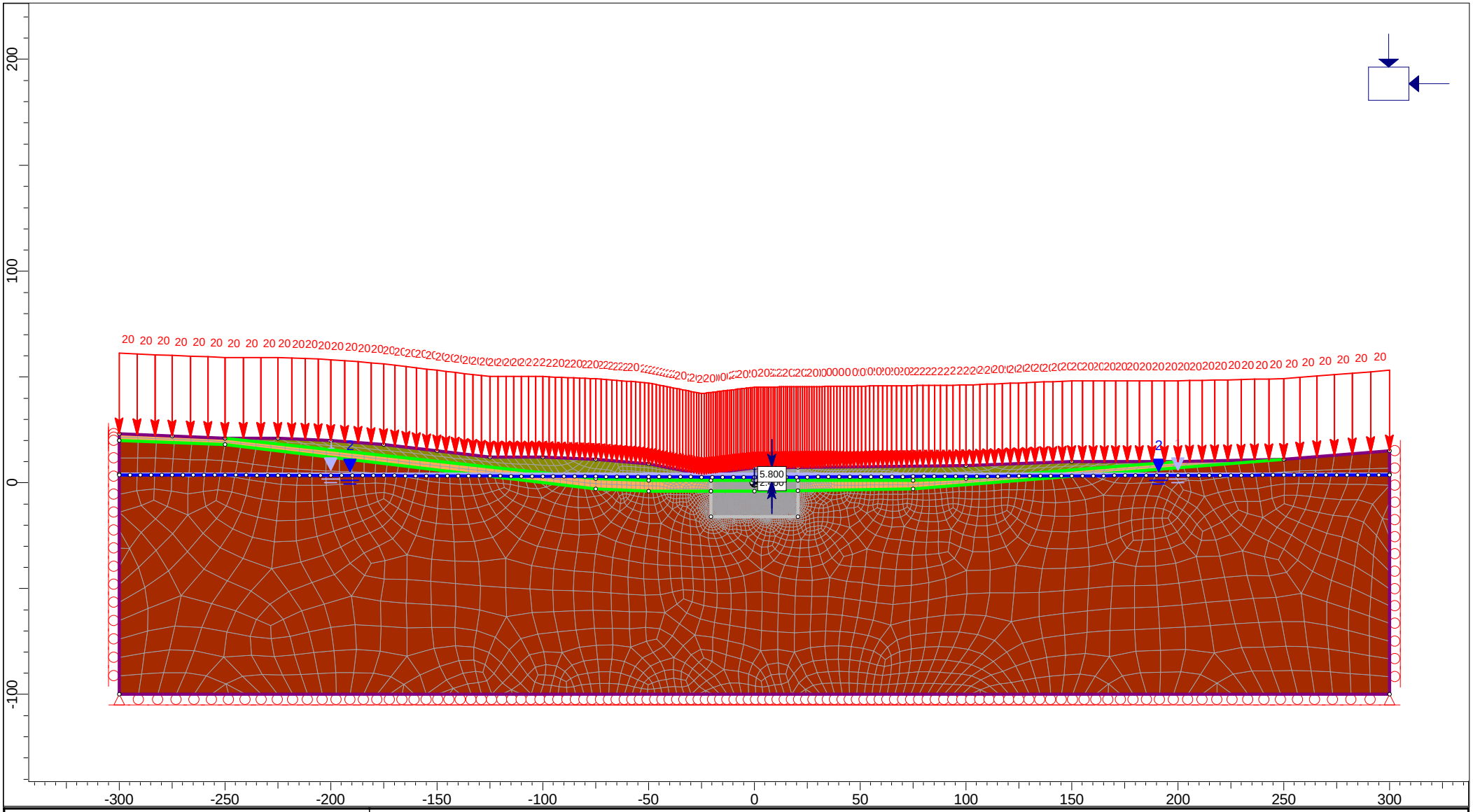

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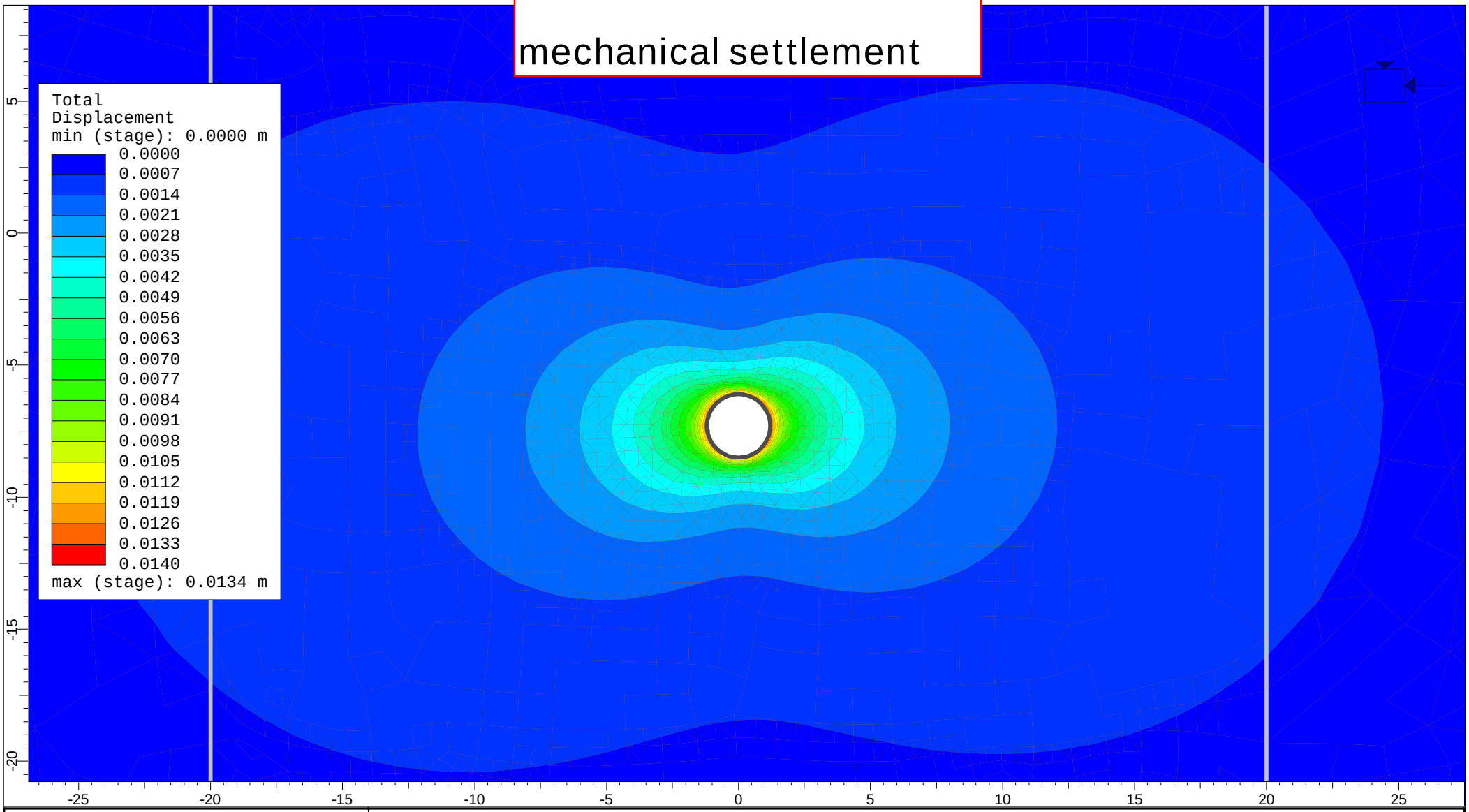
