Central Interceptor

Main Project Works Detailed Design

WATERCARE SERVICES LIMITED

Tunnel, Link Sewers and Shafts – Settlement Assessment

DSCIN-DEL-REP-T-J-100252 | 3

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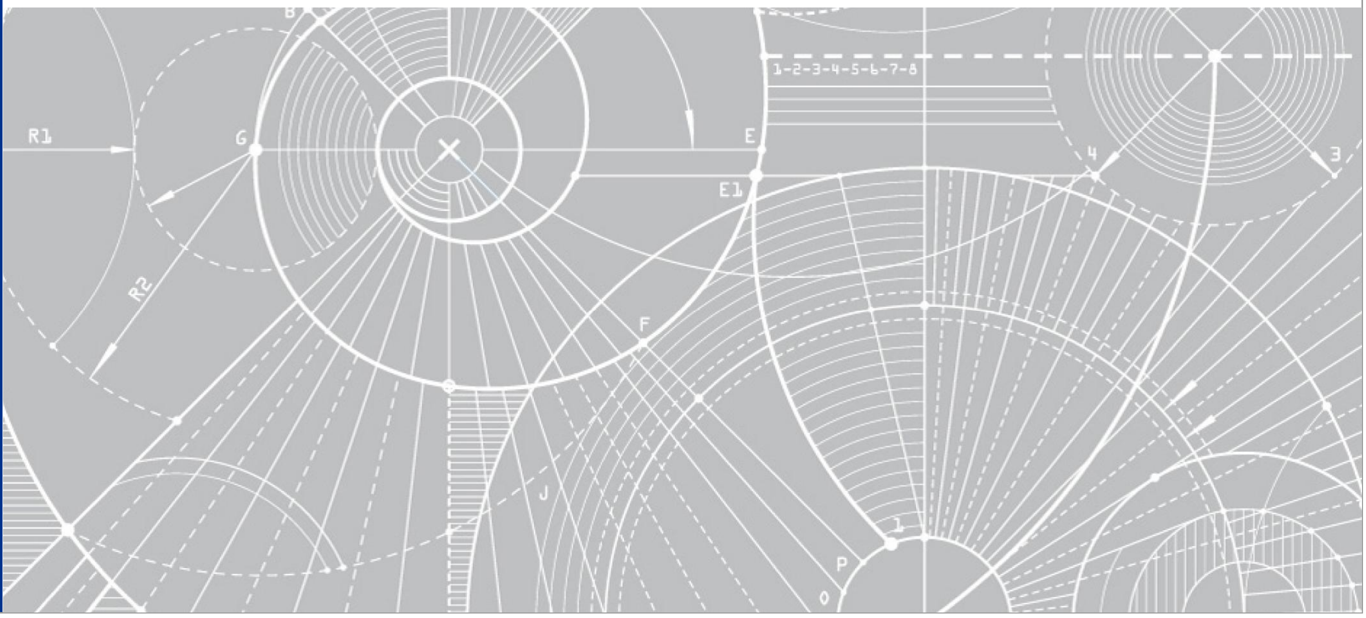

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Executive Summary

Pursuant to the CI RMA Consent Conditions, this report summarises the risk assessment undertaken to identify existing structures and utilities at risk of damage because of settlement caused by shaft sinking or tunnelling activities.

The Consent Conditions define limits for total settlement at 50mm, or differential settlement at 1:1000.

This settlement assessment considered both the mechanical settlement associated with excavation of tunnels and shafts, and the consolidation settlement that could occur as a result of dewatering during construction. Mechanical and consolidation settlements have been combined on settlement contour plots in Appendix A; these drawings also show areas where the 50mm total settlement or 1:1000 differential settlement may be exceeded.

Mechanical settlements were estimated by a combination of internationally accepted empirical methods, in some cases supplemented by numerical modelling. Groundwater drawdown was assessed by 2D and 3D numerical modelling, followed by numerical modelling of consolidation settlements.

Most the main tunnel alignment is in deep bedrock or in soils with a thick basalt cap overlying the alluvium, in which case settlement is anticipated to be negligible. Potential settlement impacts are primarily around the shafts, where soils are present and dewatering may occur.

The link sewers are predicted to have similar settlement behaviour as the main tunnel: negligible settlements from the link sewer tunnels and potential impacts around the link sewer shafts.

The alignment is generally situated under residential areas that are characterised by 1–2 storey stand-alone buildings. There are some commercial and larger 5 storey residential buildings near the Lyon Avenue shaft site. At the May Road shafts, there is a collection of commercial and industrial buildings, including the Foodstuffs complex. Other major non-residential buildings are the Cameron pool and leisure centre at the Keith Hay Park shaft site and the Mt Albert Community and leisure centre at the Mount Albert shaft site.

Utilities potentially impacted by settlement consist of water retail pipes (most commonly 100mm ID pressurised pipe); wastewater retail pipes (typically 150 to 450mm ID); stormwater pipes (225mm ID and greater); larger wholesale water and wastewater pipes that connect into the retail networks; gas lines; communication and fibre optic lines housed in conduits; and underground electrical transmission lines.

Major infrastructure above the CI alignment includes SH16 piled retaining wall and overbridge at the Western Springs on/off ramp; SH20 near Keith Hay Park; Transpower pylons in Manukau Harbour; and the Western Rail Line near Mt Albert War Memorial Reserve. Settlement of these major structures is anticipated to be negligible.

Where the consent settlement limits are anticipated to be exceeded, buildings and utilities were evaluated for potential damage using internationally accepted methods, which calculate building strains and compare with building deflection over building length, yielding damage severity that can be correlated with visual damage categories (e.g., Negligible, Very Slight, Slight, Moderate). A total of 21 buildings near shaft sites exceed either the 50mm total settlement or 1:1000 differential settlement consent criteria. Of these 21 buildings, only 2 buildings may experience damage beyond “Negligible,” specifically:

- **16 Norgrove Ave** near the Norgrove Ave Shaft on Link Sewer B. This residential home is in the “Slight” damage category.
- **22 Gregory Pl** near Keith Hay Shaft: This building is on Watercare land and will be demolished for construction of the shaft.

A damage assessment was also made for utilities at risk to settlement around the shafts for both the main tunnel and link sewers. These analyses indicated only one utility potentially at risk: **Haverstock stormwater pipe SW9** (450mm ID).

As a result of the settlement and building damage assessment, the following recommendations are made:

1. More detailed information about the existing building and utility conditions should be collected during pre-construction surveys where necessary.

2. The settlement assessment herein assumes shafts will be excavated 'in the wet' (shaft flooded) in soils subject to invert heave or excessive lateral deformations in the shaft wall. Should the contractor elect to excavate shafts for this situation in the dry, mechanical settlements will likely be more than 3 times the magnitudes predicted herein. This should be addressed in the GBR.

3. 16 Norgrove Ave will likely require a damage mitigation approach by the contractor (proactive or reactive). This should be communicated in the temporary shaft support specification.

4. Haverstock pipe SW9 (450mm ID) will likely require a damage mitigation approach by the contractor (proactive or reactive). This should be communicated in the temporary shaft support specification.

1. Introduction

This report summarises the risk assessment undertaken to identify existing buildings and structures at risk of damage due to settlement caused by shaft sinking or tunnelling activities. This is written pursuant of requirements set forth in Consent Condition 4.10 and 4.33:

4.10 The Consent Holder shall undertake a risk assessment to identify existing buildings and structures at risk of damage due to settlement caused by shaft sinking or tunnelling activities. The risk assessment process shall be set out in the M&CP required by Condition 4.6 and shall be based upon the final tunnel alignment and construction methodology, the groundwater and settlement monitoring required under this consent, and groundwater and settlement modelling completed using this data. The risk assessment shall include:

(a) identification of the zone of influence where differential settlements of greater (steeper) than 1:1,000 are predicted due to shaft sinking or tunnelling activities;

(b) identification of the building types in this zone, and their susceptibility to settlement induced damage; and

(c) identification of the buildings and structures at risk of damage due to shaft sinking or tunnelling activities.

4.33 The Consent Holder shall use all reasonable endeavours to ensure that the exercise of this consent does not cause:

(a) greater (i.e. steeper) than 1:1,000 differential settlement (the Differential Settlement Limit) between any two adjacent settlement monitoring points required under this consent; or

(b) greater than 50mm total settlement (the Total Settlement Limit) at any settlement monitoring point required under this consent.

1.1 Project Overview

The proposed Central Interceptor tunnel is a new 13 km long, 4.5m inside diameter wastewater tunnel from Western Springs to Mangere Waste Water Treatment Plant in Auckland (Figure 1-1). It will lie between 21 and 107m below ground level, and cross the Manukau Harbour at a depth of approximately 15m below the seabed. There will be 10 shafts up to approximately 80m deep on the main alignment including three large diameter working shafts, one of which will also serve as the pump station at Mangere WWTP. The project also incorporates two link sewers (referred to as Link Sewers B and C) adding a further 4.2 km of smaller diameter tunnels and seven shafts to the project. The main tunnel will be constructed using an earth pressure balance (EPB) tunnel boring machine (TBM). Shafts will be constructed using open-excavation or drilling methods.

1.2 Report Scope

This report describes the assessment of ground settlement that could result from the construction of tunnels and shafts for the Central Interceptor, the effects of these settlements on the existing buildings, services and infrastructure.

For the tunnels and shafts this report considers both the mechanical settlement associated with excavation and construction, and the consolidation settlement that could occur as a result of dewatering. Settlement will result from different aspects of the construction. Each of the sources is described in the report, along with the methodologies for analysing and combining the settlements.

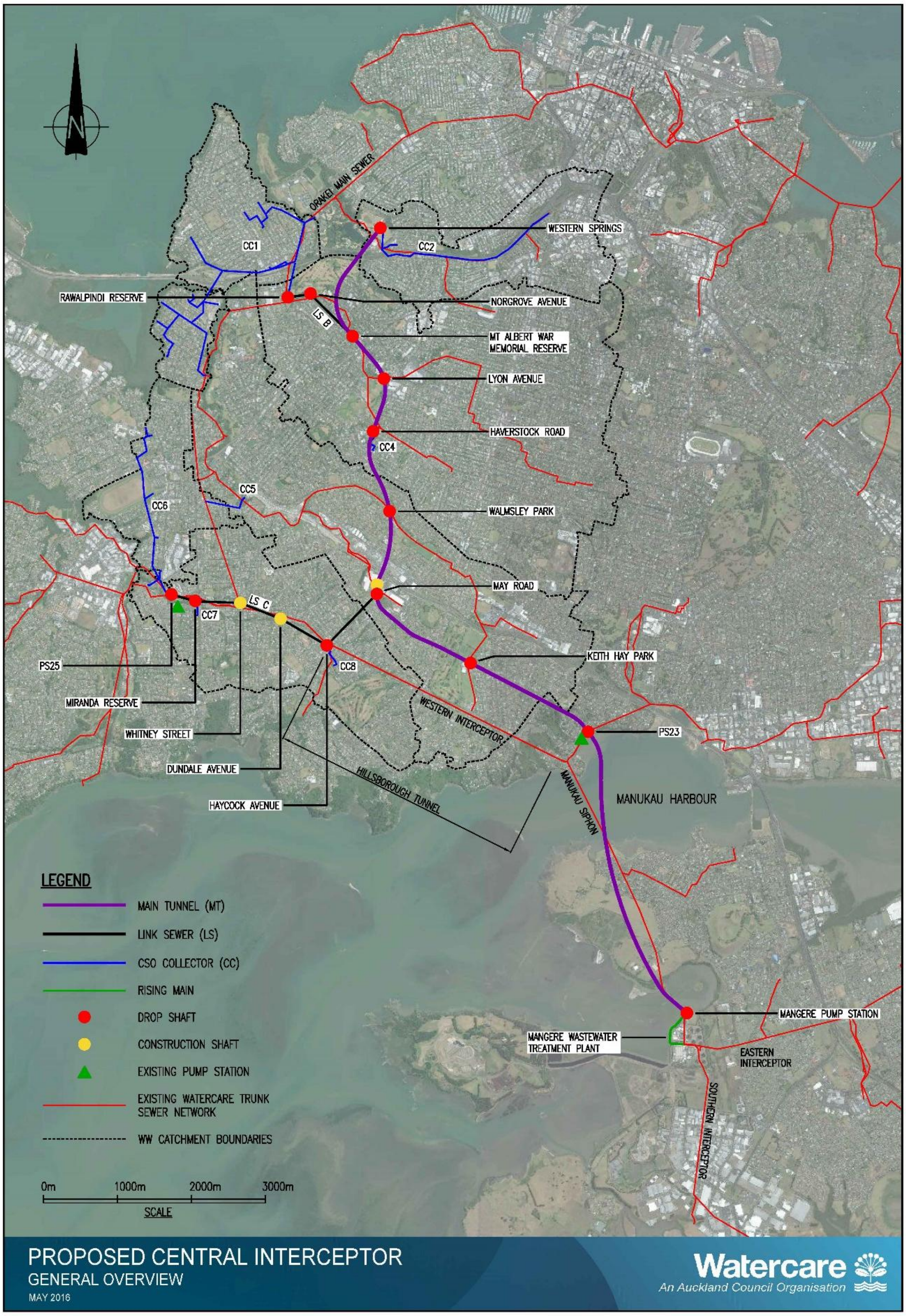

Figure 1-1: Project Overview

1.3 Abbreviations

Table 1-1: Abbreviations used in this Report

Abbrev.	Description
AVF	Auckland Volcanic Field
CI	Central Interceptor
CPT	Cone Penetration Test
DPCIN	Watercare Operations facility code for the Mangere Pumping Station
DSCIN	Watercare Operations facility code for the Central Interceptor Main Tunnel
ECBF	East Coast Bays Formation
EPB	Earth Pressure Balance
GBR	Geotechnical Baseline Report
GIR	Geotechnical Interpretive Report
GIS	Geographical Information System
HDPE	High-density Polyethylene
ID	Internal Diameter
LS	Link Sewer
MPS	Mangere Pumping Station
PE	Polyethylene
PVC	Polyvinyl Chloride
RCP	Reinforced Concrete Pipe
RMA	Resource Management Act
TBM	Tunnel Boring Machine
WSP	Welded Steel Pipe
WWTP	Wastewater Treatment Plant
ZOI	Zone of Influence

Watercare facility codes for the shafts are as shown below in Table 1-2.

Table 1-2: Watercare Central Interceptor Facility Codes

Code	Facility Name
CSO	Collector Sewer
DPCIN	Mangere Pumping Station
DSCIN	Central Interceptor Tunnel
DSCIN002	PS23
DSCIN003	Keith Hay Park
DSCIN004A	May Road (Drop)
DSCIN004B	May Road (Work Shaft)
DSCIN005	Walmsley Park
DSCIN006	Haverstock Road
DSCIN007	Lyon Avenue

Code	Facility Name
DSCIN008	Mt Albert War Memorial Reserve
DSCIN009	Western Springs
DSLSB	Drainage Sewer Link Sewer 2
DSLSB001	Norgrove Avenue
DSLSB002	Rawalpindi Reserve
DSLSC	Drainage Sewer Link Sewer 3
DSLSC001	Haycock Avenue
DSLSC002	Dundale Avenue
DSLSC003	Whitney Avenue
DSLSC004	Miranda Avenue
DSLSC005	PS25
PWCIN	Project Wide

1.4 Sources of Effect

The sources of settlement associated with the construction of the Central Interceptor project are the following:

- Mechanical settlement of the ground due to excavation of the tunnel. The relaxation of the rock and soil above the tunnel can result in settlement that occurs within a short period after the excavation is done and is concentrated over the tunnel alignment.
- Mechanical settlement of the ground due to excavation of shafts. Lateral deflection of the temporary shaft walls during excavation can result in settlement that occurs within a short period after excavation and is concentrated in the area immediately behind the wall.
- Consolidation of the ground due to extraction of groundwater. Depending on the compressibility properties of the soils, draining of the groundwater into the excavation can result in consolidation of the ground around the shafts, and resulting settlement will occur over a longer period. Watertight final linings proposed for the shafts will not allow for permanent draining of groundwater, and only assessment of ground consolidation occurring during construction (short-term draining) is considered.

1.5 Related Reports

This report refers to the following project reports:

- Basis of Design Report – Reference: PWCIN-DEL-REP-G-J-00017, 15 June 2016
- Geotechnical Interpretative Report – Reference: PWCIN-DEL-REP-GT-J-100048.4, 17 June 2016
- Geotechnical Factual Report – Reference: PWCIN-DEL-REP-GT-J-100047.4, 3 June 2016
- Assessment of Potential Groundwater Drawdown due to Shaft Construction – Reference: PWCIN-DEL-REP-GT-J-100236, 30 Sep 2016
- Assessment of Ground Settlement at Link Sewer B & C due to Shaft Construction – Reference: PWCIN-DEL-REP-GT-J-100239, 23 September 2016
- Combined Settlement Report for the Link Sewers – Reference: PWCIN-DEL-REP-GT-J-100262, 11 November 2016

- Central Interceptor Main Works, Resource Consent Conditions – Reference: STD00538.01953, 19 December 2013

2. Existing Environment

2.1 Overview

The mainline tunnel commences at Western Springs in the north, where it curves generally towards the south, terminating at Mangere pumping station. From the north, the tunnel passes under the SH16 motorway and through Mount Albert before crossing the SH20 motorway in Mount Roskill. The alignment then continues under the Hillsborough ridge before traversing under Manukau Harbour and terminating at the Mangere Pumping Station.

2.2 Geology

The subsurface geology along the north section of the CI alignment is dominated by the weak sandstones and mudstones/siltstones of the Waitemata Group rocks, in particular the East Coast Bays Formation (ECBF), with volcanic deposits (basalt flows) and Tauranga Group alluvium deposits within the present day and paleo-drainage channels cut into the Waitemata Group rocks.

