Appendix Q

Assessment of Effects on Ground Settlement – Aurecon
St Mary’s Bay and Masefield Beach Improvement

Assessment of Settlement Effects

Auckland Council

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# Document control record

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## Approval

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1 Introduction

1.1 Overview

The St Marys Bay and Masefield Beach Water Quality Improvement Project is an Auckland Council (AC) project being undertaken to mitigate the frequent combined wastewater/stormwater overflows (CSOs) that occur at St Marys Bay and Masefield beaches, by providing CSO storage within a proposed new storage pipeline located from Point Erin Park to London/New Street. This pipeline will store the CSOs from the local catchment, and return these to Watercare’s Branch 5 sewer when this existing pipeline has capacity.

The project includes the construction of three shafts (one in Point Erin park, one in St Marys Road park, and one on the intersection of New Street / London Street) and the installation of the new storage pipeline by way of tunnelling. The main construction site will be located within the Point Erin Park, where the majority of spoil will be extracted, with some open trenching required along Curran Street and Sarsfield Street to connect Engineered Overflow Points (EOPs). Further trenching or jacking will be used to install the marine discharge pipeline across Curran St and past the seawall west of Curran Street, with the marine pipeline discharging via an outlet structure some 450m from shore to the west of the Auckland Harbour Bridge.

The project specifically involves the construction of a the 1km long storage pipeline with an internal diameter of 1.8m. To implement the proposed storage pipeline from Point Erin Park through St. Marys Bay to the corner of London/New St (as shown in Figure 1), a feasible construction methodology is required. The preferred construction methodology for this project is a pipejacking technique using a tunnel boring machine (TBM) on slurry or earth pressure balance machine (EPBM).

Figure 1-1: Location of St Marys Bay storage pipeline with shaft locations
In addition, a small pump station will be installed at the lower end of the storage pipeline at Point Erin Park that pumps to Branch 5 of the Orakei Main Sewer (OMS). The new marine discharge pipeline will discharge excess overflows from the storage pipeline via an overflow weir to the Waitemata Harbour west of the Auckland Harbour Bridge.

To facilitate the tunnel construction, the three vertical access shafts will be built along the length of the storage pipeline alignment. These shafts are expected to be installed at the start of the project and each will likely take 3-4 months to construct.

The tunnel is expected to be excavated from Point Erin in the west in an easterly direction to London / New Streets. The tunnel will be excavated and constructed using a tunnel boring machine (TBM). It is anticipated all excavated materials from the tunnelling will temporarily be stored at Point Erin Park to allow continuous progress of works. It is currently envisioned that the total duration of the construction works will be 24 months although the tunnel boring process itself is likely to take up to 6 months.

1.2 Purpose

The purpose of this report is to present the results of the surface settlement analysis due to the construction of the project; the subsequent assessment of the potential effects on adjacent buildings, utilities and infrastructure, and to propose monitoring and mitigation measures to address these potential effects (where required).

This report reviews the existing environment and the infrastructure which is envisaged to be affected by the construction of the project. This includes buildings and utilities within the extent of the settlement effects. A monitoring regime and potential mitigation measures are proposed to deal with these potential effects during construction (if required).

1.3 Report Structure

This Report is structured as follows:

- Description of the project.
- Explanation of the sources and the estimated extent of the potential settlement effects.
- Overview of the environment, including geological conditions, key inputs into the assessment, and existing key structures within the estimated extent of settlement effects.
- Results of the potential settlement effects and building damage assessment, including sensitivity analysis.
- Definition of the monitoring scheme and mitigation measures to be implemented to ensure the estimated settlement effects are confirmed and the potential effects on surrounding structures are controlled during construction.

1.4 Sources of Settlement Effects

There are three potential sources of settlement associated with the construction and operation of the project. These are:

- Mechanical settlement of the ground due to physical excavation of the material during pipe jacking and mining. This is caused by the removal of the supporting ground, convergence of the annulus void, and subsequent relaxation of the ground above. The settlement will occur relatively quickly following the pipe jacking excavation and will be concentrated above the pipeline alignment.
- Mechanical settlement of the ground due to the physical movement of the retaining walls supporting the shaft excavations and trenching. This is the result of the lateral movement of the retaining walls as they take 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 immediate area behind the retaining walls. The magnitude of influence is dependent on the wall stiffness and lateral support system.

- Consolidation of the ground due to the drawdown of the groundwater table (referred to as consolidation settlement). This is caused by the reduction in pore water pressure within the soil as the water seeps into an excavation e.g. through the retaining walls. It is time dependant and based on the location and permeability of the excavation at any one time.
2. Existing Environment

This section outlines the environment surrounding the project works. This information is essential when undertaking an assessment of the potential effects on surrounding structures.

2.1 Land Use

The land use above the ground surrounding the project is generally low-rise residential, parks and local roads in the areas of Point Erin and St Marys Bay. The most significant piece of infrastructure in the project area is the State Highway 1 Northern Motorway which is located just north of the project area. The Point Erin Pool is located on the hill immediately south of the Point Erin shaft.

2.2 Topography and Geology

The topography of the project area consists of low lying and reclaimed areas at St Marys Bay and Point Erin (which are both currently recreational park areas) as well as elevated areas to the south where residential housing and other buildings are located.

The pipe jack alignment traverses a variety of geological environments below and in front of the St Marys Bay hill cliffs, most of the pipe jacking will be passing rock varying from residual to unweathered East Coast Bays Formation (ECBF), although short sections of construction through fill and reclamation soils are expected in the vicinity of the shafts at Point Erin and at the St Marys Road park. The geological conditions have been established from a series of project-wide investigations (refer Geotechnical Assessment (Memo dated 16 March 2018) for further details).

The anticipated geological sequence at each shaft is as follows:

Point Erin Shaft
- Fill and hydraulic fill: Comprising both granular and cohesive soils.
- ECFB: East Coast Bays Formation (ECBF) present at approximately 4m depth generally comprising alternating siltstone and sandstone beds with a thin weathered layer.

St Marys Road Park Shaft
- Fill and hydraulic fill: Comprising both granular and cohesive soils.
- ECFB: East Coast Bays Formation (ECBF) present at approximately 3m depth generally comprising alternating siltstone and sandstone beds with a thin weathered layer.

London/New Streets Shaft
- ECFB: Weathering profile over 5m from residual soil to moderately weathered rock. Large depth of interbedded siltstones and sandstones.

2.3 Hydrogeology and Groundwater

The hydrogeology of the area can be split into two areas as summarised below:
• For the low elevation areas - groundwater is generally present at relatively shallow depth (<1-3m below ground level) or 2mRL to 3mRL. In these areas, groundwater is primarily in an ‘unconfined’ state i.e. a water table aquifer or phreatic surface.

• For the higher elevation cliff-top areas – groundwater is encountered at varying depths, ranging between approximately 0.5 m and 5 mBGL. However, the very shallow occurrences i.e. <2mBGL, are likely associated with zones of localised perched groundwater (perched groundwater = groundwater that sits above the regional water table/piezometric surface). Occurrences of perched groundwater are typically unconfined. Beneath any perched occurrences, a phreatic surface (water table aquifer) is typically present within the ECBF. Deeper within the unit, groundwater typically becomes confined or semi-confined due to the alternate layering of sandstone and siltstone bedding.

