NZ TRANSPORT AGENCY
WAKA KOTAHI

Roads of national significance

16 20 Western Ring Route – Waterview Connection

Assessment of Ground Settlement Effects

Beca

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Quality Assurance Statement

	Prepared by : James Burr, Technical Director – Geotechnical Engineering (Beca) ; Robert Hopkins, Civil Engineer (Beca); Aaron Beer, Technical Director – Structural Engineering (Beca)
	Reviewed by: Gavin Alexander, Technical Director – Geotechnical Engineering (Beca)
	Approved for Release (Beca): Amelia Linzey, Technical Director (Beca)

NZ Transport Agency

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1. Summary Statement

This report presents the results of an assessment of the potential magnitude and effects of ground settlements (settlements) due to the construction and operation of the proposed SH20 tunnels (cut and cover and driven) and the associated retaining walls at either end. These settlements will be generated by three separate sources; the mechanical settlement of the ground due to the physical excavation of material for the driven tunnel, the mechanical settlement of the ground due to the physical movement of the retaining walls and the consolidation of the ground due to the extraction of groundwater.

The area in which the settlement will occur is above the tunnel alignments extending out several hundred metres either side of the tunnels. In broad terms the area is predominantly residential in nature with the Unitec campus at the northern end and a few commercial buildings along the proposed alignment.

The estimated settlements from the three sources were derived separately and then combined. The mechanical settlements were calculated using computer programs and the consolidation settlements derived using the groundwater drawdown values described in Technical report no. G.7 Assessment of Groundwater Effects and the linear (m_v) method of calculating settlement. The above settlements have all been calculated at a consistent series of cross sections along the alignment and combined to produce total settlement results at critical stages of construction and operation. The magnitude of the settlement estimated is considered to be conservative (i.e. to overstate the likely amount of movement and hence effects) due to the parameters used in the calculations.

The effects on buildings were assessed using an internationally accepted method, specifically prepared for tunnel construction work (Burland, 1997), which classifies the potential damage to a particular building. The effects on other features were assessed from the settlement values and gradients.

The settlements follow the anticipated trough shape along the alignment, with the greatest settlements occurring over the tunnel alignment and then reducing as they extend out several hundred metres each side.

The effects assessment predicts there will be “negligible” effects on the vast majority of buildings in the Study Area, with a limited number of areas of more than “negligible” effects. The effects on other features (such as services) were assessed as negligible or minor.

A proposed monitoring regime is described in detail in the report. It comprises horizontal and vertical monitoring of survey marks, condition assessments of nearby buildings and specific monitoring of retaining walls and services, all to be carried out by an independent team, contracted directly to the NZTA. These results will be used to compare the actual damage categories with those estimated in this report and, if different, mitigation measures agreed. These may include amending the current monitoring regime or enacting specific mitigation measures. The Settlement Effects Management Plan is appended to the report.

While the current assessment indicates that only minor mitigation is required in isolated locations, some more comprehensive mitigation measures are presented to cover the unlikely scenario of more significant damage than estimated occurring. These include grouting, water reinjection, and strengthened support (additional props, additional rock bolts and shotcrete) for the walls and tunnels. Building mitigation includes repair of non-structural defects once the settlement is complete, and the immediate repair of any issues that are structural or will affect the weathertightness of buildings. Services mitigation options depend on the type of service and its construction, but include crack repairs, diversion, relining, support and replacement. If required, road and rail settlements can be mitigated by relatively minor surface reconstruction methods and the adjacent landfills by surface recontouring works.

The conclusions of the report are as follows:

- The effects from the estimated ground settlements caused by the tunnel construction are considered to be typically negligible, with isolated areas of very slight to moderate damage predicted for buildings outside the proposed designation.

- Monitoring should be carried out to confirm the above, to quantify any actual damage and to allow for early warning of areas where the settlement effects may be greater than that predicted.

- Mitigation measures are readily available for the predicted levels of damage and in the unlikely event that greater effects than predicted do occur.

2. Project Description

2.1 Project Overview

In 2009 the NZTA confirmed its intention that the 'Waterview Connection Project' would be lodged with the Environmental Protection Authority as a Proposal of National Significance. The Project includes works previously investigated and developed as two separate projects: being the SH16 Causeway Project and the SH20 Waterview Connection. The key elements of the Waterview Connection Project (Project) are:

- Completing the Western Ring Route (which extends from Manukau to Albany via Waitakere);
- Improving resilience of the SH16 causeway between the Great North Road and Rosebank Interchanges to correct historic subsidence and "future proof" it against sea level rise;
- Providing increased capacity on the SH16 corridor (between the St Lukes and Te Atatu Interchanges);
- Providing a new section of SH20 (through a combination of surface and tunnelled road) between the Great North Road and Maioro Street Interchanges; and
- Providing a cycleway throughout the surface road elements of the Project.

2.2 Report Overview

This report describes the results from one of the group of studies that support the notices of requirement and applications for designation and resource consent to construct and operate the Project. Specifically, the purpose of this report is to assess potential settlements associated with the SH20 section of the Project, the effects of these on the existing buildings, services and infrastructure and to propose monitoring and mitigation for those effects, where required.

The assessment and reporting predominantly focus on the settlement and its effects associated with the SH20 northern portal, cut and cover tunnel, driven tunnels and southern portal, as these works will result in the most significant settlement magnitudes. However, two other proposed retaining walls that have existing buildings nearby were also assessed for their settlement and effects. These retaining walls are located on the northern side of SH16 as part of the motorway widening under the Carrington Road overbridge and on the northern side of SH20 as part of the Richardson Road overbridge construction. Other cuts and retaining walls associated with the Project are not considered to have any significant settlement effects.

This report describes the existing environment in which the effects are assessed to take place. This includes a review of the existing buildings in the area, the services, the transportation infrastructure and other features, where considered relevant.

Settlement effects will result from several different aspects of the construction and operation. Each of these sources are described in the report, along with the methodologies for analysing and combining them. That data was then used to assess the effects on buildings, services, infrastructure and other relevant features. The report then presents the results of the assessment of settlement effects for each of these items.

Finally, the report presents a proposed monitoring regime and potential mitigation measures. The monitoring regime will allow the actual magnitude of contributory causes, and of settlements and the resulting effects to be confirmed and compared with those predicted, while the mitigation provides particular measures should the monitoring indicate that these are required.

Settlements associated with the SH16 causeway works have been assessed in a separate Coastal Engineering Report as the effects related to this settlement are integrally linked with coastal processes and the specific features of the coastal environment.

2.3 Sources of Effects

There are three sources of settlements associated with the construction and operation of the Project. These are:

- Mechanical settlement of the ground due to the physical excavation of the material for the driven tunnels. This is caused by the removal of the supporting rock and subsequent relaxation of the rock and soil above. It will occur relatively quickly following the excavation of the driven tunnels and will be concentrated over the tunnel alignments.

- Mechanical settlement of the ground due to the physical movement of the retaining walls. This is the result of the lateral movement of the retaining walls as they take the load (i.e. as one side is excavated and/or the other side loaded). It will also occur relatively quickly following loading of the walls and will be concentrated in the immediately area behind the retaining walls.

- Consolidation of the ground due to the extraction of the groundwater. This is caused by the reduction in porewater pressure within the soil as the water seeps into an excavation e.g. for the driven tunnels, retaining walls or open cuts. It is time dependant and based on the location and permeability of the excavation at any one time (i.e. as the tunnel progresses the excavation changes in permeability depending on the extent and type of lining that is installed).

2.4 Expected Area of Effects

The expected area of effects extends from the southern portal tunnels entrance through to the northern end of the cut and cover tunnel. The lateral extent of these expected settlement effects is anticipated to be well within the range of the two dimensional groundwater modelling which extends out to 1 km either side of the alignment.

In terms of the Project descriptions, the area extends from approximately chainage Ch1400 through to Ch4200, within Sectors 7, 8 and 9.

The Carrington Road and Richardson Road overbridges are located in Sectors 6 and 9 respectively.

A plan showing the relevant Sector areas is included as Figure A-1 and chainage locations can be seen in Figure A-2, both in Appendix A.

3. Existing Environment

3.1 Overview

The following sections detail the main features within the expected area of settlement effects defined above. In broad terms the land use is predominantly residential in nature containing a few commercial buildings and the Unitec campus at the northern end. The buildings are typical of older residential areas of Auckland, and there are some historic structures within the Unitec campus.

Services are again typical of such an area, although some major sewers and a large watermain cross through the area. The North Auckland rail line crosses the alignment and there are two main roads (New North and Great North Roads) and many smaller local roads within the area.

Other features of interest include Oakley Creek and several historic landfills in reserve areas near the Creek.

3.2 Buildings

3.2.1 General

For the purposes of nominating a "Study Area" in which to initially consider the effects, a zone extending 500m either side of the new alignment was defined and all buildings within this zone were assessed visually to ascertain their structural characteristics. A 500m limit was adopted as the distance beyond which settlement effects were anticipated to be either nil or negligible for all building types. The consequent effects assessment has shown this zone to be sufficient for this purpose,

The majority of the building stock within the Study Area can be characterised as low-rise medium density residential housing interspersed by a limited number of low-rise commercial buildings. There are also a number of low-rise institutional complexes within the area, such as various Unitec buildings, a school, and a rest home.

The stock of residential housing comprises a number of construction types. However, for the purposes of assessing susceptibility to the effects of settlement, it was considered appropriate to group these into two dwelling types, which are defined below.

Of the larger buildings in the Study Area, a limited number of sensitive buildings or structures have been identified which will warrant special consideration as a result of their unusual construction or particular status. These are also identified below and on Figures G-1 to G-4 in Appendix G.

Also considered, were ground movements associated with two proposed retaining walls away from the main tunnel works. The walls specifically assessed include the soil nail wall associated with the widening of the east bound carriageway on SH16 under Carrington Road, and the retaining wall under Richardson Road (refer to Figure A-2).

3.2.2 Dwelling Type 1 – Masonry Construction/Brittle Clad

Houses defined in this category generally contain one or more brittle elements that may be susceptible to visual cracking in the event of differential ground movement. Visual effects are possible, even when differential ground movement is "slight"[1].

Many of the existing buildings in this category will already exhibit some signs of cracking, as this may have occurred as a result of historical or seasonal ground movement, drying shrinkage, or thermal stress relief.

The following construction types are included:

- Reinforced or unreinforced concrete block masonry that may be either painted or overlain by a concrete render (often used in basements or over the lower level of a two storey home).
- Unreinforced brick (solid or cavity wall) that may be either exposed or overlain by a concrete render or plaster.
- Single thickness brick or brick/stone veneer over a timber frame or concrete block substrate.
- Stucco or plaster over a timber frame or concrete block substrate.

3.2.3 Dwelling Type 2 – Timber Construction/Flexible Clad

Houses defined in this category are constructed in a more flexible material that is able to accommodate a certain degree of differential ground movement without any visual effects.

The following construction types are included:

- Weatherboard, either painted, stained cedar, or similar.
- Board and batten or similar timber panel type claddings.
- Fibre cement or fibreglass sheet.

3.2.4 Specific Building 1 – Unitec Building 76

This is a relatively large brick building situated approximately 100m from the proposed tunnel alignment and is shown in Figure 3.1 below. Although a detailed structural assessment has yet to be undertaken, it is likely this building is predominantly unreinforced. As a result of its construction and height, it is expected to be susceptible to visual cracking in the event of "slight" differential ground movement.

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[1] Magnitude of effects terms such as "slight" are defined in Table 4.5 of this report.

Figure 3.1 – Unitec Building 76

3.2.5 Specific Building 2 – 1510 Great North Rd, Unitec Residential Flats

This site comprises two four storey buildings that were constructed around 2001 and is shown in Figure 3.2 below. Both buildings are situated directly over the tunnel alignment. These buildings have been identified for special consideration as a result of their unique founding condition, whereby one side of the building is supported on piles and the other on shallow foundations. The response of these buildings to settlement within the underlying compressible soils is expected to be irregular as the piled portions are expected to be less prone to settlement than the shallow founded parts.

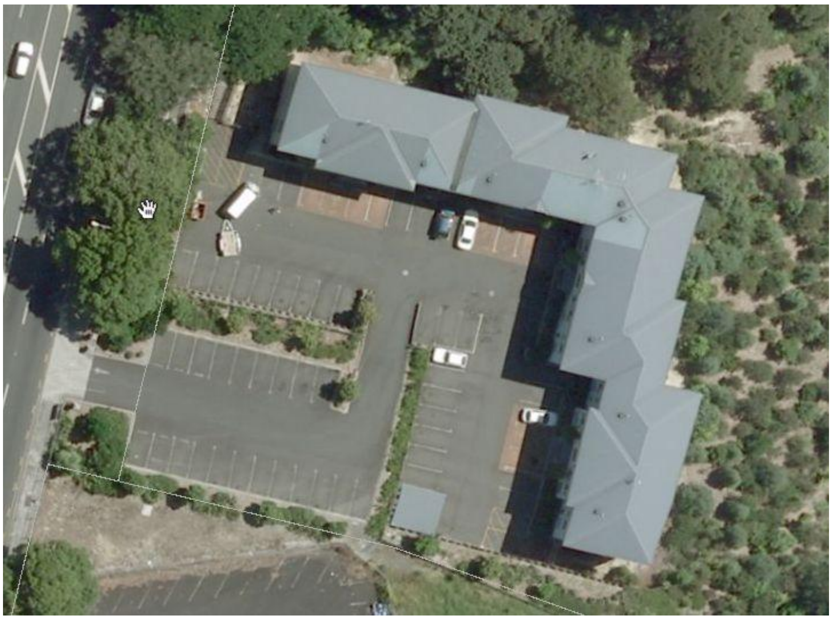

Figure 3.2 – Unitec Residential Flats

3.2.6 Specific Building 3 – Pak'n'Save Supermarket

This supermarket complex is relatively extensive, containing more than one building construction and is shown in Figure 3.3 below. Generally the buildings on this site have shallow foundations and are constructed in a manner that allows for some articulation without damage in the event of minor differential settlement. However, there are some brittle elements evident including masonry or concrete retaining walls, basement structures, and large areas of floor slab on grade where crack control will be important to the owner. For this reason this building has been indentified for special consideration.