Appendix E. Phase 2 Results

Numerical calibration for mechanical settlement



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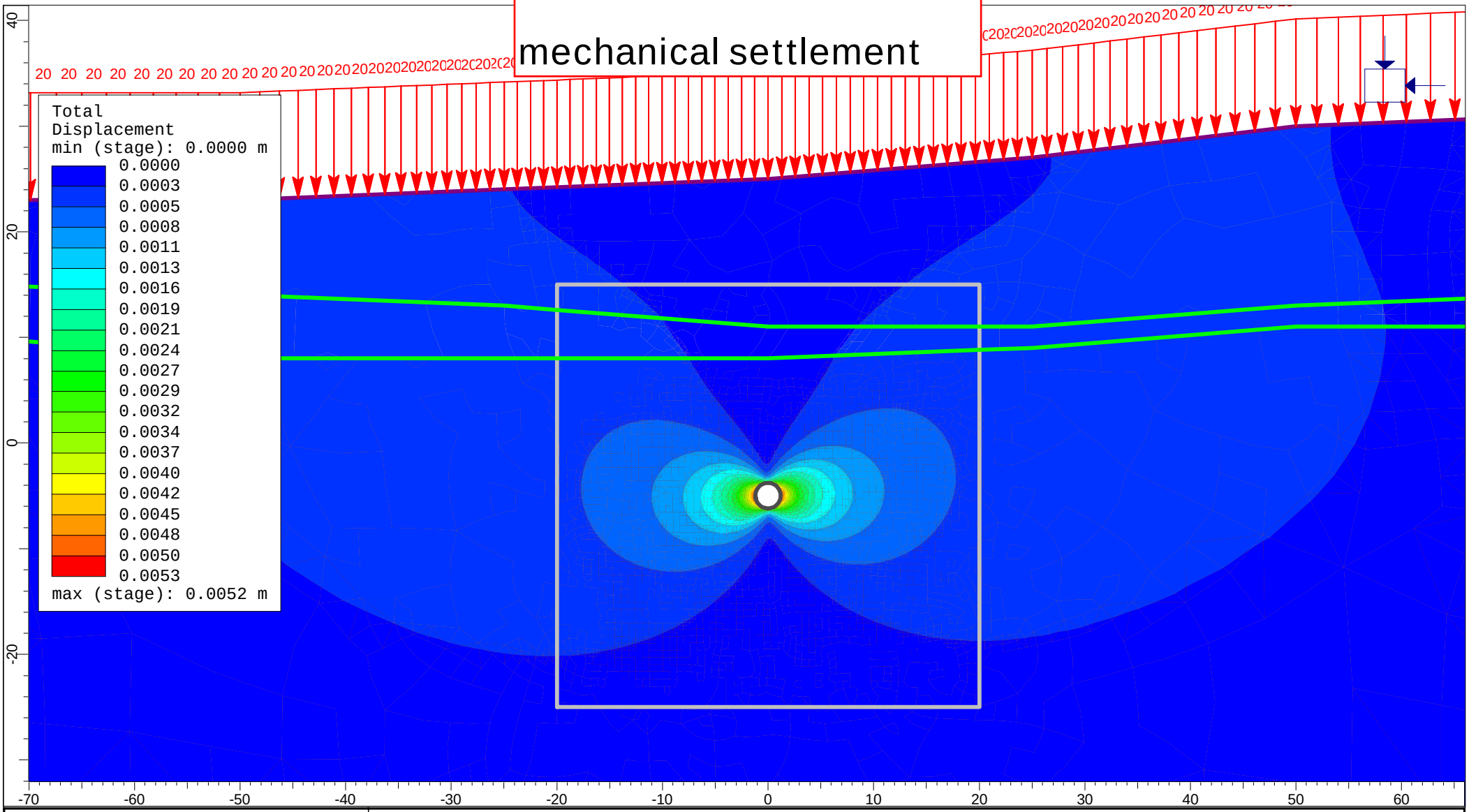
Numerical calibration for mechanical settlement



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