For further details refer to the Groundwater Effects Assessment that has been prepared by Pattle Delamore Partners Limited (PDP).

Table 2-1: Initial Groundwater Levels at the shaft locations

<table>
<thead>
<tr>
<th>Location</th>
<th>Pt Erin</th>
<th>St Marys Bay</th>
<th>London/New Streets</th>
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<tr>
<td>Initial Water Level (mRL)</td>
<td>1.5</td>
<td>2</td>
<td>21</td>
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2.4 Existing Buildings and Infrastructure

Adjacent structures to be considered in the settlement assessment are given in Table 2-2. Structures of particular concern are the Shelly Beach Road motorway off-ramp, the northern motorway itself and some local residential properties.

Table 2-2: Adjacent structures to be considered in the assessment

<table>
<thead>
<tr>
<th>Location</th>
<th>Potentially Effected Structures</th>
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<td>Pt Erin Shaft</td>
<td>Curran Street on-ramp, no adjacent buildings at this location</td>
</tr>
<tr>
<td>St Marys Bay Shaft</td>
<td>Northern motorway, no adjacent buildings at this location</td>
</tr>
<tr>
<td>London/New Street Shaft</td>
<td>Local streets and residential properties &lt;10m from shaft location</td>
</tr>
<tr>
<td>Pipe jack alignment</td>
<td>Passes immediately adjacent to the northern motorway and Shelly Beach Road off-ramp as well as many local streets and residential properties, albeit at significant depth</td>
</tr>
<tr>
<td>Curran and Sarsfield Street trenches</td>
<td>Curran and Sarsfield Streets, nearby residential properties expected to be outside the zone of settlement influence from the trenches</td>
</tr>
<tr>
<td>Marine outfall pipe</td>
<td>Curran Street and on-ramp, no adjacent buildings at this location. Underground valving structure owned by Watercare nearby.</td>
</tr>
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</table>
3 Technical Assessment Methodology

3.1 General Overview

The construction of the shafts, new storage pipeline and connections have the potential to induce surface, subsurface and lateral ground movements with resulting effects on nearby structures and their foundations. The mechanisms of surface settlement and ground movement will be discussed in more detail in the following sections.

Mechanical settlement caused by the construction of the project is expected to occur within several weeks from the start of construction and are therefore relatively instantaneous. Consolidation settlements, on the other hand, occur relatively slowly and may not be fully evident for years after completion of the project. It is usual that the effects of instantaneous settlement are more adverse than the long-term settlement. In the case of this project, investigations have been undertaken to determine the rate and extent of drawdown of the water tables and these have been used in the settlement analysis (refer Groundwater Effects Assessment by PDP).

The potential effect on buildings have been assessed based on an internationally accepted method specifically prepared for tunnel construction work (Burland, 1997). The process classifies the level of risk and potential damage to a particular building based on the estimated settlements and building structure.

It is important to note that damage classifications are based on some assumptions outlined by Burland (1997). Hence, detailed pre-construction condition surveys and construction monitoring are a vital part of the on-going process of limiting potential adverse effects of construction.

3.2 Derivation of Parameters

The geotechnical design parameters used in the assessment of settlement effects have been developed from the initial stages of ground investigation and the designers past experience with similar ground conditions and projects in the Auckland area. The preliminary geotechnical design parameters are presented in Table 3-1. Note that these are global parameters for the project and some revision of these is possible to suit the ground conditions specific to the structure being assessed.

Table 3-1: Preliminary geotechnical design parameters

<table>
<thead>
<tr>
<th>Unit</th>
<th>Typical Material Type</th>
<th>Bulk Unit Weight</th>
<th>Strength</th>
<th>Drained Stiffness</th>
<th>At-Rest Earth Pressure</th>
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<td></td>
<td>γ (kN/m³)</td>
<td>Su (kPa)</td>
<td>c' (kPa)</td>
<td>ϕ' (°)</td>
<td>E' (MPa)</td>
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<td>F</td>
<td>Varies</td>
<td>17</td>
<td>20±2z ¹)</td>
<td>2</td>
<td>28</td>
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<tr>
<td>Fh</td>
<td>Clayey sand</td>
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<td>-</td>
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<td>EW</td>
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<td>20</td>
<td>500</td>
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Notes:

¹) Where z is the depth below ground surface. Applicable for depths greater than 3m, take z = 0 when depths is less than 3m.
Geotechnical parameters used for undertaking sensitivity analysis are outlined in Section 3.3.5 of this Report. The sensitivity analysis has shown that the geotechnical parameters of the overlying soils (i.e. Tauranga Alluvium, and Fill) have some influence over the mechanical and consolidation settlements.

3.3 Settlement Estimates Methodology

The methodology used to estimate the settlements from the three sources described in Section 1.4 are discussed below:

1. Mechanical settlement due to pipe jacking – this settlement is calculated using the empirical equation as described in Section 0. The volume loss input is the sum of the face loss and convergence of the overcut from the pipe jacking operation. The convergence usually occurs in soft ground such as Tauranga Alluvium (referred to as geological unit TA). Where there is sufficient cover of unweathered East Coast Bays Formation (referred to as geological unit EU2) at the crown convergence is unlikely to occur.

2. Mechanical settlement due to shaft and trench excavation – this analysis is carried out using the finite element modelling software PLAXIS. PLAXIS is used to determine the movement of the ground, including vertical and horizontal displacements of the shaft walls. PLAXIS simulates the actual excavation and considers the stiffness of the soil and retention system (i.e. shaft piled wall). The software has the capability to extract settlement contours to be used for assessing the effects on the existing buildings. The modelling input to PLAXIS used in calculating the settlements is described in Section 3.3.2.

3. Consolidation settlement due to groundwater drawdown – the maximum groundwater drawdown has been determined according to the Groundwater Effects Assessment by PDP. The settlement has then been calculated using the one-dimensional consolidation theory, using the coefficient of volume compressibility (\(m_v\)) method. The \(m_v\) values (inverse of the stiffness, \(E'\)) for each soil type have been taken from the geotechnical parameter table above.

3.3.1 Settlement due to pipe jacking

Analysis Methodology

Settlements observed at the surface are typically in the form of a settlement trough, which commonly resembles a Gaussian normal distribution. The fundamental assumption in the Gaussian model is that the ground movements occurring from the pipe jacking (maximum within 2 diameters of the pipe jack face depending upon the support installed) are equal to the volume of a settlement ‘trough’ that occurs at surface (refer Figure 3-1). Two parameters must be estimated to use this method, the volume loss and the trough width factor. The volume loss parameter accounts for the strength of the ground, the type of tunnel excavation method, in this case pipe jacking, and workmanship.