Figure 3.3 – Pak'n'Save Supermarket

3.3 Services

3.3.1 General

The proposed tunnel alignment runs beneath a built-up residential area of Auckland which has a typical services networks associated with it. Services located within the Project and their typical construction comprise:

- Sanitary and combined sewers – brick, concrete, asbestos concrete, vitrified clay;
- Stormwater – Concrete, asbestos concrete;
- Watermains – Cement-lined steel, steel, polyethylene (PE);
- Gas – PE, steel;
- Communications – Direct buried cable, cable in PVC/PE duct, fibre optic in PVC/PE duct; and
- Power – direct buried cable, cable in PVC/PE duct.

These networks are primarily located on arterial roads such as Great North Road, New North Road and Richardson Road. Some of the areas that the proposed alignment passes through contain services that are over 60 years old and of varying condition. Meetings have been held with representatives from all service providers and investigations are ongoing into the condition of the services. Preliminary service plans have been prepared based on as-built information received. Discussions are ongoing and drawings are being refined to develop Memoranda of Understanding with each of the service providers.

Service locations are included on Figures F-1 to F-7 in Appendix F. Services of particular note are discussed in the following sections.

3.3.2 Watercare Orakei No.9 Trunk Sewer

This trunk sewer crosses the tunnel alignment 5 times between Great North Road and Richardson Road. Of particular concern are the crossings between Ch3400 and Ch3785 where the sewer is a 750mm diameter brick lined tunnel constructed in the 1940s. Discussions with Watercare Services Ltd (Watercare) are on-going regarding the condition of this sewer. It is a mixture of trenched and tunnelled construction, of various forms including brick, and has generally quite flat grades.

3.3.3 Metrowater Sewers

Metrowater owns sewers ranging from 150mm diameter to large main sewers that feed into the Watercare trunk sewers collecting local residences. They are constructed of varying materials, older pipes being largely asbestos cement and newer pipes concrete, according to the as-built information received. Some parts of the network are over 60 years old. Of particular interest is a 550x900mm sewer at approximately Ch3740. (With the advent of the new Auckland Council, all Metrowater infrastructure will fall under control of Watercare).

3.3.4 Great North Road Services

Numerous services, both local and network, run along Great North Road and cross the proposed alignment of the northern cut and cover tunnel (see figures in Appendix F). The age and condition of these services vary greatly. Many of these services will be relocated as part of the cut and cover tunnel construction.

3.3.5 Watercare Watermains

Watercare owns a number of watermains through the Project, some of which are out of service but require preserving for future use. An important watermain is the Huia No. 2, a 1300mm diameter cement-lined steel pipe installed in 1975. This pipe runs along New North Road, crossing the tunnel at around Ch2370. It is a vital supply line for Watercare.

3.4 Transportation Infrastructure

The area around the proposed alignment has a mix of typical residential and trunk transportation infrastructure comprising roading and rail. The North Auckland rail line crosses the proposed tunnel alignment at around Ch2450. This line comprises twin rail tracks running approximately parallel to New North Road.

The main roads in the area are classified in the Auckland City District Plan as follows;

- Strategic Routes: Great North Road.
- Regional Arterial Roads: New North Road, Carrington Road.
- District Arterial Roads: Blockhouse Bay Road, Richardson Road, Woodward Road
- Collector Roads: Hendon Ave, New Windsor Road, Owairaka Ave

All other roads are classified as Local Roads or Service lanes.

3.5 Other Features

3.5.1 Oakley Creek

Oakley Creek runs generally along the proposed tunnel alignment. At the southern portal the Creek will require diversions to avoid the proposed approach cut, while for the remainder of the route, the Creek will remain above or adjacent to the alignment. The Creek is described in greater detail in Technical report no. G.15 Assessment of Stormwater and Streamworks Effects and Technical report no. G.6 Assessment of Freshwater Ecological Effects.

3.5.2 Landfills

There are three closed landfills along the proposed alignment which will be subject to settlement induced by the Project. These are at Alan Wood Reserve, Harbutt Reserve and Phyllis Reserve.

These landfills are described in detail in Technical report no. G.24 Geotechnical Interpretive Report and Technical report no. G.9 Assessment of the Land and Groundwater Contamination. However, for ease of reference are briefly described here.

- Alan Wood Reserve has known and likely currently unknown localised areas of uncontrolled and construction landfill adjacent to the area of the proposed southern portal.

- The current Phyllis Reserve was historically a basalt quarry. The basalt was generally completely quarried out (although not always as indicated in some of the exploratory bores) and the underlying natural geology exposed. The basalt quarry was then backfilled with waste. Typically, the capped landfill has been found to be of up to 11m thickness in the exploratory boreholes undertaken to date. The landfill is unlined at its base and sits directly on Tauranga Group alluvial soils or weathered volcanogenic East Coast Bays Formation (ECBF) soils and remaining areas of basalt. Beneath these are the ECBF sandstones and siltstones but also a large zone of Parnell Grit. This latter unit has a higher permeability than the more typical ECBF sandstones and siltstones. The alignment in the vicinity of the Phyllis Reserve landfill passes through the Parnell Grit with the driven tunnel crowns located some 30m below the landfill floor.

- Harbutt Reserve landfill is similar in nature to Phyllis Reserve being an infilled quarry, but it likely contains less organics. It is underlain by predominantly ECBF sandstone and siltstone rather than Parnell Grit.

4. Methodology

4.1 Overview of assessment

The total settlements that will result from the tunnels have been assessed from the three individual components. These components are described in Section 2.3 and are mechanical settlements from the tunnel excavation, mechanical settlements from the retaining wall movement and consolidation of the ground due to extraction of the groundwater. The mechanical and consolidation components occur at separate times as a result of different mechanisms and have therefore been calculated individually and then combined to produce the total settlement and consequent effects at critical stages in the Project.

To assess the effects of the settlement on buildings, services and other features, the settlements have been calculated at individual locations. To produce results that are directly comparable to one another, the settlements from each source have been calculated at points located on a consistent set of cross sections along the alignment. These cross sections were chosen to provide representative examples of the relevant geology, hydrogeology and construction types proposed, while retaining a good coverage of the entire alignment. The settlements at each section have been assessed at critical stages in the Project and combined to produce total settlement results for those critical stages.

The cross sections used and their location/ construction type are shown in Table 4.1 below. The cross sections locations are also shown on Figure A-2 in Appendix A.

Table 4.1 – Settlement Analyses Cross Sections

Cross Section Chainage	Location/ Construction type
1420	South Portal, open cut in basalt
1620	South Portal, open cut with retaining walls
1780	South Portal, open cut with retaining walls
1860	Twin driven tunnels
2160	Twin driven tunnels
2450	Twin driven tunnels
2750	Twin driven tunnels
3200	Twin driven tunnels
3400	Twin driven tunnels
3785	North Portal, bored northbound tunnel and cut and cover southbound tunnel
3880	North Portal, bored northbound tunnel and cut and cover southbound tunnel
3940	North Portal, cut and cover tunnel
4160	North Portal, cut and cover tunnel

Descriptions of the methods of analysis and parameters used for the three sources are presented in the following sections, along with details of how the sources are combined and how these values are then used to assess the potential effects on the buildings, services and other features. It is important to understand that the effects of settlement on the structures and other elements are typically a result of differential movement rather than total settlement (i.e. the difference in settlement across a structure rather than the absolute values).

Conservative methodologies have been used when estimating settlements, including:

- The selection of lower bound the rock parameters for the driven tunnel mechanical settlement and the use of the lower end (i.e. less extensive) support techniques in the model; and

- The assumption that settlements occur instantaneously once the groundwater drawdowns are reached. There will be a time lag between the groundwater drawdowns and the actual settlement as porewater pressures within the soils equalise (i.e. in areas where the groundwater is drawn down and then rises back up, the soil may never “feel” the full drawdown effects and therefore achieve the estimated settlements).

A conservative approach is considered to be appropriate because, while the majority of the affected area may have lesser settlements than estimated, the inherent unknowns in geology and groundwater mean that is possible that some zones may reach the estimated values. It is therefore considered appropriate for the assessment of effects to use values that have a low risk of being exceeded.

The assessment has not used historic tunnelling data to calibrate the settlement estimates, because the existing tunnels in similar geological conditions are either too old to have surface settlement data available or were too small in diameter to result in surface projections (e.g. Vector tunnel). Other tunnels in Auckland were not in comparable geology and so their response would not provide an appropriate comparison.

4.2 General Derivation of Parameters for Analyses

Parameters adopted for analyses have been based on the results of both laboratory tests and in situ tests from a number of site investigation phases, including the recent 500-series and 700-series boreholes and most previous investigations. The derivation of the parameters is set out in Technical report no. G.24 Geotechnical Interpretive Report, supported by summary plots of the test data.

Laboratory tests were carried out generally on “undisturbed” (tube) samples taken from the boreholes, though some classification tests were performed on “disturbed” samples (bag or unconfined core).

In situ tests included standard penetration tests (SPT), cone penetration tests (CPT) and pressuremeter tests. The latter is the generic term for tests on soils using an OYO Elastmeter 100 pressuremeter, and tests in rock using a Probex Borehole Dilatometer manufactured by RocTest.

For the analysis of settlement due to consolidation, it is conventional to use compressibility parameters (inverse of stiffness). Parameters have been derived for the two commonly adopted methods of settlement calculation, namely:

- Linear-based method: Coefficient of volume compressibility (m_v) for a particular stress range, with dimensions m^2/MN.

- Log-based method: Compression index (Cc) and re-compression index (Cr), together with preconsolidation pressure (pc). The compression indexes are dimensionless and the preconsolidation pressure is in kPa. In addition, to aid interpretation and calculation, results are given for the ratios (Cc/Cr), Cc/(1+eo) and Cr/(1+eo), where eo is initial void ratio.

The pressuremeter tests have been used to determine material strengths and stiffness, predominantly in the unweathered ECBF weak rocks. Because the uncemented “sandstones” of this series are difficult to sample (for subsequent laboratory testing), these were targeted for the in situ pressuremeter testing, and consequently there is a bias towards the weaker materials.

The pressuremeter tests have been interpreted using industry recognised methods (Hughes et al., 1977). The data give parameters compatible with the triaxial testing and indicate more favourable (i.e. higher) strength and stiffness parameters than adopted for preliminary design.

For calculation of mechanical settlements over tunnels, the stiffness values (Youngs Modulus, E) have been derived primarily from the pressuremeter tests, based on the first loading or “initial” modulus (Ei). Researchers (e.g. Haberfield and Johnston, 1993) have suggested that the unload/reload modulus (Eu/r) would be more representative of in situ rock mass modulus. The pressuremeter tests show that this is approximately twice the initial modulus. The design parameters are, therefore considered to be conservative, possibly by a factor of two.

Consequently, the mechanical settlement design parameters used in this assessment have been selected with a significant conservative bias.

Tables 4.2 and 4.3 below summarise classification, strength, compressibility and stiffness parameters used for the settlement analyses.

Table 4.2 : Classification, Strength and Stiffness Parameters

Geological Unit	Classification Parameters				Strength Parameters						Stiffness Parameters			
	Bulk Density Mg/m^3	Water Content %	Atterberg Limits %		Su kPa	UCS MPa	Peak Values		Residual Values		E_i MPa	$E_{u/r}$ MPa	Poissons Ratio	
			PI	LL			c’ kPa	F’ degrees	c’ kPa	F’ degrees			υ_u	υ
Tauranga Group	1.86[1]	36	32	60	52[2]	-	3	28	-	-	10[4]	-	0.49	0.35
ECBF (Weathered)														
Weathered ECBF	1.78[1]	39.5	40	70	57[2]	-	8	30			20[4]		0.49	0.30
Weathered Parnell Grit	1.78[1]	39.5	31	60	49[2]	-	5	26			20[4]		0.49	0.30
ECBF (unweathered)														
ECBF Rock (General)	2.08[1]	20.6	-	-	-	2.2	50[3]	40			150[6]	300	0.25	0.25
Parnell Grit (General)	2.08[1]	20.6	-	-	-	7.8	50[3]	40			150[6]	300	0.25	0.25
ECBF (Tunnel Design)														
ECBF Rock Class 1							700	40	0	40				
ECBF Rock Class 2							300	40	0	40				
ECBF Rock Class 3							100	40	0	40				
Basalt Rock	2.77	-	-	-	-		200	60	-	-	10000		0.15	0.15

See Table 4.3 for Notes

Table 4.3 : Compressibility Parameters - Soils

Geological Unit	Compressibility Parameters[5]							m_v @ in situ effective stress, σ_v' (kPa)								
	c_v	p_c	c_c	c_r	c_c/c_r	$c_c/(1+e_0)$	$c_r/(1+e_0)$	20	40	60	80	120	200	400	600	800
Tauranga Group	12	s_v'+ 80	0.23	0.04	6	0.13	0.025	0.3	0.275	0.225	0.2	0.18	0.14	0.13	0.102	0.11
ECBF (Weathered) Weathered ECBF	35	s_v'+ 110	0.32	0.03	9	0.15	0.015	0.3	0.28	0.25	0.22	0.18	0.14	0.12	0.11	0.10
Weathered Parnell Grit	32	s_v'+ 110	0.28	0.033	8.5	0.13	0.015	0.3	0.28	0.25	0.22	0.18	0.14	0.12	0.11	0.10

Notes for Tables 4.2 and 4.3.

