NZ TRANSPORT AGENCY
WAKA KOTAHI

Roads of national significance

16 20

Western Ring Route – Waterview Connection

Geotechnical Interpretive Report

Tonkin & Taylor

newzealand.govt.nz

Quality Assurance Statement	
	Prepared by: Kate Williams, Senior Engineering Geologist (Tonkin and Taylor), Tim Sinclair, Technical Director (Tonkin and Taylor)
	Reviewed by: Gavin Alexander, Technical Director Beca)
	Approved for Issue by: Chris Bauld, Principal Geotechnical Engineer (Tonkin and Taylor)

Contents

Figures

Tables

Appendices

1. Introduction

In 2009, the government identified the Roads of National Significance (RoNS) and set priority for investment in these as New Zealand's most important transport routes. The RoNs are critical to ensuring that users have access to significant markets and areas of employment and economic growth.

The Western Ring Route (WRR) is identified as a RoNS. The WRR comprises the SH20, SH16 and SH18 motorway corridors and, once completed, will consist of 48km of motorway linking Manukau, Auckland, Waitakere and North Shore.

In 2009 the NZ Transport Agency (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 Project.

1.1 Scope and Purpose

In August 2009, the NZTA commissioned a consortium of design organisations, to develop a concept design for the SH20 motorway extension between the Maioro Street and Great North Road Interchanges – referred to as the "Waterview Connection Project". The purpose of the concept design is to allow the motorway designation to be defined and allow an Assessment of Environmental Effects (AEE) to be undertaken with the aim of securing resource consent for the project.

Beca Infrastructure Ltd (Beca) took the role of lead designers and project managers including undertaking the geometrics design and hydrogeological studies while Tonkin & Taylor (T&T) were tasked with the geotechnical design component.

The purpose of this report is to provide an interpretative assessment of the geotechnical/ geological conditions for the proposed alignment route of the SH20 Waterview Connection Project. The report presents geological models as well as a summary of all in situ and laboratory testing and the derivation of material parameters to be used for detailed design.

In addition, this report has been prepared to provide appropriate advice and detail for the current consenting on concept design phase of the project. We expect that further phases of design, especially detailed design, will require the development of this report in significantly finer detail. We also anticipate that further development of detailed design will require a scope of investigations significantly greater than that currently presented. This is likely to include the development and interpretation on a range of specialised geotechnical parameters that are not specifically addressed in this current report.

1.2 Project Description

The Waterview Connection Project is the final project to complete the Western Ring Route (WRR), providing for works on both State Highway 16 (SH16) and State Highway 20 (SH20) to establish a high-quality motorway link that will deliver the WRR as a Road of National Significance.

The current Manukau Harbour Crossing and recently completed Mount Roskill Extension projects on SH20 means that this highway now extends from Manukau in the south to New Windsor in the north, terminating at an interchange with Maioro Street and Sandringham Road. Through the Waterview Connection Project, the NZTA proposes to designate land and obtain resource consents in order to construct, operate and maintain the motorway extension of SH20 from Maioro Street (New Windsor) to connect with State Highway 16 (SH16) at the Great North Road Interchange (Waterview). The majority of this section of SH20 will be constructed via deep tunnels.

In addition, the Waterview Connection Project provides for work on SH16. This includes works to improve the resilience of the WRR and wider transport network; raising the causeway on SH16 between Great North Road and Rosebank Interchanges, which will respond to historic subsidence of the causeway and “future proof” it against sea level rise. In addition, the project provides for increased capacity on the SH16 corridor; with additional lanes provided on the state highway between St Lukes and Te Atatu Interchanges, and works to improve the functioning and capacity of the Te Atatu Interchange. The SH16 Northwestern Motorway Causeway Project is being undertaken separately by Aurecon and is reported on elsewhere.

The Waterview Connection Project will be the largest roading project ever undertaken in New Zealand. The project includes construction of new surface motorway, tunnelling and works on the existing SH16 (Northwestern Motorway) as well as a cycleway that will connect between the SH16 Northwestern and SH20 cycleways. The Project sector diagram (Figure 1.1) provides an overview of the extent and works for the Waterview Connection Project.

The new section of SH20 is approximately 5.5km in length and passes through or beneath the suburbs of Owairaka, Avondale, Waterview and Point Chevalier. The general horizontal alignment and area of the project is shown in Geotechnical Investigation Plan Drawings 108-119 (Appendix A). Specific details of the current vertical alignments are shown on Geological Section Drawings 252-255, 258-262 and 326-337 which are contained in Appendix B. The comments in this report relate to those alignments.

The Maioro Street Interchange marks the southern extent of the project. The southern on/off ramps are subject to a separate project by URS.

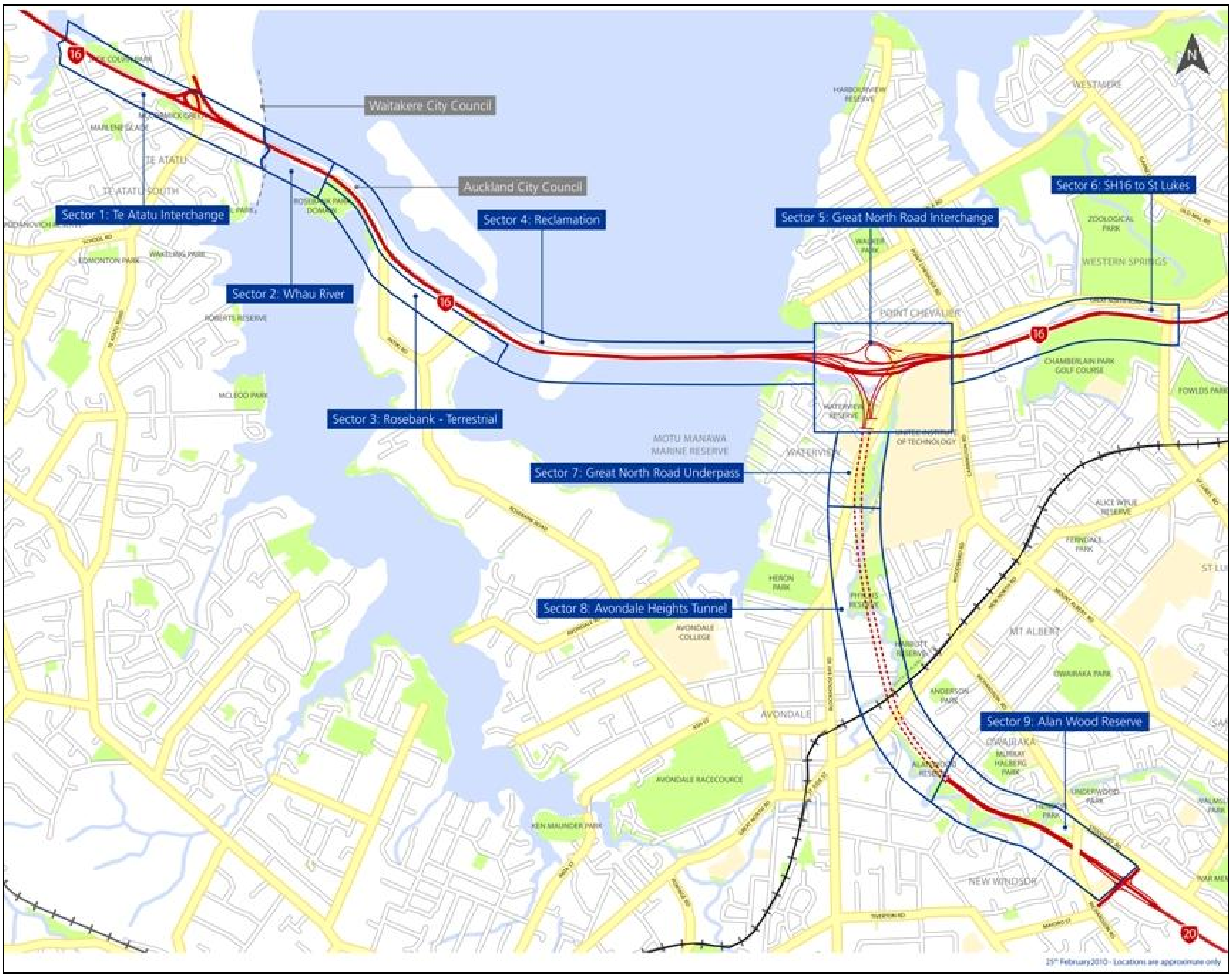

Figure 1.1: Western Ring Route: Waterview Connection Project (SH16 - SH20) – Sector Map