In the southern section of the CI alignment, the geology is dominated by Puketoka Formation (part of Tauranga Group) alluvial sediments. The Kaawa Formation sands and Waitemata Group rocks also occur within the southern section of the alignment immediately north of the Mangere Pumping Station site. Volcanic deposits (primarily basalt flows) mantle the alluvial soils in areas flanking the Mangere peninsula.

Geologic units that will be encountered along the Main CI Tunnel alignment include the Tauranga Group, Kaawa Formation and the ECBF of the Waitemata Group, including isolated lenses of the Parnell Volcaniclastic Conglomerate (PVC) of the ECBF. Shaft excavations will encounter surficial deposits of Made Ground (undifferentiated fill), Undifferentiated Tauranga Group alluvium, Kaawa Formation, Auckland Volcanic Field (AVF) basalt/tuff, residual ECBF soils and weathered ECBF rock.

A detailed geologic profile is provided on Drawing 2012061 in the GIR.

2.3 Buildings

The alignment is generally situated under residential areas that are characterised by 1–2 storey stand-alone buildings. There are some commercial and larger 5 storey residential buildings near the Lyon Ave shaft site. At the May Road shafts, there is a collection of commercial and industrial buildings, including the Foodstuffs complex. This complex is considered to be a sensitive stakeholder. Other major non-residential buildings are the Cameron pool and leisure centre at the Keith Hay Park shaft site and the Mt Albert Community and leisure centre at the Mount Albert shaft site.

Pre-construction building structure and dilapidation surveys have not yet been conducted, but in general small residential buildings are anticipated to be wood frame and masonry structures, while larger buildings are anticipated to be mixed structural systems of wood, steel frames, masonry, concrete frames, or pre-cast ‘tilt-up’ structures.

2.4 Utilities

In residential areas, utilities generally consist of smaller diameter pipes and conduits. Water retail pipes are most commonly 100mm ID pressurised pipe, made of asbestos cement or concrete lined cast iron. These pipes are buried approximately 1 meter belowground. Wastewater retail pipes are commonly polyethylene, earthenware or concrete, with a wide variety of sizes, but typically 150 to 450mm ID. Stormwater-only pipes are most commonly concrete pipes 225mm ID and greater. Both wastewater and stormwater networks are gravity fed and are typically buried 1–4 meters belowground.

Retail services connect into the network via larger wholesale pipes. Water wholesale mains are typically concrete-lined steel pipes that are pressurised and buried approximately 1 meter deep. Wastewater wholesale pipes are commonly reinforced concrete and vary in depth. These pipes can be quite deep underground as they rely on gravity flow with the occasional pumping station. Some of these are larger utilities that were installed by tunnelling methods.

Other less common utilities include the Marsden to Wiri gas lines, communication and fibre optic lines housed in conduits and underground electrical transmission lines.

The main tunnel passes under or in proximity to the Western Interceptor on the margin of Mangere Lagoon and under the Manukau Siphon, which is a key Watercare asset. Cover over the tunnel as these locations is 23m and 14m, respectively.

2.5 Other Infrastructure

Notable infrastructure along the CI main tunnel alignment includes:

- SH16 piled retaining wall and overbridge at the Western Springs on/off ramp
- SH20 near Keith Hay Park
- Transpower pylons in Manukau Harbour
- Western Rail Line near Mt Albert War Memorial Reserve

Smaller infrastructure along the alignment includes local roads, footpaths, and culverts.

3. Consent Settlement Limits

3.1 Consent Settlement Limits

The settlement limits are defined per Consent Condition 4.33 as:

- Differential Settlement Limit: 1:1,000 between any two adjacent settlement monitoring points required under the consent; or
- Total Settlement Limit: 50mm at any settlement monitoring point required under the consent

The consent holder must undertake all reasonable endeavours to ensure these limits are not exceeded.

3.2 Consent Alert and Alarm Levels

The Consent Conditions define Alert and Alarm Levels as the following:

- The 'Alert Level' is the Differential Settlement Limit or Total Settlement Limit set at a threshold less than the Alarm Level, at which the consent holder shall implement further investigations and analyses (as will be described in the Contractor's *Monitoring and Control Plan*) to determine the cause of settlement and the likelihood of further settlement. The contract specifications will set the Alert Level as a percentage of the Alarm Level.
- The 'Alarm Level' is the Differential Settlement Limit and Total Settlement Limit set in Consent Condition 4.33, or has the potential to cause damage to buildings, structures and services. At that time, the contractor shall immediately stop dewatering the site and cease any activity that has the potential to cause deformation to any building or structure or adopt the approved contingency measures.

3.3 Damage Trigger Levels

3.3.1 Buildings

Each structure within the zone of predicted settlement was evaluated for potential distortion due to settlement. The intent was not to precisely quantify the effect of settlement, but to determine which buildings are potentially at risk to damage and thus require further evaluation.

Criteria for allowable settlement of structures were originally a topic related to foundation engineering. The initial motivation for studies of building settlement and the degree of damage was to establish a basis for design of building foundations. The classic works and most comprehensive studies that set the early engineering precedents were by Skempton and MacDonald (1956) and Polshin and Tokar (1957). Additions to the experience base and summaries of world-wide practices took place over the years (such as by Bjerrum, 1963) and later in the United States, in particular by Wahls (1981). These studies concluded that differential settlement was a key factor influencing observed building damage. Since most of the observed building damage appeared to be related to distortional deformations, 'angular distortion' (β) was used as a critical index of damage. Angular distortion is a measure of differential settlement. Limiting angular distortions and potential types of damage are given in Table 3-1 below.

Table 3-1: Limiting Angular Distortion

Category of Potential Damage (after Wahls,1981)	β=δ/L [(a)]
Danger to machinery sensitive to settlement	1/750 (0.0013)
Danger to frames with diagonals	1/600 (0.0017)
Safe limit for no cracking of buildings [(b)]	1/500 (0.002)
First cracking of panel walls Difficulties with overhead cranes	1/300 (0.0033)
Tilting of high rigid buildings becomes visible	1/250 (0.004)
Considerable cracking of panel and brick walls Danger of structural damage to general buildings Safe limit for flexible brick walls, L/H >4b	1/150 (0.0067)
Notes: [(a)] β = angular distortion, δ = differential settlement, H = building height and L = span length of beam or building. [(b)] Safe limits include a factor of safety.	

On recent urban tunnelling projects, angular distortion criteria on the order of 1/500 to 1/600 have been used as threshold values for decisions regarding settlement mitigation measures.

Subsequent to the work of Bjerrum (1963) and Wahls (1981), tunnels and deep excavations for tunnel construction promoted substantial research regarding the effects on existing structures of excavation-induced ground movements. The work of Mair et al. (1996), also referred to as the ‘Burland Method’, added the additional effects of horizontal ground movement to the effects of angular distortion as a further refinement to building damage prediction. Their work, backed up by world-wide settlement data derived from actual field measurements of low-rise buildings, has gained worldwide acceptance in engineering practice.

3.3.2 Utilities

Each pipeline within the zone of predicted settlement was evaluated for potential distortion due to settlement. This distortion predominantly depends on pipe material and diameter, and the settlement profile. The trigger values shown in Table 3-2 are 80% of the maximum slope calculated in in Table 5-4.

Table 3-2: Utility Deformation Trigger Values

Utility Type	Utility Dia. (mm)	Trigger Level
WSP	-	1:55
Cast-in-situ Concrete	-	1:75
PVC & HDPE	-	1:30
RCP	-	1:290
Ductile Iron Pipe	-	1:290
Vitrified Clay Pipe		1:290
Cast Iron Pipe	150	1:65
	200	1:80
	300	1:110
	400	1:150
	500	1:200

Utility Type	Utility Dia. (mm)	Trigger Level
	600	1:270
	750	1:330
Notes: WSP= Welded Steel Pipe RCP = Reinforced Concrete Pipe		

4. Settlement Assessment Methodology and Results

4.1 Expected Areas of Effect

Settlement assessments were carried out on areas where tunnel excavation may result in excavation-related settlements exceeding measurable levels. These areas include lengths of tunnel where:

- The tunnel crown is in alluvium or residual soil and where there is no basalt or thin basalt cap (i.e. less than 1.5m thick).
- The tunnel crown is within a few metres (less then 3m) of the top of the ECBF rock (unweathered to highly weathered), which is overlain by alluvium and/or residual soils without a basalt cap (less than 1.5m).

Sections of the tunnel that do not meet the above criteria (i.e. most the main tunnel alignment) are excluded from detailed settlement assessment because of favourable geological conditions that will result in negligible settlement.

Applying the exclusion criteria above to the main tunnel, the areas of effect extend along the following chainages:

- Tunnel: CH 10000 to CH 10600 – Mangere WWTP to Ambury Park
- Tunnel: CH 12800 CH 13700 – Manukau Harbour Crossing
- Tunnel: CH 23050 to CH 23067 – Mixed-face conditions in Western Springs area

All shafts were analysed along the mainline tunnel and the link sewers. No exclusion criteria are applied for shafts. The major component settlement is expected to be consolidation settlement due to groundwater drawdown at the shafts. Here, the zone of influence is expected to extend beyond site boundaries at some shaft sites. Sites that have alluvium deposits near the surface will be more at risk of this type of settlement.

Both May Road and Lyon Avenue shaft sites have a basalt layer in close proximity to the shaft, in conjunction with alluvial deposits. Similarly, the Haverstock shaft site sits on alluvium with residential properties to the east across Meola Creek, and commercial buildings upslope to the west. Alluvial deposits likely to exhibit some degree of settlement are also present at the Walmsley Park and Western Springs shaft sites. Residential properties surround the Walmsley Park shaft site, while greenfield conditions (sports fields) are indicative of the Western Springs shaft site.

4.2 Main Tunnel & Link Sewer Tunnels Mechanical Settlement Assessment

4.2.1 Assumed Construction Methods

The CI main tunnel is required to be constructed using an earth pressure balance (EPB) tunnel boring machine (TBM) and a single-pass segmental lining. The EPB TBM must be able to apply a positive pressure to the tunnel face, balancing the earth and groundwater pressures at all times to effectively control the ground and prevent groundwater inflows into the tunnel.

The one-pass gasketed precast concrete segmental lining system is erected in the tail of the TBM concurrent with TBM advance. The annulus between the erected segmental lining and the excavation perimeter will be completely filled with grout. Annulus grouting provides continuous and intimate contact between the excavated ground and the precast concrete segmental lining and must be performed in a timely manner to reduce the risk of settlement resulting from closure of the annular tail shield void. Annulus grouting is also required to control the flow of water along the annulus, which may result in consolidation-related settlements.

The link sewer tunnels will be excavated and supported by pipe jacking methods, with the type of TBM selected by the contractor. The jacked pipe serves as permanent tunnel support.

Resource Consent Conditions limit impacts on the groundwater regime because of concerns about dewatering-induced settlement and environmental impacts. Accordingly, systematic dewatering methods are not allowed for groundwater control in the main tunnel or link sewer tunnels.

4.2.2 Methodology

The method used for calculation of tunnel settlements follows industry accepted practices pioneered by Peck (1969), and more recently updated by Mair et al. (1996). This method is well recognized for predicting settlements and has been successfully used on recent tunnel projects in Auckland and internationally. In addition, it provides a rational estimate of expected settlements and settlement trough width parallel to the structure, and the calculations are simple enough that multiple calculations can be readily performed for the varied conditions and geometries along the project alignment.

The tunnel settlement method used assumes that the shape of the settlement trough generated by tunnel construction follows a Gaussian distribution (see Figure 4-1). The volume of the settlement trough is assumed to be equal to the total volume of lost ground during tunnelling, which is usually given as a percentage of the excavated area. Lost ground is defined as the volume of all ground movements taking place around a tunnel. The maximum settlement and width of the settlement trough are a function of the volume of lost ground, depth of the tunnel, and geotechnical characteristics of the soils.

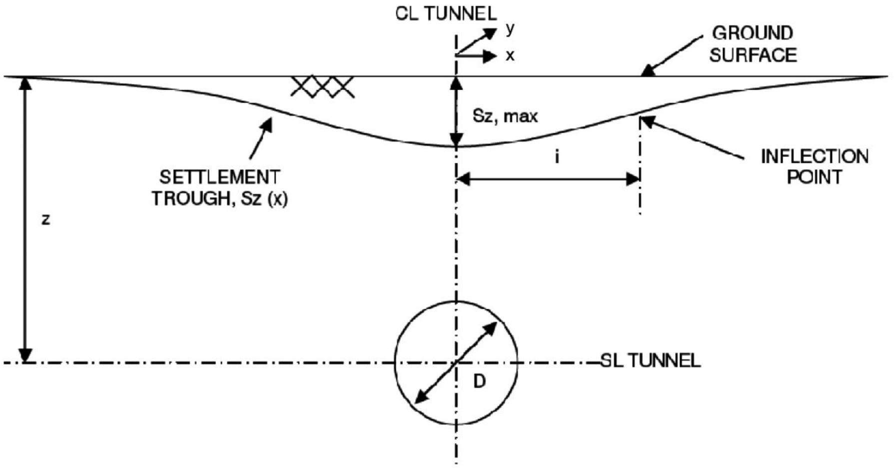

Figure 4-1: Gaussian Settlement Trough from Tunnelling

Settlement troughs presented herein are perpendicular to the tunnel alignment. Longitudinal settlements parallel to the tunnel, which are caused by the advancing tunnel and/or differing ground conditions over a short reach, were not examined because the impact of this type of settlement is typically transitory, levelling off as the tunnel passes. Also, the slope of the settlement profile parallel to the tunnel tends to be less severe than the profile perpendicular to the tunnel.

Two of the more important parameters in estimating settlement include average ground loss and settlement trough width parameter, and are discussed below.

4.2.2.1 Estimation of Ground Loss

The average ground loss is a function of many factors, including expected ground conditions, presence of groundwater, construction means and methods, and overall workmanship. Ground loss is typically caused by a combination of three general sources: face loss, shield loss, and tail loss. These sources can be summarized as follows:

- **Face Loss:** Ground loss at the heading of the tunnel, often caused by unsupported unstable ground conditions at the face (ravelling, running or flowing ground conditions). Over-excavation of material due to the presence of boulders or hard inclusions can also lead to face losses.

- **Shield Loss:** Ground loss at the shield of the TBM, often caused by movement of surrounding material into the overcut annulus caused by the cutterhead. Steering overcuts (ploughing and yawing, for example) to either excavate curves or to make steering corrections can increase the volume of this annular space.

- **Tail Loss:** Ground loss that occurs as the shield passes, often caused by movement of surrounding material into the annulus between the outside skin of the shield and the outside surface of the primary ground support. Deflection of the primary support under loads can also lead to some additional tail losses.

The method of excavation will impact the ability to control the ground and thus control deformation at the surface. By selecting an EPB TBM for tunnel excavation, the probability of significant deformation is minimised. EPB TBMs use the pressure of excavated materials inside the cutterhead to counteract earth pressure around the excavation. Thus, volume loss—which is a key factor affecting the magnitude of ground deformation—can be significantly reduced. Because of the nature of the ground, the method of tunnel excavation, and permanent lining installation as excavation proceeds, it is expected that most ground deformation will occur during tunnel excavation. Once the permanent lining is installed, there is expected to be very little if any additional ground movement. Impacts related to ground deformation include settlement of existing facilities, leading to cracking and other damage.

As described in Section 2.2, the local geology, for the most part, constitutes ECBF or alluvial materials. If excavated using well-controlled face-pressure TBM excavation, the ECBF is typically not susceptible to significant deformation and ground loss, and thus significant surface settlement is not anticipated. A 1% ground loss is widely accepted in practice as a conservative value for a pressure-face TBM in soils; 0.5% has been used/achieved for previous Auckland projects in ECBF conditions. Therefore, ground losses of 0.50% and 0.75% for the excavated area were assumed for the main tunnel settlement assessment. These assumptions of ground loss are considered to represent typical past tunnelling performance and yet are not overly optimistic.

Ground losses assumed for the link sewer tunnels (1% for pipe jack) is documented in the report *Combined Settlement Report for the Link Sewers – Ref: PWCIN-DEL-REP-GT-J-100262.*

4.2.2.2 Settlement Trough Width Parameter

The settlement trough width parameter is a function of many factors, including ground conditions, presence of groundwater, tunnel depth, and magnitude of the ground loss at the tunnel. The concept of the width factor (K) was developed by O'Reilly and New (1982), and was found to fit most field data well. This factor has been observed from past experience to typically vary from about 0.3 to 0.7, with the lower range of values usually associated with coarse-grained soils above the groundwater table, and the higher range of values usually associated with fine-grained soils or clean coarse-grained soils below the groundwater table. For the ground conditions anticipated along the tunnel alignment, values of K were assessed based on these factors and are detailed in Table 4-1.

Table 4-1: Trough Width Factor

Chainage	Location	Trough Width Factor, K (dimensionless)
10000	Mangere WWTP	0.32
10100	Mangere WWTP	0.32
10200	Mangere WWTP	0.31
10300	Mangere WWTP	0.30
10400	Mangere WWTP	0.30
10500	Mangere WWTP	0.30
13400	Mangere Inlet	0.38
13500	Mangere Inlet	0.36
13600	Mangere Inlet	0.37
Link Sewers	All	0.5 (see note 1)

Note 1: per *Combined Settlement Report for the Link Sewers – Ref: PWCIN-DEL-REP-GT-J-100262*

4.2.3 Summary of Results

The results are summarised in Table 4-2 for the main line tunnel. Maximum settlement along the tunnel is very small, ranging from 6 to 12mm. Maximum differential slope is also small, ranging from 1 in 2440 to 1 in 770 (steepest).