(1) Modified values were adopted for retaining wall design (see Section 4.4.1)
(2) Undrained strengths at 25th percentile
(3) Constant cohesion adopted for all rock classes in retaining wall design
(4) Undrained modulus values
(5) All compressibilities taken at median values
(6) Constant rock mass modulus taken for all rock classes

4.3 Driven Tunnel Mechanical Settlement Methodology

4.3.1 Derivation of Parameters

Soil properties adopted for analyses of the driven tunnels are generally the typical values developed and agreed for the Project (refer to Technical report no. G.24 Geotechnical Interpretive Report). These are also provided on Tables 4.2 and 4.3 in Section 4.2.

The mechanical ground surface settlements are largely dependent on the properties of the ECBF rock (the unit through which the tunnel will be excavated). Sensitivity analyses have shown that the parameters of the overlying soils have very little influence on the mechanical settlements.

The parameters on which the mechanical settlement analyses have been based for the ECBF rock are E = 150MPa, $\phi' = 40^{\circ}$, and c' = 200kPa. These represent a material midway between Classes 2 and 3 in Table 4.2.

4.3.2 Analysis Methodology

4.3.2.1 Ch1780 to Ch3785

Analysis of the vertical and horizontal movement due to the construction of the driven tunnels from the southern portal at Ch1790 through to the northern portal of the southbound tunnel at Ch3785 has been undertaken using the finite element geotechnical software Phase^2 (Version 7), which was specifically developed for tunnel design. Phase^2 allows for a staged construction sequence to be modelled simulating as closely as possible the actual tunnel construction as well as the long term conditions.

The driven tunnel portion of the alignment comprises two tunnels running side by side with a gap ranging between 10m and 20m between them. The tunnels are approximately 15m wide and 12m high and have a semi elliptical (horse shoe) profile. The depth of soil above the tunnel roof at the southern portal is approximately 10m. The vertical alignment of the tunnel dips down to a maximum depth of 50m below the ground surface at approximately the mid length of the tunnel and then rises up again towards the northern end. At the northern end of the tunnel (at Ch3785) the depth of soil above the roof of the southbound tunnel is approximately 5m and is thereafter shallow enough to change over to a cut and cover construction methodology. The depth of cover above the northbound tunnel at this location is approximately 15m and therefore the driven northbound tunnel continues up to Ch3930 before the vertical position of the tunnel is shallow enough to change over to a cut and cover construction methodology. Apart from a very short section at the southern portal, the tunnel alignment runs completely within the underlying ECBF rock for this particular section.

As each tunnel progresses, a temporary support comprising installation of rock bolts around the circumference and a shotcrete lining will be installed. Several different classes of temporary support have been designed to accommodate the anticipated range of variability in the rock strength along the tunnel alignment. A permanent concrete or shotcrete liner will be installed to provide the long term support.

Descriptions of the key design assumptions and inputs to the model are presented in Appendix C.

4.3.2.2 Ch3785 to Ch3930

This section of the tunnel is the approximately 150m length at the northern end where there is only one driven tunnel (the northbound tunnel) and the southbound traffic is carried within an adjacent cut and cover tunnel. This single driven tunnel section has also been analysed using the finite element geotechnical software Phase[2].

At the northern end of the tunnel (at Ch3785) the depth of soil above the roof of the southbound tunnel is approximately 5m and is thereafter shallow enough to change to a cut and cover construction methodology. The depth of cover above the northbound tunnel at this location is approximately 15m and therefore the driven northbound tunnel continues up to Ch3930 before it is shallow enough to change over to a cut and cover construction methodology. From Ch3785 to Ch3930 the bottom half of the northbound tunnel alignment typically runs within the underlying ECBF rock whereas the upper half of the tunnel typically runs within the overlying residual ECBF soils and possible ancient landslide debris.

For this portion of the driven northbound tunnel alignment, the temporary support includes installation of canopy tubes. These are circular steel tubes (later infilled with grout) installed around the roof and sides of the tunnel in advance of the tunnel excavation to provide support to the surrounding ground when the tunnel is excavated. A shell of shotcrete lining is installed behind the excavation to support the free end of the canopy tubes. A permanent concrete or shotcrete liner will be installed to provide the long term support.

The southbound cut and cover design is discussed later in this report. The cut and cover section of the tunnel has been assumed to be constructed before the driven tunnels to enable access for the driven tunnels construction. The change in ground stresses due to the tunnel construction for both tunnels (the northbound driven tunnel and the southbound cut and cover tunnel) interact, and therefore the mechanical settlement of the individual tunnels cannot be calculated separately and then combined. Therefore, the southbound cut and cover section has been modelled in Phase[2], to simulate the appropriate stress conditions before analysing the mechanical ground surface settlement analyses.

Descriptions of the key design assumptions and inputs to the model are presented in Appendix C.

4.4 Retaining Wall Settlement Methodology

Drawings for the retaining walls are included in Appendix B.

4.4.1 Derivation of Parameters

The derivation of parameters used for the design of the retaining walls is summarised in Section 4.2. The analyses referred to in this section of the report were, however, generally undertaken prior to the completion of much of the laboratory testing. Hence, there is some variation between the bulk density values used for analyses and those recommended in Section 4.2. These differences are detailed in Table 4.4 below and are considered to be nominal and to have little effect on the analysis results. All remaining design parameters used for analysis are as provided in Tables 4.2 and 4.3.

Table 4.4 – Bulk Densities for Retaining Wall Analysis

Geological Unit	Bulk Density, γ (kN/m^3)	
	Adopted for preliminary retaining wall design - completed to date	Statistical analysis of densities from the Geotechnical Interpretive Report
Tauranga Group	17	18.6
Weathered ECBF	18	17.8
ECBF Rock	22	20.8
Basalt Rock	25	27.7

4.4.2 Analysis Methodology

4.4.2.1 Sector 6 – Carrington Road Overbridge

Ground movements associated with the proposed 6m high cut and soil nail retention along the northern side of SH16 extending under Carrington Road overbridge have been assessed using the empirical methods described in the following references.

- Soil Nailing: Best Practice Guidance, 2005, Ciria publication C637, (page 108)

- Geotechnical Engineering Circular No. 7, March 2003 - Federal Highways Administration, U.S. Department of Transportation, Report No. FHW A0-IF-03-017, (page 105)

These methods are similar, and are based on an assessment of the soil type (in this case a low to moderately plastic clay) and a relationship between typical deformations and overall wall heights.

4.4.2.2 Sector 7 – Cut and Cover Tunnel Retaining Walls

Analyses of the vertical and horizontal movements due to the excavation and retention for the cut and cover tunnel and northern portal retaining walls have been undertaken using the finite-difference geotechnical software FLAC. FLAC allows for a staged construction sequence to be modelled simulating the actual construction and long term conditions.

The proposed excavation for the cut and cover tunnel extends over a distance of approximately 500m. The southbound structure extends from the southbound driven tunnel portal at Ch3785 (with an invert depth of 21m) to the northern cut and cover portal at Ch4280. The northbound structure extends from the northbound driven tunnel portal at Ch3930 (with an invert depth of 24m) to the northern cut cover portal at Ch4280. From Ch3785 to Ch3930 the cut and cover excavation will be for a single southbound structure (the northbound tunnel being driven). From Ch3930 northward both the northbound and southbound alignments will be accommodated within one single cut and cover box.

The cut and cover tunnel design allows for fully tanked (essentially water proof), 1200 mm thick diaphragm walls constructed top down using both permanent and temporary props for lateral support as required. Base drainage will be installed beneath the floor slab to relieve groundwater pressures.

The maximum depth below the existing ground level to the road elevation is up to approximately 20m deep for the single tunnel box at Ch3870 and up to 24m deep for the combined tunnel at Ch3930. From Ch3970 north the excavation depths decrease, reducing to approximately 8m at the cut and cover tunnel northern portal.

The excavation widths are typically 15m for the single southbound tunnel excavation and 40m for the combined tunnel box excavation.

Tauranga Group and/or residual ECBF soils are present at the surface with the maximum thickness of these soils expected to be approximately 22m. These soils, and the underlying ECBF rock, will be retained with reinforced diaphragm walls, with a maximum retained height of approximately 20m. Surface earthworks will be required to reduce the retained height to 20m from the maximum depths referred to above.

At Ch3785 to Ch4050 the base of the tunnel excavation is expected to be into ECBF rock, and then from Ch4050 northward, the base of the tunnel excavation will be founded in Tauranga Group or Weathered ECBF. The toe of the diaphragm walls are likely to be founded within the ECBF rock.

The diaphragm walls will be supported with permanent and temporary props in order to limit deflections. These props will be installed at approximately 3 to 5m depth intervals depending on soil conditions and geometric constraints.

Descriptions of the key design assumptions and inputs to the models are presented in Appendix D.

4.4.2.3 Sector 7 - Northern Portal Retaining Walls

Analyses of deflections for the northern portal retaining walls have been completed using the retaining wall analysis software WALLAP. Empirical relationships were then used to derive ground settlements from lateral displacements at increasing distance from the retaining wall alignment.

The northern portal retaining walls extend beyond the cut and cover tunnel at Ch4280 and support excavations on the eastern/southbound (Retaining Wall RW706) and western /northbound (Retaining Wall RW707) sides of the portal. The first 18m of the northern portal walls comprise tied-back 1.0m thick diaphragm retaining structures. North of the diaphragm walls, the northern portal retention comprises 600mm diameter reinforced concrete piles at 1.0 to 1.8m centre to centre (c/c) for a further 12m (RW706) to 15m (RW707).

The maximum height of the northern portal diaphragm walls is 8m at the tunnel portal, reducing to 4m at a distance of 18m from the portal. The diaphragm walls will be partly embedded in ECBF rock where the retained height exceeds 6m, then within weathered ECBF soils. Multi-strand ground anchors will be installed along the top of RW706 and RW707 in order to limit wall deformations. The anchors will be installed at an angle of 45° below horizontal to bond within ECBF rock.

The 12 to 15m lengths of 600mm diameter reinforced concrete cantilevered piles extending north of the diaphragm walls will be installed top down with reinforced shotcrete arches between the piles. The retained height of the 600mm diameter piles varies between 4m and 1m.

All remaining retention structures in Sector 7 (RW704 and RW705), north of the Northern Portal Retaining walls, comprise comparatively small (1 to 3.5m high) cantilevered structures offset more than 25m from the nearest road (Great North Road) or property boundary. The displacement of these structures and associated ground settlements are expected to be less than 30mm at the wall and negligible at a distance of 20m of more. Hence no specific analyses of these structures were considered necessary.

Descriptions of the key design assumptions and inputs to the models are presented in Appendix D.

4.4.2.4 Sector 9 - Southern Portal Retaining Walls

Analysis of the vertical and horizontal movement due to the excavation and retention at the southern portal has been undertaken using the finite-difference geotechnical software FLAC.

The proposed approach excavation to the southern portal extends for approximately 580m, from Ch1200 to Ch1780, reaching a depth of 22m at the southern portal. For the deeper half of the approach, the base of the excavation is expected to found within the underlying ECBF rock.

As shown in the elevation drawings (refer to Drawings in Appendix B) for the walls, both sides of the excavation extend through a depth of up to 15m of surface basalt in some areas. It is proposed that the basalt will be excavated at a slope of 1 Horizontal to 4 Vertical (1H:4V) and retained with rockbolts.

In other areas there is no basalt, with Tauranga group or residual ECBF soils present at the surface. The maximum thickness of these soils is expected to be approximately 13m. These soils, and the underlying ECBF rock, will be retained with a secant-pile wall, with a maximum wall height of approximately 15m.

The secant-pile wall will be tied back with ground anchors (up to 25m long), installed at a steep angle (45˚) into the underlying ECBF rock. Preliminary design has been based on a single row of anchors in the capping beam for wall heights up to 5m. For wall heights of 5-10m a second row of anchors is proposed through a waler beam at 4m below the top of the wall. For wall heights of 10-15m a third waler and row of anchors is proposed at 4m below the second row.

Descriptions of the key design assumptions and inputs to the models are presented in Appendix D.

4.4.2.5 Sector 9 – Richardson Road Cut

Analysis of the vertical and horizontal movements due to the excavation and retention at the Richardson Road Cut has been undertaken using the finite-difference geotechnical software FLAC.

Vertical and horizontal ground displacements are anticipated to be insignificant outside the proposed designation except for RW907 which is located along the boundary of the Modern Chairs building between approximately Ch740 to Ch800 on the northern side of the motorway. Detailed vertical and lateral displacements have been assessed at this section.

The Richardson Road cut allows the motorway to pass under the proposed Richardson Road Overbridge, which will be constructed at the current road level. The cut is up to 10.5m in height and is retained by anchored bored pile retaining walls on both sides.

RW907 varies in height from 4.0m at Ch740 to 7.0m at Ch800 over the area in which settlements have been assessed. It comprises a bored pile wall with shotcrete arched between piles and permanent facing panels. The retaining wall will be constructed in a "top down" methodology.

In this location ECBF soils are present at the ground surface with ECBF Rock typically encountered 6m to 7m beneath existing ground level.