1.2.1 Sectors

Sectors 1 to 4 are included in the SH16 Northwestern Motorway Causeway Project being undertaken separately by Aurecon and reported on elsewhere.

Sectors 5 to 9 are included in the SH20 Waterview Connection Project and described below. For SH20, between Great North Road Interchange (with SH16) and Maioro Street Interchange, a new state highway alignment will be provided over a length of approximately 5km and a future capacity for three traffic lanes in each direction. This interpretative report covers the following main sectors within the overall project and is subdivided into:

Sector 5: Great North Road Interchange

- A new interchange will be built at Great North Road Interchange to provide motorway-to-motorway connections between the Northwestern Motorway (SH16) and SH20, while maintaining the existing connections between Great North Road and the Northwestern Motorway (SH16) at this interchange.

- The existing cycleway will be retained and the existing interchange will be reconfigured to maximise land to the north of the interchange.

- A cycleway connection between the existing Northwestern Cycleway along SH16 and the SH20 cycleway (that terminates at Maioro Street) will also be provided (Sectors 5 through to 9).

Sector 6: SH16 to St Lukes

- Capacity enhancements will be made to the Northwestern Motorway (SH16) between Great North Road Interchange and St Lukes Interchange (in the east), which will include an additional lane in each direction over this section of the Northwestern Motorway.

- Additional lanes, a cycleway and a bus priority lane will be provided between Great North Road Interchange and the St Lukes Interchange.

Sector 7: Great North Road Underpass

- The northern portals of the cut and cover tunnel in Waterview Reserve.

- From Great North Road Interchange, the alignment will be two cut-cover tunnels (some 2.5m apart) beneath Great North Road to connect to the deep tunnels.

Sector 8: Avondale Heights Tunnel

- The alignment for the two 'deep tunnels' (one in each direction) from the cut-cover tunnel beneath Great North Road through to Alan Wood Reserve (adjacent to the Caravan Park), passing beneath Avondale Heights/ Springleigh, the North Auckland Rail Line and New North Road.

Sector 9: Alan Wood Reserve

- The southern portals of the deep tunnel will emerge at grade approximately halfway along the length of Alan Wood Reserve.

- The alignment between the Avondale Heights Tunnel (Sector 8) and Maioro Street Interchange (Sector 10) is 'at-surface', alongside the existing land set aside for rail (the Avondale Southdown Line Designation).

- Richardson Road will be bridged 'at grade' across the State Highway.

- In designing the Waterview Connection Project, accommodation has been made for a future Avondale – Southdown Rail Line (double track with electrification), from Maioro Street Interchange to the southern tunnel portal in Alan Wood Reserve.

Sector 10: Maioro Street Interchange

- New north-facing ramps will be built at Maioro Street Interchange to provide local traffic access to SH20 northbound. The interchange will be extended with a bridge to connect with Stoddard Road on the northeast side of SH20. This will provide both a vehicle and pedestrian connection between New

Windsor and Mount Roskill / Wesley, and will also allow for the local road network to connect to SH20 and access SH1 and SH16.

1.3 Geotechnical Investigation

Several separate phases of ground investigation have been undertaken along various corridor routes for the project. Generally investigations involved sub surface drilling consisting of fully cored NQ to PQ sized boreholes, various groundwater instrumentation installed, core sampling for various laboratory testing schedules and in situ testing where appropriate. All investigation locations were undertaken following approvals and the appropriate consents from the local regulatory bodies.

All available data has been compiled into various factual reports for the project. This interpretive report must be read in conjunction with the factual reports.

1.3.1 Previous Investigations

Numerous options for extending SH20 motorway through Mount Albert to connect into SH16 have been developed over several years by NZTA (formerly Transit New Zealand). As a result numerous geotechnical investigations and studies have been undertaken. The locations of sub-surface investigations are presented in Drawings 108 – 119 (Appendix A). The following list provides a summary of the ground investigation information made available for this report.

- Woodward-Clyde (NZ) Ltd, March 1999: SH20 Extension, Geotechnical Investigation Factual Report, Volumes 1&2, for Transit New Zealand.
- Beca Carter Hollings and Ferner Ltd., December 2001: SH20 Avondale Extension, Geotechnical Factual Report, for Transit New Zealand Contract No. PA1871.
- Meritec Ltd., June 2002: SH20 Mount Roskill Extension, Geotechnical Investigation Factual Report, for Transit New Zealand.
- Meritec Ltd., October 2002: SH20 Mount Roskill Extension, Geotechnical Investigation Factual Report, Volume 1, for Transit New Zealand, Contract PA2084.
- Beca Carter Hollings and Ferner Ltd., May 2003: SH20 Avondale Extension, Geotechnical Factual Report, Stage 3, Volumes 1 & 2, Revision B, for Transit New Zealand Contract No. PA1871.
- Maunsell Ltd., December 2003: SH20 Mount Roskill Extension Western Termination, Geotechnical Factual Report, for Transit New Zealand Contract No. PA2084.
- Beca Infrastructure Ltd, March 2006, Western Ring Route - Waterview Connection (SH20 Avondale Extension), Factual Report Stage IV, for Transit New Zealand, Contract No. PA1871.
- Westlink, December 2006: Western Ring Route, Waterview Connection, ARC Sourced Borehole Logs, for Transit New Zealand.
- Connell Wagner, February 2007: Driven Tunnel Concept Feasibility Report, Western Ring Route, Waterview Connection, Preliminary Geotechnical Information, for Transit New Zealand.

- Connell Wagner, April 2007: Geotechnical Data Report, Volumes 1 to 6, SH20 Waterview Connection, Driven Tunnel Concept, for Transit New Zealand.
- Beca Infrastructure Ltd, December 2007, Western Ring Route - Waterview Connection, Factual Report, Regional Hydrogeological Bores, for Transit New Zealand.
- Beca Infrastructure Ltd, May 2008, SH20 Waterview Connection Factual Geotechnical Report – Driven Tunnel Hydrogeological Bores, for Transit New Zealand.
- Beca Infrastructure Ltd, May 2009, SH20 Waterview Connection Factual Geotechnical Report – Additional Tauranga Group Investigations, for NZTA.
- Aurecon, November 2009: Preliminary Geotechnical Report for the Causeway SH16 Waterview to Royal Road Widening, Revision 2, for NZTA.
- Tonkin & Taylor Ltd, August 2009: Draft Geotechnical Factual Report Waterview Connection – Maioro Interchange, for NZTA.
- Opus International Consultants, December 2009: SH16 Waterview Interchange, Geotechnical Review Report GS 09/131, for Tonkin & Taylor Ltd.
- Beca Infrastructure Ltd, April 2010: SH20 Waterview Connection CST Option - Geotechnical Factual Report – BH500 series, for NZTA.
- Beca Infrastructure Ltd, March 2010: SH20 Waterview Connection CST Option - Geotechnical Factual Report – BH700 series: Part 1, for NZTA.