Table 4-2: Mechanical Tunnel Settlement Results – Main Line Tunnel

Chainage	Location	Maximum Settlement (mm)		Maximum Slope		ZOI (m)
		Vol. Loss 0.5%	Vol. Loss 0.75%	Vol. Loss 0.5%	Vol. Loss 0.75%	(Vol. Loss 0.5% & 0.75%)
10000	Mangere WWTP	6	9	1:2440	1:1610	49.9
10100	Mangere WWTP	6	9	1:2220	1:1490	48.0
10200	Mangere WWTP	6	10	1:1920	1:1280	44.6
10300	Mangere WWTP	6	9	1:1960	1:1300	45.0
10400	Mangere WWTP	7	10	1:1820	1:1210	43.2
10500	Mangere WWTP	6	9	1:1960	1:1300	45.0
13400	Mangere Inlet	8	12	1:1280	1:860	36.5
13500	Mangere Inlet	8	12	1:1150	1:770	34.6
13600	Mangere Inlet	8	12	1:1220	1:810	35.5

Note:

ZOI = Zone of Influence. This is the settlement trough width. The centreline is located at half this value, and settlement is symmetric on either side.

The link sewer results presented in Table 4-3 and Table 4-4 have been extracted from the report *Combined Settlement Report for the Link Sewers – Ref: PWCIN-DEL-REP-GT-J-100262*.

Maximum settlements along the link sewer tunnels are very small, ranging from 0.5 to 7.1mm. Maximum differential slope is also small, ranging from 1:172,000 to 1:875 (steepest).

The link sewer tunnels only exceeded the 1:1000 or 50mm settlement criteria at chainage 3050. This is due to the presence of a stream. The incision of the stream (poorer ground conditions) is also coincident with low cover above the tunnel crown. There are no buildings or services with in the area that this condition is present.

Table 4-3: Mechanical Tunnel Settlement Results - 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
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

Chainage (m)	VL (%)	K	i	Depth bgl (m)	Max Settlement (mm)	Zone of influence (m)	Ave Differential Settlement (1 in …)
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
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

Table 4-4: Mechanical Tunnel Settlement Results - 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

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

4.3 Main Tunnel Shaft Mechanical Settlement Assessment

4.3.1 Assumed Construction Methods

The shaft excavations for the CI tunnel project include 10 shafts along the main CI alignment, of which three will be working shafts during construction.

The contractor shall select shaft excavation methods to be compatible with the ground conditions and ground behaviour anticipated. Except for the Keith Hay Park and Walmsley Park shafts, shaft excavation methods are generally anticipated to consist of conventional excavation in overburden soils; conventional excavation in weaker bedrock conditions such as ECBF, Parnell Grit and rubbly basalt; and blasting in harder, thick basalt flows. Because of the smaller size and significant depth of the Keith Hay Park and Walmsley Park shafts, it is anticipated that the Contractor will elect to excavate these shafts by drilling methods to avoid manned entry.

Mangere Pumping Station is housed in a dual cell shaft that will be supported with a diaphragm slurry wall (D-wall). The D-wall is a stiff support system founded in rock, and is in a greenfield area, on Watercare land. Buildings currently on this site will be demolished prior to construction of the D-wall.

For all other shafts, the contractor shall is responsible for design of shaft temporary excavation support systems subject to the requirements in the specifications, compatible with the expected ground conditions and ground behaviours. Soil support systems anticipated include sheet piles with ring beams, steel pipe casings, secant piles and concrete caissons. Rock support systems include rock bolts, shotcrete and/or rock mesh.

Settlement at the shafts is primarily dependent on excavation support rigidity in overburden soils; settlement at shafts is dictated by the degree the shaft wall can flex inwards and allow for soil movement. Steel pipe casings, caissons and secant piles limit this movement and are considered rigid, whereas shafts constructed with sheet piles will result in more settlement. The anticipated rigidity of each shaft is given in Table 4-5.

Table 4-5: Assume Shaft Rigidity

Shaft No.	Shaft Name	Anticipated Soil Support Type	Rigidity
DSCIN009	Western Springs	Secant piles	Rigid
DSCIN008	MT Albert Memorial Reserve	Caisson	Rigid
DSCIN007	Lyon Avenue	Caisson	Rigid
DSCIN006	Haverstock Road	Sheet piles	Flexible
DSCIN005	Walmsley Park	Pipe casing	Rigid
DSCIN004	May Road	Secant piles	Rigid
DSCIN003	Keith Hay Park	Pipe casing	Rigid
DSCIN002	PS23	Sheet piles	Flexible
DPCIN	Mangere Pumping Station	D-wall	Very Rigid

FLAC sensitivity modelling indicates larger settlements unless measures are taken to stabilize the ground and minimise the ground loss during the shaft excavation in soils. Excessive upward displacements from invert heave are predicted unless mitigation measures such as excavation of soil 'in the wet' (i.e., shaft flooded with underwater grab) are utilised to provide support pressure in soils prior to reaching the top ECBF bedrock. Once excavation reaches ECBF bedrock, the construction method switches to dry excavation.

4.3.2 Methodology

Shaft settlement was assessed using methods recommended by Clough and O'Rourke (1990), calibrated with soil-structure interaction numerical modelling results for Western Springs and Lyon Avenue shafts.

The FLAC software was used to model the settlement profile for the Western Springs and Lyon Avenue shafts. A 2D axisymmetric model was used for these shafts, as it is specifically designed to model a cylinder-like shaft because the mesh is viewed as a unit-radian section, where horizontal displacement is in the radial direction.

The empirical Clough and O'Rourke (1990) approach to calculating ground movements associated with open-cut construction is based on the methodology originally defined in Peck (1969). Subsequent field performance monitoring has refined methods for estimating ground movements by considering not only the excavation depth, but also the geologic conditions and stiffness of the excavation support system.

The calibration between the FLAC model and the empirical Clough and O'Rourke method was performed on the Western Springs shaft, and is shown in Figure 4-2 below. The method was then used to predict the settlement at Lyon Avenue. This output is shown in Figure 4-3. The method predicts the maximum settlement to within 7% of what is predicted through finite difference modelling. As shown is Figure 4-3, the empirical method over-predicts the settlement nearer to the excavation and under-predicts it further away.

The calibrated empirical method was than applied to the remaining shafts, except for Mangere Pumping Station. The method was altered at the PS23 and Haverstock shaft sites to reflect the flexible shaft wall (sheet pile). The predicted settlement at each site is shown in Figure 4-4 to Figure 4-9.

Maximum settlement at Mangere Pumping Station was calculated and the Clough and O'Rourke settlement profile were applied to this.