The estimates of settlement assume a volume loss of 3% in the low-lying areas where soils dominate and 1.5% in the cliff areas. A trough width parameter, \(K\), of 0.5 is also assumed. These values take into account the mixed geological face conditions encountered by the pipe jacking machine in the weathered East Coast Bays Formation (referred to as geological unit EW). Given recent successful pipe jacking experience in Auckland and that the pipe jacking is predominantly within the ECBF (Auckland’s weak sedimentary bedrock) this is considered reasonably conservative. The surface settlement estimated using this method is up to 10mm which not considered sufficient to cause any damage to the existing buildings and infrastructure identified in Section 2.4.
Figure 3-1 Settlement trough distribution due to pipe jacking

### 3.3.2 Settlement due to shaft excavation

Three sections have been analysed which represent the three proposed shafts and a further three sections for the trenches using the finite element analysis method implemented in the software package PLAXIS. The analysis is carried out in either a plane strain condition which assumes an infinitely long wall or an axisymmetric model.

PLAXIS is able to model the whole construction process, the behaviour of the soil and the soil-structure interaction can be captured. This includes additional surcharges from traffic, construction plant and adjacent buildings (Refer Figure 3-2 below). The settlement can be extracted directly from PLAXIS (refer Figure 3-3 and Figure 3-4 below) for use in the building assessments.

![Figure 3-2 Typical excavation geometry input into PLAXIS (axisymmetric model)]
3.3.3 Consolidation Settlement

Derivation of Parameters
Hydrogeological modelling has been undertaken at the shafts to determine the drawdown effects on the groundwater tables. The Groundwater Effects Assessment and drawdown extent by PDP is used in this analysis. The assessment has interpreted the testing data to define parameters for the permeability of each geological unit, as well as expected groundwater drawdown profiles during construction. This information has been used in this settlement analysis.

Analysis Methodology
It is assumed that the pipe jacking operation will be undertaken by a Tunnel Boring Machine (TBM) with the capability to operate with full face support. The pipe jacking operation in full face mode will restrict ingress of water into the tunnel, hence negligible water drawdown will occur along the pipe jack.
alignment. Therefore, groundwater drawdown has been ignored and the settlements calculated are only due to volume loss.

Ground consolidation settlement effects due to groundwater drawdown around the shafts have been estimated based on the stiffness and permeability of the ground, as well as the estimated groundwater levels and drawdown profile as discussed in the Groundwater Effects Assessment by PDP. One-dimensional consolidation settlement analysis has been completed using the formula shown in Figure 3-5 below.

\[
s_c = m_v \cdot d_v \cdot z
\]

where
- \(m_v\) is the coefficient of volume compressibility
- \(d_v\) is the dissipation of pore water pressure due to dewatering
- \(z\) is the thickness of each soil layer

**Figure 3-5 One-dimensional consolidation settlement formula**

The consolidation calculations have been made assuming a hydrostatic pore pressure distribution from the initial groundwater levels interpreted by PDP. The groundwater drawdown at each structure has the potential to cause consolidation of the fill, Tauranga Alluvium (TA) and ECBF Residual Soils (ER). The weathered and unweathered ECBF units (EW and EU2) are considered to be rock and therefore it is assumed that they will not consolidate further. The consolidation settlement extent is based on the Groundwater Effects Assessment by PDP which has determined expected groundwater drawdown profiles for each shaft. An example is shown in Figure 3-6 below.
3.3.4 Combination of Settlement Effects

The total settlements at the ground surface will result from a combination of values from the three settlement sources. Although the two mechanical sources will produce relatively quick settlements following excavation and consolidation will take a comparatively longer time to occur, the combination of settlement effects has been assessed as a long-term case, i.e. assuming full settlement has occurred for all sources. The method used to combine the settlements is a simple superposition of the settlement values from each individual source. A typical graph showing an example of estimated surface settlements from each source for a shaft excavation is shown in Figure 3-7.

![Graph showing estimated settlements](image)

Figure 3-7 Example graph of surface settlement for each settlement source (London/New Street shaft)

3.3.5 Sensitivity Analysis

Derivation of Parameters

A major aspect of the shaft and trench retention design is the estimation and control of ground movements resulting from wall deflections, groundwater drawdown, and pipe jacking. In this regard, the most important geotechnical parameters are as follows:

- Wall deflections are predominantly controlled by the at-rest earth pressure coefficient (K₀) and the structural stiffness of the wall system.
• Consolidation settlement is controlled by (1) soil permeability and depth of the ground water table, which controls the extent of the drawdown curve, and (2) the pre-consolidation pressure and compression indices of the soils \((C_c/C_s)\), which controls the magnitude of settlement.

• Settlements due to pipe jacking are controlled by (1) the ground modulus which influences the shape of the settlement trough method of ground/face support (open/closed face mode), and (2) the volume loss that occurs during pipe jacking due to overcut. The volume loss parameter and trough width factor are adopted from previous Auckland projects. The mean parameters adopted are taken as moderately conservative values.

In order to gauge the risk of larger ground settlements occurring as a result of natural variation in the ground properties, sensitivity analyses relating to the above parameters have been carried out at critical locations.

The modifications that have been made to the recommended parameters for the purpose of the sensitivity analysis are as follows:

• 25\% increase in the at-rest earth pressure coefficient \((K_o)\).

• Ground water table depth taken as the maximum credible depth below ground at each analysis location.

• 20\% increase in the compression index \((C_c)\) and swelling index \((C_s)\).

• 20\% decrease in drained Young’s Modulus \((E')\).

• 20\% decrease in Effective Strength \((c' \text{ and } \tan \phi')\).

The modified parameter set is considered to produce conservative settlement estimates.

A further sensitivity case was also considered for the St Marys Road park shaft which considered the possibility of weaker geological conditions. In this case a lower ECBF level and the addition of a Tauranga Group (TA) layer was analysed.

**Analysis Methodology**

For the purposes of this sensitivity analysis, the St Marys Road park shaft section has been assessed as the critical section since it is located in geology that is most susceptible to ground settlement. The sensitivity analysis has been carried out assuming long term settlement effects, i.e. the combination of poor ground conditions and increased drawdown is assessed together.

The ground surface settlement due to pipe jacking used in the assessment of settlement effects is already considered to be an upper bound and therefore no further sensitivity analysis has been done on the calculated values.

### 3.4 Methodology for Effects Assessment

#### 3.4.1 Buildings

The Burland Building Damage Classification was adopted to categorise buildings based on the effects from settlement. The limiting tensile strain criteria in the aforementioned classification are presented in Table 3-2.
Table 3-2 – Building damage assessment criteria

<table>
<thead>
<tr>
<th>Category of Damage</th>
<th>Description of degree of damage</th>
<th>Limiting Tensile Strain %</th>
<th>Max. Slope of Ground</th>
<th>Max. Settlement of Building (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Negligible</td>
<td>Less than 0.05</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Very Slight</td>
<td>0.05 to 0.075</td>
<td>Less than 1:500</td>
<td>Less than 10</td>
</tr>
<tr>
<td>2</td>
<td>Slight</td>
<td>0.075 to 0.15</td>
<td>1:500 to 1:200</td>
<td>10 to 50</td>
</tr>
<tr>
<td>3</td>
<td>Moderate</td>
<td>0.15 to 0.3</td>
<td>1:200 to 1:50</td>
<td>50 to 75</td>
</tr>
<tr>
<td>4</td>
<td>Severe</td>
<td>Greater than 0.3</td>
<td>1:200 to 1:50</td>
<td>Greater than 75</td>
</tr>
<tr>
<td>5</td>
<td>Very Severe</td>
<td>Greater than 1:50</td>
<td>Greater than 75</td>
<td></td>
</tr>
</tbody>
</table>

In a stage 1 assessment, buildings that are located outside the 10mm settlement contour and are subject to less than a 1/500 settlement slope are identified and classified as having negligible effects. These have not been further assessed as part of this analysis.