In addition to the above information, additional sub-subsurface geotechnical investigations (700 Series: Part 2) were being undertaken for the current route alignment at the time of writing this report. Investigations that were available at the time of issuing this report have been incorporated as much as possible. The existing sub-surface investigations have been used to develop a geological model from which geological long sections and several typical cross sections for the proposed route alignment have been produced and these are included in Appendix B. Several interpretative reports have also been produced for various alternative schemes and routes but have not been referenced as they are generally for different design concepts and alignments.

1.3.2 500 Series Investigation (2009)

This investigation series was undertaken to develop the geological and hydrogeological model for the proposed SH20 motorway extension between Richardson Road and the Great North Road Interchange alignment route as at August 2009. The investigations included 30 rotary machine boreholes (BH501–BH509, BH511–BH519, BH521–BH532 and BH542 inclusive), 13 cone penetration tests (CPT571–CPT583), 22 percussion boreholes (PB541–PB562) and 38 test pits (TP201–TP218, TP301–TP308, TP311–TP316 and TP321–TP326). Various standpipe (28) and vibrating wire (13) piezometers were installed along the route within the investigation boreholes.

1.3.3 700 Series: Part 1 Investigation (2009-2010)

As a result of an alignment change (December 2009) to shorten the length of the tunnel section the 700 series investigations were undertaken to update the geological model along the new tunnel alignment and determine the hydrogeological effects associated with the Phyllis Reserve closed landfill. In addition specific investigations were undertaken at the cut and cover/ deep tunnel interface and in areas of significant geological uncertainty. The 700 series: Part 1 investigations included 30 rotary machine boreholes (BH701–BH717 exclusive BH715), 10 cone penetration tests (CPT750–CPT760 exclusive CPT755) and 10 test pits (TP401–TP410). Various standpipe (17) and vibrating wire piezometers (5) were installed along the route within the investigation boreholes. A groundwater pumping test was undertaken at a borehole (BH709b) located within Phyllis Street Reserve with additional local bores used as observations wells. Further investigations for the 700 series: Part 2 is currently ongoing.

1.3.4 Environmental Investigation (2009-2010)

Various boreholes and test pits have been undertaken for environmental testing and analysis throughout the project alignment by Beca. Some (BH542, TP201-TP218, TP301-TP308, TP311-TP-321 and TP401-TP410) of these investigations have also been logged for geotechnical conditions and this information has been incorporated into this report. The environmental investigations are assessed and reported on in the Waterview Connection Contamination Assessment - Interpretive Report (Beca, 2010d).

1.3.5 Geological/ Geotechnical Logging

The field logging of materials recovered from boreholes and inspection pits were undertaken in accordance with the New Zealand Geotechnical Society Field Description of Soil and Rock: Guidelines for the Field Classification of Soil and Rock for Engineering Purposes (NZGS, 2005).

1.3.6 Instrumentation

Various standpipe and vibrating wire piezometers were installed throughout the 500 and 700 series borehole investigations (Refer to factual reports for details). These installations are primarily related to understanding the regional groundwater to further define the hydrogeological model for analysis. These instruments have been monitored on a regular basis and are reported in the Waterview Connection Project SH16/ SH20 – Groundwater Assessment Report (Beca, 2010c).

Blank PVC pipes have been installed in various boreholes for future geophysical testing.

1.4 In situ Testing

In situ testing of materials along the proposed alignment route was carried out during the investigation stages of the project and includes standard penetration tests, dilatometer testing, pressuremeter testing, groundwater testing and geophysical testing.

1.4.1 Dilatometer Testing

Dilatometer testing was undertaken within boreholes located north of New North Road through to Great North Road. The tests were all performed within Waitemata Group rock of SPT N values greater than 50+ above and within the alignment route of the proposed tunnel. All data is presented in reports prepared by URS and collated in the 700 series: Part 1 factual report (Beca, 2010b).

1.4.2 Pressuremeter Testing

Pressuremeter testing (OYO Elasmeter 100 pressuremeter probe) was undertaken within weathered Waitemata Group (soils) at selected sites within the project route. All data is presented in reports prepared by Geotechnics Ltd and collated in the factual reports (Beca, 2010a & b).

1.4.3 Groundwater Testing

The groundwater testing covers a range of tests including packer permeability, rising head and pumping tests. All factual (Beca, 2010a & b) and interpretative information regarding groundwater is reported on separately. Refer to Waterview Connection Project SH16/ SH20 - Groundwater Assessment Report prepared by Beca for the interpretative groundwater discussions associated with this project (Beca, 2010c).

1.4.4 Geophysical Testing

Geophysical testing is yet to be commissioned. Seven boreholes drilled during the 700 investigations series: Part 1, for in situ dilatometer testing have been equipped with 65mm PVC pipes for future downhole geophysical logging. This testing will include Acoustic Televiewer, Natural Gamma, Full-wave Sonic logging to provide high resolution imaging of bedding and fractures, stratigraphy, physical properties and strength.

1.5 Laboratory Testing

Selected soil and rock samples recovered from the boreholes were tested at IANZ accredited geotechnical soils laboratories: Geotechnics Ltd, Geotest Ltd and Landcare Research Ltd.

The laboratory testing results are presented in the factual reports for the 500 and 700: Part 1 series borehole investigation (Beca, 2010a & b). Geotechnical design parameters have been determined from all available laboratory test results. These parameters are discussed in section 3.

1.6 Geological Model

A 3D Geological Interpretative Model of the Waterview Connection Project has been generated from an excel database of all investigation data (over 400 locations) known along the alignment route and surrounding area. Due to an increased understanding of geological unit distribution across the project route some reinterpretation of older geological data has been made within the database. The 3D Geological Model for the Waterview Connection Project was developed principally for use by hydrogeologists to produce sections at specific locations. Geological long sections and cross sections through the project route have also been produced from the 3D model and adopted for geotechnical analysis (Appendix B and C). The sections provided in this report are generalised to simplify presentation.

Contour plans have been developed for the following surfaces:

1. Base of Basalt based on borehole data

2. Top of Weathered Waitemata Group based on borehole data

3. Waitemata Group SPT N values >50+ i.e. top of ‘rock’ based on borehole data

Tauranga Group materials are not continuous across the proposed alignment therefore no specific contours have been developed. It is assumed that where present the Tauranga Group effectively lies at the ground surface and/ or between the base of basalt and top of weathered Waitemata Group contours, where a space exists.

Contour plans of the model surfaces are presented in Appendix C (Figure C.1 to C.3).

1.7 Assumptions and Applicability

This interpretation of ground conditions is based on the data presented in the SH20 Waterview Connection CST Option - Geotechnical Factual Reports (Beca, 2010a & b). A significant quantity of other factual data has been collected from other sources and has been included in the interpretation.

It should be noted that interpretations of ground conditions are made on the basis of observations at discrete locations and the continuity of ground conditions between observation sites cannot be guaranteed. Recommendations and opinions in this report are based on data from the various investigations. The nature and continuity of subsoil away from these investigations are inferred but it must be appreciated that actual conditions could vary from the assumed model.