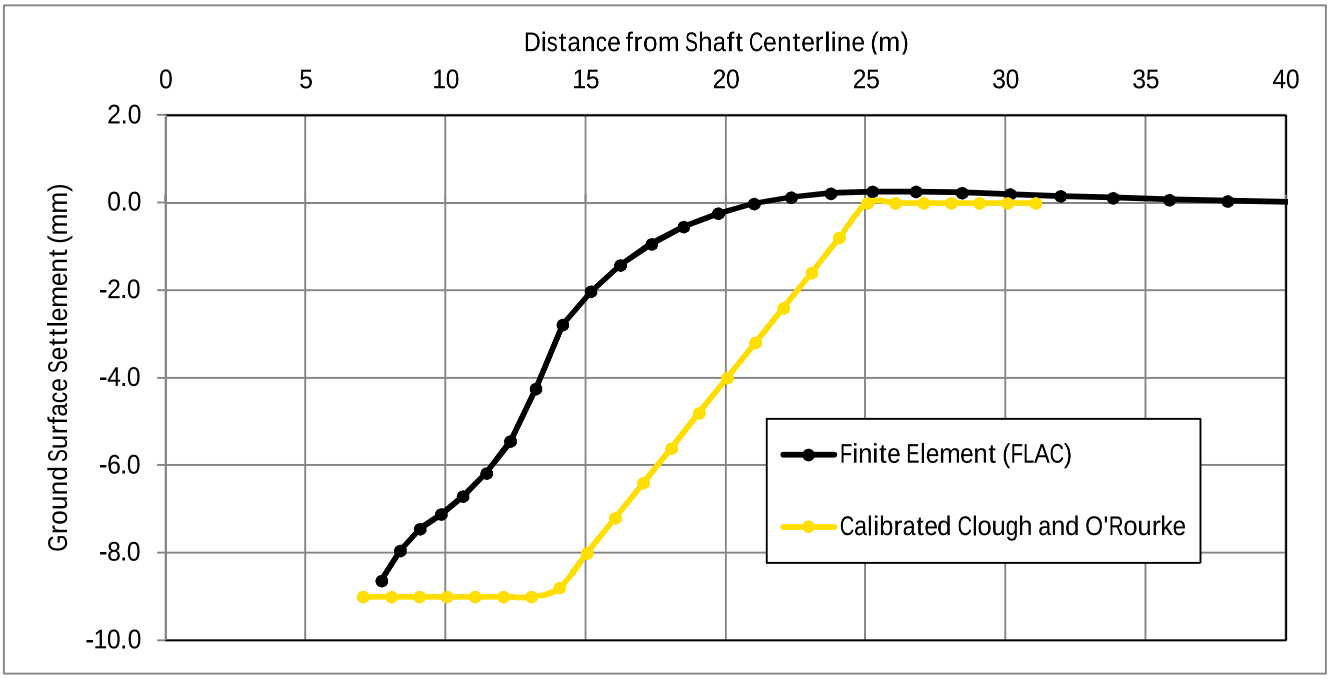

Figure 4-2: Western Springs Predicted Mechanical Settlement Profile

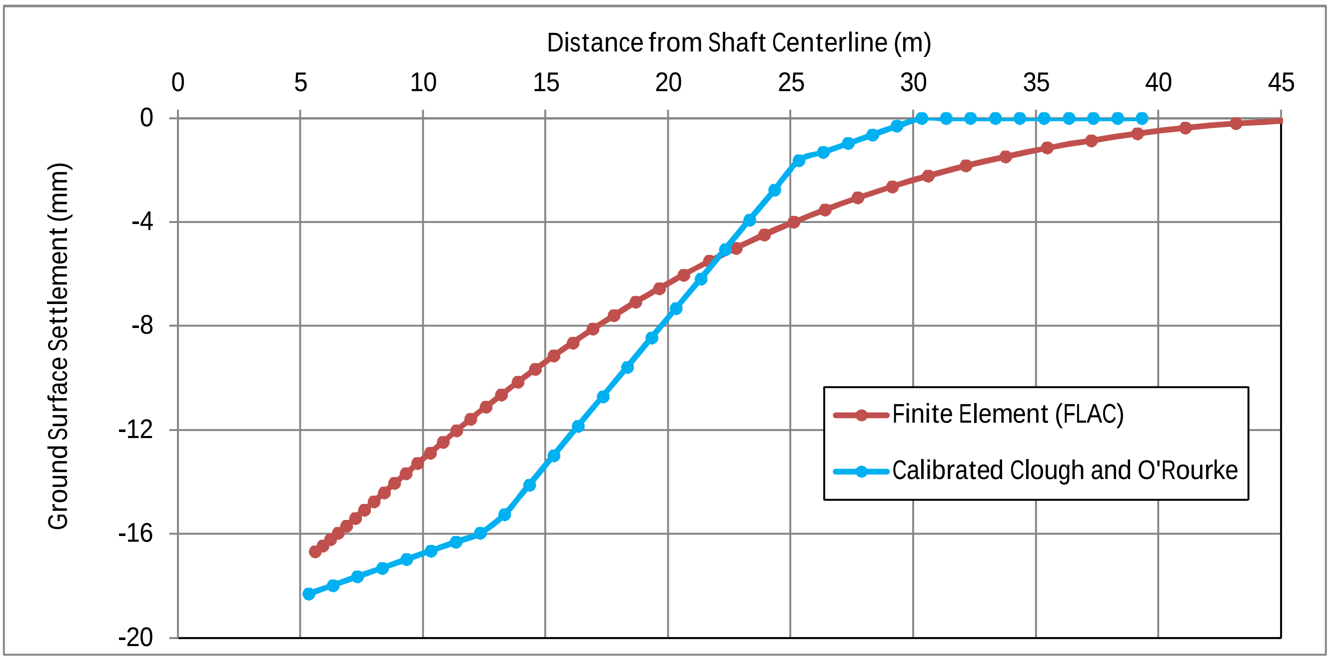

Figure 4-3: Lyon Avenue Predicted Mechanical Settlement Profile

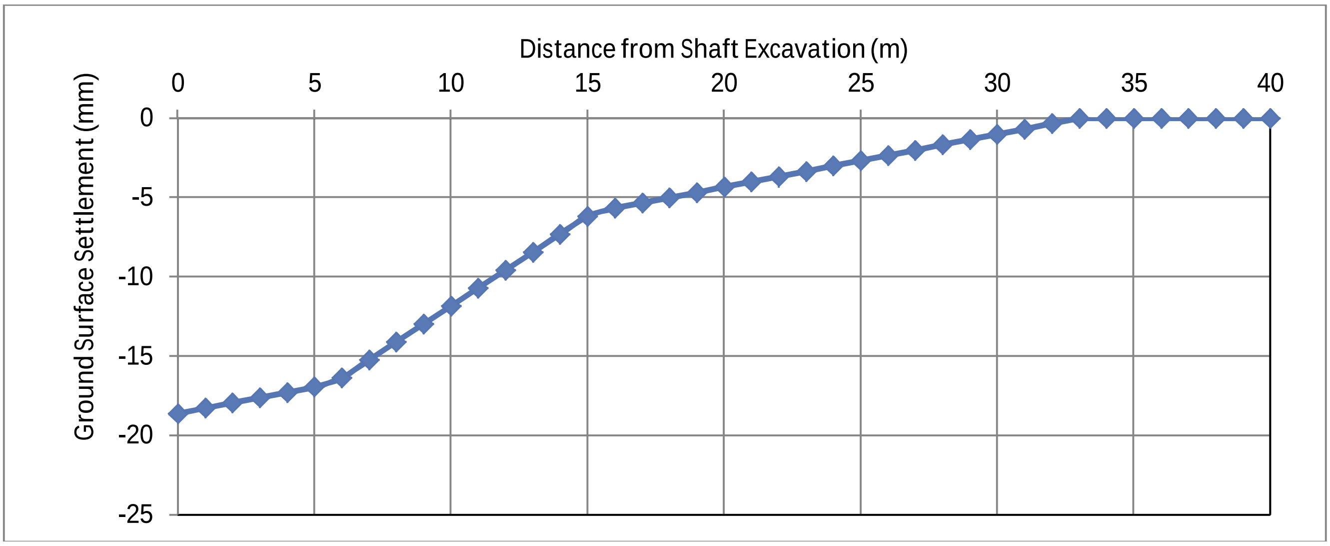

Figure 4-4: Mt Albert Shaft Predicted Mechanical Settlement Profile

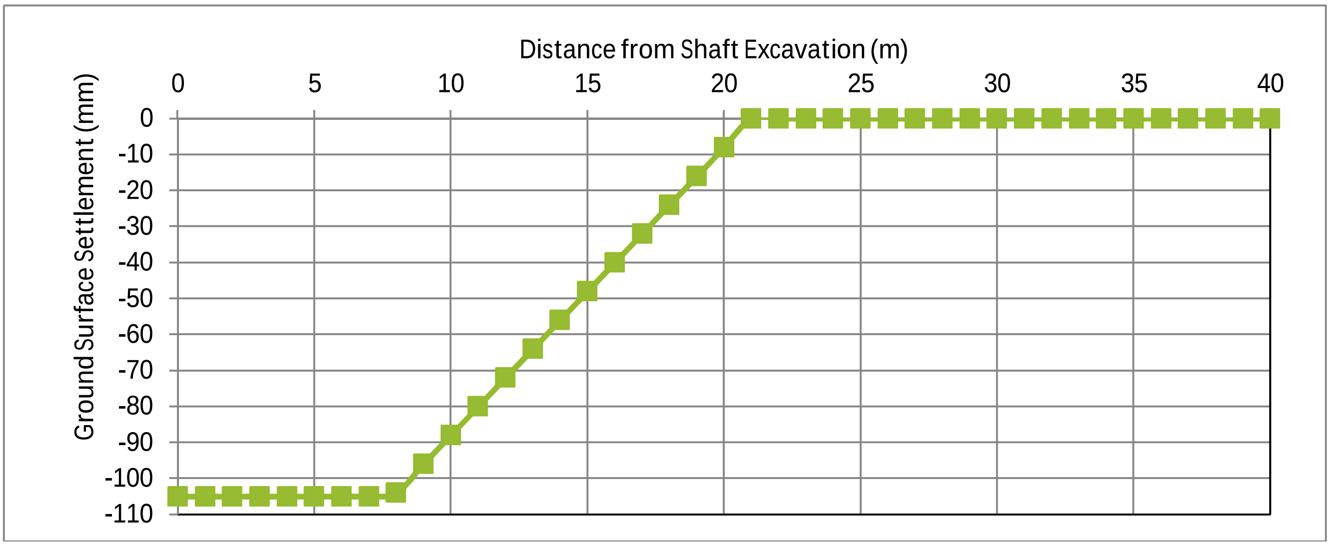

Figure 4-5: Haverstock Shaft Predicted Mechanical Settlement Profile

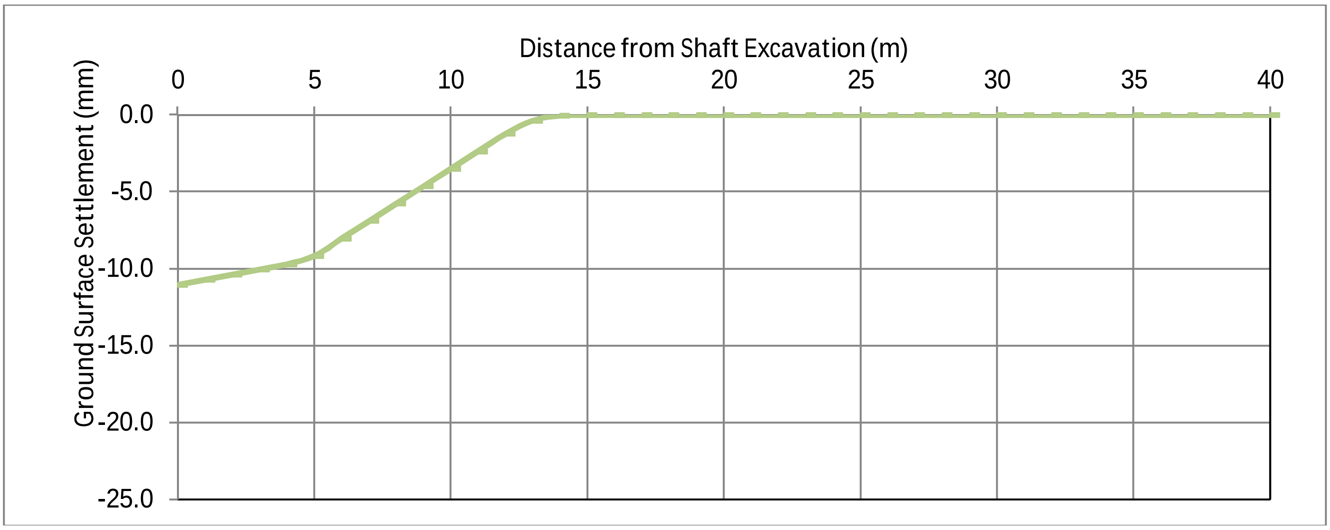

Figure 4-6: Walmsley Park Shaft Predicted Mechanical Settlement Profile

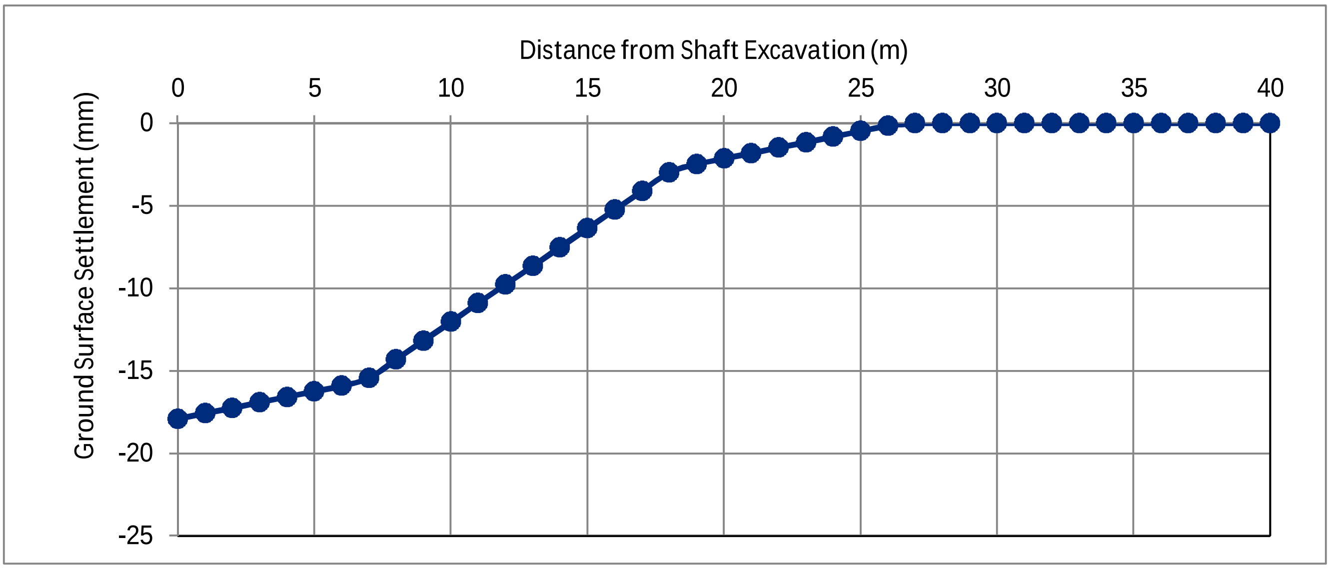

Figure 4-7: May Road Shaft Predicted Mechanical Settlement Profile

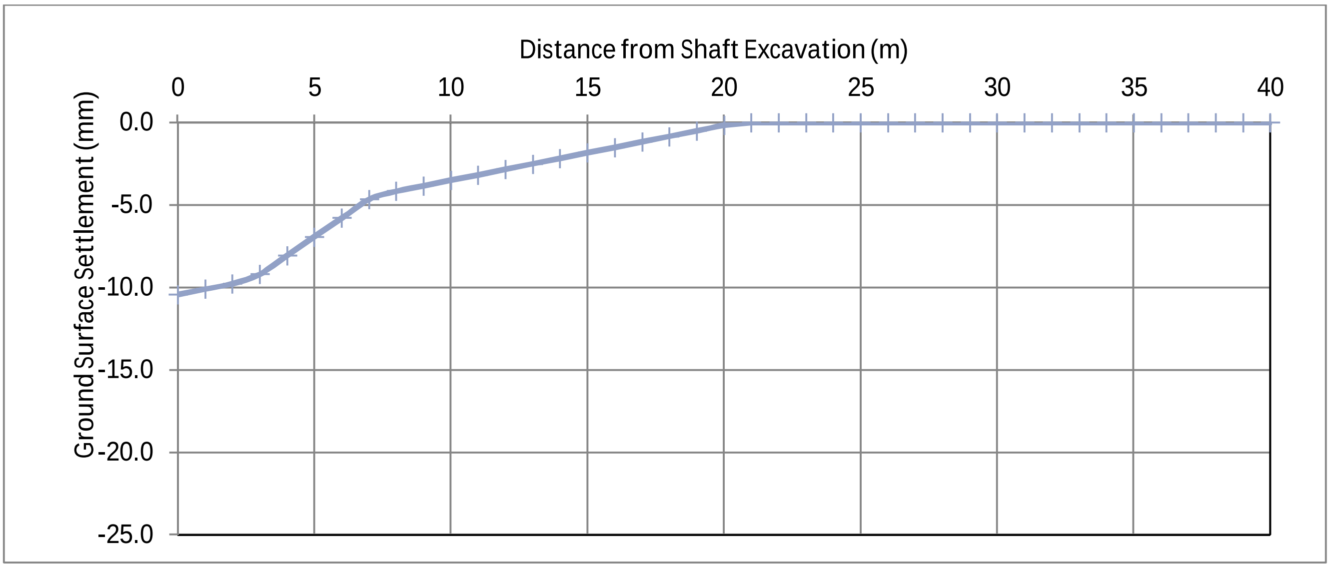

Figure 4-8: Keith Hay Park Shaft Predicted Mechanical Settlement Profile

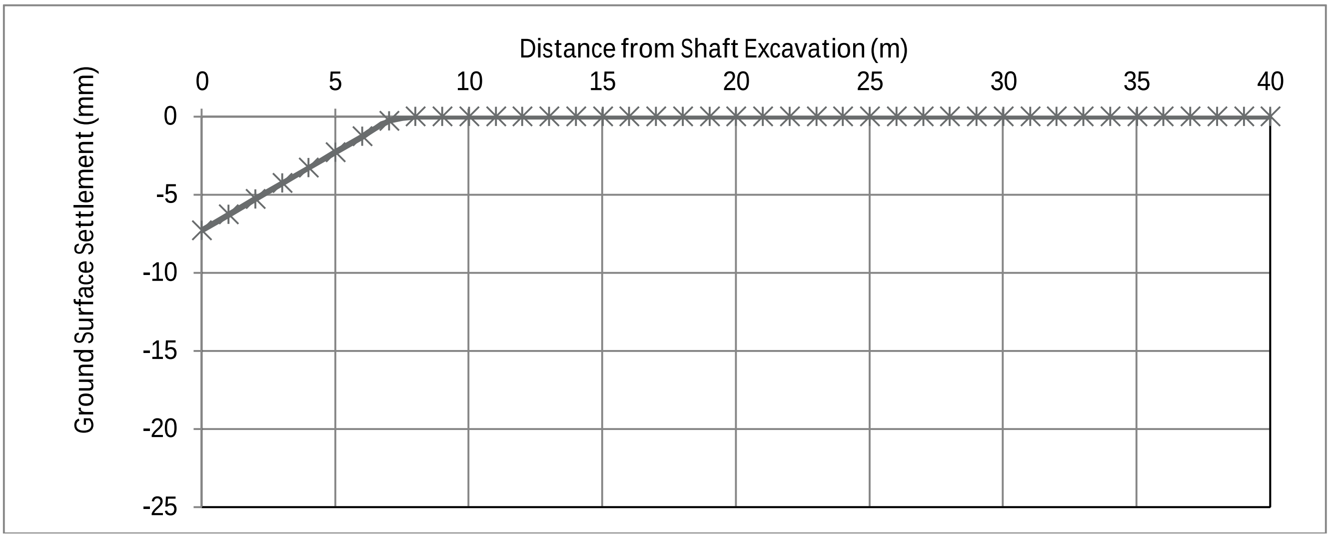

Figure 4-9: PS23 Shaft Predicted Mechanical Settlement Profile

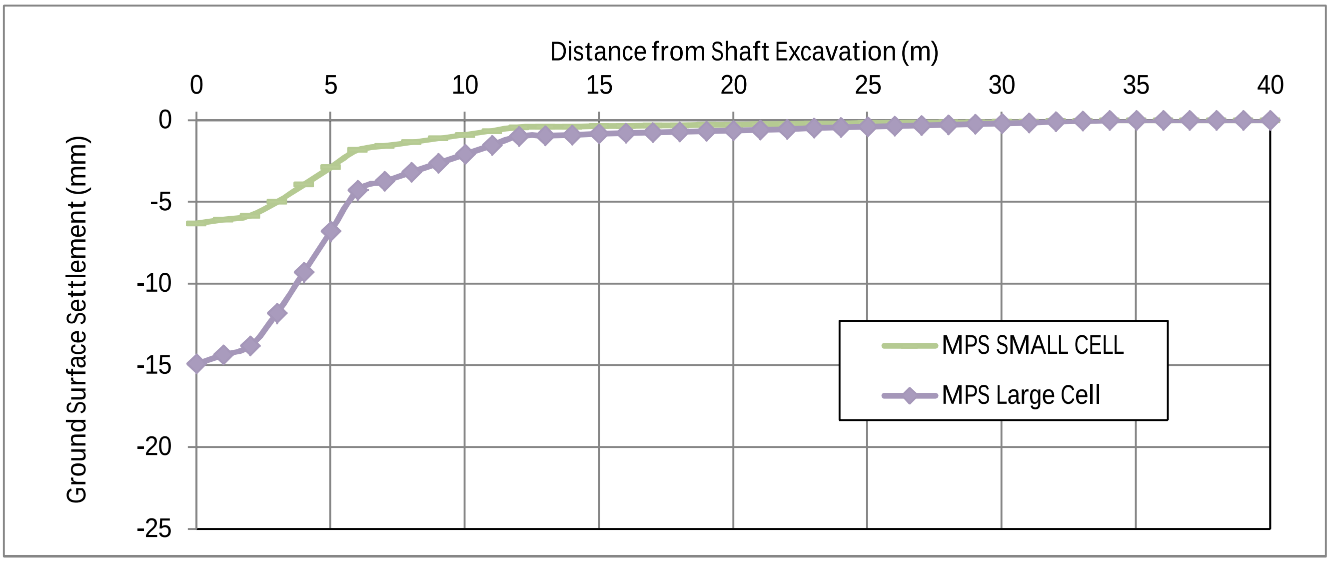

Figure 4-10: Mangere Pumping Station Shaft Predicted Mechanical Settlement Profile

4.3.3 Summary of Results

The results from the shaft mechanical settlement are summarised below in Table 4-6: and Table 4-7. Green cells indicate the calculated settlement is within the settlement limits defined in Section 3.1, and pink cells indicate this is exceeded.

The predicted settlement is larger at the Haverstock shaft site due to the flexible shaft wall (sheet pile) assumption.

Table 4-6: Shaft Mechanical Settlement Estimates – Main Line Tunnel

Code	Shaft Name	Settlement	
		Maximum Settlement (mm)	Maximum Differential Settlement
DSCIN009	Western Springs	-9	1:650
DSCIN008	Mt Albert Memorial Reserve	-19	1:900
DSCIN007	Lyon Avenue	-18	1:700
DSCIN006	Haverstock Road	-105	1:400
DSCIN005	Walmsley Ave	-11	1:900
DSCIN004	May Road – Drop Shaft	-18	1:900
DSCIN004	May Road – Construction shaft	-16	1:900
DSCIN004	May Road – Combined	-19	1:900
DSCIN003	Keith Hay Park	-10	1:1000
DSCIN002	PS23	-7	1:1250
DPCIN	MPS – Small Cell	-7	1:950
DPCIN	MPS – Large Cell	-15	1:400

Table 4-7: Shaft Mechanical Settlement Estimates – Link Sewer Tunnels

Code	Shaft Name	Maximum Settlement (mm)
DSLSB002	Rawalpindi Reserve	-5
DSLSB001	Norgrove Avenue	-10
DSLSC005	PS25	-7
DSLSC004	Miranda Reserve	-6
DSLSC003	Whitney Street	-4
DSLSC002	Dundale Avenue	-4
DSLSC001	Haycock Avenue	-5

4.4 Consolidation Settlement Assessment

Assessment of consolidation settlements was done in two stages:

1. Predict potential groundwater drawdown due to shaft construction (Report Reference: PWCIN-DEL-REP-GT-J-100236), followed by

2. Assessment of ground settlement due to predicted groundwater drawdown associated with shaft construction (Report Reference: PWCIN-DEL-REP-GT-J-100230).

The details of the respective analyses and assessments of these referenced reports are summarised below.

4.4.1 Methodology for Groundwater Drawdown Prediction

Seven 3D groundwater models were developed to cover the various indicative shaft hydrogeologic conditions and configurations. In addition, a 2D model was developed for a 'worst case' main tunnel condition. The shaft models were carried out using the MODFLOW software, and the tunnel was modelled by the SEEP/w software. These models were analysed for transient conditions while a conservative approach was taken from the point of view of construction methodology. Recharge from rainfall and watercourses was considered where appropriate.

Excavation support systems in soils were modelled to have a low conductivity, which will impede groundwater flow directly into the shafts. The excavated shafts were modelled as 'open' for a construction period of up to 3 years, before the permanent impermeable linings are installed and groundwater conditions return to pre-existing levels.

4.4.2 Methodology for Groundwater Drawdown Settlement Assessment

Dewatering settlement of the soils surrounding each shaft was analysed individually via analytical 3D modelling methods. The 3D soil models were constructed using Settle3D (v3.018) software by Rocscience based on geological sections comprising borehole data, hand augers, test pits and cone penetrometer test results. The output of Settle3D was used to obtain contours of the dewatering consolidation settlements via Surfer (v10.7.972) software. Soil settlement was calculated using the following method:

$$d = m_v \, Ds' \, H$$

Where d is settlement, m_v is the one-dimensional volume of compressibility (m^2/kN) and $\Delta\sigma'$ is change in vertical effective stress at mid height of the compressible layer depth H. The value of m_v was derived from CPT plots and therefore reflects the in-situ soil stiffness.

4.4.3 Summary of Results

Groundwater drawdown modelling results indicate maximum groundwater drawdown will tend to occur as the shaft invert levels are reached and before the shafts are permanently lined; for the base of the alluvium/basalt layers, at 50m from the shaft edges, groundwater drawdown will range between 2.6 and 6.3m, depending on the shaft. The drawdown cone at each of the shafts will tend to dissipate completely within 400m of the shafts. After a permanent lining system is installed in these shafts, groundwater levels near the shafts will start to rise. The shafts with the highest drawdown values are in the northern part of the alignment, which are Western Springs, Lyon Avenue, Haverstock Road, and May Road. The shaft sites with the most extensive drawdown cones are Western Springs, May Road, Walmsley Park, and Haverstock Road.

An analysis of the 'worst case' tunnel section near Keith Hay Park shows that, if the open tunnel wall remains unlined for a period of 3 days (e.g. 45m length), the maximum groundwater inflow rate into the tunnel will be about 1.63 L/s, which results in very small drawdown, and therefore negligible risk of consolidation settlements.

The maximum estimated consolidation settlements associated with groundwater drawdown are presented in Table 4-8 below. Contours of consolidation settlement around shafts are presented in report PWCIN-DEL-REP-GT-J-100230.

Table 4-8: Shaft Dewatering Settlement Estimates

Name of Shaft	Maximum Groundwater Drawdown (m)	Maximum Drawdown Consolidation Settlement (mm)
Western Springs	18	70
Mt Albert Memorial Reserve	15	30
Lyon Avenue	8	100
Haverstock Road	9	60
Walmsley Park	7	70
May Road	9	50
Keith Hay Park	4	30
PS23	9	-
Rawalpindi Reserve	10	10
Norgrove Avenue	8	45
Haycock Avenue	30	30
Dundale Avenue	17	15
Whitney Street	19	15
Miranda Reserve	10	15
PS25	2	15

4.5 Combined Settlement

Tunnel mechanical, shaft mechanical and consolidation settlements are theoretically cumulative and therefore can be combined arithmetically. Tunnel mechanical settlements were very small and did not occur near any shaft related settlement so have not been plotted. The shaft mechanical and consolidation settlements have been plotted as combined settlement contours on project plan drawings provided in Appendix A. The areas that exceed 50 mm of settlement are shown, as well as the 1:1000, 1:500 and 1:200 contour lines.