Stage 2 assessments are undertaken on buildings within the 10mm settlement contour to determine the expected level of effects based on the estimated tensile strains and differential settlements or slope of the ground moment under the subject buildings, categorising them accordingly using Table 3-2. Total settlement has not been utilised in the stage 2 phase, recognising that it is relative deflection or angular distortion under a building that causes adverse effects rather than the magnitude of the total settlement.

The categorisation is developed from a masonry brick or block building founded on shallow foundations, for which tensile strains induced in a building and associated ground slopes are derived and compared against limiting values to assess the risk category and degree of damage. Piled buildings may also be categorised on the basis of the Burland Building Damage Classification, to address potential ground settlement impact on any slab on grade structures.

A further sensitivity assessment is undertaken on the affected buildings by scaling the estimated effects on the building up to the Moderate Damage Classification as per Table 3-2. This assessment is governed by the tensile strains estimated for each building.

The above assessment method ignores the interaction between the ground and foundations (greenfield conditions). All buildings will inherently have some degree of resistance against bending imposed by the ground. As a result, the effects estimated in Table 3-3 can generally be considered conservative.

It should be noted that the degree of damage description in terms of Slight (category 2) and Moderate (category 3) in Table 2-2 and Table 3-3 below do not necessarily correspond to building owner's perceptions and are based on published back analysis of data that provides a benchmark for classification of building damage. However, such terminology is widely accepted and the classification system illustrated has been adopted by organisations such as the Institution of Structural Engineers London, the Institution of Civil Engineers and the British Research Establishment (BRE), and internationally.

It is further noted that the division between category 2 and 3 is considered particularly significant, as damage up to category 2 can potentially be caused by a variety of sources within the building itself, including seasonal ground movements or the buildings sensitivity.
Stage 2 Building Risk Assessment for Shallow Foundations

Settlement profiles were developed at each retention structure in accordance with the methods outlined above, and the building cross sections plotted against these. The resulting hogging and sagging strains under the building footprint have been estimated, according to the method of Burland and Wroth, considering bending, diagonal and horizontal strains. Allowance has been made for the structural stiffness of the building through use of a structural stiffness parameter that is dependent on the flexibility of the building. The estimated critical strain was then compared with the Burland criteria of Table 3-2 and the expected building damage classification in accordance with Table 3-3.

Table 3-3 - Building damage classifications

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

Overview
Damage to utilities due to settlement is considered less likely than damage to buildings as the allowable slopes are much lower for buildings. However, utilities are likely located much closer to the proposed works than any of the buildings considered.

The utilities which have the highest risk of damage are utilities which run at a 90° angle to the excavation (i.e. perpendicular to the settlement contours). Furthermore, as the slope of settlement increases as distance from the excavation walls decreases, damage to utilities is most likely in the area closest to the excavation walls.

Damage to utilities due to settlement is usually the result of tension created in the walls of the utility. This typically manifests as either opening of joints (for jointed pipes/ducts) or cracks in the cables or ducts.

Minor Utilities
For the purposes of this report, minor utilities are defined as those with a diameter of 600mm or less and which are located near the existing ground level. While many of the utilities will be able to accommodate high levels of deflection, others will be more susceptible to damage. This assessment has been based on a maximum allowable slope value which is applicable for the utilities most susceptible to damage and which will be conservative for other utilities.

A paper by O’Rourke & Trautmann (1985) recommends a maximum slope of 1:140 for cast iron pipes and brittle utilities with a diameter of 200mm or greater. While cast iron is considered the material most susceptible to damage, some of the utilities are expected to be very old and as such a safety factor has been applied to give a maximum allowable slope of 1:250. This value has been applied to all utilities.

Major Utilities
Major utilities in the project area are the 810 diameter and 700 diameter water supply trunks located near the marine discharge pipeline at Curran Street and a 1200 diameter stormwater pipe approximately 10m east of the St Marys Road Park shaft. The 1:250 maximum allowable slope applied to minor utilities will also be used as a stage 1 assessment for these major utilities. The actual service will need to be further checked during detailed design for its condition and tolerance to the predicted settlement, with specific mitigation measures developed if the tolerances are approached.

Roads and Motorways
The assessment of effects on the roads and infrastructure comprises overlaying the estimated settlement contours over the roads, and determining changes to the gradients of those assets. The effect of those changes in gradient on each road 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. Based on the designers past experience with similar project works, the following allowable deflection limits shown in Table 3-4 have been applied when assessing the settlement effects on local roads.
### Table 3-4 Allowable Deflection Limits for Roads and Motorways

<table>
<thead>
<tr>
<th>Measure</th>
<th>Tolerable limit</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Roads</td>
</tr>
<tr>
<td>Total settlement</td>
<td>20mm</td>
</tr>
<tr>
<td>Differential Settlement</td>
<td>1:500</td>
</tr>
</tbody>
</table>
4 Effects Assessment

4.1 Effects Overview

The settlement effects which have been analysed in this report are based on the sources of settlement discussed in Section 1.4. These effects have been combined to produce the overall long term estimated settlement effects due to the construction of the Project.

These settlement effects will predominantly occur during the construction period. The settlement due to mechanical effects is likely to occur quickly following the ground excavation. However, some consolidation settlement due to dewatering will likely occur after the pipe jacking operation is completed. The magnitude of this on-going settlement is considered negligible due to the length of construction (greater than one year), the permanent structures being designed as undrained and the logarithmic nature of the timing of consolidation effects.

Monitoring and mitigation methods to measure and deal with the potential settlement effects during construction are discussed in Section 5.

The estimated settlement effects interpreted contour plans are provided in Appendix A.

4.2 Settlement Estimates

4.2.1 Settlement due to shaft and trench construction

The estimated surface settlement contours due to the construction of the shafts are shown on drawings 255303-0002-DRG-JJ-1001 to 255303-0006-DRG-JJ-1001 in Appendix A. These settlements show the combined effects of the mechanical and consolidation settlement due to the shaft excavations.

The mechanical settlements around the shafts are up to 14mm and extend up to 20m away from the shaft walls, intruding into surrounding building properties. The largest mechanical settlement occurs at the Pt Erin park shaft where 14mm of settlement is estimated to occur at the face of the wall. This level of settlement has the potential to cause some effect to the structures adjacent to the shafts which will need to be assessed. The utilities surrounding the shafts will also need to be assessed based on the settlements occurring in these locations.