This report has been prepared for the benefit of NZTA with respect to the particular project brief. The interpretations contained herein may not be used or relied on in any other context or for any other purpose without our prior review and agreement. Should there be any doubt as to the content or understanding of this report, the underlying geological assessment and geotechnical engineering recommendations for design as described herein, it is essential that these issues are discussed with NZTA and the authors before proceeding with any work based on this document.

2. Site Interpretation

2.1 Topography

The Waterview Connection Project extends between SH20 Maioro Street Interchange in the south, through Mount Albert joining into SH16 Great North Road Interchange in the north.

The southern end of the alignment, at Richardson Road, is located on an elevated ridgeline (54 m RL). The ground slopes downwards in a northerly direction towards Hendon Park and Alan Wood Reserve. The topography is relatively flat through these two reserve areas with an elevation of approximately 42.5 m RL at the southern end and 40.5 m RL at the north extent. Moderately sloping ground rises up to the west of the reserve area, with gently sloping ground to the east rising steeply in the area immediately surrounding the Mount Albert volcano. Oakley Creek follows a meandering path through these reserves where some man-made modifications to the creek are evident within the Hendon Park/ Alan Wood Reserve area.

On the northern side of New North Road, the surface topography drops down into a steep sided gully where the alignment (deep tunnel section) crosses under the North Auckland Rail Line (37.5 m RL) and Oakley Creek (30 m RL), rising back up the northern side of the gully to a maximum 46 m RL at the eastern end of Powell Street. From here the deep tunnel alignment follows beneath gently to steeply undulating surface topography alongside the western side of the Oakley Creek gully passing the eastern end of Cradock Street at approximately 40 m RL. Here the alignment passes beneath Oakley Creek to the eastern side and below Phyllis Reserve (previously an old landfill site) crossing beneath Oakley Creek again near Waterview Downs. The surface topography undulates where the deep tunnel alignment crosses beneath the deeply incised Oakley Creek. A 5.5 m high waterfall occurs within Oakley Creek north of Waterview Downs. The topography through this section reflects the eroding path of Oakley Creek with large steep embayment's along its western side where the deep tunnel interface sits beneath Oakley Creek Esplanade at lower points in the topography.

The alignment then follows a cut and cover tunnel underpass beneath Great North Road which is on relatively flat ground with a steeply incised valley to the east conveying Oakley Creek down towards the Great North Road Interchange. The ground level falls from approximately 25 m RL at the Great North Road Underpass to approximately 15 m RL at the northern portal in Waterview Reserve. Oakley Creek crosses Great North Road from east to west immediately south of the existing Great North Road Interchange and transitions into a tidal estuary (Oakley Inlet) on the western side of Great North Road.

The alignment to the east of the Great North Road Interchange and beyond Carrington Road over bridge is through a ridge, previously cut during the construction of the SH16 motorway. From Carrington Road the topography is relatively flat as it progresses towards St Lukes Road Interchange, crossing over Meola Creek with Chamberlain Park towards the south and Western Springs towards the north.

2.2 Geological Setting

The generalised regional geology of the Auckland area presented in Appendix C (Figure C.4) is based on published geologic maps (Kermode, 1992 and Edbrooke, 2001). An overview of the regional geological setting of the project area is provided below.

The Waterview Connection Project is situated within what is referred to as the Waitemata Basin, formed between 24 and 18 million years before present, which extended from North Waikato to Whangarei (Isaac et al, 1994). The Waitemata Basin at this time was a shallow marine subsiding (sinking) basin receiving sediments from eroding landforms to the west including the Manukau volcanoes. The sediment was dominated by inter-bedded silts and muddy sands with some coarser grained volcaniclastic sediments within the sequence.

As the basin subsided the sediments were buried to greater depths with some intra-formational slumping of semi consolidated sediments as delta fronts progressively developed and migrated over the basin. The basin is understood to have subsided to depths of between 1000 and 3000 metres (Isaac et al, 1994). The sediments infilling the basin were consolidating and in places cemented to form a thick sequence of inter-bedded weak siltstone and muddy sandstones, with inter-fingering deposits of volcaniclastic sandstone units (Parnell Grit Member). This geological sequence is now collectively referred to as the Waitemata Group.

Subsequent to this the Waitemata Basin has undergone uplift, faulting, folding and erosion over the past 18 million years. This has resulted in the distribution of Waitemata Group rock mapped today along the coast of North Shore and around the Waitemata Harbour. These rocks weather to form residual soils and completely to highly weathered material. Waitemata Group sediments form the ridges and valleys along the western and southern extents of the project alignment, in other areas they are intermittently overlain by thin terrestrial and shallow marine alluvial sediments of the Tauranga Group.

The Tauranga Group deposits were emplaced during the Pleistocene period (1.8 million years ago to 10,000 years ago) in topographically low areas. Tauranga Group soils, comprising Puketoka Formation and recent (<10,000 years ago) undifferentiated alluvium, are present intermittently within low-lying natural gullies across the site, and particularly within the main paleovalley system stretching from the Northwestern Motorway (SH16) in the vicinity of Oakley Creek towards the south. The thickness of the Tauranga Group sediments is highly variable, increasing within paleovalley systems and decreasing up valley flanks. Old stream courses, particularly those lower down the Oakley Creek valley, are often completely infilled by these sediments. During the course of deposition, fluctuating sea levels have resulted in both variations in the types of sediments deposited (sand, clay/silt and peat) and repeated cycles of deposition and erosion. This discontinuous and variable depositional environment is reflected in the changeability of the deposits. Recent colluviual soils have now accumulated in places along the stream banks of Oakley Creek overlying the various material types.

More than 30,000 years ago, (Searle & Mayhill, 1981) basalt lava, tuff and ash erupted through the Waitemata Group and/or Tauranga Group materials from the Mount Albert Volcano. The Basalt lava flowed down the flanks of the volcano and into the Oakley Creek paleovalley system to the southwest and west where it spread towards the flanks of the Waitemata Group ridges and infilled the valleys, then it spread out across the valley floor towards the north. The Oakley Creek paleovalley system is thought to have dammed against the new basalt lava at the southern end of Alan Wood Reserve at the time of the eruption (Kermode, 1992). Oakley Creek then overflowed and established a new course along the western most edge of the lava flow from Stoddard Road in the south to the Unitec property and the Northwestern Motorway in the north where it now flows into the marine estuary. Ash and tuff deposits produced in the final stages of the volcanic eruption cover some areas of the lava flows for up to 1 km from the vent.

The Mount Roskill and Three Kings Volcanoes (outside the project area) erupted sometime after Mount Albert Volcano, producing lava flows and ash deposits. In the upper reaches of Oakley Creek in the south (Richardson Road area), lava from Mount Roskill flowed into the valley however was likely blocked by the previous Mount Albert lava at the head of the valley (Searle & Mayhill, 1981). Lava flows from Three Kings were widespread filling deep valley systems, in particular the Meola Valley where it extends beneath Chamberlain Park/ SH16/ Western Springs out to the Waitemata Harbour.

2.3 Stratigraphy

A summary of the geology encountered at mapping and investigation sites over the project is presented in Appendix C (Figure C.4). In general, the geology encountered on site is in accordance with published geological maps of the area (Kermode, 1992 and Edbrooke, 2001). The site specific geology is explained in greater detail in the Section 2.7.

Materials that underlie the project route are known to vary considerably in terms of geologic complexity, physical properties, lateral extent and thickness. The following geological units have been mapped and/ or logged within the site (Table 2.1).