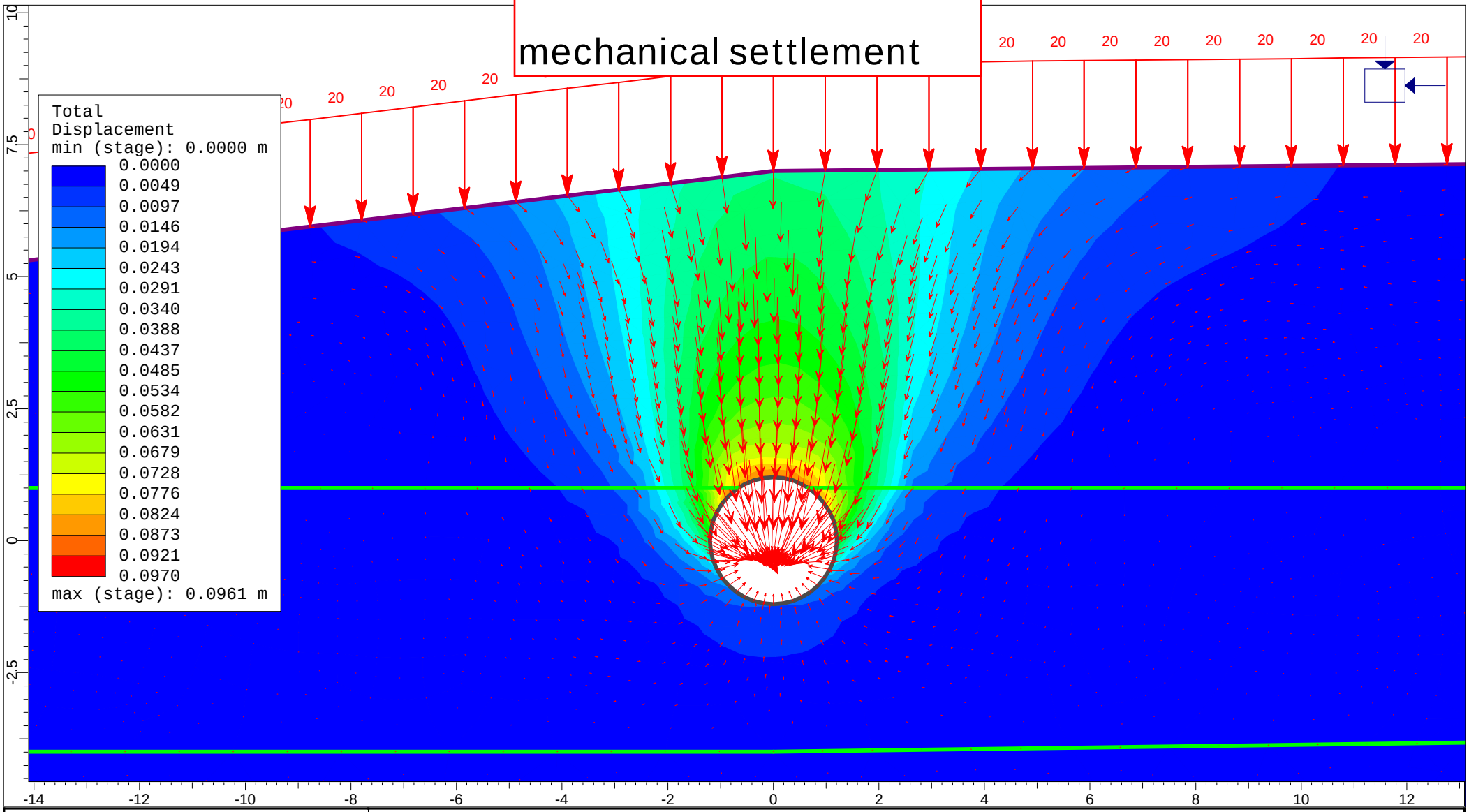
INTERPRET 9.016

Numerical calibration for mechanical settlement



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