5. Effects Assessment

Each structure and identified utility within the zone of predicted settlement was evaluated for potential damage. Differential settlement is a key factor influencing predicted damage.

5.1 Potential for Damage to Buildings

The procedure to assess building damage is summarized as a two-step process:

1. Identify all buildings in settlement zones exceeding 1:1000 differential or 50mm total settlement per consent conditions.

2. Perform a potential damage assessment on these buildings per the Burland Method described in Section 3.3.1.

The results from the first two steps are summarized in Table 5-1.

Table 5-1: Building Settlement Screening

No. Bldgs.	Address	Shaft	50mm+	1:1000 – 1:500	1:500 – 1:200	1:200 +
1	27 Morning Star Plc	Lyon Ave	-	✓	-	-
2	28 Morning Star Plc	Lyon Ave	-	✓	-	-
3	96A Haverstock Rd	Haverstock	-	✓	-	-
4	3 O'Donnell Ave	Walmsley	-	✓	-	-
5	9 O'Donnell Ave	Walmsley	-	✓	✓	-
6	11 O'Donnell Ave	Walmsley	-	✓	-	-
7	13 O'Donnell Ave	Walmsley	-	✓	-	-
8	3-5 Roma Rd	May Rd	✓	-	-	-
9	38 Roma Rd	May Rd	✓	-	-	-
10	44-52 Roma Rd	May Rd	✓	-	-	-
11	54 Roma Rd	May Rd	✓	-	-	-
12	101 May Rd	May Rd	✓	-	-	-
13	47 Marion Ave	May Rd	✓	-	-	-
14	49 Marion Ave	May Rd	✓	-	-	-
15	51 Marion Ave	May Rd	✓	-	-	-
16	53A Marion Ave	May Rd	✓	-	-	-
17	55A Marion Ave	May Rd	✓	-	-	-
18	22 Gregory Place	Keith Hay	-	✓	-	-
19	66C Dundale Ave	Dundale	-	✓	-	-
20	66D Dundale Ave	Dundale	-	✓	-	-
21	16 Norgrove Ave	Norgrove	-	✓	-	-

It should be noted that the Foodstuffs complex present at 60 Roma Street near the May Road shafts is not predicted to experience any settlement exceeding 1:1000 or 50 mm. Similarly, the Cameron pool and Leisure Centre near the Keith Hay Park shaft has maximum total and differential settlements of 25mm and 1:2600 respectively, and is not predicted to experience any damage.

27 Verona Ave was described in the report *Assessment of Ground Settlement at Link Sewer B & C due to Shaft Construction – Ref: PWCIN-DEL-REP-GT-J-100239* report as exceeding the 1:1000 criterion. However, subsequent analysis of combined settlements indicates the structure impacted is ancillary and is better than the 1:1000 consent threshold for differential settlement. Therefore 27 Verona Ave is not considered at risk to damage greater than negligible (see below).

The results in Table 5-1 above show that 21 buildings need further analysis by the Burland Method, which determines the anticipated damage classification relative to the magnitude of tensile strain. A description of the damage associated with the degrees of severity is provided in Table 5-2: below.

Table 5-2: Building Damage Criteria

Category of Damage	Normal Degree of Severity	Description of Typical Damage [Building Damage Classification after Burland (1995), and Mair et al. (1996)]	General Category (after Burland - 1995)
0	Negligible	Hairline cracks.	Aesthetic Damage
1	Very Slight	Fine cracks easily treated during normal redecoration. Perhaps isolated slight fracture in building. Cracks in exterior visible upon close inspection. Typical crack widths are up to 1mm.	
2	Slight	Cracks easily filled. Redecoration probably required. Several slight fractures inside building. Exterior cracks visible; some repainting may be required for weather tightness. Doors and windows may stick slightly. Typical crack widths are up to 5mm.	
3	Moderate	Cracks may require cutting out and patching. Recurrent cracks can be masked by suitable linings. Brick pointing and possible replacement of a small amount of exterior brickwork may be required. Doors and windows sticking. Utility services may be interrupted. Weather tightness often impaired. Typical crack widths are 5 to 15mm or several greater than 3mm.	Serviceability Damage
4	Severe	Extensive repair involving removal and replacement of walls required especially over doors and windows. Window and door frames distorted. Floor slopes noticeably. Walls lean or bulge noticeably. Some loss of bearing in beams. Utility services disrupted. Typical crack widths are 15 to 25mm but also dependent on the number of cracks.	
5	Very Severe	Major repair required involving partial or complete reconstruction. Beams lose bearing; walls lean badly and require shoring. Windows broken by distortion. Danger of instability. Typical crack widths are greater than 25mm but also dependent on the number of cracks.	Stability Damage

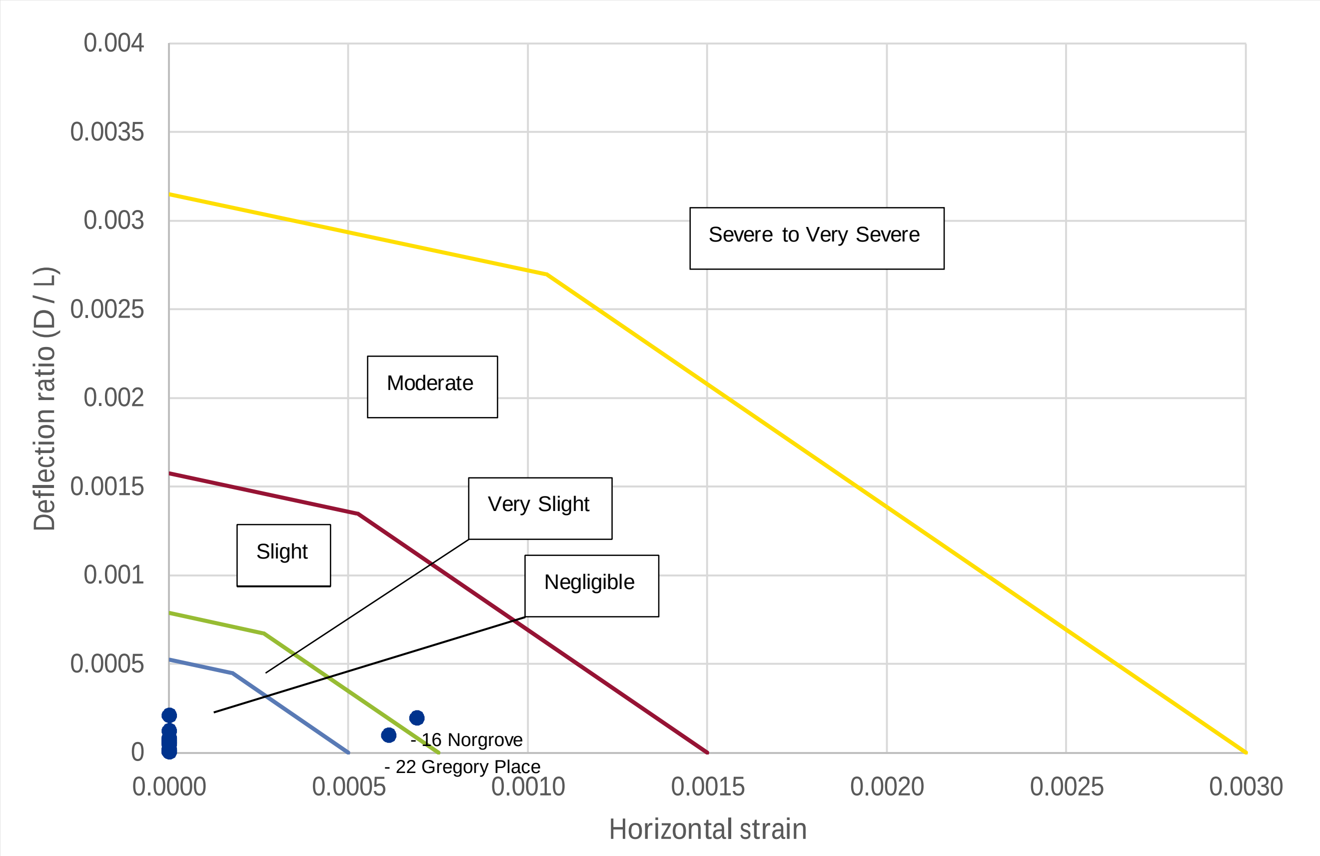

Figure 5-1: Predicted Damage for Buildings that Exceed Consent Settlement Limits (per Burland Method)

As shown in Figure 5-1, of the 21 buildings that exceed the consent settlement criteria, two buildings may experience negligible to slight damage, specifically:

- **22 Gregory Pl** near Keith Hay shaft: this building is on Watercare land and will be demolished for construction of the shaft.
- **16 Norgrove Ave** near the Norgrove Ave shaft on Link Sewer B: this residential home is in the Slight damage category.

5.2 Potential for Damage to Utilities

Three types of settlement impacts typically affect buried pipeline utilities, as summarized in O'Rourke and Trautmann (1982):

- Tensile pull-apart at joints, caused by relative tensile axial movements along the pipeline.
- Opening of joints between pipe segments, due to relative rotation between two pipe segments.
- Straining of pipe caused by flexural deformations, and lateral deformations that lead to rupture or intolerable deformation.

The first two impacts focus on failures occurring at well-defined joints, and would be more likely to occur in fairly rigid, jointed pipe such as reinforced concrete pipe. The third type of impact is caused by differential settlements and lateral ground movements, and is most likely to occur in flexible pipelines with well-designed rigid joints that can take significant rotation, such as welded steel pipelines. See schematics of each of these three modes of failure.

The maximum allowable value for each of the above deformation modes is given in Table 5-3: for each pipe material, based on the recommendations of O'Rourke and Trautmann (1982).

Table 5-3: Utility Deformation Criteria

Utility Type	Utility Dia. (mm)	Allowable Joint Displacement (mm)	Allowable Joint Rotation (□)	Allowable Tensile Strain (μ mm/mm)
WSP	-	NA	NA	600
Cast-in-situ Concrete	-	NA	NA	300
PVC & HDPE	-	NA	NA	2000
RCP	-	10.2	0.250	300
Ductile Iron Pipe	-	10.2	0.250	600
Cast Iron Pipe	150	2.1	1.140	400
	200	2.1	0.930	400
	300	2.1	0.670	400
	400	2.0	0.490	400
	500	1.8	0.370	400
	600	1.6	0.270	400
	750	1.6	0.220	400

Notes:

RCP = Reinforced Concrete Pipe

WSP= Welded Steel Pipe

The tunnel alignment was screened for any sensitive services, such as the Marsden to Wiri gas line, fibre optic lines and water wholesale mains. Damage to these utilities has a much higher consequence, so they are screened separately. No gas lines, fibre optic lines or water wholesale mains were identified within the zones of settlement.

All services that intersected a zone of settlement exceeding 1:1000 differential or 50mm total settlement were then tabulated. As can be expected, several large-diameter wastewater or combined stormwater and wastewater mains were identified within settlement zones because the CI drop shafts will be intercepting many of these wastewater flows. These pipes are not under pressure have been analysed in the same manner as all the other services.

Using the utility deformation criteria in Table 5-3:, maximum slopes for utilities at risk around shafts were back-calculated based on typical utility lengths. Based on this analysis a 1:500 maximum slope was identified irrespective of utility type and diameter to screen these utilities for analysis. Five utilities were found to exceed the values detailed in Table 5-4. These utilities are listed in Table 5-5:

Table 5-4: Utility Deformation Maximum Slopes

Utility Type	Utility Dia. (mm)	Maximum Slope
WSP	-	1:41
Cast-in-situ Concrete	-	1:58
PVC & HDPE	-	1:22
RCP	-	1:
Ductile Iron Pipe	-	1:229
Vitrified Clay Pipe		1:229
Cast Iron Pipe	150	1:50
	200	1:62
	300	1:86
	400	1:117
	500	1:155
	600	1:212
	750	1:260
Notes: WSP= Welded Steel Pipe RCP = Reinforced Concrete Pipe		

Table 5-5: Summary of Pipe-specific Analyses for Damage Settlement

Pipe No.	Shaft	Shaft Code	Diameter	Material (Auckland council GIS)	Analysis Material	Analysed Diameter	Slope Zone (greenfield)
SS1	Lyon Ave	DSCIN007	381	Out of service	Reinforced Concrete	400	1:500 – 1:200
SS2	Lyon Ave	DSCIN007	650	Concrete lined steel	Cast Iron	750	1:500 – 1:200
SS7	Lyon Ave	DSCIN007	150	Earthenware	Vitrified Clay	NA	1:500 – 1:200
SW9	Haverstock	DSCIN006	450	Unknown	Worst Case	500	1:200 +
SS2	Walmsley	DSCIN005	525	Reinforced Concrete	Reinforced Concrete	NA	1:500 – 1:200

The utilities identified in Table 5-5 were then subjected to pipe-specific analyses (Table 5-6). Following this, only one pipeline was predicted to experience damage due to settlement. This is a 450mm diameter stormwater pipe experiencing a predicted maximum settlement grade of 1:230.

Table 5-6: Limit Summary for Haverstock Pipe SW9

	Joint Displacement	Allowable Joint Rotation (°)	Tensile Strain	Bending Strain
Limit	1:29	1:229	1:58	300μ
1:230 Slope	✓	Fails	✓	326μ (fails)
Note: L* = Length of individual pipe segments.				

The main tunnel is not anticipated to damage the Western Interceptor at Mangere Lagoon or Manukau Harbour Crossing. Under the Manukau Siphon (harbour crossing) the tunnel is situated in weathered rock (mixed-face conditions) with a basalt cap above characterised up to 4.5m thick, predicted settlements and angular distortions at these location is 12mm and 1:750, and 10mm and 1:1200 respectively. The predicted ground movements result in negligible predicted damage levels.

5.3 Potential for Damage to Infrastructure

Major infrastructure above the CI alignment includes SH16 piled retaining wall and overbridge at the Western Springs on/off ramp; SH20 near Keith Hay Park; Transpower pylons in Manukau Harbour; and the Western Rail Line near Mt Albert War Memorial Reserve. These structures are not within the settlement zone of influence for any shafts. The tunnel under these structures is either deep, situated in bedrock, or offset horizontally. Settlement of these major structures is therefore anticipated to be negligible.

6. Monitoring and Mitigation

6.1 Preconstruction Monitoring

Surface settlement monitoring is a requirement of the RMA Consent Conditions prior to, during and after construction. Pre-construction monitoring has been undertaken by Watercare to establish baseline ground surface movements associated with seasonal variations in soil moisture content and associated shrink/swell behaviour unrelated to construction of the project. The monitoring has been undertaken using a deep level monitoring pin as a datum and several shallow monitoring points near each shaft site over a minimum period of 12 months.

6.2 Construction Monitoring

6.2.1 Surface Settlement Monitoring

Complementary to the preconstruction monitoring, the contractor is required to develop and implement a surface settlement monitoring programme in accordance with the Consent Conditions.

6.2.2 Shaft Monitoring

Shaft instrumentation is anticipated to consist of shaft convergence and/or ground movement measurements in soils, e.g. inclinometer monitoring in the deeper soil profiles around shafts.

The Mangere Pumping Station shaft has required inclinometer monitoring of the ground surrounding the D-walls, which will be shown on the Drawings. This monitoring is required to verify the design and stability of the D-walls.

Minimum shaft monitoring requirements will be provided in the geotechnical instrumentation and monitoring specification.

6.2.3 Utilities Monitoring

Utilities should be monitored for settlement in areas at higher risk for settlement and utility damage (as described in Section 5.2 above). Minimum utility monitoring requirements will be provided in the geotechnical instrumentation and monitoring specification.

6.2.4 Tunnel Convergence Monitoring

In-tunnel instrumentation will consist of instruments installed to monitor convergence of the precast segmental lining, as required to verify the design and stability of the lining. A typical instrumented section of tunnel will consist of an array of convergence survey reference points, which will be shown on the Drawings. Monitoring requirements in the tunnel will be provided in the geotechnical instrumentation and monitoring specification.

6.3 Proactive Mitigations

6.3.1 Pressurisation of the TBM

The requirement for an EPB TBM with annular grouting of the segmental lining through the TBM tail shield will minimise mechanical settlements related to tunnelling. Operation of the TBM in closed-mode or partial-mode will prevent dewatering around the tunnel, thus minimising or eliminating risk of consolidation settlements due to dewatering.

6.3.2 Watertight Shafts

Where watertight or very low permeability shaft support systems are specified (e.g. MPS D-walls and shaft support systems in soil such as secant piles, sheet piles, steel casings or caissons), dewatering of soil materials will be minimised. Therefore, consolidation settlements resulting from dewatering will be reduced significantly.

6.3.3 Groundwater Recharge

A mitigation measure to counteract groundwater drawdown is to artificially recharge the groundwater table through injection wells. While this can be effective at reducing groundwater drawdown and related consolidation settlements, it requires accessibility for installation of injection wells, and must be done carefully so as not to create unwanted groundwater table rise in areas where this can be detrimental, e.g., flooding of basements or low elevation areas. In addition, the effectiveness of injection wells can be limited where vertical and horizontal soil/rock permeability varies greatly or there is poor hydraulic connectivity. Also, since the shafts are essentially large wells, injection wells can exacerbate shaft inflows and create construction problems.

6.3.4 Shaft Support Measures While Excavating in Soils

The numerical modelling described in Section 4.3.2 assumed that excavation in soils will be done ‘in the wet’ (shaft flooded with underwater grab) until the excavation reaches the ECBF and thereafter preformed using dry excavation techniques. This process is followed to minimise invert heave and lateral shaft wall movements in weak soils, which in turn will minimise shaft mechanical settlements. This mitigation measure is only effective where flexible shaft temporary support measures are used to support weak soils.