Table 2.1: Mapped Geological Units

Geological Units		Location
Man-made Deposits	Fill/ Landfill Refuse	SH16 Northwestern Motorway/ Phyllis Reserve Landfill/ Harbutt Reserve Landfill/ Alan Wood Reserve/ Hendon Park/ Maioro Street Interchange
Holocene Alluvium	Recent Stream Alluvium/ Estuarine Deposits	Oakley Creek/ Estuarine Deposits SH16 Northwestern Motorway (West) – not identified within SH20 Project
Auckland Volcanic Field	Basalt Tuff/ Ash	Eastern side of Oakley Creek
	Basalt Lava	Eastern side of Oakley Creek/ Great North Road Interchange/ Chamberlain Park
Tauranga Group	Undifferentiated Alluvium	Waterview Reserve/ Northern end Great North Road, Great North Road Interchange/ SH16 Meola Creek and lenses underlying the Basalt lava

Geological Units		Location
	Puketoka Formation	SH16 Northwestern Motorway (West) – not identified within SH20 Project
Weathered Waitemata Group	Residual Soils/ Weathered ECBF	Ridge crests along western side of Oakley Creek and underlying Basalt towards the east
	Weathered Parnell Grit Member	Lenses and inter-fingering unit through Waitemata Group particularly around Phyllis Reserve and Waterview Downs
Waitemata Group Rock	East Coast Bays Formation	Underlies whole site (best exposed in base of Oakley Creek and in deep boreholes).
	Parnell Grit Member	Lenses and inter-fingering unit through Waitemata Group particularly around Phyllis Reserve and Waterview Downs

These geological units are described below in more detail.

2.4 Geological Units

2.4.1 Man-made Deposits

Various areas of known man-made fill and landfill refuse deposits are located along the proposed route. Auckland City Council (ACC) is responsible for managing the closed landfills within the project limits and these are identified within the ACC Closed Landfills asset management plan 2009/2010 (ACC, 2009).

Environmental investigations and test pits have been undertaken in more detail by Beca and are assessed and reported on in the Waterview Connection Contamination Assessment - Interpretive Report (Beca, 2010d).

2.4.1.1 Landfill Refuse

a) Alan Wood Reserve Closed Landfill – This landfill contains known and likely unknown areas of localised uncontrolled landfill and construction debris through the area adjacent to the proposed southern tunnel portal. Some fill materials were observed in environmental test pits alongside the eastern and southern banks of Oakley Creek. Some contaminated land and fill materials were found in test pits in Hendon Park at the southern end of Alan Wood Reserve.

b) Phyllis Reserve Closed Landfill – This landfill officially closed in the late 1980's, having received primarily municipal solid waste since 1945. The area of the landfill comprises an abandoned basalt quarry of which the base lies between 8 m and 11 m below ground level. The quarry has been infilled with ACC parks waste, cleanfill and domestic refuse.

c) Harbutt Reserve Closed Landfill – This landfill is located directly to the south of Phyllis Reserve Landfill. The landfill was operated by a private contractor on behalf of ACC as an uncontrolled tip for clean fill probably between 1945 and the late 1980's.

For both Phyllis Reserve and Harbutt Reserve there was no preparatory or engineering work within the quarry areas before accepting waste. Therefore, there is no designed barrier to isolate the landfill from the surrounding geological environment.

The landfill refuse in both Phyllis Reserve and Harbutt Reserve has been found in investigations to comprise the following:

- Cleanfill: Basalt boulders (up to 0.5 m in diameter), demolition materials (concrete and brick gravels), broken shells, sand, clay and silt.

- Uncontrolled Fill: Silty clay, gravel, basalt rock, concrete, glass, timber, and steel

- Urban Refuse: Rubber, cloth, glass, plastic, basalt rock, wood, paper, steel, pottery, batteries, copper wire, shoes, bones.

Strong decomposing organic and hydrocarbon odours were noted in the refuse materials and methane gas monitoring was undertaken and encountered throughout the known landfill during investigations. Land contamination issues have been addressed and reported on by Beca Environmental (Beca, 2010d).

The Phyllis Reserve Closed Landfill was remediated approximately 10 years ago. The remediation works comprised construction of rock buttresses and gabion basket walls along the toe of the slope (immediately above the Oakley Creek Walkway), re-grading of the slope and capping of the landfill. It is understood the landfill remediation capping comprises approximately 150mm of topsoil underlain by a clayey material to a depth of 0.5 to 1m below ground level.

2.4.1.2 Engineered Fill

Sections of the Northwestern Motorway (SH16) around the Great North Road Interchange contain areas of localised construction fill comprised mainly of engineered recompacted clay fill.

2.4.2 Holocene Alluvium

Actively eroding stream banks along Oakley Creek are depositing Holocene (recent) stream alluvium over the low-lying areas of Oakley Creek and along the stream banks. The material being deposited is generally derived from erosional sediments along the stream bank and from the surrounding Waitemata Group within the Oakley Creek catchment. The stream alluvium commonly comprises soft, unconsolidated silts and clays and loose silty sands with organic lenses and it is generally thinly distributed.

Holocene marine alluvium (intertidal marine muds) is exposed at low tide in the northern section of Oakley Inlet/ Creek at Great North Road Interchange. These materials are commonly derived from coastal erosion and deposited by wave/ tidal action within the intertidal zone. Marine alluvium generally overlies Waitemata Group sediments in this area, although these recent materials were not observed in the 500 and 700 series borehole investigations within the alignment route as no drilling was undertaken at this stage in the northern section of the project. This area of the project and material parameters will be discussed in detail from reporting undertaken by Aurecon for the SH16 Causeway Project.

2.4.3 Auckland Volcanic Field

2.4.3.1 Basalt Ash and Tuff

Ash and /or tuff material mantles the basalt lava flows across some areas of the project route. It is described as reddish brown, firm to very stiff sandy silt (ash) and brownish orange, loose to medium dense silty gravel (tuff) comprised of scoria and lapilli. Thicknesses of tuff and ash vary from less than 0.5 m up to 2.0 m although this is highly variable depending on proximity to the vent and distribution along the project alignment route. Thicknesses recorded are typically less than 1.0 m.

2.4.3.2 Basalt Lava

The basalt lava is described as slightly weathered to unweathered, grey, often vesicular, very weak to strong basalt rock. The lava tends to be dense and non-vesicular in the centre of the thick lava flows and highly vesicular along the outer regions, with areas of rubble material recorded at the top and base of the lava flows.

The lava encountered in boreholes across the alignment is assumed to have erupted from the Mount Albert eruptive centre flowing west infilling the Oakley Creek paleovalley with lava being trapped by the western side of the valley creating an upper lava surface at 40 m to 42 m RL across the valley. Thickness of the lava varies considerably with recorded depths of up to 22 m. The lava flow thickness around the southern tunnel portal ranges between 8 m to 15 m.

No basalt lava or ash/tuff materials have been identified on the southern or western side of Oakley Creek.

Basalt lava from the Mount Roskill and Three Kings Volcanoes was not encountered in the locations along the recently investigated series (500 and 700: Part 1) however it is expected any basalt encountered from these centres will be of a similar nature to that already described.