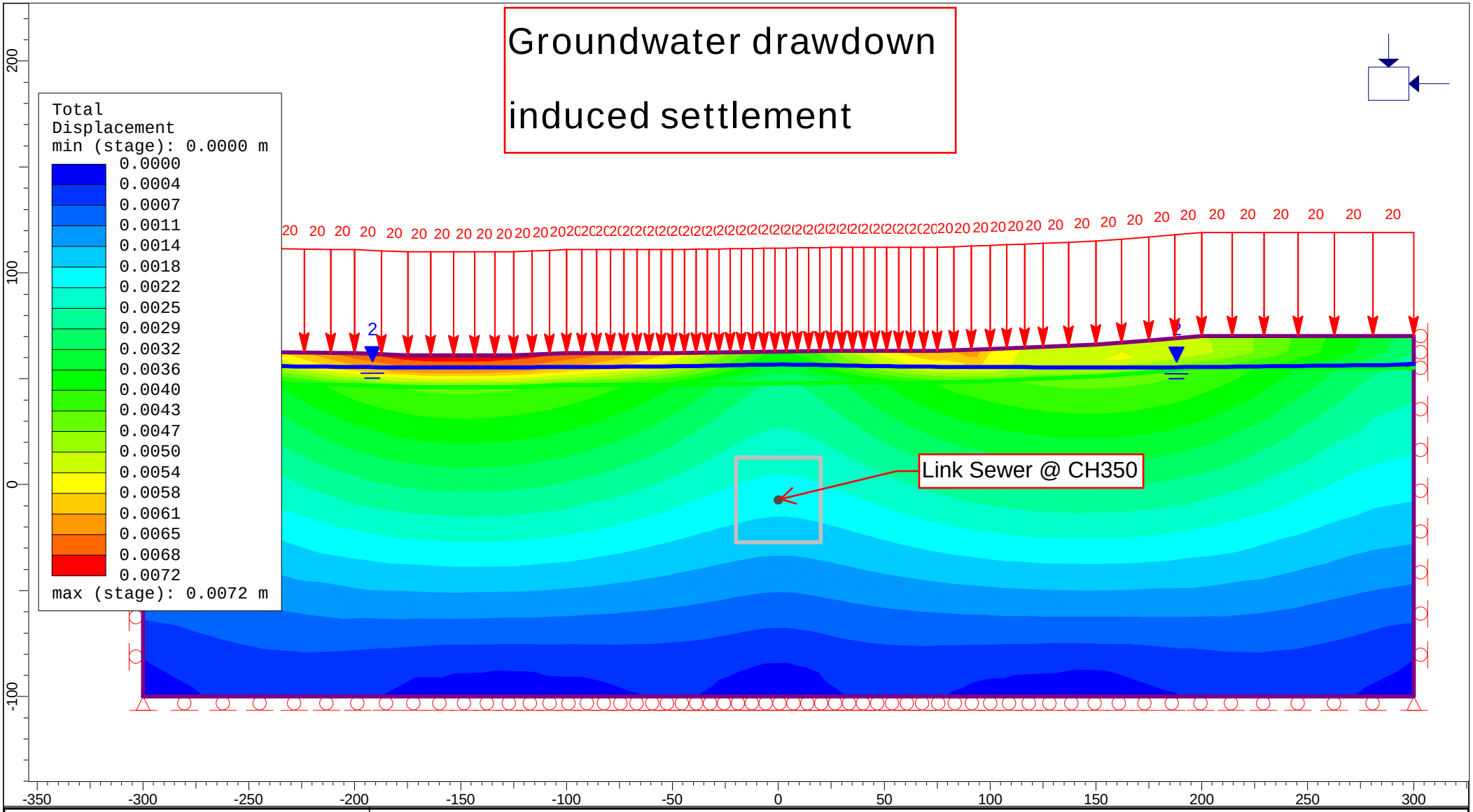
Groundwater drawdown induced settlement



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