6.3.5 Building Protection Measures

Possible methods for building protection include underpinning, permeation grouting, compaction grouting, and compensation grouting. Building and utility protection methods will be the responsibility of the contractor based on the selected construction means and methods.

7. Recommendations

1. The settlement assessment was conducted based on available geotechnical data and information available via Auckland Council's geographical information system. More detailed information about the existing building and utility conditions should be collected during pre-construction surveys where necessary.

2. The settlement assessment herein assumes shafts will be excavated 'in the wet' (shaft flooded with underwater grab) in soils subject to invert heave or excessive lateral deformations in the shaft wall. This applies to shafts with flexible wall support systems. should the contractor elect to excavate shafts for this situation in the dry, mechanical settlements will likely be more than 3 times the magnitudes predicted herein. This should be addressed in the GBR.

3. 16 Norgrove Ave will likely require a damage mitigation approach by the contractor (proactive or reactive). This should be communicated in the temporary shaft support specification or GBR.

4. Haverstock stormwater pipe SW9 (450mm ID) will likely require a damage mitigation approach by the contractor (proactive or reactive). This should be communicated in the temporary shaft support specification or GBR. As the pipe is situated in an open space, a simple mitigation approach would be to execute before and after pipe condition surveys, and then replace any effected areas of pipe that exceed acceptable limits.

8. References

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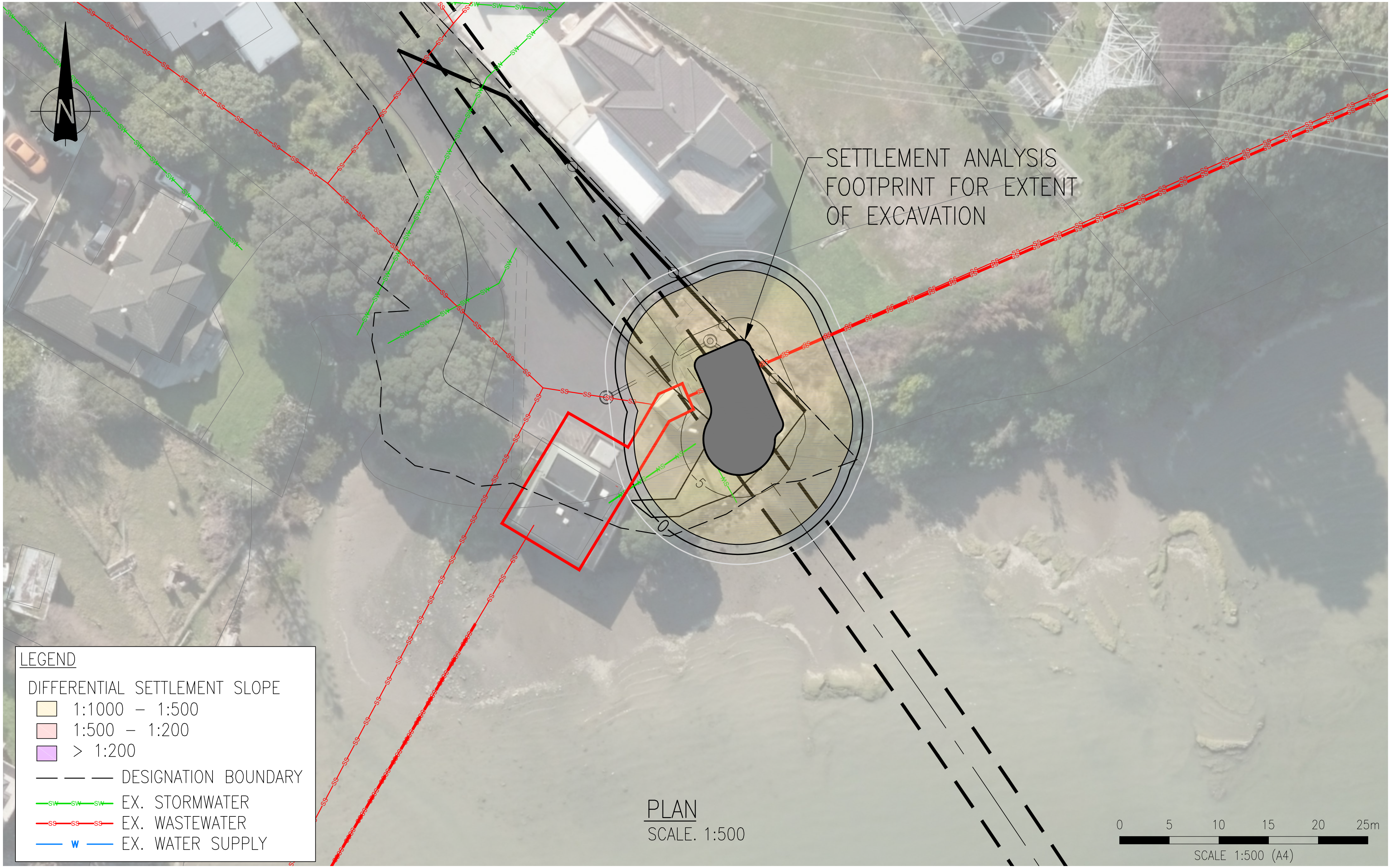
Appendix A. Drawings

Main Line Shaft Contour Plots



JACOBS AECOM McMILLEN JACOBS ASSOCIATES

FIGURE 1.0
SETTLEMENT – DSCIN002 PS23

Watercare

Plot Date: Oct 10, 2016 - 2:13pm R:\Deliverables\3D GeoCIVIL BIM\05 Sketches\Settlement\ Settlement Figure.dwg

Plot Date: Oct 10, 2016 - 2:14pm R:\Deliverables\3D Geo\CIVIL BIM\05 Sketches\Settlement\Settlement Figure.dwg

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FIGURE 2.0

SETTLEMENT – DSCIN003 KEITH HAY PARK

Watercare

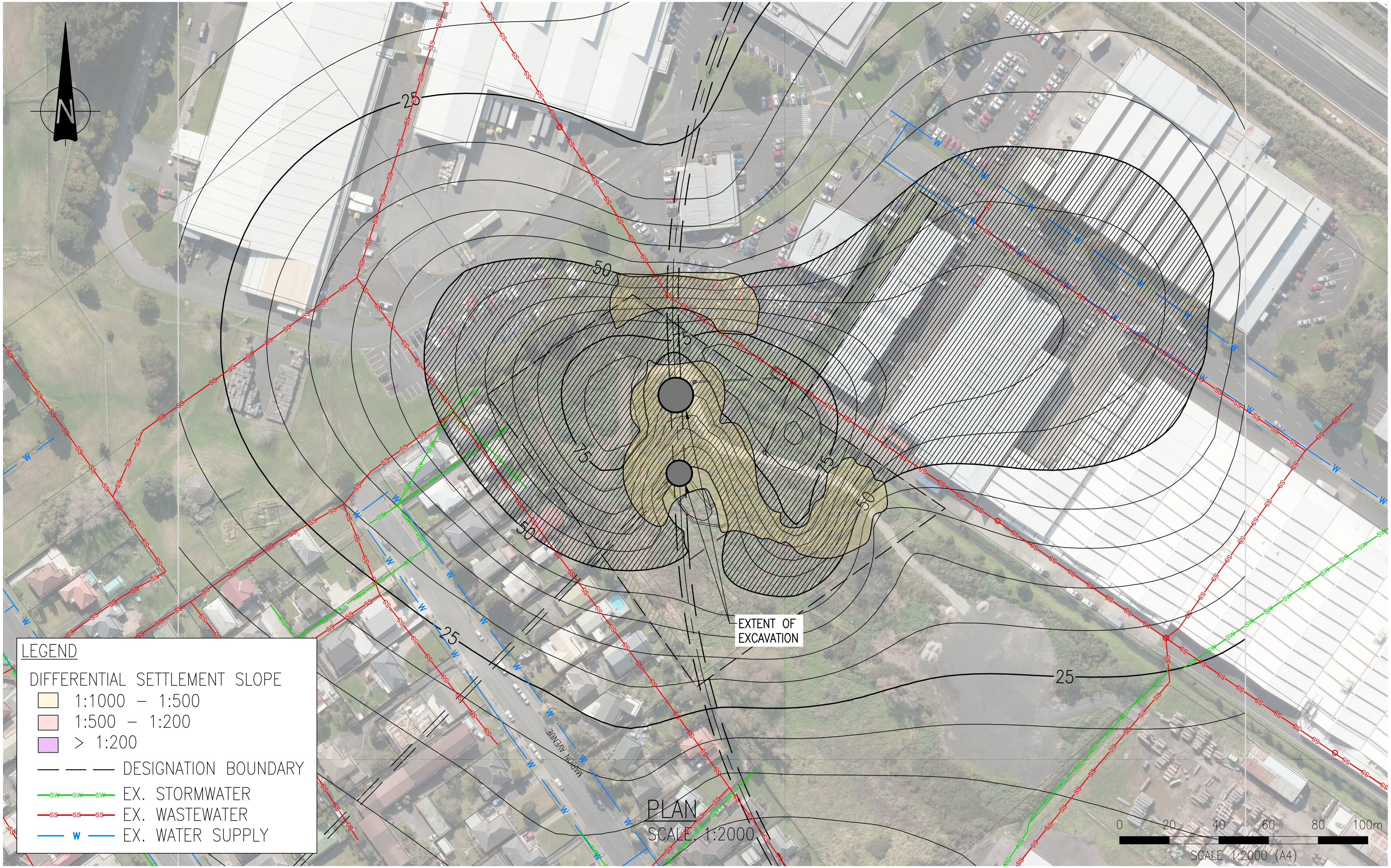

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FIGURE 3.0
SETTLEMENT – DSCIN004 MAY ROAD

Watercare

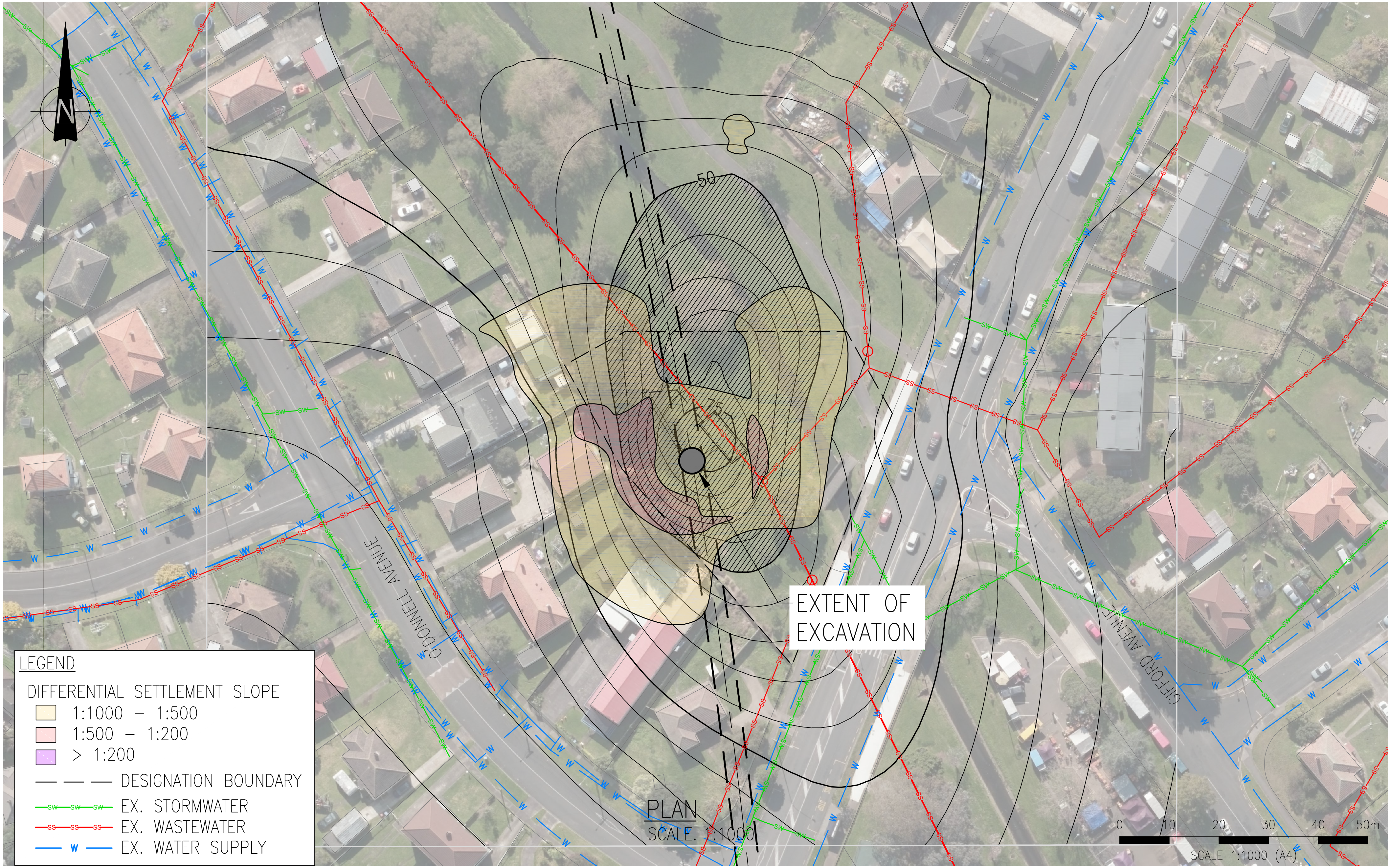

FIGURE 4.0

SETTLEMENT – DSCIN005 WALMSLEY PARK

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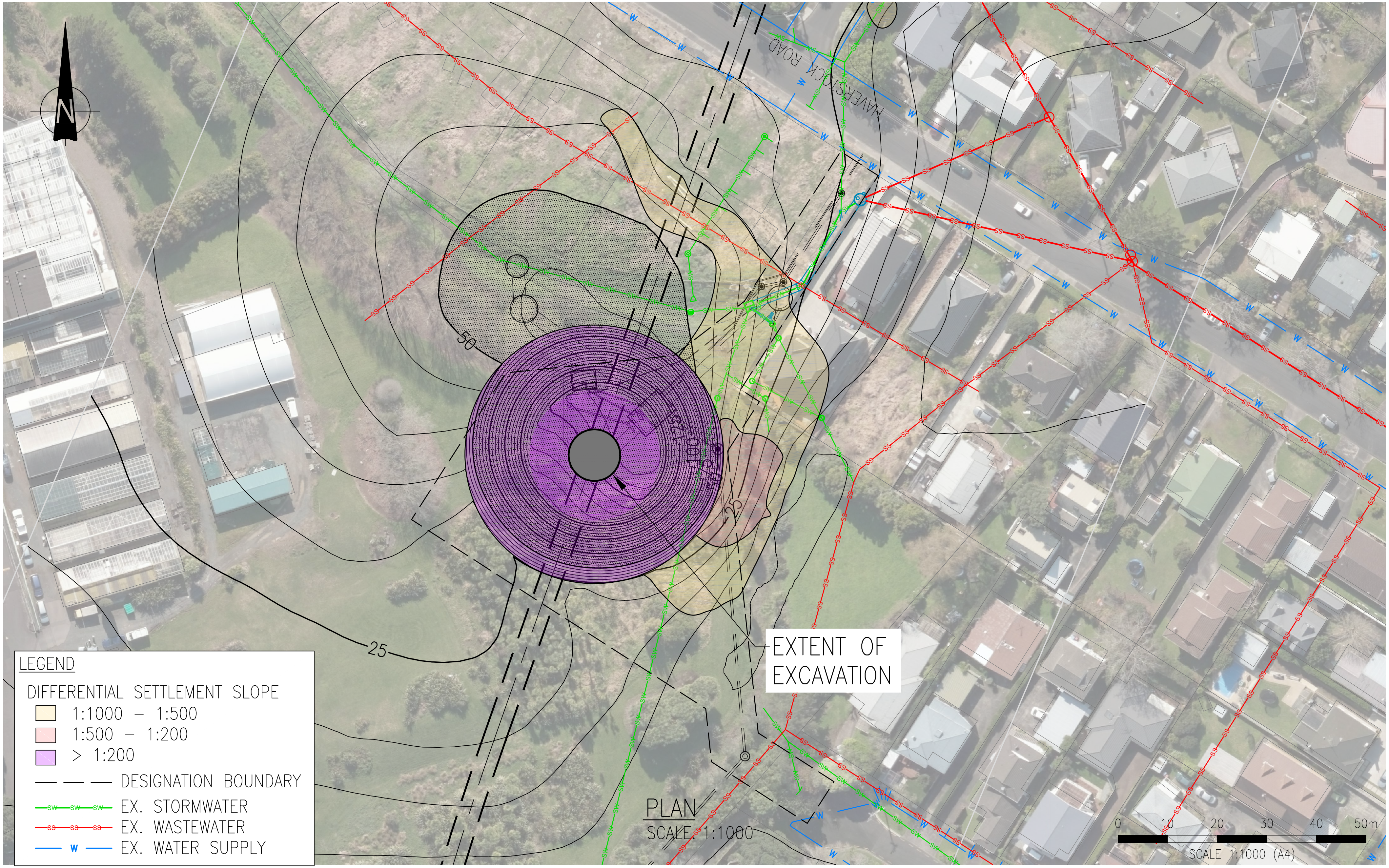