At the assessed location RW907 comprises 900mm diameter piles at 2.7m centres with multi-strand ground anchors approximately 15m in length at 5.4m centres. The piles are embedded into ECBF Rock with the ground anchors being installed at a steep angle (45°) and the bond length installed in ECBF Rock.

The FLAC analysis was primarily carried out to assess vertical and lateral ground movements behind the retaining wall. The retaining wall design was carried out using the computer software programme WALLAP. WALLAP provides horizontal displacement outputs but does not assess settlement behind the retaining wall. The FLAC and WALLAP outputs have been compared and generally provide consistent results.

Descriptions of the key design assumptions and inputs to the models are presented in Appendix D.

4.5 Groundwater Settlement Methodology

4.5.1 Derivation of Parameters

The derivation of the compressibility parameters for analysis of groundwater drawdown settlement is discussed in Technical report no. G.24 Geotechnical Interpretive Report with values reproduced in Tables 4.2 and 4.3 in Section 4.2 of this report.

The analyses have assumed that there will be no settlement from the ECBF rock, basalt or landfill materials due to groundwater drawdowns. The ECBF rock and basalt are rock and so their particle matrix will substantially carry the change in load, and the geotechnical investigations indicate that the landfill materials are above the groundwater level.

4.5.2 Analysis Methodology

The settlement that occurs due to the groundwater drawdown has been estimated using one dimensional consolidation theory, the results of the groundwater modelling (refer Technical report no. G.7 Assessment of Groundwater Effects) and parameters from the geotechnical investigations (refer Section 4.2). The analyses have been carried out at each of the thirteen cross sections shown in Figure A-2 (Appendix A), using the coefficient of volume compressibility (m_v) method. The m_v values for each soil type were derived from laboratory consolidation testing. The value of m_v varies with effective overburden pressure thereby inherently incorporating the effects of overconsolidation of the soils.

The two dimensional groundwater modelling has been carried out for each of the cross sections and these analyses provide the values of groundwater pressure change that are a result of seepage into the tunnel and portals. That modelling is described in Technical report no. G.7 Assessment of Groundwater Effects, and the resulting pressure changes have then been used directly in the settlement analysis.

At each of the cross sections, ground surface settlements were calculated at 25m intervals for the central 200m of the cross section (i.e. 100m either side of the alignment reference line) and then at 100m intervals beyond to a distance of 400m to 700m either side of the alignment, depending on the extent of drawdown and settlement.

For the purposes of comparison, and as a check on the settlement results settlements have also been assessed using the compression index method. For these analyses preconsolidation pressures and recompression index were determined for each soil type based on the laboratory data. These analyses produced results that were generally smaller than (25% to 60%) those from the m_v method. As the m_v settlement values were greater and the method uses less input parameters (i.e. leading to less potential total error) it was decided to use the m_v values for estimating settlements.

4.6 Combinations of Settlement Prediction

As previously stated, the total settlements at the ground surface will result from a combination of values from the three sources. Therefore, the analyses must consider the relevant cases for each source and how to combine them to provide the most appropriate total settlement values. The two mechanical sources will produce relatively quick settlements following their trigger mechanisms (tunnel excavation and excavation in front of the retaining wall), while consolidation will take a comparatively longer time to occur.

The method used to combine the settlements from the individual sources is simple superposition of the component values. This is achieved by combining the individual source values together in a spreadsheet to produce the total settlement for each cross section. The selection of which case from each source to combine to calculate the total settlements is based on the proposed construction programme.

For the twin driven tunnel sections (Ch1860 through Ch3400) there are two critical scenarios which have been assessed. These are:

- Mechanical settlement from the first (northbound) tunnel constructed combined with the groundwater drawdown just before this tunnel is lined. This is typically at “Day 13”, although at Ch3400 the “Day 3” groundwater drawdown is used as the local ground permeabilities produce greatest drawdown at that time (The “day” terminology comes from the groundwater modelling and refers to the length of time from first excavation of that part of the tunnel). While this scenario does not produce the greatest overall settlement values, it does produce the steepest settlement gradient (i.e. differential settlement) in the surface area adjacent to the first tunnel (approximately 10m to 40m either side of the tunnel). The effects of settlement are typically related to the settlement gradient so this scenario is particularly relevant to that localised area.

- Mechanical settlement from both tunnels combined with the groundwater drawdown just before the second (southbound) tunnel is lined, which occurs at “Day 43”. This scenario produces the greatest total settlements and typically the steepest settlement gradients in the remaining area.

Both of these driven tunnel scenarios will occur during the construction phase of the Project. Once the construction of these tunnels is complete, the surrounding groundwater levels will rise again to near original values. The greatest ground settlements will occur during construction (at the time of greatest drawdown). The potential rebound of the soils as a result of the long term return in groundwater levels has not been considered, because any rebound in the soils will be quite small relative to the initial settlement and is likely to slightly reduce settlement gradients. Therefore this long term scenario has not been assessed for the driven tunnels.

In the southern portal, for sections at Ch1620 and Ch1780, one critical scenario has been assessed. This scenario combines the retaining wall mechanical movements with the greatest estimated drawdowns to produce the maximum settlements and gradients. As the floor of this portal remains drained the greatest drawdowns occur in the long term.

For the section at Ch1420, only the greatest (long term) groundwater drawdowns were used to estimate settlements. The retaining wall mechanical displacements were not included for this section as the walls are relatively small (5m max) and completely in basalt, meaning that these displacements will be relatively minor and concentrated directly adjacent to the rear of the cut slopes where there are no structures.

For the northern portal area (i.e. Ch3785 to Ch4160), one critical scenario has been assessed. This scenario combined the final tunnel and retaining wall mechanical movements with the greatest drawdowns to produce the maximum settlements and gradients. In this area the groundwater drawdown results are more complicated, with greatest drawdowns occurring at different times over each cross section (refer to Technical report no. G.7 Assessment of Groundwater Effects). Therefore, for this area the greatest drawdowns at each calculation point at each cross section have been used to estimate the settlements.

4.7 Methodology for Assessment of Effects

4.7.1 Buildings

The method described by Burland (Burland, 1997) was used to assess the effects of settlement on buildings. The approach upon which this paper is based remains the most commonly used and recommended method in international references.

The concept of Limiting Tensile Strain (presented in the above paper) has been used, which enables a classification of the expected severity of damage, of an "idealised" building, at each location where vertical and horizontal ground movement data is available.

At each location along the alignment where settlement data are available, an arbitrary building is assumed to "bend" to follow the predicted ground shape, whether it is a hogging or sagging profile. The maximum tensile strain arising in the building as a result of this profile is calculated and combined with the predicted horizontal strain at the same location, using the method described by Burland 1997.

The building parameters adopted in the analysis are analogous to a continuous masonry wall façade, rectangular in elevation, and this can be varied in scale and aspect ratio to be broadly representative of the typical buildings in the Study Area.

The resulting maximum tensile strain is then compared to the limiting strains that correspond to thresholds or categories of damage. Table 4.5 below shows the limiting strains adopted and overviews an objective system for the classification of damage. This system assigns a description of typical damage, severity, and ease of repair to each of the categories described.

It is important to note this method of assessment and classification is specifically relevant to buildings of brick or block masonry construction (i.e. analogous to Dwelling Type 1 as described in Section 3.2.2). Buildings comprising more flexible construction types, such as timber clad dwellings, are considered less likely to exhibit visible effects (particularly at the low damage categories predicted for the majority of buildings) given the same categorisation.

The above assessment method ignores the interaction between the building foundations and the ground. All buildings will exhibit a degree of restraint against a bending action imposed by the ground and this restraint will be a function of the building stiffness and continuity. For this reason, the effects predicted in Table 4.5 can generally be taken as conservative.

This method of assessment has been used previously in New Zealand and is widely used in the UK. It has been used to enable a broad analysis of the possible degrees of damage to the buildings in the Study Area. This level of assessment is considered conservative and the actual damage is likely to be less than the assessed damage category.

Figure G-5 (Appendix G) provides an example of the graphs that have been prepared to determine the damage categories across each chainage location. This graph has been formulated for Ch3400 day 3, based on the procedure outlined above and assuming a 10m wide by 3m high building, analogous to a single storey residence (note: a sensitivity analysis undertaken has found there is little change in the outcome if the building length is doubled to 20m). The graph plots horizontal strain versus surface deflection ratio for each data point at 2m intervals perpendicular to the tunnel alignment. A separate plot is provided depending on whether the ground is hogging or sagging. From these graphs, and with reference to Table 4.5 below, those areas where adverse effects on buildings may occur can be established.

In Appendix G, a contour plan (Figures G-1 to G-4) is provided which shows the extent (or boundaries) of the assessed damage categories along proposed the tunnel alignment. This plan is used to identify those buildings lying within an area where the settlement modelling estimates that greater than "negligible" effects (i.e. damage category 0) are possible.

In all areas with buildings assigned a damage category of "1 – Very Slight" or greater, the buildings have been observed (from the street) to determine their general construction and to establish what further study, inspection, and/or monitoring/mitigation is appropriate.

Table 4.5: Building/Structure Damage Category (after Burland, 1997)

Damage Category	Category of Damage	Description of Typical Damage (Ease of Repair is Underlined)	Approx Crack Width (mm)	Limiting Tensile Strain (%)
0	Negligible	Hairline cracks	< 0.1	< 0.05
1	Very Slight	Fine cracks that can be easily treated during normal decoration. Perhaps isolated slight fracture in buildings. Cracks in external brickwork visible on inspection.	< 1	0.05 – 0.075
2	Slight	Cracks are easily filled. Redecorating probably required. Several slight fractures showing inside of building. Cracks are visible externally and some repointing may be required externally to ensure weather tightness. Doors and windows may stick slightly.	< 5	0.075 – 0.15
3	Moderate	The cracks require some opening up and can be patched by a mason. Recurrent cracks can be masked by suitable linings. Repointing of external brickwork and possibly a small amount of brickwork to be replaced. Doors and windows sticking. Service pipes may fracture. Weather tightness often impaired.	5-15 or a number of cracks > 3	0.15 – 0.3
4	Severe	Extensive repair work involving breaking out and replacing sections of walls, especially over doors and windows. Windows and door frames distorted, floor sloping noticeably. Walls leaning and bulging noticeably, some loss of bearing in beams. Service pipes disrupted.	15-25 but also depends on number of cracks	> 0.3
5	Very Severe	This requires a major repair job involving partial or complete rebuilding. Beams lose bearing, wall lean badly and require shoring. Windows broken due to distortion. Danger of instability.	Usually > 25 but depends on number of cracks	

Table Notes:
- In assessing the degree of damage, account must be taken of its location in the building or structure.
- Crack width is only one aspect of damage and should not be used on its own as a direct measure.
- The table is based on buildings of brick/blockwork masonry construction.

4.7.2 Services

In order to assess the effects of settlement on existing underground services, the predicted settlement contours were overlain on a plan of the existing services in the area (Figures F-1 to F-7, Appendix F). From this, settlement gradients along services have been calculated, allowing an assessment of effects on each particular service. If particular services are identified as having a high risk of damage, discussions are being held with the service owner regarding potential damage, proposed monitoring regimes and potential options for mitigation.

The sensitivity of a service to settlement depends on its construction, age and condition. The main services of concern are expected to be older piped services constructed of brick, glazed earthenware and asbestos cement which are likely to have the least amount of ability to absorb movement.

4.7.3 Transportation Infrastructure

The assessment of effects on the roads and railway track comprises overlaying the estimated settlement contours over the roads and rail, and determining changes to the gradients of those assets. The effect of those changes in gradient on each road and the railway was then assessed, and monitoring and potential mitigation options proposed, if required.

The most significant effect for the roads is likely to be the potential for changing the surface water flow regime by affecting the existing drainage grades.

4.7.4 Other Features

4.7.4.1 Oakley Creek

The effects of the estimated settlement on Oakley Creek were assessed by overlaying the total settlement values over the existing Creek profile and considering the predicted changes.

4.7.4.2 Landfills

The effects of the estimated settlement on the landfills were assessed based on the total settlement values and considering the known and likely details of the landfills.

5. Effects Assessment

5.1 Effects Overview

The settlements will occur over the tunnel and portal construction period and, in some areas, continue on into the operational phase of the Project, albeit at ever reducing rates. As there is no simple or distinct transition of the effects from construction to long term operation, the settlements and their effects have been assessed as a whole, rather than attempting to artificially break them into construction and operational parts.

While the mechanical settlements (tunnel and retaining walls) will occur during the construction phase, the consolidation settlements are dependent on the groundwater drawdowns which vary (and recover) with time. There will be a time lag between the drawdowns and the consequent settlements as the internal porewater pressures in the soils equalise. As the northern cut and cover tunnel slab and southern portal floor slab will remain drained in the long term, a new steady state groundwater regime will need to be in place before settlements in these areas are complete.

This assessment considers the potential effects based on the combination of estimated settlements that give the highest risk of damage. For many projects, these typically occur at the “end” of the settlement process but as shown for the driven tunnel sections at “Day 13”, for this Project a critical case can occur part way through the construction.

The monitoring regime described later in this report will provide on-site confirmation of the rate of settlement as well as the magnitude. In addition, Technical report no. G.7 Assessment of Groundwater Effects provides details of the proposed groundwater monitoring regime, which will provide an earlier indication of the likely effects.

Overviews of the design and proposed construction methodology for the driven tunnels and each section of retaining walls are provided in Appendices C (driven tunnels) and D (retaining walls).