2.4.4 Tauranga Group

2.4.4.1 Undifferentiated Alluvium

Tauranga Group soils, comprising recent (<10,000 years ago) undifferentiated alluvium, are present within low-lying natural gullies across the site, and particularly within the main paleovalley system stretching from the Northwestern Motorway in the vicinity of Oakley Creek towards the south. Within the main paleovalley gully system, which is likely offset to the east of the current location of Oakley Creek, these soils have locally been found to be up to 14 m thick, although they average up to 5 m thick and are sometimes not present within borehole investigations.

At the northern tunnel portals at Waterview Reserve, up to 16 m thickness of undifferentiated alluvium has been recorded. This area is interpreted to be related to an infilled basin forming an alluvial terrace at about 10 m to 15 m above sea level which likely occurred during Pleistocene sea level fluctuations.

These sediments unconformably overlie the Waitemata Group materials and predominantly comprise estuarine and terrestrial sediments in the northern section of the route and typically more terrestrial sediments in the south and southeastern areas of the project. These soils are generally soft to very stiff mixed soils comprising sandy silt, clay-silt and silty clay with variable organic content. Peat is locally present along with some lenses of sensitive pumiceous silt and sand.

The discontinuous and variable depositional environment of these materials is reflected in the changeability and variable thicknesses of this unit where encountered in investigation boreholes.

2.4.4.2 Puketoka Formation

The Puketoka Formation of the Tauranga Group overlies the Waitemata Group in some areas around the Auckland region, although these materials were not observed in the recent borehole investigations (500 and 700: Part 1) across the alignment route. It is expected these materials are more likely to be found along the SH16 section of the alignment, which is discussed in detail within reporting completed by Aurecon for the SH16 Causeway Project.

2.4.5 Waitemata Group

2.4.5.1 East Coast Bays Formation

The East Coast Bays Formation (ECBF) of the Waitemata Group forms the basement unit throughout the site and only outcrops in the base of Oakley Creek along sections of the route. This formation comprises interbedded extremely weak to weak sandstone and siltstone that has been gently folded and faulted. These rocks weather to form residual soils and completely to highly weathered material.

2.4.5.1.1 Weathered ECBF

Residual soils weathered from the underlying ECBF rock are typically 1 m to 5 m in thickness overlying less weathered ECBF with thicknesses of up to 10 m. The soils comprise orange brown to grey brown mottled, stiff to very stiff, silty clay and clayey silt of intermediate plasticity and loose to medium dense, fine to medium silty sand. With depth, the relict structure of the original rock mass is evident.

Depth of weathering can vary in areas where there has been recent erosion uncovering rock at the surface e.g. areas of Oakley Creek. The most extensive area of weathered ECBF encountered in investigation boreholes was at a discrete location along Great North Road and east though to Oakley Creek, where weathered materials of up to 37 m thickness were encountered.

2.4.5.1.2 ECBF Rock

ECBF rock consists of grey extremely weak to weak interbedded muddy sandstone and siltstone with minor mudstone. Individual beds of siltstone or sandstone are commonly between 0.2 m and 1.8 m thickness but have been noted to be as thin as 20 mm and as thick as 3 m. Individual beds are typically persistent for several hundreds of metres, although continuity of bedding between boreholes is difficult to establish.

The sandstone is grey, slightly silty, fine to medium grained and often coarsens to medium to coarse grained towards the base of individual beds. Additionally, rare 1 mm to 10 mm thick black carbonaceous laminations within the sandstones are observed. The siltstone beds are grey, with rare carbonaceous laminations and very rare 1 mm to 2 mm thick sandstone lenses. Siltstone beds typically become sandy towards their base.

The ECBF is poorly indurated and cementation is variable along the route. The very weakly (partially cemented) to uncemented sandstone has a UCS of less than 1 MPa and technically lies in the soil strength domain. However these materials are included as ‘rocks’ but are also given a soil description.

2.4.5.2 Parnell Grit Member

Fine to coarse grained volcaniclastic conglomerate/ gritstone can occur as channelised deposits within the ECBF, derived from submarine mass flow deposits from the Manukau volcanoes in the west. This volcaniclastic material is best observed at Red Bluffs near Campbells Bay and behind the Parnell Baths in Judges Bay where it takes its name as the Parnell Grit Member of the East Coast Bays Formation. A number of Parnell Grit deposits are present across the alignment route. These gritstones weather to soils of reddish pink sand and clay and occur in localised areas across the site.

Where encountered in rock outcrops along Oakley Creek, the Parnell Grit is described as moderately to highly weathered, very weak to weak, brownish grey, fine to medium grained conglomerate/ gritstone, with angular to subrounded volcaniclastic and lithic gravel clasts. The unweathered gritstone is dark bluish grey, mottled red, green and is weak to moderately strong.

The distribution of larger bed thicknesses of Parnell Grit appear confined to the area located beneath Phyllis Reserve/ Waterview Downs, in and around Oakley Creek. In this area, the Parnell Grit is interbedded with grey siltstone and is locally up to 20 m thick.

The inter-fingering nature of the lower and internal contacts in the Parnell Grit suggest that the grit beds were deposited by subaqueous mud flows (Keunen, 1950). The influx of the mudflows most likely caused overloading on graded subaqueous slopes in the area of Waitemata Group sediments and thereby initiated gravity slides (Brothers, 1953). This would explain the local exposures of intensely deformed and buckled Waitemata beds developed in association with Parnell Grit beds. The correlation between steeply inclined ECBF beds and the presence of Parnell Grit in the Auckland region has been observed by others (Brothers, 1953).

2.5 Geological Structure

Published maps of regional geology around the Waterview – Mount Albert area (Kermode, 1992 and Edbrooke, 2001) indicate structural patterns in the underlying geology dominated by structural features trending southwest to northeast and southeast to northwest. No faults are mapped in the project area as a part of the present study, however several lineaments have orientations similar to the regional sub-rectangular fault system (NE-SW to N-S trending faults).

Specific comment on structural detail for rock mass defects are presented in the following section.

2.5.1 Rock Mass Defects

Various defect sets are present throughout the Waitemata Group rock mass as observed in borehole drill core and exposures within Oakley Creek. Rock mass defects consisting of bedding plane partings, joints and crush/ shear zones, were observed within the drilled rock material where iron staining and some polished surfaces were recorded. The joints are generally open to tight, planar to undulating and have quartz, calcite or clay infill. The majority of joints recorded within ECBF rock at depth are predominantly clean.

Prominent subvertical joints are observed within drilled Parnell Grit material. The joints are either open or contain some mineral infill. Similar joint sets are observed in cliff outcrops of ECBF/ Parnell Grit exposures around Auckland, which also exhibit an open nature and iron staining and/ or mineral infill. Cementation indicates a passage of fluids through the joints which may indicate a high joint conductivity.

2.5.1.1 Bedding Defects

Bedding is the most prominent and well developed defect within the ECBF. In general, average bedding dips in logged boreholes are around 0 to 20 degrees (average 5 to 10 degrees) from the Harbutt Reserve area towards the south along the alignment (approximate chainage 0-2900). To the north of New North Road the bedding becomes more inclined in some areas with recorded bedding dip magnitudes of up to 80 degrees (approximate chainage 2900-3600; Figure 2.1). It is possible that bedding may be locally disturbed around intra-formational slump features within the ECBF and the Parnell Grit deposits. Variable dips are noted particularly adjacent to Parnell Grit beds in the Phyllis Reserve/ Waterview Downs/ Oakley Creek area. Here bedding dips vary between 25 and 80 degrees (See Figure 2.1).