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FIGURE 5.0

SETTLEMENT – DSCIN006 HAVERSTOCK ROAD

Watercare

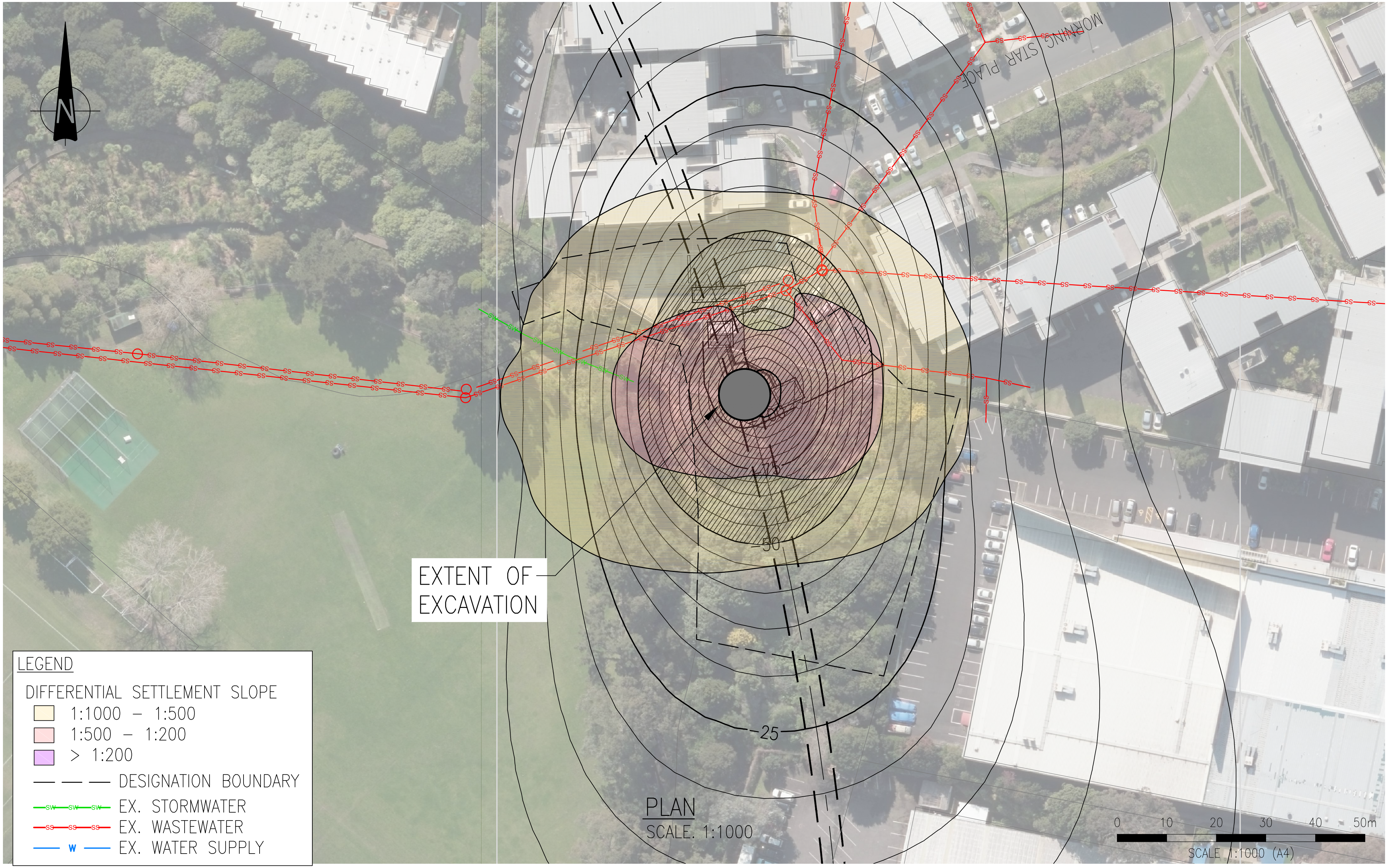

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FIGURE 6.0
SETTLEMENT – DSCIN007 LYON AVE

Watercare

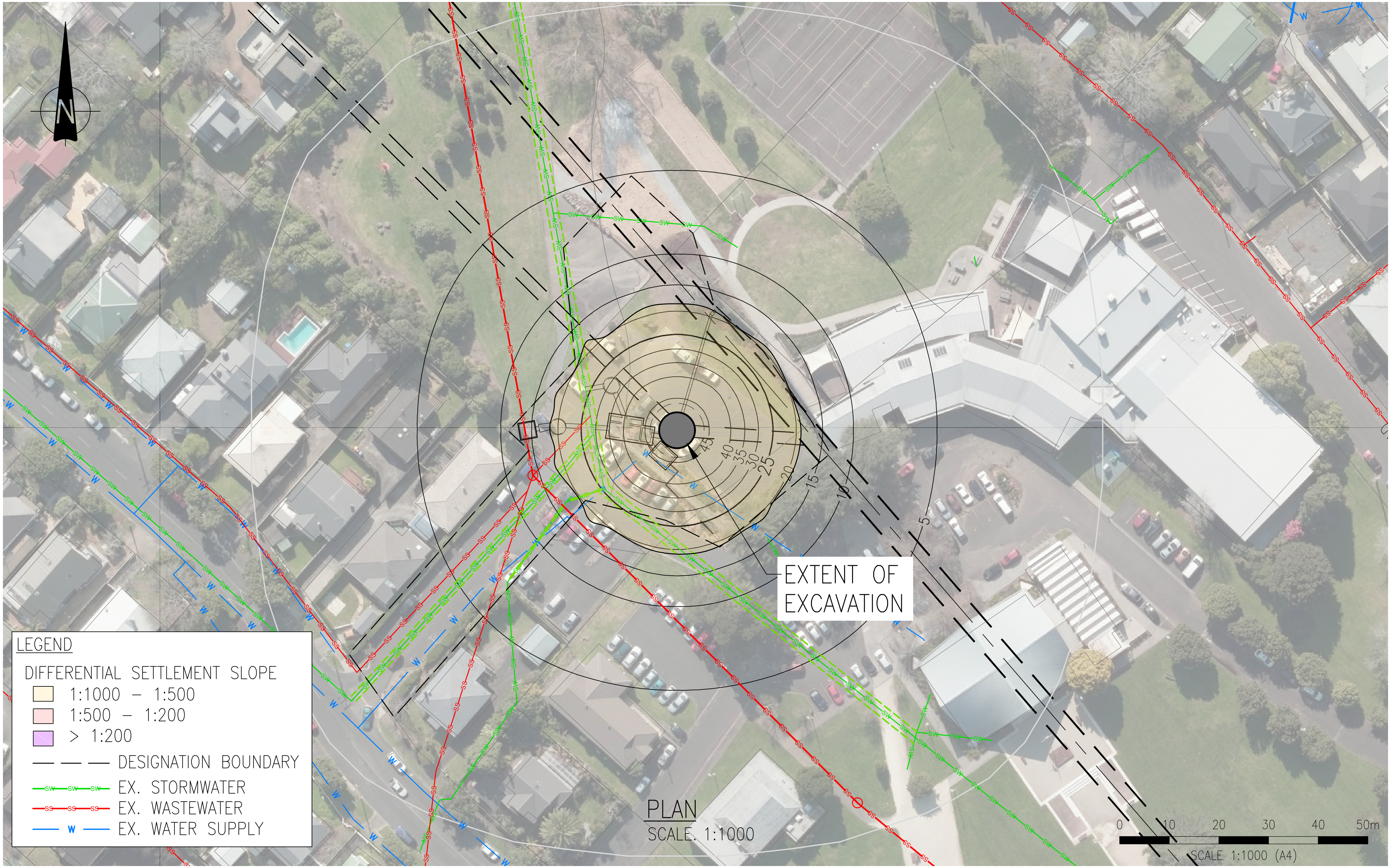

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FIGURE 7.0
SETTLEMENT – DSCIN008 MT ALBERT

Watercare

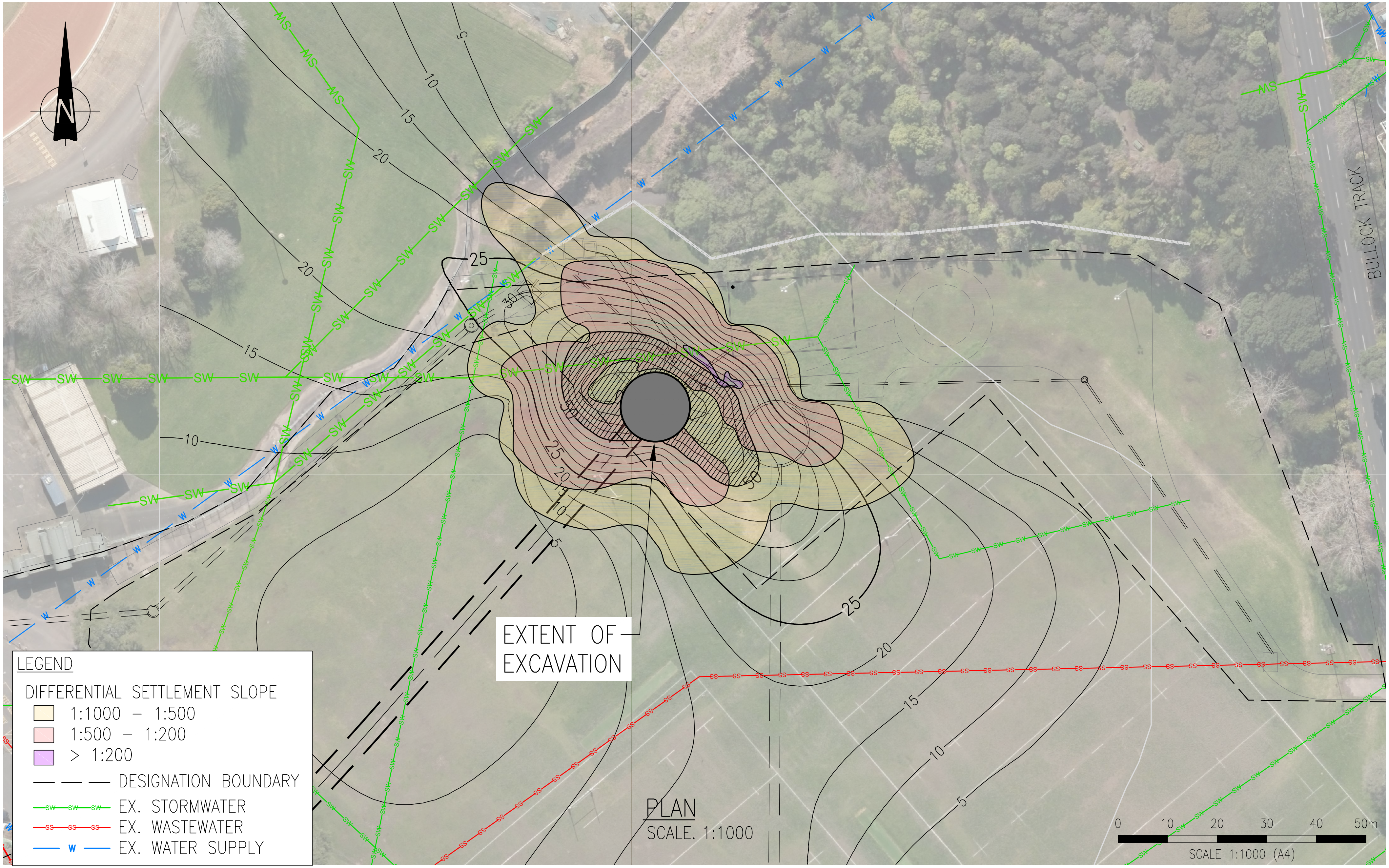

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FIGURE 8.0

SETTLEMENT – DSCIN009 WESTERN SPRINGS

Watercare

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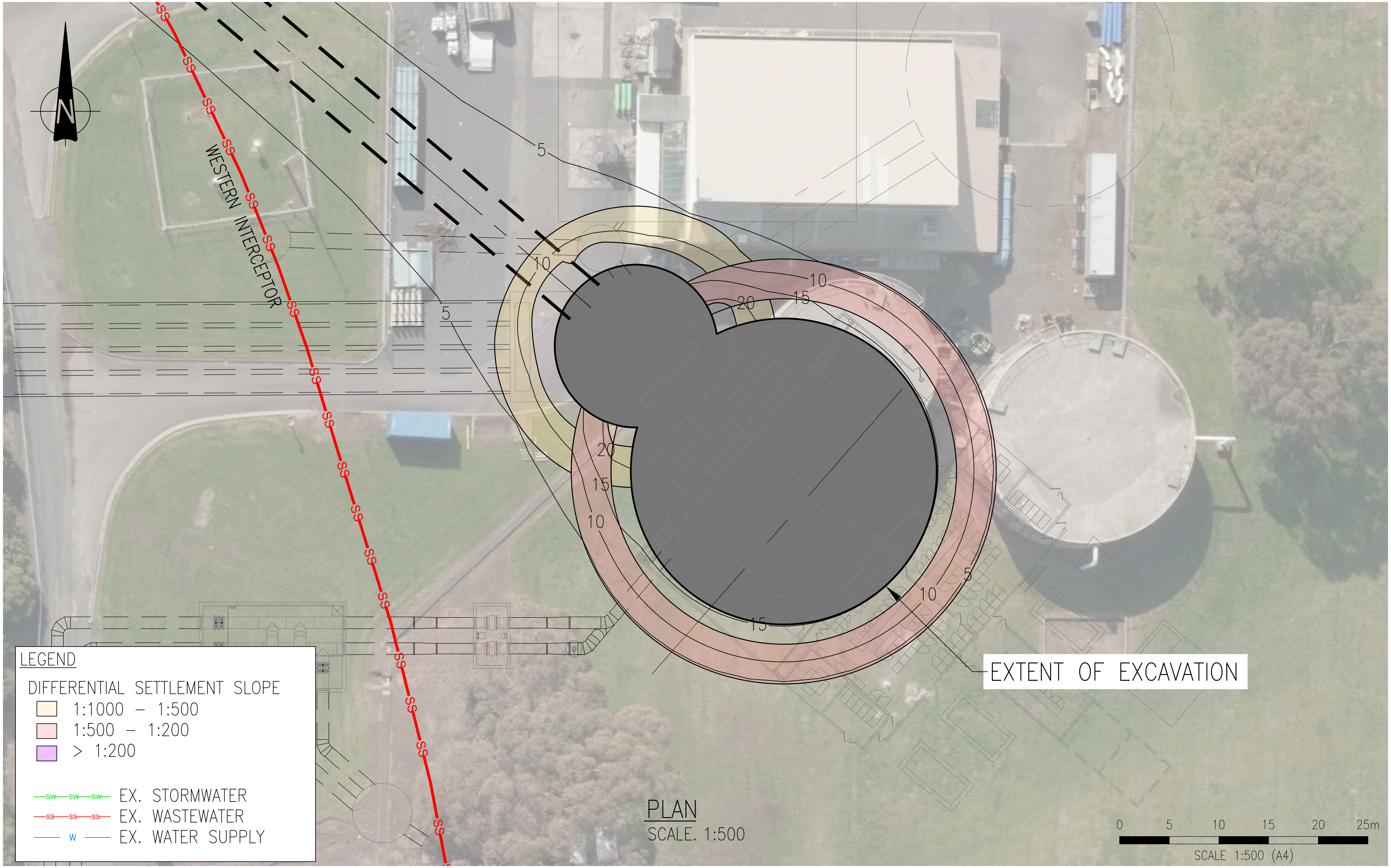

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FIGURE 9.0

SETTLEMENT – DPCIN MANGERE PUMPING STATION

Watercare

Plot Date: Mar 06, 2017 - 4:43pm R:\Deliverables\3D GeoCIVIL BIM\05 Sketches\Settlement\Settlement Figure_DPCIN.dwg