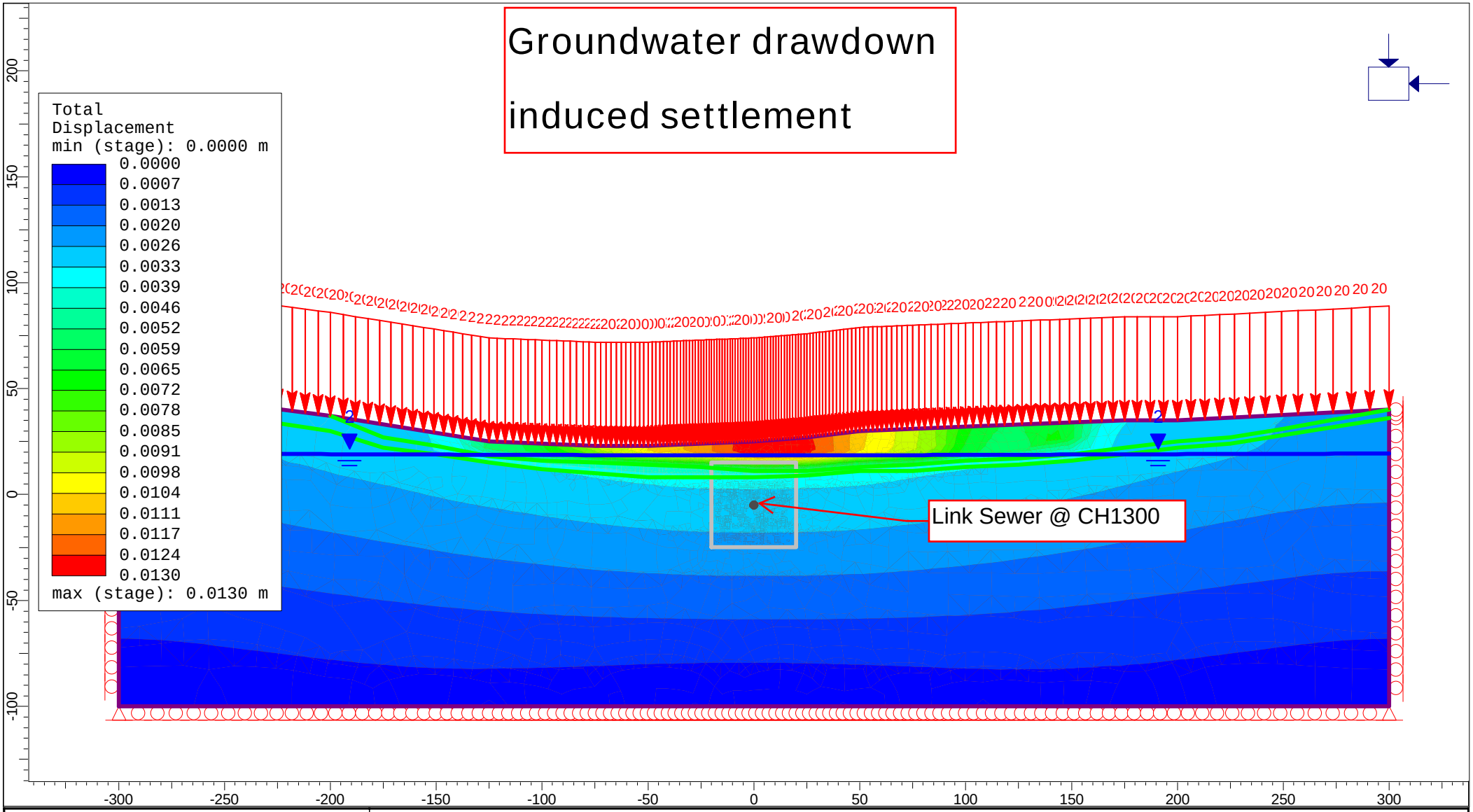
Groundwater drawdown induced settlement



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