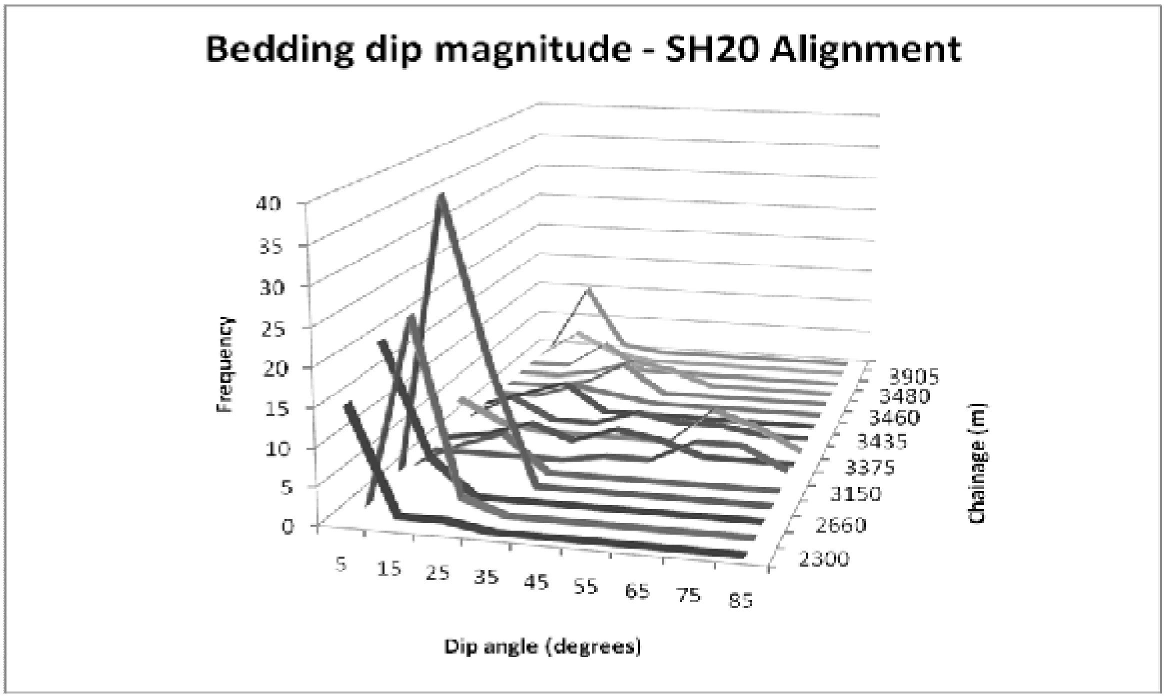

Figure 2.1: Dip magnitude of ECBF bedding along the SH20 tunnel alignment

Bedding dip direction within outcrops in the northern reaches of Oakley Creek (around the waterfall) are approximately 20 to 25 degrees to the south-southwest. Further borehole investigations are scheduled for this area to identify and model the areas of the geological uncertainty. No orientated or inclined core drilling has been undertaken at this stage of the project. Bedding shear surfaces and partings have developed along some of the interbedded layers as a result of tectonic movement. Evidence of possible shear planes has been observed within several boreholes. These provide preferential planes of weakness for instability and structural defects.

Table 2.2: Observed Shear Planes in Boreholes

Borehole ID	Depth (m) below ground level	Depth RL (m)	Comments
504a	24.45	16.84	Shear Zone; 15 mm
504a	49.6	-8.31	Shear Zone; 20 mm, 10 degrees
505a*	52.91	-11.88	Shear Zone; 20 mm, 15 degrees
507a*	58.2	-10.21	Fault Zone; 80 degrees
508*	31.85	20.14	Shear Zone; 60 degrees
508*	60.85	-8.86	Fault Zone; 65 degrees
508*	62.05	-10.06	Shear Zone; 5 mm, 10 degrees
508*	67.8	-15.81	Shear Zone; 10 mm, 10 degrees
509a*	31.45	9.47	Shear Zone; 10 mm, 10 degrees
517	40.0	2.14	Crush Zone; numerous >40m depth
524	22.35	18.75	Fault Zone; gouge offsets
526	22.0	19.78	Shear Zone; 3 degrees
526	31.45	10.33	Shear Zone; 20 mm, 5 degrees
529*	21.2	16.29	Shear Zone; 40 mm, 25 degrees
529*	45.7	-8.21	Shear Zone; 45 degrees
703a	47.2	-1.1	Fault, 60 degrees
703a	49.8	-3.7	Shear Zone; 5 mm, 30 degrees
703a	60.9	-14.8	Crush Zone; 15 mm
704	17.3	6.6	Shear Zone; 2mm, 22 degrees
704	22.25	1.45	Shear Zone; 2mm, 50 degrees

704	40.4	-16.7	Shear Zone, 8mm, 22 degrees
706a	31.9	-2.4	Fault Zone; 60 degrees
706a	55.8	-26.4	Fault Zone; 70 degrees
707a	45.9	-15.9	Fault Zone; 50 degrees
709c-1*	24.5	5.45	Shear Zone; 90mm
709c-1*	33.0	-3.05	Shear Zone; 20mm, 25 degrees
709c-1*	37.2	-7.3	Fault Zone; 5mm, 45 degrees
710a	44.5	-17.4	Fault Zone; 20mm, 70 degrees
711	30.9	-14.65	Shear Zone; 20mm, 40 degrees
712	29.3	-13.0	Crush Zone; 100mm
712	39.9	-23.6	Shear Zone; 15mm, 5 degrees
713	18.55	-3.25	Crush Zone; 50mm, 10 degrees
713	20.1	-4.8	Fault Zone; <5mm, 70 degrees
713	20.9	-5.6	Fault Zone; <5mm, 80 degrees
713	22.2	-6.9	Shear Zone; 25mm, 10 degrees
717b	3.2	13.85	Shear Zone; 20mm, 25 degrees

* Denotes >20m off alignment

Further structural analysis will be undertaken following further borehole investigations including inclined drilling along with downhole geophysical logging providing high resolution imaging of defects and bedding.

2.6 Seismicity

Auckland is in one of the least seismically active regions of New Zealand (Stirling et al, 2000). No active faults are known to exist in close proximity to the study area. Geological and Nuclear Sciences (GNS) have published a database of known active faults (Active Faults Database, 2010) which indicates that the closest proven active fault is the Wairoa North Fault located some 50 km southeast of the site. The Kerepehi Fault, though located outside the Auckland Region is classed as active, with a 5,000 – 10,000 year return period.

An additional study (Williams et al, 2006) of the potential for additional active faulting within the Auckland Region indicates that the Drury Fault on the western margin of the Hunua Range, South Auckland, should also be considered active. This fault is considered to have last ruptured approximately 45,000 years before present, with minimum uplift rates estimated at between 0.01 and 0.03mm/year.

Depths to rock do not exceed 25 m throughout the project alignment. The soils are generally within the firm to stiff range and hence the site subsoil seismic classification would be Class C (shallow soil) for all locations, in accordance with the definitions of NZS1170.5:2004.

2.7 Distribution of Geological Units – Sector Specific Conditions

The distribution of geological units along the SH20 alignment is discussed in more detail below and presented in the interpretive geological long sections in Appendix B – Drawings 252-255, 258-262 and 326-337. Only descriptions and depths for the geology along the alignment are discussed. The geology and interface depths may vary away from the alignment. The general horizontal alignment and area of the project is shown in Drawings 108-119 (Appendix A). Specific details of the current vertical alignments are shown on Drawings 252-255, 258-262 and 326-337 which are contained in Appendix B.