Link Sewer Shaft Contour Plots

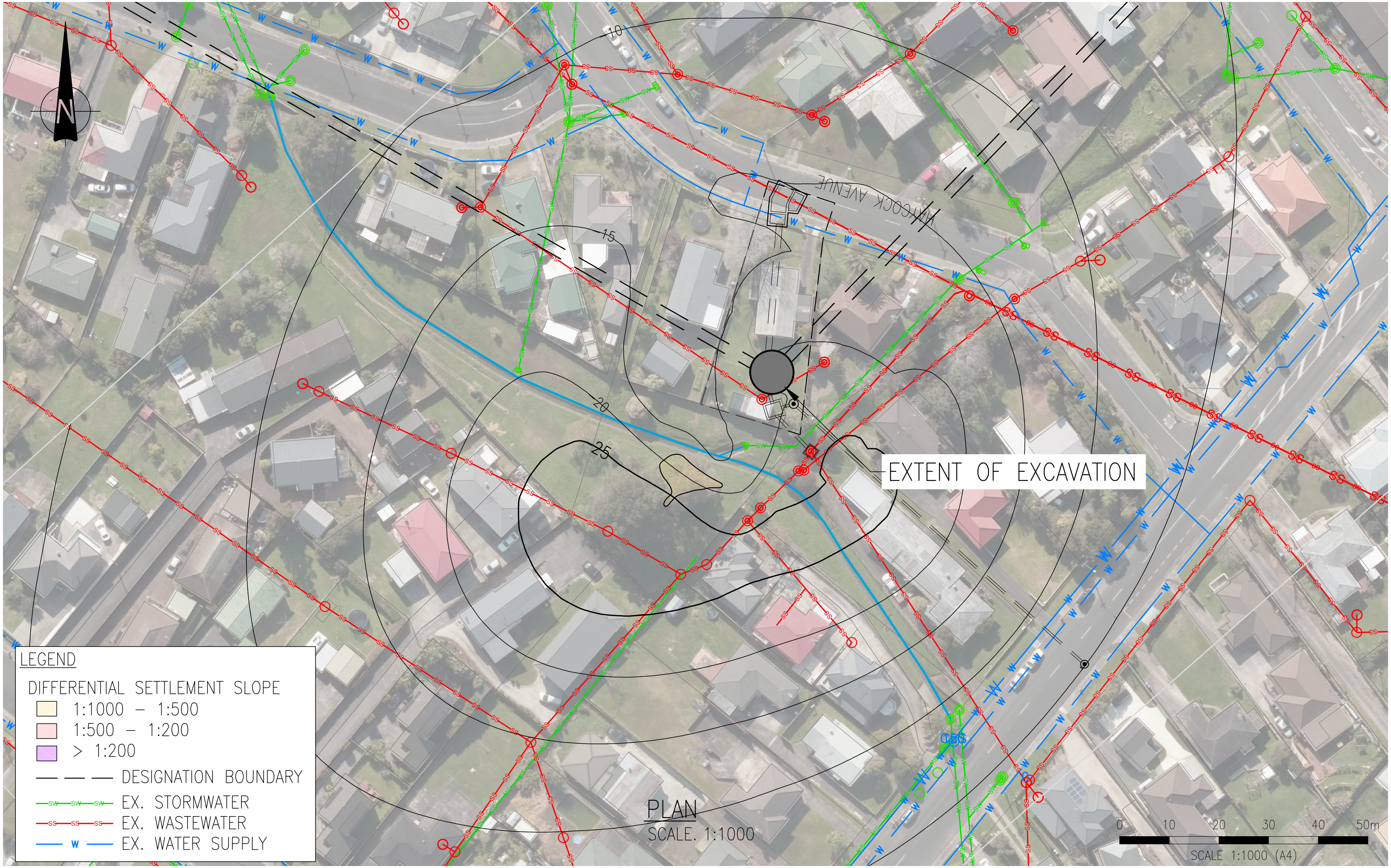

FIGURE 1.0

SETTLEMENT – DSLSC001 HAYCOCK AVENUE

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FIGURE 2.0

SETTLEMENT – DSLSC002 DUNDALE AVENUE

Watercare

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FIGURE 3.0

SETTLEMENT – DSLSC003 WHITNEY AVENUE

Watercare

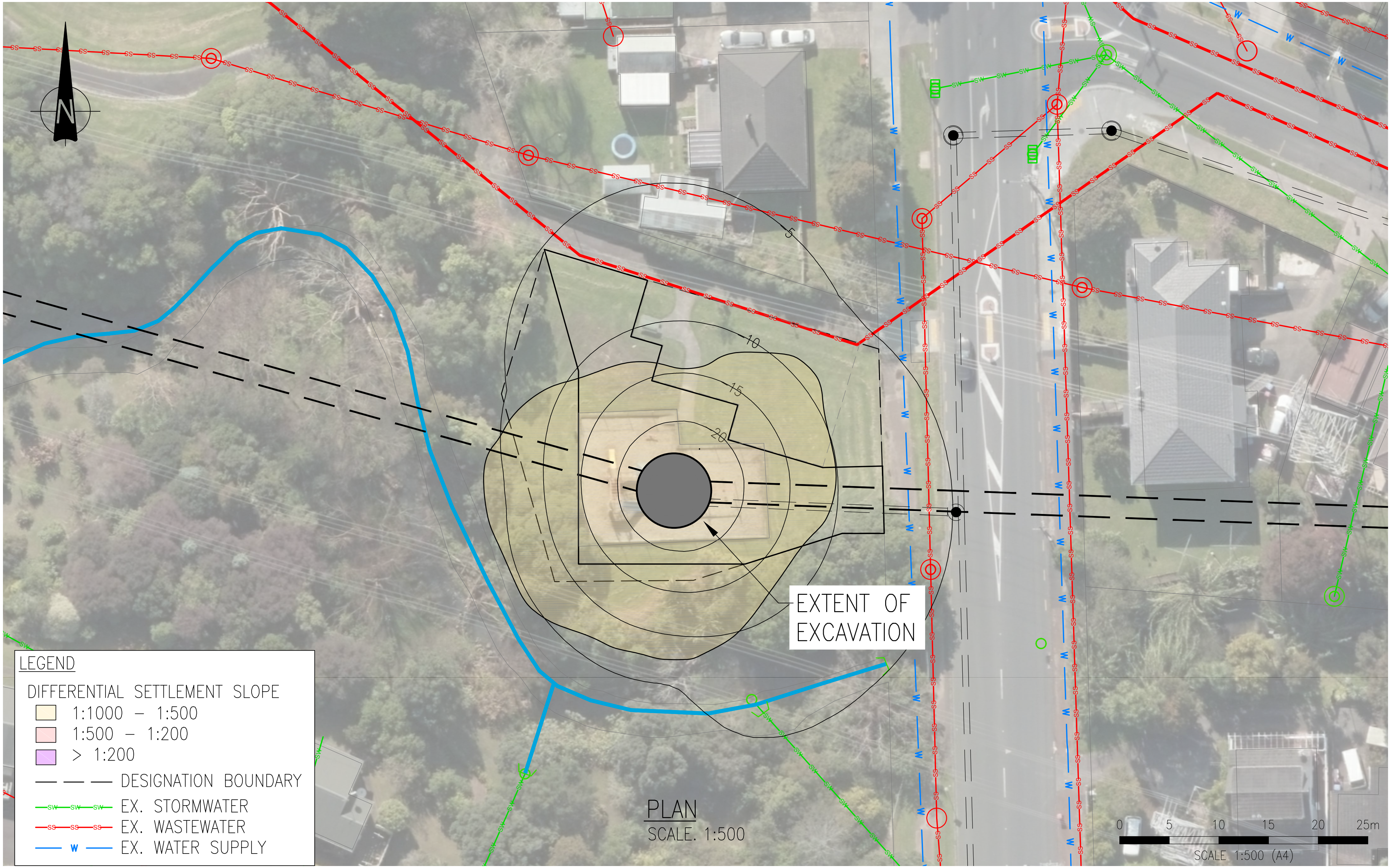

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FIGURE 4.0
SETTLEMENT – DSLSC004 MIRANDA AVENUE

Watercare

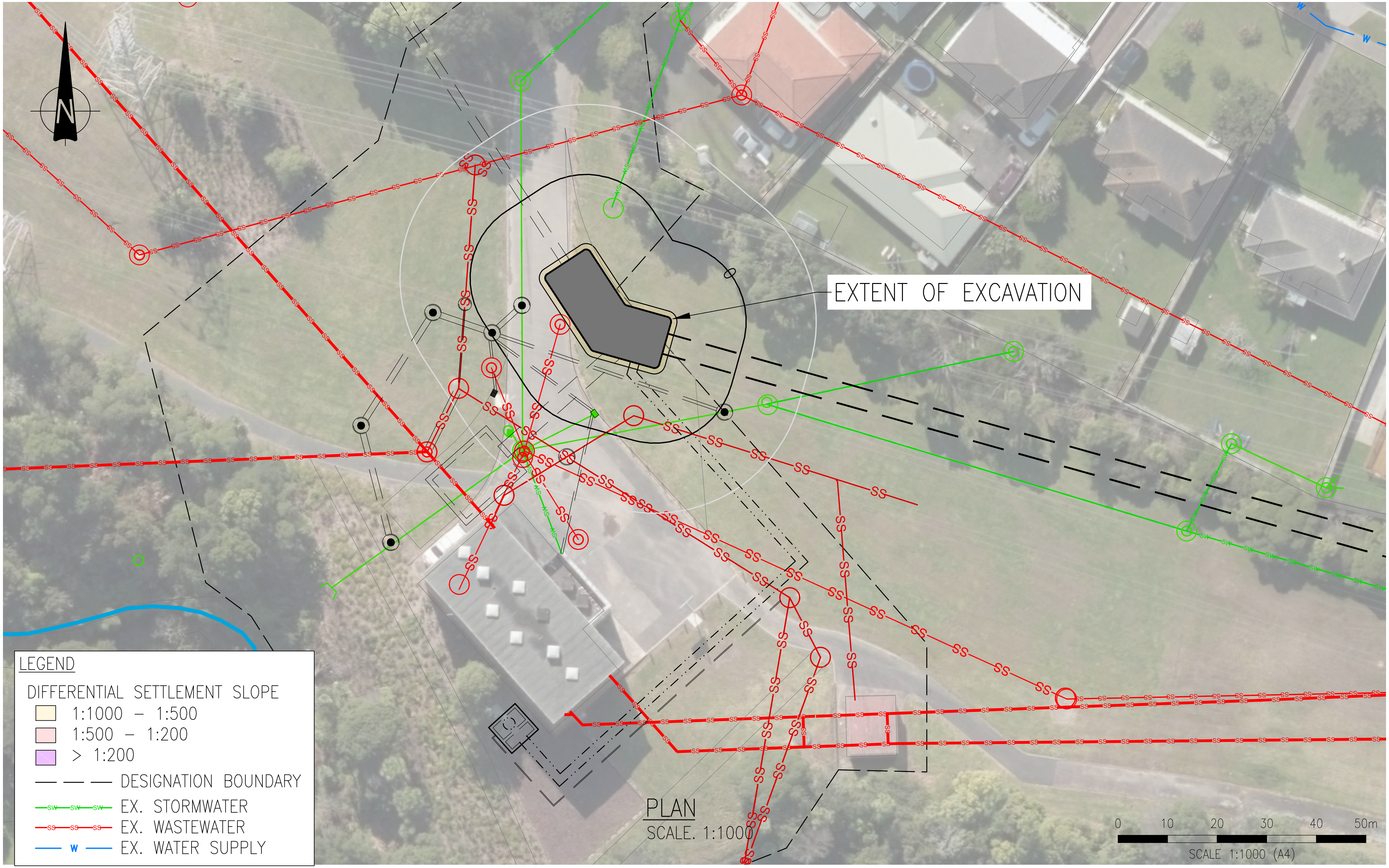

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FIGURE 5.0

SETTLEMENT – DSLSC005 PS25

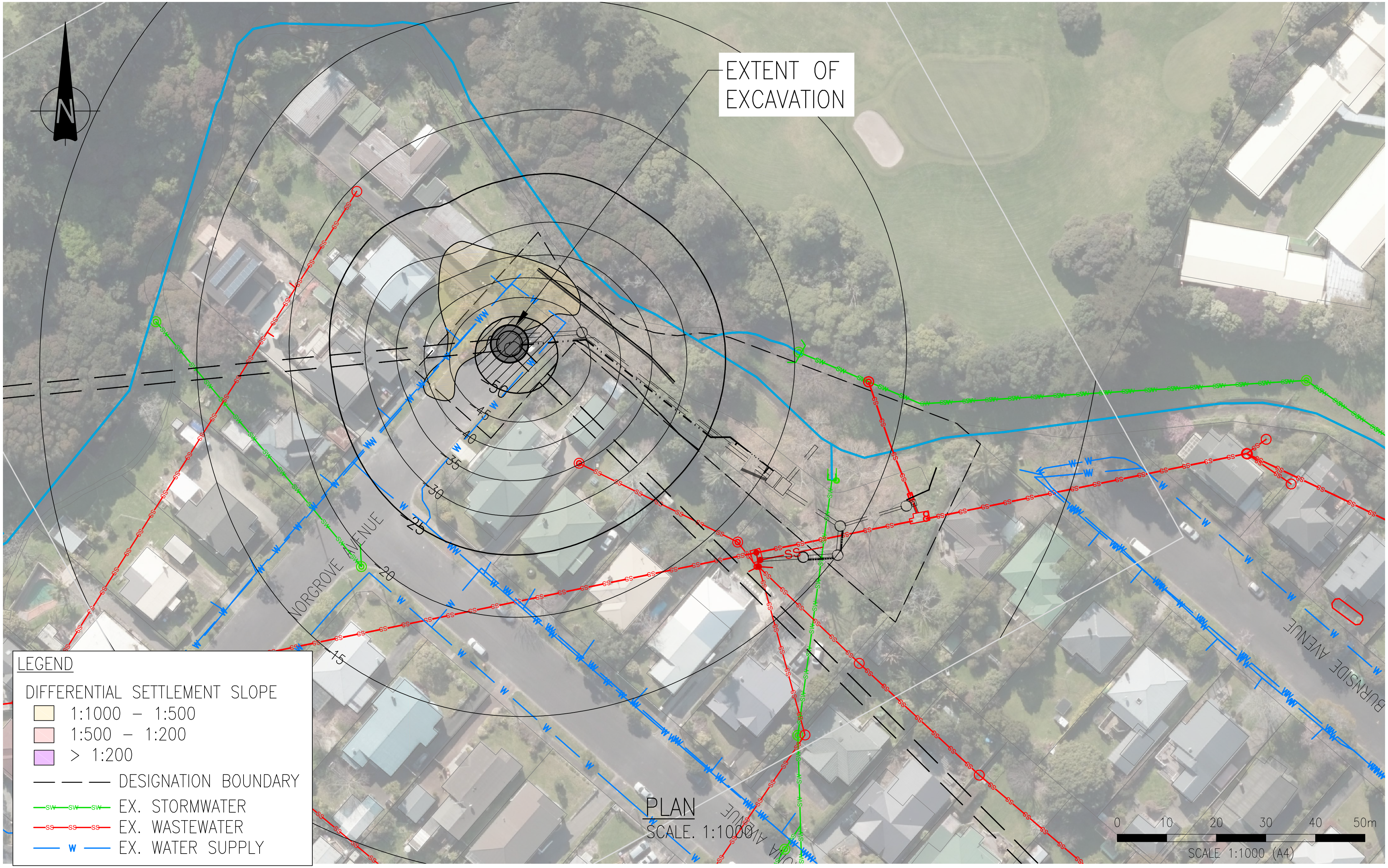

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FIGURE 6.0
SETTLEMENT – DSLSB001 NORGROVE AVENUE

Watercare

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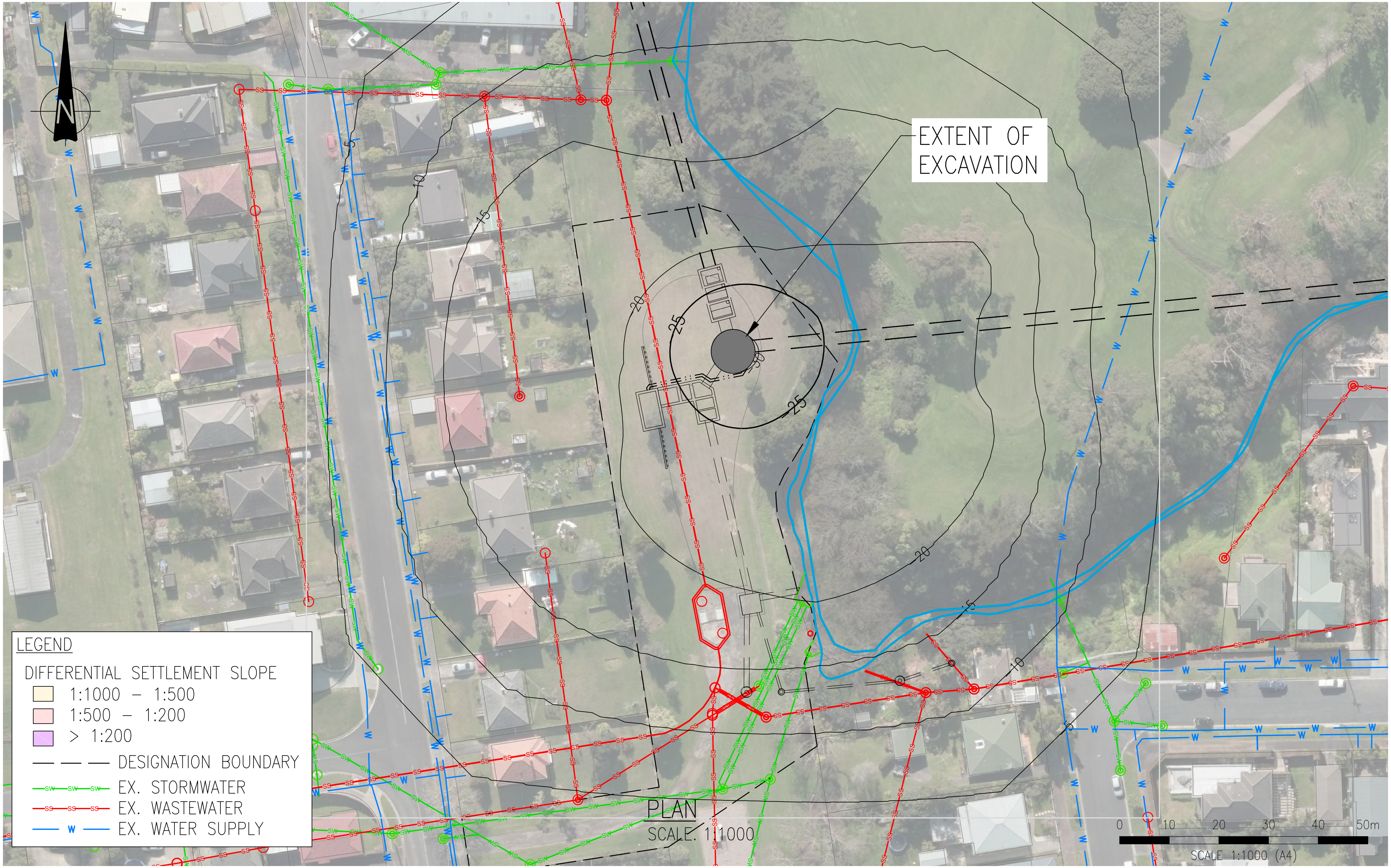

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FIGURE 7.0
SETTLEMENT – DSLSB002 RAWALPINDI AVENUE

Watercare