5.2 Settlement Estimates

5.2.1 Settlement due to Driven Tunnel Construction

The calculated vertical and horizontal ground surface displacements for the mechanical tunnel settlements are shown on the plots (Figures C-1 to C-8) in Appendix C. The summary plots present the ground surface displacements for the last stage of the first tunnel construction and the last stage of the second tunnel construction. These stages have the highest differential, as well as total ground surface settlements. The plots show how the ground surface displacements diminish with increasing distance away from the tunnels. The mechanical ground surface displacements also show the sensitivity to the in situ horizontal to vertical total stress ratio within the ECBF rock for three different stress ratios, 0.7 (the expected lower bound) 1.0 (expected stress ratio) and 1.3 (the expected upper bound).

The mechanical settlement ground surface displacement summary plots show that the vertical displacement is generally highest at the centre line of the two tunnels, further away from the tunnel. From Ch1780 to Ch3785 the maximum ground surface displacement (directly above the tunnels) ranges between 10 to 50mm (for the cases where the stress ratios are taken at unity). Generally, 100m from the centreline of the two tunnels the mechanical ground surface displacements are less than 5mm. The horizontal displacement of the ground surface is generally towards the tunnels and is generally at the maximum value near the outer two sides of the tunnels (i.e. on the sides next to the left lanes). The horizontal displacements are often asymmetrical about either side of the tunnel, and this is mainly due to the topographic effects. The maximum ground surface horizontal displacements are in the order of 5 to 30mm.

The sensitivity of the horizontal to vertical stress ratio in the ECBF rock is reasonably significant. Generally the vertical ground surface displacements increase by 25% if the stress ratio is decreased to 0.7 and decrease by 25% if the stress ratio is increased to 1.3. Conversely, the horizontal ground surface displacements decrease if the stress ratio is decreased to 0.7 and increase if the stress ratio is increased to 1.3 (particularly at the southern and northern ends where the tunnels are close to the ground surface.

The computed vertical ground surface displacements increase with increasing tunnel depth. This is contrary to normal experience because, as the depth of cover increases, the ground surface displacements would be expected to decrease. The reason that the Phase^2 settlement analyses is calculating increased settlement for the deeper tunnel sections, is because a constant lower bound rock modulus parameter has been used for the ECBF rock. The in situ stress state in the ECBF rock increases with depth. Therefore, as the depth of the tunnel increases, the stress relief in the rock, caused by the tunnel construction also increases which, if multiplied by a constant modulus value, results in increasing settlement with increasing tunnel depth.

However, while the test data show a clear increase in stiffness with strength there is no discernable increase in strength or stiffness with depth. Therefore, the use of a constant modulus with depth is considered to be appropriate at this stage. If later testing and analyses indicate that the rock modulus does increases with depth, then this would result in more constant (or even reducing) ground surface displacements with increased tunnel depth.

A few minor anomalies are observed at the sections analysed at the two ends of the tunnels (where the tunnels are close to the ground surface). At Ch1860 the settlement results indicate that the settlement trough caused by the construction of the first tunnel construction is slightly reduced (i.e. the ground surface above this tunnel rebounds slightly) after the construction of the second tunnel. This is mainly due to a combination of topographic slope effects and stress relief effects caused by the second tunnel interacting with the first tunnel. The stress relief causes the floor of the first tunnel to heave slightly and because of the limited depth of cover, the tunnel lining transmits the heave displacements through to the roof and pushes up the ground slightly. In reality the rebound modulus is typically between 2 to 5 times stiffer than the loading modulus. Because a linear elastic-plastic soil model has been used (which only has a single modulus parameter), the increased unloading stiffness has not been included in the models. Therefore, the heave observed in the analyses is likely to be less than predicted (and probably negligible). The same phenomenon occurs at Ch3785.

From Ch3850 to Ch3930, the bottom half of the northbound tunnel alignment typically runs within the underlying ECBF rock whereas the upper half of the tunnel typically runs within the overlying residual ECBF soils and possible landslide debris[2]. The residual soil strength and stiffness parameters are approximately an order of magnitude less compared to the underlying ECBF rock, resulting in much higher ground surface settlements directly above the tunnel (in the order of 200 to 300mm). However, these displacements are limited in extent and occur almost entirely directly above the tunnel.

5.2.2 Settlement Due to Retaining Wall Construction

5.2.2.1 Sector 6 – Carrington Road Overbridge

Ground movements in the proposed soil nailed cut section beneath the Carrington Road overbridge have been assessed using empirical methods, as described previously in this report. These settlement results are presented in Appendix D.

Based on the highest cut being 6m high and near vertical, with a 2m high batter above, the estimated vertical wall mechanical deflection is approximately 20mm. This deflection is reduced to 5mm at the end of the soil nail embedment length which is around 8m behind the wall. These deflections are approximately in proportion to the retained height (i.e. a 3m high section will give settlement estimates of half the above values). Horizontal and vertical mechanical displacements have been shown to be approximately equal in magnitude.

5.2.2.2 Sector 7 – Cut and Cover Tunnel Retaining Walls

FLAC output plots of estimated maximum mechanical displacements for each analysis section are included in Appendix D for the static case after construction is complete. The analyses indicate up to 65mm of lateral ground movement and a similar magnitude of vertical settlement for the western wall supporting the deepest excavation at Ch3940. These ground movements reduce to less than 5mm at a distance of approximately 25 from the edge of the wall.

A summary of the maximum vertical and horizontal mechanical ground movements are shown in Tables 5.1 and 5.2 below.

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[2] The landslide debris is described in Technical report no. G.24 Geotechnical Interpretive Report, and the influence of this material has been considered in the analysis by using a full depth of soil to the base of the disturbed zone and representative in situ strength parameters for those materials.

Table 5.1 – Cut and Cover Tunnel - Calculated vertical mechanical movements

West Wall		Chainage	East Wall	
Maximum Settlement, d	Horiz offset from wall to point where, d < 5mm		Maximum Settlement, d	Horiz offset from wall to point where, d < 5mm
< 5mm	0m	3785	< 5mm	0m
45mm	50m	3880	10mm	5m
67mm	25m	3940	< 5mm	0m
27mm	20m	4160	15mm	20m

Table 5.2 – Cut and Cover Tunnel - Calculated horizontal mechanical movements

West Wall		**Chainage**	**East Wall**	
Maximum Horizontal Displacement ρ	Horiz offset from wall to point where, ρ < 5mm		Maximum Horizontal Displacement ρ	Horiz offset from wall to point where, ρ < 5mm
7mm	10m	3785	7mm	5m
58mm	55m	3880	42mm	16m
65mm	30m	3940	52mm	40m
23mm	18m	4160	5mm	10m

5.2.2.3 Sector 7 – Northern Portal Retaining Walls

Plots of estimated maximum mechanical horizontal displacements and vertical settlement for each analysis section are included in Appendix D for the static case after construction is complete. The analyses indicate up to 45mm of lateral ground movement and a similar magnitude of vertical settlement where the wall is cantilevered at Ch240 (Ramp 4 chainage). These ground movements reduce to less than 5mm at a distance of approximately 25m from the edge of the wall.

A summary of the maximum vertical and horizontal mechanical ground movements are shown in Tables 5.3 and 5.4 below.

Table 5.3 – Northern Portal Walls - Calculated vertical mechanical movements

West Wall (RW707)		**Ramp 4 Chainage**	**East Wall (RW 706)**	
Maximum Settlement, d	Horiz offset from wall to point where, d < 5mm		Maximum Settlement, d	Horiz offset from wall to point where, d < 5mm
25mm	15m	220	25mm	15m
41mm	6m	240	41mm	6m
20mm	4m	250	20mm	4m

Table 5.4 – Northern Portal Walls - Calculated horizontal mechanical movements

West Wall (RW707)		Ramp 4 Chainage	East Wall (RW 706)	
Maximum Horizontal Displacement ρ	Horiz offset from wall to point where, ρ < 5mm		Maximum Horizontal Displacement ρ	Horiz offset from wall to point where, ρ < 5mm
20mm	27m	220	20mm	27m
45mm	13m	240	45mm	13m
20mm	7m	250	20mm	7m

5.2.2.4 Sector 9 - Southern Portal Retaining Walls

Plots of estimated maximum mechanical displacements for each analysis section are included in Appendix D for the static case after construction is complete.

A summary of the maximum vertical and horizontal mechanical ground movements are shown in Tables 5.5 and 5.6 below.

Table 5.5 – Southern Portal - Calculated vertical mechanical movements

West Wall (RW909)		Chainage	East Wall (RW 911)	
Maximum Settlement, d	Horiz offset from wall to point where, d < 5mm		Maximum Settlement, d	Horiz offset from wall to point where, d < 5mm
12mm	30m	1780	14mm	25m
11mm	6m	1690	Not analysed	
Not analysed		1655	32mm	22m
0mm	n/a	1620	30mm	25m

Table 5.6 – Southern Portal - Calculated horizontal mechanical movements

West Wall (RW909)		Chainage	East Wall (RW 911)	
Maximum Horizontal Displacement ρ	Horiz offset from wall to point where, ρ < 5mm		Maximum Horizontal Displacement ρ	Horiz offset from wall to point where, ρ < 5mm
24mm	51m	1780	28mm	35m
4mm	n/a	1690	Not analysed	
Not analysed		1655	15mm	25m
2mm	n/a	1620	15mm	25m

Additional analyses to determine ground movement from mechanical displacement were undertaken at Ch1690 (west wall) and Ch1655 (east wall) as the retaining wall geometry was different to the control sections at Ch1780 and Ch1620. As shown in Tables 5.5 and 5.6, the ground movement results for Ch1690 and Ch1655, are either relatively minor or are similar to the next closest chainages at Ch1780 and Ch1620. Analysis of the settlement due to groundwater drawdown has therefore not been undertaken for Ch1690 and Ch1655.

5.2.2.5 Sector 9 – Richardson Road Cut

Plots of estimated maximum displacements for the tied back bored pile retaining wall RW 907 at Ch780, for the static case following construction, are attached in Appendix D. The analysis shows approximately 5mm horizontal movement and less than 5mm vertical movement at the wall location reducing to zero at a distance of approximately 20m away from the wall.

5.2.3 Settlement Due to Groundwater Drawdown

5.2.3.1 Driven Tunnels

Ground surface settlements resulting from groundwater drawdown have been estimated at each cross section for the relevant scenarios using the methodology and parameters described previously. These scenarios are the "Day 13" and "Day 43" results for the driven tunnel sections (i.e. directly following the temporary lining of the first and then second tunnel). The results of these analyses are presented in Figures E-4 to E-9, in Appendix E. These figures also show the mechanical and combined total settlements at these sections.

The results for the driven tunnel section show the expected shape with a trough of settlement centred on the proposed tunnel alignment. The magnitude and extent of this trough depends on the local geology and groundwater conditions. The results generally show a greater extent on the eastern (i.e. Mt Albert) side of the driven tunnel alignment and an increasing extent to the north. The peak settlement values are similar for cross sections Ch1860 through Ch3200 (50 to 70mm), while the northern end (Ch3400) is substantially greater at around 300mm.

The difference in peak magnitudes between the "Day 13" and "Day 43" results is typically relatively small. The extent of the trough does not vary significantly between the two scenarios. Comparison between the two graphs for each of Ch2450, Ch2750 and Ch3200 indicates that at some particular points the settlement is expected to reduce between "Day 13" and "Day 43". This occurs because the settlement analyses are based on the groundwater model results at each of those particular times. As the ongoing tunnel construction is modelled the drawdowns can reduce slightly, although they predominantly increase between these two days. While the analyses have not assumed that the soils will actually rebound with increasing groundwater pressures, these slight discrepancies do not significantly affect the estimates or the assessment on which they are based.

The results at Ch3400 are substantially greater in magnitude (300mm) and extent (200m west to 600m east) than the other driven tunnel cross sections and this is a direct result of the local geological conditions in that area. This area is underlain by Parnell Grit, a volcanogenic unit of the ECBF rock that is assessed as having a higher permeability and therefore induces a greater magnitude and extent of drawdown than the typical ECBF rock (over the time periods considered). In addition, the thickness of the weathered soils in this area (particularly to the east) is greater than for the other cross sections, and this also increases the magnitude of settlement.

5.2.3.2 Sector 6 – Carrington Road Overbridge

There is not considered to be a significant effect on groundwater due to the construction of the wall under the northern side of the Carrington Rd overbridge. The wall will simply be cutting back the existing cut slope and does not result in a significant deepening of it. Any effect on the groundwater regime is anticipated to be negligible, and therefore settlements are also considered to be negligible.

5.2.3.3 Sector 7 - Northern Cut and Cover Tunnel and Portal

Ground surface settlements have been estimated for at each cross section for the single scenario using the methodology and parameters described previously. The results of these analyses are presented in Figures E-10 through E-13, in Appendix E. These figures also show the mechanical and combined total settlements at these sections.

The northern portal results are a little more variable than the others, but follow a logical set of trends. The driven tunnel and retaining walls result in drawdown of the groundwater extending out both east and west. As a result of these drawdowns, typically small (less than 50mm) settlements extend out around 300m to the east. A small area of up to 100mm settlement is estimated adjacent to the eastern edge of the tunnel at Ch3940 (between the tunnel and Oakley Creek).

To the west, between Ch3785 and Ch3940 the settlements are less than 50mm and extend to a distance of around 50m. At Ch4160 the settlements are small (less than 25mm) but extend out up to 300m. These Ch4160 results are due to the increasing thickness of compressible soils to the north.

The portal walls that extend to the north of the cut and cover tunnels (i.e. north of approximately Ch4280) are considered to have a negligible effect on groundwater drawdowns and therefore negligible settlements associated with them. These open cuts start at approximately the average level of the recorded groundwater levels at that location. The groundwater levels then reduce to the north as the cuts rise up, meaning the drawdown effect is minor. In addition, seasonal variations will have resulted in groundwater levels falling below those average levels and so the surrounding soils will have already historically "felt" the small magnitude of potential drawdown from these cuts.