4.2.2 Settlement due to pipe jacking

The mechanical settlement caused by the volume loss from the pipe jacking operation will cause a settlement trough at the surface above the pipe jack alignment. The highest assumed volume loss of 3% causes a maximum mechanical settlement of 10mm above the centreline of the pipe jack alignment. This settlement will only occur in the low-lying areas where the pipe jack alignment is located within soil or has shallow cover. Where the pipe jack is below existing buildings, the geology is unweathered ECBF rock and there is significant cover, thus 1.5% volume loss is assumed and resulting settlement is in the order 5mm. These small settlements are unlikely to have an effect on the structures or utilities discussed previously.

4.2.3 Settlement due to groundwater drawdown

Surface settlements due to dewatering have been assessed at the shafts and trenches. It is assumed that the pipe jacking operation will be undertaken by a Tunnel Boring Machine (TBM) with the capability to operate with full face support, which will restrict ingress of water into the tunnel. Hence negligible
water drawdown will occur around the tunnel and therefore groundwater drawdown has been ignored and the settlements calculated are only due to volume loss (mechanical).

All three shafts and then trenches are excavated through layers of fill and residual soils (ER). These geological units have the potential to undergo consolidation under the change of stresses imposed by the drawdown of groundwater. Some drawdown of the ground water table expected at all the shafts and trenches. Predicted drawdowns are in the order of 2-3m. Drawdown of the ground water within the moderately weathered to unweathered ECBF is predicted, but will not induce any consolidation settlements in these materials.

### 4.2.4 Combined Settlement

The vertical settlements described above were combined to assess the total effects. The combined surface settlements are shown on the Settlement Contour drawings in Appendix A, and are summarised in Table 4-1 and Table 4-2.

#### Table 4-1 Estimated vertical settlement due to shaft excavations

<table>
<thead>
<tr>
<th>Location</th>
<th>Maximum Vertical Settlement (mm)</th>
<th>Distance from Wall (m)</th>
<th>5mm Settlement</th>
<th>10mm Settlement</th>
<th>20mm Settlement</th>
</tr>
</thead>
<tbody>
<tr>
<td>London/New Street Shaft</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- All directions</td>
<td>7</td>
<td>2.4</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>St Marys Road Park Shaft</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- All directions</td>
<td>9</td>
<td>2.8</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Point Erin Shaft</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- All directions</td>
<td>14</td>
<td>7.0</td>
<td>5.6</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

#### Table 4-2 Estimated vertical settlement due to trench excavations

<table>
<thead>
<tr>
<th>Location</th>
<th>Maximum Vertical Settlement (mm)</th>
<th>Distance from Wall (m)</th>
<th>5mm Settlement</th>
<th>10mm Settlement</th>
<th>20mm Settlement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Curran Street</td>
<td>7</td>
<td>2.2</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Sarsfield Street</td>
<td>25</td>
<td>5.8</td>
<td>5.4</td>
<td>3.7</td>
<td></td>
</tr>
<tr>
<td>Outfall option A1</td>
<td>18</td>
<td>10.8</td>
<td>5.8</td>
<td>1.9</td>
<td></td>
</tr>
</tbody>
</table>

#### Table 4-3 Estimated vertical settlement due to pipe jacking

<table>
<thead>
<tr>
<th>Location</th>
<th>Maximum Vertical Settlement (mm)</th>
<th>Distance from tunnel centreline (m)</th>
<th>5mm settlement</th>
<th>10mm settlement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pipe jack (high areas)</td>
<td>5</td>
<td>3.7</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Pipe jack (low areas)</td>
<td>13</td>
<td>2.7</td>
<td>1.2</td>
<td></td>
</tr>
<tr>
<td>Outfall Option A1</td>
<td>13</td>
<td>1.2</td>
<td>0.6</td>
<td></td>
</tr>
<tr>
<td>Outfall Option A2</td>
<td>8</td>
<td>1.4</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>
In some cases, the horizontal movement can govern the building damage assessment, therefore the horizontal displacements were also analysed for the shaft excavations and pipe jack operation. The horizontal components of the settlements are summarised in Table 4-4.

Table 4-4 Estimated horizontal movement due to excavations

<table>
<thead>
<tr>
<th>Location</th>
<th>Maximum Horizontal Movement (mm)</th>
<th>Distance from Wall (m)</th>
<th>3mm Movement</th>
<th>5mm Movement</th>
<th>10mm Movement</th>
</tr>
</thead>
<tbody>
<tr>
<td>London/New Street Shaft</td>
<td>- All directions</td>
<td>&lt;1</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>St Marys Road Park Shaft</td>
<td>- All directions</td>
<td>&lt;1</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Point Erin Shaft</td>
<td>- All directions</td>
<td>4</td>
<td>1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Trenches</td>
<td>Curran Street</td>
<td>3</td>
<td>1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Sarsfield Street</td>
<td>7</td>
<td>5</td>
<td>4</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Option A1</td>
<td>5</td>
<td>6</td>
<td>5</td>
<td>-</td>
</tr>
</tbody>
</table>

4.2.5 Sensitivity analysis

Sensitivity analysis was carried out on a number of geological parameters as described in Section 3.3.5. The input parameters have been modified (assuming an extreme case) in order to analyse the effect of encountering poor ground and adverse groundwater drawdown conditions. Surface settlement predictions have been calculated using these factored inputs and have resulted in the estimated settlements shown in Table 4-5. These settlements have been assessed to not cause any increase in effect on adjacent structures.

Table 4-5 Sensitivity analysis of settlements due to shaft excavations

<table>
<thead>
<tr>
<th>Location</th>
<th>Maximum Vertical Settlement (mm)</th>
<th>Distance from Wall (m)</th>
<th>5mm Settlement</th>
<th>10mm Settlement</th>
<th>20mm Settlement</th>
</tr>
</thead>
<tbody>
<tr>
<td>St Marys Park Shaft (critical location)</td>
<td>26</td>
<td>3.6</td>
<td>2.4</td>
<td>1.1</td>
<td></td>
</tr>
</tbody>
</table>

4.3 Assessment of Effects on Buildings

4.3.1 London/New Street Shaft

As stated above, this shaft is located within a residential area, where properties are located within 10m of the shaft walls. However, predicted settlement at the shaft is less than 10mm at the shaft and reduces to less than 5mm within 2.4m. The effect on any adjacent structures is therefore considered to negligible.
4.3.2 St Marys Road Park and Point Erin Shafts

There are no existing buildings located within the area of settlement influence at St Marys Road Park or Point Erin Shafts.

4.3.3 Curran and Sarsfield Trenches

The trenching along Curran and Sarsfield Streets is predicted to generate up to 25mm settlement. The trenches are located alongside Point Erin Park, with residential housing located on the opposite side of Sarsfield Street. The 10mm settlement contour is 5.4m from the trench walls and does not cover the housing. The effect on any adjacent structures is therefore considered to be negligible.

4.3.4 Marine Discharge Pipeline Trench

There are no existing buildings located within the area of settlement influence of the marine discharge pipeline trench works.