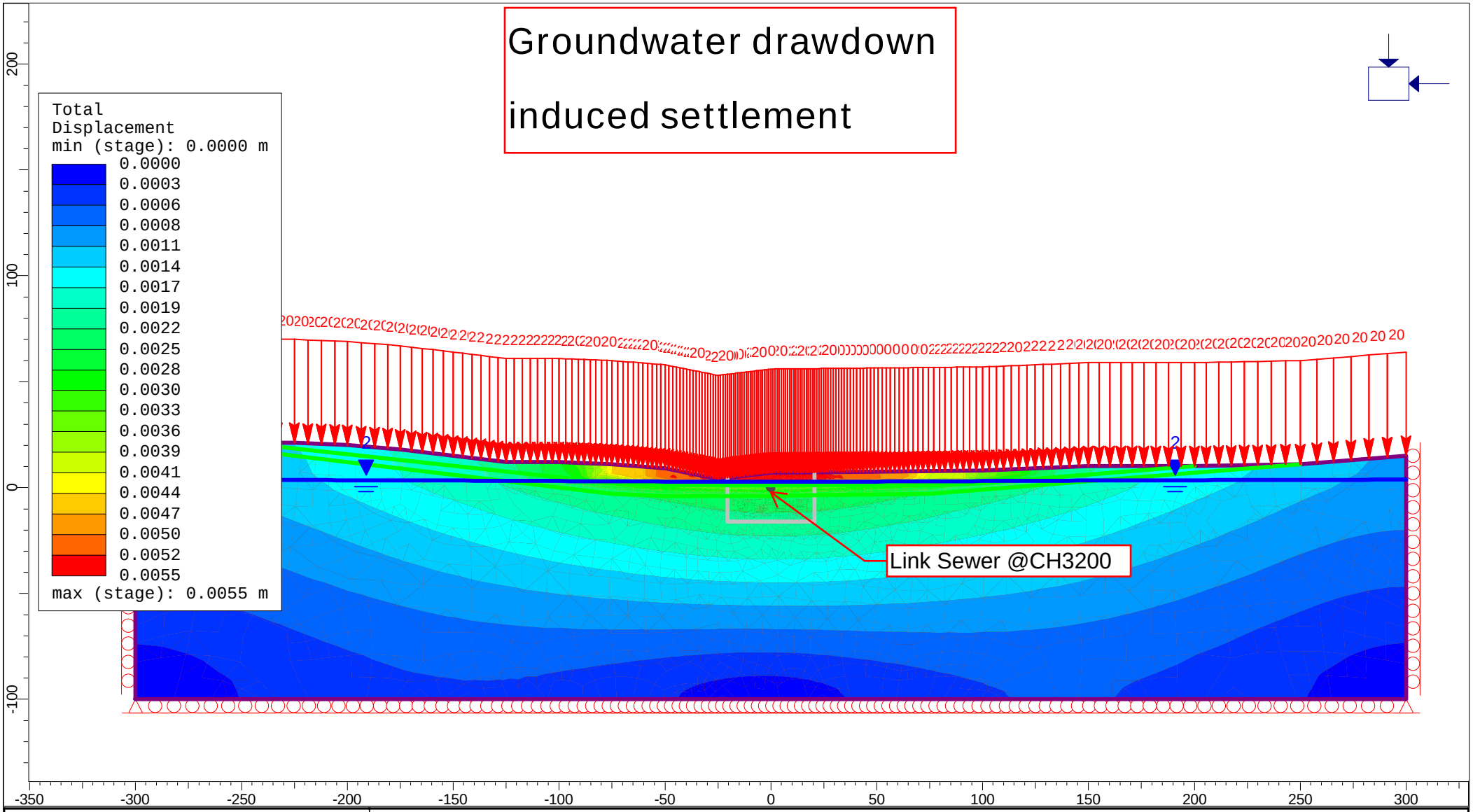
Groundwater drawdown induced settlement



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