5.2.3.4 Sector 9 - Southern Portal

Ground surface settlements have been estimated for at each cross section for the single scenario using the methodology and parameters described previously. This scenario uses the long term results as the floor slab in this portal is left permanently drained. The results of these analyses are presented in Figures E-1 to E-3, in Appendix E. These figures also show the mechanical and combined total settlements.

The results for these cross sections show the expected shape with a trough of settlement centred on the cut for the portal. The magnitude of the settlement increases up to 100mm to the north as the portal cut deepens and therefore the groundwater drawdown increases. The trough extends up to 200m to the west and 500m to the east.

The figures show that the trough extends further to the west at cross sections Ch1420 and Ch1780 (around 200m) compared to Ch1620 (less than 100m) and this can be explained by the topography and geology at each section. Sections Ch1420 and Ch1780 have the Oakley Creek flood plain to the west of the portal cut and this area will be affected by drawdown and settlement to a greater amount. To the west of section Ch1620 is the adjacent foothill and a deep cut through the overlying soils for the Oakley Creek diversion. The removal of the soil required for this diversion cut significantly reduces the settlement in this zone.

5.2.3.5 Sector 9 – Richardson Road Cut

The excavation for the Richardson Road overbridge is considered to result in negligible settlements from groundwater drawdown. The piezometers in the surrounding area indicate that for the alignment adjacent to the Modern Chairs building the regional groundwater levels are within the underlying ECBF rock. This means that the soils above will not settle as they are already "dewatered". Dewatering the rock is not expected to result in measureable settlement.

5.2.4 Combined Settlement

The settlements estimated from the above sources were combined to allow the assessment of their effects. The combined settlements are presented in Figures E-1 to E-13, in Appendix E, which also show the constituent settlements. The mechanical tunnel settlements shown in these results are those for an ECBF rock stress ratio of 1.0.

The estimated total settlements for the tunnel and portal alignment have also been plotted in plan in Figure E-14 (Appendix E). This figure presents contours of maximum estimated total settlement at each of the cross sections analysed along with interpolated results for the areas in-between.

The results for Ch1860 in Figure E-14 have been manually modified to account for the three dimensional effects from the nearby southern tunnel portal (Ch1790), which has a drained floor. This drainage source will increase the drawdowns to the north and therefore the settlements. The results were modified by considering the extent and magnitude of this additional drawdown and the local geology.

The magnitude and extent of the total settlement in Figures E-1 through E-14 generally follow the trends of the groundwater settlement results. The mechanical settlements from the driven tunnels increase the magnitude in the zone directly above the tunnels and the groundwater settlements control the extent. At the portals, the mechanical settlements generally significantly affect the area within a few tens of metres of the wall and the groundwater settlements dominate further afield.

The groundwater settlement effects for the Carrington Road overbridge wall, the northern portal open cut walls and the Richardson Road overbridge wall are considered to be negligible, so only the mechanical settlements have been included at these locations.

5.3 Assessment of Effects

5.3.1 Overview

The following sections present the assessment of effects from the estimated settlements described above. The assessments are presented for each of the main features described in Section 3 of the report, namely typical and specific buildings, services, transportation infrastructure, existing landfills and Oakley Creek.

5.3.2 Effects on Buildings

A contour plan is presented in Appendix G (Figures G-1 to G-4) which indicates the damage categories derived as a result of the settlements estimated to occur on the Project. When read in conjunction with Table 4.5 in Section 4.7.1, it is possible to establish the approximate nature and scale of any resulting effects. Each of the buildings that are shown to have a greater than "negligible" damage category is listed in Table 5.7 and the effects on these buildings are discussed in the following sections.

Table 5.7 – Buildings in a zone with a greater than "negligible" building damage category

Road Name	Property Number	Damage Category
Great North Road	1437*	2
	1439*	3
	1441*	3
	1443*	3
	1445*	3
	1449*	3
	1467*	4
	1471*	4
	1479*	3
	1481*	4
	1510	2
	1550	2
	1552	2
	1578	2
	1590	2
	1590A	3
Oakley Avenue	2*	3
Waterview Downs	8	2
	10	2
	12	2
	14	2
	16	3
	18	3
	20	3
	22	3
	26	3
	28	3
	30	3
	32	3

*Properties within the designation

Road Name	Property Number	Damage Category
Hendon Avenue	73	1
	75	1
	77	2
	79	2
	81	2
	83	2
	85	2
	87	2
	103A	1

*Properties within the designation

5.3.2.1 Dwelling Type 1 – Masonry Construction/Brittle Clad

The assessment predicts there will be "negligible" effects on the vast majority of buildings in the Study Area. In a limited number of areas, "more than negligible" effects have been predicted, however, as far as buildings are concerned, these are typically either "very slight" or at worst "moderate".

It is recommended that all Type 1 Dwellings that have been assessed to be subject to "more than negligible" effects be inspected prior to construction commencing to identify any pre-existing defects or sensitive features. These buildings are also proposed to be inspected periodically during the critical phases of construction.

Any Type 1 Dwellings assessed to be subject to "slight" or greater effects are proposed to be inspected as above and also be the subject of settlement and/or wall inclination monitoring.

For a more detailed description of the monitoring regime proposed, refer to Section 6 – Monitoring and Mitigation.

5.3.2.2 Dwelling Type 2 – Timber Construction/Flexible Clad

As noted in Section 4.7.1, the damage category plan provided has been derived assuming masonry or brittle construction. Accordingly, it is expected that Type 2 Dwellings will be subject to a reduced risk of visible effects. In general terms, a Type 2 Dwelling falling within a zone assessed to be subject to "very slight" effects is expected to exhibit "negligible" effects in terms of the descriptions in Table 4.5.

Notwithstanding, it is recommended that all Type 2 Dwellings falling within a zone assessed to be subject to "more than negligible" effects be inspected prior to construction commencing to identify any pre-existing defects or sensitive features. Type 2 Dwellings falling within a zone of "slight" effects are to be inspected periodically during the critical phases of construction.

5.3.2.3 Specific Building 1 – Unitec Building 76

The modelling predicts ground settlement in the vicinity of this building, however, the settlement is estimated to be relatively uniform across this site. An assessment at the location of this building suggests a settlement profile or “slope” of the ground surface in the order of 1:4000, without significant hogging or sagging. This is a low level of movement and hence “negligible” effects are predicted. However, as this building is substantial in size, historic in nature, and will be sensitive to any significant tensile strains, a monitoring programme has been recommended. Refer to Section 6 – Monitoring and Mitigation.

5.3.2.4 Specific Building 2 – 1510 Great North Rd, Unitec Residential Flats

Due to the combination of piled foundations and pad foundations this building is likely to experience some differential settlement due to the piled areas becoming ‘hard spots’ as the rest of the structure settles.

When the construction method of this building is taken into account, the assessment undertaken predicts this building may be subject to effects generally falling within the “slight” category. Accordingly it is recommended this building be the subject of a monitoring programme. Notwithstanding this, damage occurring in this building is expected to be non-structural in nature and repairable, hence this study envisages the building will remain able to be occupied throughout the Project.

5.3.2.5 Specific Building 3 – Pak’n’Save Supermarket

The investigations undertaken to date indicate this building is founded on shallow footings and hence the building complex is expected to generally follow the ground settlement profile.

A damage category of “negligible” has been assessed at this location on the basis of a masonry or brittle construction type. Accordingly, no significant visible effects are expected. However, due to the size and complexity of the complex, it is recommended that a monitoring programme be adopted.

5.3.2.6 Specific Building 4 – BP Petrol Station

The proximity of the works to the BP Petrol Station building and its associated fuel tanks on Great North Road has been considered. The assessment has shown that the area concerned is not subject to significant ground movements, and therefore the risk is considered to be negligible. However, due to the sensitive nature of the materials stored on the site, it is recommended that a monitoring programme be adopted.

5.3.2.7 Waterview Downs

Eight houses within Waterview Downs have been identified as falling within damage category 3 and four within damage category 2. These houses are understood to be of timber framed construction with a mixture of stucco and weatherboard cladding. The weatherboard clad buildings are considered to be Type 2 “flexible” Dwellings in terms of this assessment while the stucco clad houses are Type 1 (refer Section 3.2).

As noted in Section 4.7.1, the categorisation system and description of effects presented in Table 4.5 is relevant to the more brittle Type 1 Dwellings. The assessment methodology adopted is also recognised as being conservative, and is primarily used to identify those buildings where additional study or monitoring is warranted. Therefore, it is considered that the actual effects on the Type 2 Waterview Downs houses are likely to be no more than those in damage category 2 (slight) of Table 4.5.

While the Type 1 dwellings may be subject to damage closer to that in damage category 3, this will largely affect the more brittle stucco cladding rather than the structural elements, as the timber frame will remain more flexible. Damage to the cladding can be mitigated by regular inspection and crack sealing to maintain weathertightness, as required. Any damage to the cladding can then be permanently repaired at the completion of the Project.

5.3.2.8 Carrington and Richardson Rd Retaining Walls

The expected ground surface movement due to the horizontal displacement (and any draw-down effect) of these walls has been calculated. The calculations indicate total displacements immediately behind the wall in the 5mm range, decreasing significantly further behind the wall. At the nearest building line, the predicted settlements are such that negligible effects are expected.

5.3.2.9 Properties Within the Designation

The assessment has shown that some of the properties along Great North Road from Alford Street northwards fall in to damage category 3 and greater. These buildings are within the proposed designation for the Project. Those buildings that are to remain in place, will require ongoing detailed assessments and monitoring to establish the extent of the effects and any mitigation required.

5.3.3 Effects on Services

Settlement can affect services due to the change in grade and horizontal strain (i.e. elongation). The ability of a service to withstand this change in grade and strain is highly dependant on the materials it is constructed from, the type of joints used to join components of the service together, and the general working condition of the service.

In the assessment of the effects of settlement on the existing services, it has been assumed that the estimated settlements will have negligible effects on cabled and electrical services. Assessments have concentrated on pressure and gravity services, as these are typically more susceptible to damage. Discussions are ongoing with the service providers regarding the existing condition of their assets, their ability to tolerate the predicted settlement values and monitoring and mitigation options.

The physical construction and excavation proposed at the cut and cover tunnel and north and south portals will require relocation and/or active protection of many services, regardless of the estimated settlement effects. The locations affected by this are detailed in the following sections.

The predicted settlement zones are shown on Figures F-1 to F-7 (Appendix F). The assessed effects on each of the main services identified previously are presented below.

5.3.3.1 Watercare Orakei No. 9 Trunk Sewer

This Watercare sewer weaves along the length of the driven tunnel alignment. The sewer passes through the zone of higher predicted settlement at approximately Ch3500, adjacent to Waterview Downs. At this location the sewer is a 750mm diameter brick sewer with a concrete base (based on as-built information and from discussions with Watercare). The analyses estimate a change in grade in this area of up to 1:500. This is considered to have potential to result in minor effects on the sewer, such as slight cracking of concrete and mortar, but is not anticipated to cause disruption to its operation. Along the remainder of the sewer alignment, estimated change in grades due to settlement are typically flatter than 1:1,000 and are expected to cause negligible to no damage.

5.3.3.2 Metrowater 550x900mm Sewer

This sewer runs from Waterview, across Great North Road and connects to the Watercare Orakei No. 9 sewer at approximately Ch3750, near the northern tunnel portals. It is understood to be an egg-shaped brick sewer (from discussions with Metrowater). Estimated changes in grade imposed on the sewer due to settlement range from 1:2,000 to 1:10,000. Based on this the effects from settlement on the sewer are anticipated to be minor, comprising minor cracking and slight damage to mortar.

5.3.3.3 Great North Road Services

The services along Great North Road include sewer, water, stormwater, gas, power and communications. At approximately Ch3900 some services pass through an area with settlement gradients as steep as 1:300. However, damage is expected to be negligible as the services run approximately perpendicular to the settlement gradients (i.e. the actual gradients along the line of the services are much flatter). In addition, many of these services may need to be relocated to allow the cut and cover tunnel to be physically constructed.

5.3.3.4 Albie Turner Field Services

Albie Turner Field contains stormwater and sewer networks that discharge to Oakley Creek and the Watercare Orakei No. 9 sewer, respectively. The sewer is of glazed earthenware construction and is predicted to encounter changes in grade of up to 1:500. This change in grade is not expected to cause damage to the sewer; however there is potential for the formation of a low point in the sewer at Ch3350. Additional as-built information is being collated in the area to more accurately assess the risk.

The stormwater network, of mixed construction including reinforced concrete and PVC, is estimated to encounter settlement gradients of up to 1:500. The network is not expected to experience any damage but further as-built information is required for a firm conclusion to be reached. Some leads from field catchpits may form low points and require raising.

The settlements and their effects on the networks in this area will be monitored during the construction and mitigation measures can be carried out if required

5.3.3.5 New North Road Services Including Watercare Huia No. 2 Watermain

Services along New North Road include sewer, stormwater, gas, power, communications, local watermains and the Watercare Huia No. 2 watermain (1300mm diameter cement-lined steel pipe). Estimated settlement gradients along New North Road vary up to 1:1300. This value is not expected to cause any damage to services in the area.

5.3.3.6 South Tunnel Portal Services

Services in the vicinity of the southern tunnel portal include water, stormwater and sewer. The majority of these services will be relocated as part of the proposed above ground works for the construction of the portal, tunnel and the proposed railway corridor. The predicted settlement gradients are therefore expected to have no adverse effects on the services in this area.