4.3.5 Pipe jack

The pipe jack alignment passes underneath many residential properties however the predicted settlement at these locations is typically 5mm or less. The effect on the properties is therefore considered to be negligible.

4.4 Assessment of Effects on Utilities

The assessment of effects on minor utilities has not identified and assessed the effects on any particular utility, rather a worse case assessment has been conducted by determining the critical differential settlement in the nearest 5m to the shaft or trench walls. Note that this is also conservative because, for any utility to experience this differential movement, it would need to be running perpendicular to the shaft or trench walls and be already isolated or otherwise treated as part of the piling works.

This differential settlement has then been compared with the allowable differential settlement for utilities given above to determine the effects on utilities (whether there are in fact utilities present in this critical location or not).

The differential settlement calculated for each shaft for use in the utilities assessment is given in Table 4-6.

Table 4-6: Predicted Differential Settlements

<table>
<thead>
<tr>
<th>Location</th>
<th>Differential Settlement</th>
</tr>
</thead>
<tbody>
<tr>
<td>London/New Street shaft</td>
<td>1:1400</td>
</tr>
<tr>
<td>St Marys Road Park shaft</td>
<td>1:600</td>
</tr>
<tr>
<td>Point Erin shaft</td>
<td>1:1000</td>
</tr>
<tr>
<td>Curran Street trench</td>
<td>1:1300</td>
</tr>
<tr>
<td>Sarsfield Street trench</td>
<td>1:700</td>
</tr>
<tr>
<td>Marine Discharge pipeline trench</td>
<td>1:2400</td>
</tr>
<tr>
<td>Pipe jack</td>
<td>1:750</td>
</tr>
</tbody>
</table>
Based on the above differential settlements, the allowable limits given above, the effects on existing minor utilities can be considered Negligible to Very Slight.

For major utilities, it should be noted that rotation of joints has not been assessed as the present information does not give sufficient detail for this level of assessment. However, the rigid response of the utilities would give a worst case and it is generally accepted that a maximum of 0.5° rotation of a joint is allowable (dependent on the current state of the utility). A 0.5° rotation represents a differential slope of about 1:140, which is within the envelope of settlements presented in this report. The actual service will need to be further checked during the detailed design for its condition and tolerance in agreement with the asset owners.

4.5 Assessment of Effects on Roads and Infrastructure

4.5.1 Pipe jack shafts
Construction of the London/New Street shaft may cause some damage, e.g. minor pavement cracking, of to the streets however this will only occur in close proximity to the shaft and will be simply repaired as part of the reinstatement works when the shaft is completed.

Both the St Marys Bay and Point Erin shafts are located sufficiently clear of the roads and motorways that the potential effects of resulting from settlement are considered negligible.

4.5.2 Trenches
The trenching works will have the largest effect, e.g. potential for minor pavement cracking, on local roads, being mostly located within the road corridor. Settlements may exceed the defined limit of 20mm at Sarsfield Street, Curran Street and the Curran Street on-ramp (for the Option A1 trench). The area exceeding 20mm settlement is predicted to be localised within 2-4m of the trench walls however and will be repaired as part of the reinstatement works when the trenching is completed.

4.5.3 Pipe jack
The pipe jack alignment passes underneath several local roads and close to the northern motorway but the predicted surface settlements are not large enough to be of concern to these routes (<20mm). However, where the alignment passes underneath Shelly Beach Road, it is in very close proximity (within 5.5m) to the base of the piles supporting the Shelly Beach Road overbridge. In consideration of this, a PLAXIS model has been developed to analyse the potential sub-surface settlement induced in these piles. The modelling found the piles to be outside of the zone of influence of the bridge piles and therefore a negligible effect on the bridge due to the pipe jack works.
Figure 4-1: Output from PLAXIS model for interaction between the pipe jack and the Shelly Beach Road Bridge
5 Mitigation and Monitoring

5.1 Monitoring

Monitoring of the buildings, utilities and actual ground and groundwater movements is essential to demonstrate the performance of the shafts and pipe jack, to confirm the estimations for the assumed ground conditions and to control the effects on the adjacent structures. Monitoring will be required before construction commences, during construction and following completion to confirm the effects of the construction of the Project.

Full details of the required monitoring will be provided in a Groundwater and Settlement Monitoring and Contingency Plan (GSMCP). The GSMCP will provide details of the following project controls:

- Groundwater and settlement monitoring including type and location of monitoring points and monitoring programme
- Proposed trigger levels for the above monitoring points and methodology for their determination
- Contingency and mitigation measures to be implemented should trigger levels be exceeded
- Pre- and post-construction surveys of buildings and services
- TBM methodology

A summary of the proposed methodology for monitoring is provided in the following sections of this report.

5.1.1 Monitoring Types

A series of ground movement marks will be installed at specified intervals radiating out from the shafts and pipe jack and regularly monitored so the actual surface settlements can be quantified and compared against the estimated settlements. The markers will generally be set out adjacent to sensitive structures and along the streets in the project area to match the cross sections that have been used for the settlement analysis (refer drawing 255303-0002-DRG-CC-1029 in Appendix B). Groundwater monitoring boreholes will also be used to measure the depth and extent of the groundwater drawdown.

Baseline readings will be taken of all the monitoring points prior to commencement of construction of the project for a minimum of three months in order to capture the magnitude of background movements prior to any construction influence. The ongoing frequency of monitoring will vary depending on the current stage of construction.

5.2 Reporting and Further Assessments

Pre-construction baseline monitoring will be compiled in a factual report prior to the commencement of construction with any irregular results further assessed. The outcomes will form part of the input for the construction phase assessments.

Monitoring during the construction phase will be used to verify the design analyses by comparing the actual movements with those estimated. The monitoring data will be used to reassess the building damage classifications at the critical locations and will be compared to those assessed in this report. If these reassessments indicate that the damage classifications have increased significantly then additional analyses or increased monitoring may be required. Mitigation options discussed in Section 5.6 may also be required to be implemented.

During construction of the project the monitoring results shall be reported and discussed at a daily review meeting with Auckland Council and other client representatives. These reports will then be
provided to Auckland Council and other relevant parties on a two-monthly basis. If there are any significant increases in the ground movements exceeding the specified trigger levels or significant increases in the assessed building damage then, following a more detailed review of the data, the affected parties will be notified and mitigation measures agreed and implemented.

5.3 Summary of Monitoring

Pre-construction:

- Baseline readings of the horizontal and vertical movements over a period of time leading up to the commencement of the construction.
- The monitoring will be compiled into a factual report to form part of the input for the next phases of construction.

During construction:

- Frequency of monitoring at each marker within the “Active Construction” zone to be daily.
- Reduced monitoring outside the “Active Construction” zone.
- The monitoring will be reviewed at daily meetings with Auckland Council and other Client representatives.
- Further assessments may be required where specified triggers levels are exceeded.

Post-construction:

- Monitoring on completion of all construction works to continue until ground movements stabilise.
- Monitoring to continue at locations where movements are greater than expected and/or where trigger levels have been exceeded.

5.4 Condition Surveys

Pre- and post-construction asset condition surveys will be undertaken on all assets (buildings, utilities, roads) determined to be susceptible to damage by the settlement assessment. This is based on the following criteria:

- Buildings within the estimated 10mm settlement contour.
- Other buildings adjacent to the Project construction works as deemed necessary by the Contractor.

This scope is intentionally more extensive than the schedule of buildings considered to be at risk to act as a buffer allowing for actual effects being greater than those estimated.

5.4.1 Pre-construction Condition Surveys

The initial survey will comprise an inspection of each asset to establish and record its existing condition. Each survey will produce a written description including photographs of any identified existing damage. These surveys will be carried out prior to the commencement of the project construction and will provide a baseline of the condition of each asset. Assets identified as being in a state of dilapidation may require further engineering involvement in the form of detailed assessments and possible strengthening prior to the commencement of the construction works.
5.4.2 During Construction Condition Surveys
Monthly visual inspections may be required to be undertaken during this phase of the project if alarm triggers are exceeded. The purpose of the inspections will be to look for any evidence of effects, with reference to the initial baseline condition survey.

5.4.3 Post-Construction Condition Surveys
Within three months of the completion of construction, post-construction condition surveys are proposed for assets where a condition survey was undertaken prior to and during the construction phase. The survey report shall include an assessment of the cause if any damage is noted. Surveys may not be required if the asset owner does not require it, and therefore evidence is available to Auckland Council that the current owner of that building has agreed they do not require such survey.

Where a post-construction condition survey confirms that the asset has been damaged as a result of construction, remedial work will be required to rectify the damage. Such repairs shall be undertaken as soon possible and by agreement with the owner of the asset.

5.5 Utilities Monitoring
No additional monitoring is proposed for the utilities at the surface since the settlement model predicts that deflections will be within the allowable limits. The monitoring which is implemented for surface movements will be used to check that the deflection of utilities is within the range estimated by the analysis.

However, if the monitoring which is proposed for the ground movements identifies that settlement levels are greater than expected additional, monitoring for utilities may need to be implemented as part of the review process of exceeding alert/alarm levels.

5.6 Mitigation

5.6.1 Overview
The mitigations discussed in this section are general recommendations only. Mitigation measures will be required to be discussed and agreed with the contractor as part of the interactive tender process.

5.6.2 Shafts
The mitigations below are recommended in the case that the actual surface settlements breach the “alarm” trigger level for the monitoring points defined on the monitoring drawings (refer drawing 255303-0002-DRG-CC-1029 in Appendix B). In this case all relevant information should be reviewed with Auckland Council and the Engineer to determine the cause and effect. Following this review the mitigations outlined below or other mitigations agreed with Auckland Council and the Engineer may be implemented (if required):

- Increase the size of the struts/walers being installed to reduce wall deflections.
- Reduce depth of excavation advances and install struts earlier or at closer vertical spacing to reduce wall deflections.
- Implement ground strengthening around the excavation by various methods.
- Reduce surcharge loads on adjacent walls.
5.6.3 Pipe Jacking

The mitigations below are recommended in the case that the actual surface settlements breach the “alarm” trigger level for the monitoring points defined on the monitoring drawings (refer drawing 255303-0002-DRG-CC-1029 in Appendix B). In this case construction works should halt and all relevant information should be reviewed with Auckland Council and the Engineer to determine the cause and effect. Following this review the mitigations outlined below or other mitigations agreed with Auckland Council and the Engineer may be implemented (if required):

- Review TBM face pressure drive parameters including slurry density
- If adequate controls cannot be obtained (and this is extremely unlikely) consider the use of ground improvement ahead of the pipe jack.

5.6.4 Buildings

Mitigation effects are not expected to be required based on the settlement effects assessments. However, this section outlines the process should the measured effects require mitigation.

Mitigation Measures - Non-structural

Where a during construction or post-construction building condition survey confirms that the building has been damaged as the result of construction or operation works relating to the Project, remedial work will be required to rectify the damage. General repairs may include repainting and redecoration. In severe cases some repair of bricks and possible replacement of a small amount of exterior brickwork may be required. The timing of the repairs would depend on the owner’s requirements, stage of construction and degree of damage.

Depending on the extent and degree of damage more regular monitoring may be implemented for the relevant buildings.

Mitigation Measures - Structural

The settlement effects assessment contained in Section 4.3 has not identified any buildings with a Building Damage Criteria greater than the ‘Negligible’ damage category, which is limited to the possibility of minor aesthetic damage. As such structural building damage is unlikely and not expected on this project. If any effects of a structural nature are identified during the condition surveys then a detailed assessment will be required by a qualified structural engineer. Any recommendations for repair and increased monitoring arising from this assessment will then be implemented. In the highly unlikely event where local repairs are not sufficient, then additional works such as underpinning, strengthening or propping of the building may be required.

Utilities

If deflections are identified in the monitoring which are close to the allowable deflection values, it is proposed that the utilities be exposed. This will isolate the utilities from the adjacent soil so they will not be affected by settlement.

If it is believed that damage to a service may have occurred, investigation of the utility and the surrounding area shall be carried out. This will include an assessment of the site, consultation with the affected utility operator, and an intrusive investigation to confirm the source of the damage. Remedial works shall be agreed with the utility operator and carried out as soon as possible. These could include:
- The provision of a new utility along a new alignment, which may be either permanent or temporary depending on the acceptability of the location to the utility operator. This could also be via an unused duct if available.
- Lining of the damaged pipeline using a cured-in-place pipe (CIPP) method.
- Replacement of the damaged section of the utility.
- Local repair of the cable via other methods such as crack injection, epoxy resin, sleeve, or other approved method.

5.6.5   Roads and Motorways

If trigger levels are exceeded at monitoring points at the location of nearby roads, a condition survey will be undertaken to determine the level and extent of any damage. Should the survey find that damage has occurred as a result of the construction works, Auckland Council will be notified and supplied with a methodology to repair the damage (resealing or reinstatement of pavement) and to prevent further damage, including timeframes.

Any repairs will be undertaken at the cost of the consent holder as soon as practicable. The timing and extent of repairs may vary depending on the asset owner's requirements or alternative agreement.
6 Conclusions

6.1 Settlement

Settlement analysis was undertaken for the project which included analysis of the proposed pipe jacking, trenching and excavation of the three shafts. Sections were taken at key locations which were then assessed to determine the magnitude of settlement from three sources.

The sources of settlement effects are; mechanical settlement caused by deflections of the piled walls during the excavation of the shafts, mechanical settlement caused by volume loss during pipe jacking, and consolidation settlement caused by dewatering of the surrounding ground.

Each of these sources of effects was individually assessed and the results combined by simple superposition to obtain the total estimated settlement effects (provided in Appendix A). The resulting maximum vertical settlements were found to be typically less than 10mm over the pipe jack alignment, and 25mm at the shafts and trenches. The vertical settlement reduces to 5mm (considered very minor movement) within a maximum distance of 7m of the shaft walls. These settlements were then used to define the building damage category for the surrounding structures.