5.3.4 Effects on Transportation Infrastructure

5.3.4.1 Roads

The assessment of effects indicates that the changes in road gradients as a result of the estimated settlement will typically be negligible with almost all changes less than 1 in 1,000. The only road where this very low gradient change is exceeded is the central section of Waterview Downs, where gradient changes of up to 1 in 800 are estimated. At this location, the road has an existing gradient of around 1 in 6 and falls in the same direction as the gradient change from the settlements (i.e. the settlements will make the gradients marginally steeper). Therefore the effect on the already steep road is also assessed to be negligible. These changes are expected to occur as a smooth profile over the existing roads with no sharp changes in grade.

5.3.4.2 Rail

The estimated settlement along the railway line will result in changes to the existing gradient of no greater than 1 in 1,000, and typically less than this. These changes are expected to occur as a smooth profile over the existing track with no sharp changes in grade. Therefore, the settlement effect on the railway line is considered to be negligible.

5.3.5 Effects on Other Features

5.3.5.1 Oakley Creek

The settlements also result in some changes to the Oakley Creek streambed levels and therefore the water velocities. The effects of this settlement have are described in detail in Technical report no. G.6 Assessment of Freshwater Ecological Effects, and are summarised below.

The total settlement contours in Appendix E were used to determine the change in streambed profile along Oakley Creek. The section of greatest effect was assessed to be between Ch3400 and Ch3785, where the streambed slope may change by up to 7%. This maximum slope change was then used in a computer model to assess the effect on the stream velocity for a range of flowrates (1 to 9m^3/sec) and the results indicate potential changes in velocity of between 1.8% and 4.4% (depending on flowrates). As the change in average velocity will be less than 0.1m/s, which is considered to be minor, no significant effect on the stream ecology is anticipated.

5.3.5.2 Landfills

Alan Wood Reserve

Each of the landfill pockets in the vicinity of the portal area within Alan Wood Reserve will be subject to the relevant estimated settlements. Due to the discreet nature of the landfill material, settlement of these areas is expected to be relatively uniform and so is not considered to have any detrimental effect.

Phyllis Reserve

The differential settlement across Phyllis Reserve landfill is calculated to be in the region of 300mm with total settlements being assessed as approximately 120mm at the western limit of the landfill, 400mm at the top of the slope above Oakley Creek and 410mm adjacent to the creek. These are considered to be upper bound estimates as the groundwater drawdown will not be at the worst case level for a long period of time and total settlement is time-dependant.

The settlement profile adjacent to the proposed tunnels will be gradual rather than showing sudden increases. The settlement gradient across the landfill is calculated to be approximately 1 vertical in 250 horizontal from west to east and typically in the region of 1 vertical in 500 horizontal from south to north (due to variations in geology and the location of the landfill relative to the tunnels). These gradients and the calculated total settlements across the landfill are not considered to be large enough to compromise the integrity of the surface capping of the landfill or decrease the stability of the landfill slope. There is no lining or leachate or gas collection system to be affected by this order of settlement.

Harbutt Reserve

Due to tunnel depth and underlying geology, the predicted settlements of 100mm at Harbutt Reserve are much less than at Phyllis Reserve. Consequently, they have been assessed as having negligible effect on the landfill stability.

6. Monitoring and Mitigation

6.1 Monitoring

An important part of the Project will be to monitor the actual ground settlement to both compare with the estimated values and to confirm the effects on buildings and other structures. Monitoring will need to occur before the tunnel construction starts, during construction and following completion to provide a comprehensive assessment of the effects. The following sections present the proposed settlement monitoring procedures for this Project. Details of the monitoring are also included in the appended Settlement Effects Monitoring Plan (Appendix H).

6.1.1 Survey of General Monitoring Points

A series of survey marks will be installed and regularly monitored to provide information to compare to the settlement estimates. The marks will radiate out from the tunnel alignment and be placed, as far as is practical, to match with the cross sections that have been used for the settlement estimates. The number of marks at each cross section will depend on the location of buildings or other features relative to the section (i.e. where there are more buildings there will be more regular marks and where there is open land the spacing of marks may be increased) and access to those locations for surveying. The marks will generally extend out to a maximum distance of around 400m either side of the tunnel alignment as the analyses indicate very little to no settlement at this distance. In the vicinity of Ch3400, they will extend out further to the east to cover this more extensive zone of estimated settlement. In the more critical areas the marks may be duplicated to provide some redundancy in the case of an individual survey marker being damaged.

In addition to the above, survey monitoring marks will be placed on or around buildings or features that are considered to be particularly sensitive. This will include surface marks for shallow services and marks on manholes to assess the effects on deeper services. The number and layout of these marks will be specific to each building or feature.

If required, a series of datum points will be established for the later surveys. These will be located well outside the area expected to be affected by the settlement and will be well protected.

Two hierarchies of monitored survey marks will be used in the Project. A number of framework marks will be installed along each section (typically 4-6) and in other locations considered appropriate and these will serve as the main monitoring points. A greater number of intermediate marks will be installed between and around the framework marks to provide additional detail when required. The monitoring associated with each of these marker types is discussed below.

The framework marks will be installed initially and monitored for vertical and horizontal movement with four sets of baseline values taken during the 12 months prior to tunnel construction commencing. The four sets comprise the initial installation survey and the three subsequent monitoring rounds and would take place on a quarterly basis.

The intermediate marks will be installed prior to the start of the construction and monitored once to provide vertical and horizontal baseline values.

The ongoing frequency of monitoring will then vary depending on the stage of construction. At the start of the Project construction, each framework mark will be monitored for vertical movement on a monthly basis, with selected framework marks monitored for horizontal movement also on a monthly basis. The selected framework marks will be those located closer to the alignment of the tunnels and therefore more likely to experience horizontal displacements.

As the active construction stage starts to affect the relevant section, all marks (framework and intermediate) will be monitored weekly for vertical movement. Horizontal movement will continue to be monitored monthly at selected framework marks. For this Project, “active construction” can be defined as:

- Starting when the advancing tunnel face comes within 150m and ending when the final tunnel lining has been installed 150m beyond the section, and

- Starting when excavation in front of a retaining wall comes within 100m of a section (i.e. the wall coming under load) and ending when the permanent wall supports are in place beyond a distance of 100m.

Once the active construction for each section is complete, the monitoring can then reduce to the pre-active construction frequency (i.e. monthly monitoring of all framework marks vertically and selected framework marks horizontally), if the results indicate that the settlements and effects are within a satisfactory range. Following a six month period of this monthly monitoring and if results indicate that the settlements and effects are still within a satisfactory range, then the framework marks will be monitored on a six monthly basis for an additional period of at least 2 years.

If the monitoring results indicate that movements are outside the expected range, or if there are other reasons for concern, then the monitoring frequency and/or extent can be increased to cover those areas of concern. For example, the monthly monitoring of framework marks pre and post active construction could be increased to weekly and/or the intermediate marks could be monitored as well. Other combinations of frequency, marker types and vertical/horizontal monitoring can also be used to address a specific concern.

The following summarises this recommended survey monitoring regime:

Pre-construction

i) Framework Markers - Horizontal and vertical at quarterly intervals, starting at least 12 months prior to construction commencing.

ii) Intermediate Markers - Horizontal and vertical once.

During Construction

i) Framework Markers - Vertical on a monthly basis.

ii) Selected Framework Markers only - Horizontal on a monthly basis.

During Active Construction

i) Framework and Intermediate Markers – Vertical on a weekly basis.

ii) Selected Framework Markers only - Horizontal on a monthly basis.

6.1.2 Building Condition Assessments

Individual structural condition assessments will be carried out on buildings:

- where the property is within the Substrata Designation;
- where total settlements are estimated to be greater than 50mm;
- in any zone assessed to have greater than "negligible" risk of damage; and
- for any others specifically identified as requiring particular attention. These specifically identified buildings are currently:
    - Unitec Building 76;
    - 1510 Great North Rd, Unitec Residential Flats (two buildings);
    - Pak'n'Save Supermarket;
    - Metro Football Clubhouse, Phyllis St;
    - Building at 1550 Great North Rd;
    - BP Service Station at 1380 Great North Rd (building and tanks);
    - Modern Chairs Building (Richardson Rd); and
    - Waterview Primary School.

The initial assessment will comprise an inspection of each building and significant structure on the property to establish and record its condition. Each assessment will produce a written description including photographs of any existing damage and a copy of this report will be provided to the owner. These assessments will be carried out prior to the commencement of the tunnel excavation, retaining wall construction and any dewatering associated with those activities to provide a baseline of the condition of each building.

In addition monthly visual assessments of the following buildings will be carried out during the "active construction" phase of the Project ("active construction" is defined above).

- All Type 1 Dwellings within a zone where "more than negligible" effects have been predicted;
- All Type 2 Dwellings within a zone where "slight" effects or greater have been predicted;
- Unitec Building 76;
- 1510 Great North Rd, Unitec Residential Flats (two buildings); and

- Pak'n'Save Supermarket.

The purpose of the assessment will be to look for any evidence of effects, with reference to the initial condition (baseline) survey. If mitigation is required, possible options for action are set out in Section 6.2 below.

Assessments of other buildings, or on a more frequent basis would also be carried out if the monitoring indicated that there may be significant settlement effects or potentially at the request of a building owner. All inspections would be subject to the approval of the owner to enter their property.

It is also proposed that the following dwelling types and specific buildings be the subject of level and/or wall inclination surveys on a monthly basis during the "active construction" phase of the Project.

- All Type 1 Dwellings within a zone where "slight" effects or greater have been predicted;
- Unitec Building 76; and
- 1510 Great North Rd, Unitec Residential Flats (two buildings).

The purpose of the level or wall inclination survey will be to provide a basis for evaluating the rate of any movement and to enable a correlation with the visual survey. If mitigation is required, possible options for action are overviewed in Section 6.2 below.

6.1.3 Retaining Wall Monitoring

The retaining walls for the portals and the cut and cover tunnel will be specifically monitored for movement, in addition to the settlement survey monitoring detailed above. The walls will be monitored using inclinometers and surface survey to determine actual displacements during and post construction. These values will be compared to the estimated values and if the results indicate movements beyond those anticipated, a series of actions will be carried out, depending on the nature of the result.

The final locations for the instrumentation and trigger levels for actions will be determined during final design when the retaining wall details are confirmed.

At the southern portal, the effectiveness of the grout curtain will need to be monitored by means of piezometers either side of the curtain. The grout curtain will be installed in 3 stages, with primary, secondary and tertiary stages of grouting. At each stage the effectiveness of the grouting will be assessed in accordance with the drawdown anticipated from the design, and the amount of grouting adjusted accordingly. Areas where the basalt is more fractured, or contains more voids will require more grout than in areas where the basalt is more massive and contains less joints and voids.

6.1.4 Services Monitoring

In addition to the survey marks monitoring described above, CCTV inspections of some services will be carried out to assess the effects of the settlement. For services identified as being susceptible to damage or particularly critical, an initial preconstruction CCTV inspection will be carried out to provide a baseline for assessing any future damage. As the construction progresses, additional CCTV inspections may be carried out depending on the results of the survey monitoring and feedback from the service providers.

For the Watercare Orakei No. 9 trunk sewer, regular CCTV inspections specific to this service will be required due to its criticality, depth and susceptibility to damage. The depth makes surface monitoring less applicable to the service and as a brick lined structure it is less tolerant of movement than other services.

6.1.5 Assessment and Reporting

Preconstruction monitoring will be carried out as described above and reported following the collection of the final set of data, prior to the start of construction. This data will be factual in nature, with assessment only required for any anomalous results. The report will form part of the input for the construction phase assessments.

The monitoring during the construction is anticipated to be the responsibility of an independent monitoring team contracted directly to the NZTA. They will continue to provide this service to the NZTA in the post construction phase.

The monitoring data will be processed and compared to the design analyses. Once construction starts, the data will be used to reassess the building damage categories along each of the cross sections and these categories will then be compared to the results in this report. The effects on services will also be assessed from the settlement gradients. If this reassessment indicates that the damage category has increased by a significant amount then additional analyses or more frequent monitoring may be required and the affected buildings identified for potential mitigation work. Similarly, an increase in estimated effects on the services will require additional review and potentially amended monitoring and mitigation. Consideration may also need to be given to modifying the construction approach to reduce ground settlements, if rapid movements occur in response to ground excavation.

Reporting will be determined by the stage of construction and actual results. During the active construction stage it is anticipated that initial internal review of the monitoring results will take place shortly after receipt of the processed data. As long as the results show there are no significant anomalies or assessed significant increased risk to buildings, these monitoring results would be passed on to other parties on a monthly basis. If there are any significant anomalies or assessed significant increased risk to buildings, then following a more detailed review of the data, those parties would be notified and mitigation measures agreed. The results of this more detailed work and the outcomes and way forward would then be subject to review.

The post active construction stage results (monthly and six monthly) will be reviewed and reported shortly after receipt of the processed data. Where any significant anomalies or assessed significant increased risk to buildings occurs, then the reporting would follow the process for the active construction stage described above.

6.2 Mitigation

The following sections present details of potential mitigation measures available for the construction types and the affected features. A variety of measures are available and the most appropriate measure will be determined for each specific case should it be required. Details of the mitigation are also included in the appended Settlement Effects Monitoring Plan (Appendix H).

6.2.1 Retaining Walls

Where the monitoring results indicate that the settlement effects are greater than those estimated, and an intervention is required, the following mitigation measures are available to be implemented as considered necessary. It is anticipated that the mitigation measures would be installed in a staged approach with those measures which would provide the most efficient solution implemented first and additional measures implemented if those initial measures do not resolve the situation. The number and extent of measures taken at each stage would need to be assessed during design and construction as appropriate and specific for each site.