6.2 Estimated Extent of Settlement Effects

The estimated influence zone of the potential settlement effects extends along the pipe jack alignment from Point Erin to the corner of London and New Streets, the pipe jack and/or trench from Point Erin to the seawall, and the length of trenching works along Curran and Sarsfield Streets. For the purposes of this settlement analysis, the settlement extent was estimated down to a 5mm settlement contour, which is considered to be very minor movement. The settlements resulting from the pipe jacking were found to be less than 10mm over most of the pipe jack alignment and to reduce to less than 5mm within 14m of the pipe centreline. The vertical settlements around the shafts were estimated to be up to 20mm, typically reducing to less than 5mm within 7m of the shaft walls. The settlements due to trenching works were found to be less than 25mm and reduce to less than 5mm within 11m of the trench walls. These settlements were used to assess the potential damage category for the surrounding buildings or other structures.

Based on the building Damage Assessment Criteria (Burland, 1997, see also Section 3.4.1 of this Report) the effect on surrounding structures is expected to be “Negligible” if the vertical settlement is below 10mm. Therefore, only the buildings within the 10mm settlement contour are required to be further assessed for potential damage. Although the pipe jack alignment passes under a number of buildings, settlement effects are typically estimated to be less than 10mm, as such the effects to structures along the pipe jack alignment are expected to be negligible.

Settlement Contour Plans showing the estimated settlement extent are attached in Appendix A.

6.3 Buildings

The potential effects on the buildings were assessed using an internationally accepted method (Burland, 1997). The method determines the curvature and strain in a building and plots the values against a series of criteria to assess the likely effect on the structure. The classification of potential effects was described in this report. In summary the assessments estimate that there will be “Negligible” effects on the surrounding buildings.
A proposed monitoring plan has been described in this report which includes building and ground movement monitoring and condition surveys. These results will be used to compare the actual damage categories with those estimated in this report.

Building mitigation includes repair of non-structural defects on completion of construction and the immediate repair in the unlikely case of any issues that are structural.

### 6.4 Utilities

The settlement analysis indicates that deflections of adjacent utilities, as a result of settlement, will be within the acceptable range for utilities. Any effects of the works on utilities at the surface are expected to be negligible and managed without difficulty.

Monitoring specifically for utilities is proposed only if the estimated settlement levels are exceeded. This would be determined by monitoring ground deflections immediately adjacent to the shafts.

### 6.5 Roads and Infrastructure

The assessment has determined that the settlement resulting from the works will be within the acceptable ranges for the adjacent roads. A specific analysis was also undertaken for the Shelly Beach Road Bridge and found negligible effects on the bridge due to the project works. Any damage a road occurring from the excavations (particularly at London/New Street and Sarsfield Street) is planned to be remedied by reinstatement of the road pavement immediately following completion of the works.
7 References


Appendices
Appendix A
Settlement Contour Plans
This page has been intentionally left blank.
NOTES:

1. SETTLEMENT DUE TO TUNNELLING BASED ON AN ASSUMED VOLUME LOSS OF 1.3% TO 2.3% FOR PIPE-JACKED TUNNELS
2. SETTLEMENT DUE TO SHAFT CONSTRUCTION IS BASED ON PRELIMINARY ANALYSIS OF HORIZONTAL WALL DEFLECTION AND Dewatering EFFECTS
3. SETTLEMENT SHOWN ASSUMES GOOD CONSTRUCTION PRACTICE BY AN EXPERIENCED CONTRACTOR
NOTES:

1. SETTLEMENT DUE TO TUNNELING BASED ON AN ASSUMED VOLUME LOSS OF 1.5% TO 3.0% FOR PIPE JACKED TUNNELS
2. SETTLEMENT DUE TO SHAFT CONSTRUCTION IS BASED ON PRELIMINARY ANALYSIS OF HORIZONTAL WALL DEFLECTION AND DEWATERING EFFECTS
3. SETTLEMENT SHOWN ASSUMES GOOD CONSTRUCTION PRACTICE BY AN EXPERIENCED CONTRACTOR

5mm SETTLEMENT
10mm SETTLEMENT
20mm SETTLEMENT

LEGEND

- PROPOSED STORAGE PIPELINE
- INDICATIVE CONSTRUCTION CORRIDOR
- SHAFT LOCATION
- AUP MANA WHENUA OVERLAY
- EXISTING STORMWATER
- EXISTING WASTEWATER
- EXISTING WATER SUPPLY
- EXISTING COMMS
- EXISTING GAS
- EXISTING SURFACE CONTOURS
- MARINE DISCHARGE PIPELINE

NOTES:

1. SETTLEMENT DUE TO TUNNELING BASED ON AN ASSUMED VOLUME LOSS OF 1.5% TO 3.0% FOR PIPE JACKED TUNNELS
2. SETTLEMENT DUE TO SHAFT CONSTRUCTION IS BASED ON PRELIMINARY ANALYSIS OF HORIZONTAL WALL DEFLECTION AND DEWATERING EFFECTS
3. SETTLEMENT SHOWN ASSUMES GOOD CONSTRUCTION PRACTICE BY AN EXPERIENCED CONTRACTOR
**ST. MARYS PARK SETTLEMENT CONTOURS AND BUILDING LOCATION PLAN**

**LEGEND**
- PROPOSED STORAGE PIPELINE
- INDICATIVE CONSTRUCTION
- STORAGE PIPE LINE
- SHAFT LOCATION
- AUP MANA WHENUA OVERLAY
- EXISTING STORMWATER
- EXISTING WASTEWATER
- EXISTING WATER SUPPLY
- EXISTING COMMS
- EXISTING GAS
- EXISTING SURFACE CONTOURS
- MARINE DISCHARGE PIPELINE

**NOTES:**
1. SETTLEMENT DUE TO TUNNELING BASED ON AN ASSUMED VOLUME LOSS OF 1.5% TO 3.0% FOR PIPE JACKED TUNNELS
2. SETTLEMENT DUE TO SHAFT CONSTRUCTION IS BASED ON PRELIMINARY ANALYSIS OF HORIZONTAL WALL DEFLECTION AND DEWATERING EFFECTS
3. SETTLEMENT SHOWN ASSUMES GOOD CONSTRUCTION PRACTICE BY AN EXPERIENCED CONTRACTOR

**Scale:** 1:250

**Legend Details:**
- 5mm SETTLEMENT
- 10mm SETTLEMENT
- 20mm SETTLEMENT

**Client:**
- **Date:** 3/16/2018
- **Location:** Auckland

**Drawn By:**
- P. Nicolaien
- J. Lokes
- R. Alea

**Prepared By:**
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- J. Lokes
- R. Alea

**Revision Details:**
- B 04/04/18 FINAL
NOTES:

1. Settlemen due to tunnelling based on an assumed volume loss of 1.5% to 3.0% for pipe jacked tunnels.
2. Settlement due to shaft construction is based on preliminary analysis of horizontal wall deflection and centring effects.
3. Settlement shown assumes good construction practice by an experienced contractor.
Appendix B
Buildings and Utilities Monitoring Drawings