Groundwater drawdown contingency measures

The diaphragm and secant pile walls are intended to be essentially constructed as water-tight structures. Due to the depth of the secant piling there is a risk that the piles may "wander" off alignment during construction and that gaps will be exposed between the piles which will increase the "leakiness" of the wall. In addition, seepage is expected through the base of the excavation. In the event of groundwater drawdown exceeding the anticipated levels, the following actions may be taken:

- Install additional grout holes along the grout curtain to reduce inflow through the basalt at the southern portal;
- Grouting between and behind the secant pile wall in areas of excessive seepage;
- Seal any fracturing/jointing of the ECBF rock at the base of the excavation that shows signs of excessive seepage; and
- Re-inject water through boreholes back into surrounding ground or the basalt aquifer to mitigate the change in groundwater level (groundwater recharge).

Excessive Retaining wall deflection contingency measures

If the retaining wall deflections exceed the anticipated limits, a review of the retaining design model will be carried out to assess the increased load in the piles and existing props. If required, the following actions may be taken:

- Remove surcharges close to the wall;
- Place a berm of soil in front of the wall; and

- Install additional or stronger/stiffer props or ground anchors.

6.2.2 Driven Tunnels

There is a series of mitigation measures available should the monitoring results indicate that settlement effects are greater than those anticipated. The number and extent of measures taken at each stage would need to be assessed during construction as appropriate and specific for each site. These measures include:

Groundwater drawdown contingency measures

The tunnels are intended to be excavated through a relatively low permeability rock and any areas of higher permeability could result in larger drawdowns than anticipated and therefore settlements. During excavation the rock conditions will be monitored and if areas of excessive groundwater inflows are encountered or expected, then the following actions may be taken:

- Grouting of the rock mass in front of the excavation face to reduce inflows; and
- Re-inject water through boreholes back into the ground above to mitigate the change in groundwater level (groundwater recharge).

In addition, any areas of the lining that show excessive seepage will be remediated with additional shotcrete (of temporary liner only) or grouting up of the rock mass behind the seepage area.

Excessive deflection (convergence) contingency measures

If the tunnel wall deflections exceed the anticipated limits, additional temporary support will be added to reduce those deflections. These temporary support options include:

- Additional rock bolts;
- Additional shotcrete; and
- Reduced advance length (i.e. extent of unsupported excavation).

6.2.3 Buildings

6.2.3.1 Mitigation – Non-Structural Effects

If damage results from the tunnelling works, then general repairs may be required, such as repointing, repainting, and redecorating. In severe cases, repairs may require some partial rebuilding work, although this is considered highly unlikely. The timing of such repairs would depend on the stage of the tunnel construction, the owner's wishes and the degree of damage.

Depending on the nature and severity of the damage, a more regular monitoring regime may be implemented for the subject building, and this may extend to settlement surveying if appropriate.

6.2.3.2 Mitigation – Structural Effects

To date the assessment has not identified any buildings beyond the designation that are situated in a zone where "severe" or greater effects are predicted. However, if any effects of a structural nature are identified during the course of the monitoring programme, then a detailed evaluation will be required by a Structural Engineer. Any recommendations for repair and/or an increased level of monitoring arising from this evaluation will then be implemented.

In extreme cases where a local repair or re-construction is not sufficient, additional measures such as underpinning or strengthening may be required (although no extreme cases are envisaged on this Project).

6.2.4 Services

If initial settlement monitoring indicates damage to a service may have occurred, investigation of the area and affected services will occur promptly. This will include a detailed examination of the site, coordination with the relevant service providers to ascertain what effects their network is experiencing, and an assessment of what remedial action is required. Any remedial works will be carried out as soon as possible. If the investigation reveals no sign of immediate damage, the services will continue to be monitored closely until all parties are satisfied no damage has occurred.

If damage to the service has occurred, numerous methods are available to mitigate the damage. The final procedure employed would depend on the type of service, location and severity of the damage and would be subject to the agreement of the service provider. Possible options include:

- Diversion through another nearby service and abandon the original service line (either temporarily or permanently). This would only occur if the nearby service has the required capacity. Another variation of this option would be to temporarily divert a service through an above ground line until the original is repaired.

- Crack injection. Where access is available and the movement has effectively ceased, cracks in pipes may be repaired by injecting an appropriate product to fill the crack and provide structural continuity.

- Lining of a pipeline. A damaged pipeline could be repaired by placing a new liner on the inside of the pipe to prevent leakage and provide the structural strength lost from the damaged pipe. There are several options for lining including slip lining and Cured in Place Pipe.

- Support measures. For some large diameter pipes a mitigation option may be to underpin or otherwise support the pipe if settlements are considered too great.

- Replacement of the service. In cases of severe damage an entire service line (or part of it) may be completely replaced with new materials.

6.2.5 Transportation Infrastructure

While the assessment indicates that there will be no significant effects on the transportation infrastructure, there are mitigation options available should the actual effects differ. For roads, these options include overlaying the road surface to raise it, reconstructing kerb and channels and footpaths if they suffer from adverse differential settlement or changes in gradient, and additional stormwater catch pits and associated piping if water is found to be ponding in new areas.

For rail, the primary mitigation option would be to lift and re-ballast the tracks to bring them back to the desired level, if this is required.

6.2.6 Other Features

6.2.6.1 Oakley Creek

The assessment indicates that no significant effect is anticipated on the Creek or its ecology. However, if the settlement is significantly greater than estimated mitigation options for the Creek could include construction of artificial structures to reduce the velocity or recontouring of the streambed.

6.2.6.2 Landfills

Should any adverse settlements affect the landfills, mitigation options include:

- Reworking/reconstruction of the “clay cap” to minimise surface water infiltration. This could involve reconstruction of any cracks and/or recontouring to remove low spots that could pond water.

- Additional stability works if the settlement was considered to be causing instability in the landfills. Typical retaining wall options could be used to improve the stability in this scenario.

7. Summary and Conclusions

This report presents the results of an assessment of the potential magnitude and effects of ground settlements due to the construction and operation of the proposed SH20 tunnel and associated retaining walls at either end. These settlements are generated by three separate sources: mechanical settlement of the ground due to the physical excavation of the material for the driven tunnel; mechanical settlement of the ground due to the physical movement of the retaining walls; and consolidation of the ground due to the extraction of groundwater.

The area in which the settlement will occur is the zone above the proposed alignment extending out several hundred metres either side. In broad terms the area is predominantly residential in nature with the Unitec campus at the northern end and a few commercial buildings. The buildings are typical of older residential areas of Auckland, and there are some historic structures within the Unitec campus.

Services are again typical of a suburban location, although some major sewers and a large watermain cross through the area. The western railway line crosses the alignment and there are two main roads (New North and Great North Roads) and many smaller residential roads within the area of interest. Other features of interest include Oakley Creek and several historic landfills in reserve areas near the creek.

Geotechnical parameters used in this assessment are typically those recommended in Technical Report G.24 Geotechnical Interpretive Report, and any variations to those are detailed. The settlements from the three sources are derived separately and then combined. The driven tunnel mechanical settlements are calculated using the computer program Phase^2, which allows the user to model the construction and support methodology for the excavated tunnel. The tunnel is proposed to be constructed in three headings with rock bolting and shotcrete for temporary support and a full concrete liner as the permanent support mechanism.

The retaining wall mechanical settlements have typically been calculated using the computer program FLAC, supplemented with other programs and empirical or published hand methods for specific cases. The modelling has taken into account the proposed construction methodology and the temporary and permanent props and anchors that will be used to support these walls. The groundwater consolidation settlements have been derived using the groundwater drawdown values described in Technical Report G.7 Assessment of Groundwater Effects and the linear (m_v) method of calculating settlement.

The above settlements have all been calculated at a consistent series of cross sections along the alignment. These cross sections were selected to provide representative examples of the relevant geology, hydrogeology and construction types proposed, while retaining a good coverage of the entire alignment. The settlements at each section have been assessed at critical stages in the Project and combined to produce total settlement results for those critical stages.

For the driven tunnel there are two critical stages, one when the first tunnel has been excavated and shotcreted ("day 13") and the other when the second tunnel has been excavated and shotcreted ("day 43"). The first stage produces the greatest effects in the surface zone between the two tunnels and the second stage for the remainder of the area. The portals and other retaining walls have only a single critical stage, which is the long term when all the settlements are complete.

The critical part of the assessment is to estimate what effects those settlements will have on the existing environment. The effects on buildings were assessed using an internationally accepted method, specifically prepared for tunnel construction work (Burland, 1997). The method determines the curvature and horizontal strain in a building and plots these values against a series of criteria to assess the likely effect on the structure. The classification of potential effects (damage category) is described in the report. This method has been derived for unreinforced masonry buildings and so can be considered conservative for timber framed and reinforced concrete structures. The effects on the local services and transportation infrastructure were assessed by calculating the change in their gradient as a result of the settlement and then determining whether that could cause damage to each item being assessed. The effects on Oakley Creek and the old landfills were assessed by considering both the total settlement and the changes in gradient.

The magnitude and extent of each settlement source and the combined values are presented in the report, along with a contour map of total settlements. These follow the anticipated trough shape along the alignment, with the greatest settlements occurring over the tunnel alignment and then reducing as they extend out several hundred metres each side. The greatest magnitude and extent of estimated settlements are around Ch3400, where a combination of a higher permeability rock (Parnell Grit) and a deeper zone of compressible soils occur. Settlements are less in the cut and cover tunnel area and also reduce in the southern portal as the excavation reduces in depth.

The effects of these settlements have been assessed using the methodology described above. The damage category was assessed and the results of this are plotted in Figures G-1 to G-4, appended to the report. In summary the assessment predicts there will be “negligible” effects on the vast majority of buildings in the Study Area. In a limited number of areas, “more than negligible” effects have been predicted, however, as far as buildings outside the proposed designation are concerned, these are typically either “very slight” or at worst “moderate”. A small number of buildings have been identified for specific monitoring based on their classification, their construction or status and these are listed in the report.

The settlement effects on the majority of services were determined to be negligible with potential for some minor effects on a few specific services. These will be subject to further assessment of their current status and ongoing consultation and preparation of a Memorandum of Understanding with each of the service providers. Many services in the southern and northern portal areas may need to be relocated as part of the physical wall construction and excavation work. The effect on transportation infrastructure has been assessed as negligible.

The estimated settlements have been assessed as having no significant effects on Oakley Creek or the landfills. The construction of the retaining walls at the Carrington Rd and Richardson Rd overbridges are assessed as having negligible effects on the nearby buildings.

The proposed monitoring regime is described in detail in the report. It comprises horizontal and vertical monitoring of both framework and intermediate survey marks located along the analysed cross sections and in other relevant locations. The frequency of this monitoring will vary depending on the proximity of each section to the active construction area and to the actual result of the survey.

Condition assessments will be carried out on buildings where the property is within the Strata Designation, where total settlements are estimated to be greater than 50mm, in any zone assessed to have greater than “negligible” risk of damage and for any others specifically identified as requiring particular attention. These assessments will be carried out prior to the start of construction and then again either during construction, at the end of construction or as required, depending on the nature of and assessed risk to the property.

The retaining walls and services will also be specifically monitored to allow comparison of the predicted values to those actually occurring in the field. This will include CCTV recordings for pipe services and inclinometers for retaining walls.

The monitoring will be carried out by an independent team, contracted directly to the NZTA. These results will be used to compare the actual damage categories with those estimated in this report and, if different, mitigation measures agreed. These may include amending the current monitoring regime or enacting specific mitigation measures.

While the current assessment indicates that only minor mitigation is required in isolated locations, more comprehensive mitigation measures are presented to cover the unlikely scenario of more significant damage than estimated occurring. For retaining walls these include grouting of leaks, water reinjection, additional props and berms in front of the walls. Measures for the driven tunnels include grouting of the rock, water reinjection, additional and rock bolts and shotcrete.

Building mitigation includes repair of non-structural defects once the settlement is complete and the immediate repair of any issues that are structural or will affect the weathertightness of the building. Services mitigation depends on the type of service and its construction, but includes crack repairs, diversion, relining, support and replacement. Road and rail settlements could be mitigated by relatively minor surface reconstruction methods and the landfills by physical retaining works if required. Details of the monitoring and mitigation are also included in the appended Settlement Effects Monitoring Plan (Appendix H).

The conclusions of the report are as follows:

- The effects from the conservatively estimated ground settlements caused by the tunnel construction are considered to be typically negligible, with isolated areas of very slight to moderate damage predicted for buildings outside the proposed designation.

- Monitoring should be carried out to confirm the above, to quantify any actual damage and to allow for early warning of areas where the settlement effects may be greater than that predicted.

- Mitigation measures are readily available for the predicted levels of damage and in the unlikely event that greater effects than predicted do occur.

8. References

- Burland, J.B. (1997), “Assessment of risk of damage to buildings due to tunnelling and excavation”, Earthquake Geotechnical Engineering, Ishihara (ed.), Balkema, Rotterdam, 1997.

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APPENDIX A – Sector and Cross-Section Location Plans

APPENDIX B – Portal and Retaining Wall Drawings

APPENDIX C – Driven Tunnel Mechanical Settlement Analyses and Outputs

APPENDIX D – Portal and Retaining Wall Settlement Analyses and Outputs

APPENDIX E – Combined Settlement Figures and Contour Plan

APPENDIX F – Effects on Services Settlement Plan

APPENDIX G – Effects on Buildings Plan

APPENDIX H – Settlement Effects Management Plan