2.7.1 Sector 5: Great North Road Interchange

The geology of the Great North Road Interchange comprises basalt flows from Mount Albert Volcano overlying Tauranga Group alluvium and ECBF sandstones and minor siltstones. Localised areas of man-made fill are also present.

The Mount Albert basalt forms a continuous flow of varying thickness extending west along the northern side of Oakley Creek under the existing SH16 motorway/ Great North Road Interchange. The flow continues towards Oakley Inlet into the Waitemata Harbour for a limited distance. The basalt is up to 9 m thick in the centre of the flow and varies in strength and structure with some large voids encountered in previous investigations for the existing SH16 motorway. In this area beneath the Mount Albert basalt are discontinuous lenses of Tauranga Group alluvial sediments up to 5 m thick.

From Waterview Reserve (northern portal) to Great North Road Interchange the Tauranga Group alluvial sediments overlies weathered ECBF. The alluvial sediments are present at ground level in the south and west of the sector and have been recorded (in previous investigations) up to 16 m thick. The alluvial deposits thin towards the east and are absent between the existing Great North Road and Carrington Road over bridge on SH16.

Weathered ECBF sediments have been partly removed as a result of erosion prior to the deposition of Tauranga Group alluvial sediments. As a result, there is a minimal thickness of weathered ECBF rock (typically <3 m) underlying the Tauranga Group sediments in the south and west of the area. The thickness of weathered ECBF increases to the east between the overpass at Great North Road and the Carrington Road over bridge on SH16, where up to 8 m is present.

Areas of fill are present in this area as a result of the SH16 motorway construction. The most significant thickness is found in the embankment forming the overpass at Great North Road (up to 7.5 m thick), with minor thicknesses along the SH16 motorway corridor and existing ramps.

2.7.2 Sector 6: SH16 to St Lukes

From just east of the Carrington Road over bridge it is assumed that a thickness >10 m of Tauranga Group alluvial sediments are present in and around Meola Creek extending to the east where it underlies the Three Kings basalt flow at Chamberlain Park through to St Lukes Interchange. The Three Kings lava flow extending beneath Chamberlain Park extends from Meola Creek in the west through to St Lukes Interchange with thicknesses of up to 5 m recorded in previous investigations along the alignment.

2.7.3 Sector 7: Great North Road Underpass

From Waterview Reserve towards Great North Road along the alignment, Tauranga Group undifferentiated alluvium is present at ground level and overlies weathered ECBF at depth. A thick sequence of Tauranga Group overlies weathered ECBF rock in Oakley Creek Esplanade to the east of Great North Road. The undifferentiated alluvium is locally up to 13 m thick and comprises clay, silt and sand. The weathered ECBF is up to 12 m thick and typically comprises interbedded medium dense to dense fine sand and firm to stiff silt.

The Tauranga Group sediments observed in Oakley Creek Esplanade in the vicinity of Great North Road thin out southwards and are absent at the deep tunnel connection. The weathered ECBF comprises grey, dense interbedded fine sand and silt, which typically grade into grey unweathered ECBF siltstones and sandstones at depth. The ECBF rock appears disturbed and softened up to a depth of 23 m in the immediate vicinity of the northbound cut and cover section at the underpass/ deep tunnel interface (BH716 and BH717), with shallow intact unweathered ECBF blocks overlying deeper disturbed and softened zones. Rock strength has been reduced in these zones and sandstones typically range from very dense uncemented sand to extremely weak sandstone. The disturbed fabric indicates that the rock in this area has been displaced, possibly by faulting or block landslide movement. Further drilling investigations will be required in order to determine both the extent and cause of the disturbed rock.

2.7.4 Sector 8: Avondale Heights Tunnel

The geology varies considerably along the tunnel alignment and is presented here in four separate sections for ease of description.

2.7.4.1 Oakley Creek Esplanade to Phyllis Reserve

The geology in Oakley Creek comprises weathered ECBF sandstones and siltstones overlying gritstone (Parnell Grit). A deep weathered profile of up to 21 m is present under Oakley Creek, where the gritstone has been completely weathered to clayey silt and sand. The tunnel will intercept unweathered well cemented weak to moderately strong Parnell Grit under Oakley Creek. Bedding is typically moderately to steeply dipping along the creek (25-55º) and prominent open subvertical joints are observed in core. The limited lateral persistence of gritstone beyond Oakley Creek and the steep bedding suggests that the Parnell Grit in this area may be structurally controlled, although there is no direct evidence to support this. Further targeted drilling investigations are scheduled in this critical area to determine the control on the distribution of Parnell Grit.

2.7.4.2 Phyllis Reserve

The geology of the Phyllis Reserve area comprises Mount Albert basalt overlying Tauranga Group alluvium and interbedded ECBF siltstones and sandstones together with gritstone (Parnell Grit). The Mount Albert basalt flow is between 4 m and 8 m thick in the south of the reserve. In the north the basalt has been historically quarried and subsequently backfilled with landfill. The landfill material placed in the north of Phyllis Reserve is highly variable and up to 11 m thick. The underlying Tauranga Group alluvium decreases in thickness from 13 m in the south of the reserve to 2.5 m under the landfill in the north of the reserve. The alluvium comprises clayey silt and sand with some amorphous organic (peat) lenses. Below the Tauranga Group, interbedded ECBF sandstone and siltstone inter fingering with Parnell Grit is encountered where the deep tunnel alignment passes through.

2.7.4.3 Phyllis Reserve to New North Road

The geology between New North Road and Phyllis Reserve comprises a thick sequence of weathered ECBF overlying unweathered ECBF rock. The weathered ECBF mantle is up to 30 m thick and comprises loose to dense fine sand and firm to stiff clayey silt with localised zones of weathered volcanogenic clayey silt and sand (weathered Parnell Grit) up to 7 m thick.

The deep tunnel level in this area comprises cemented very weak to extremely weak interbedded ECBF siltstones and sandstones. Common 1 m to 4.5 m thick partially cemented extremely weak sandstone layers are found within the ECBF both in and below tunnel level in the vicinity of Harbutt Reserve and the southern part of Phyllis Reserve. A 5 m thick zone of well cemented weak to moderately strong gritstone (Parnell Grit) is found at the crown level of the tunnel in the vicinity of Harbutt Reserve/ Cradock Street. The lateral extent of this gritstone zone is inferred from borehole data.

2.7.4.4 New North Road to Alan Wood Reserve

This area is mantled by a layer of strong unweathered Mount Albert basalt lava between 5 m and 13 m thick. The thickest area of the basalt flow is found at the junction of New North Road and Hendon Avenue. The basalt flow overlies Tauranga Group undifferentiated alluvium and ECBF rock and has preserved the undulating paleo-topography of the underlying Tauranga Group alluvial surface. The Tauranga Group alluvium comprises stiff silt and clay of medium to high plasticity with some organic (peat) and pumiceous content. The Tauranga Group thickness is variable with a maximum of up to 10 m thick under Alan Wood Reserve.

The tunnel level intersects cemented very weak to extremely weak, subhorizontal to gently inclined interbedded ECBF siltstones and sandstones. Some thin <1 m zones of partially cemented to uncemented sandstone are found within the tunnel level and above the crown level but these are typically interbedded with cemented sandstones and siltstones. A 5 m thick zone of weak to moderately strong gritstone (Parnell Grit) is found within the tunnel level beneath the Pak N Save carpark on New North Road. Bedding within the gritstone is between 15 º and 20º. The lateral extent of this zone is unknown but is likely to be channelised and not as extensive as the sandstone units within the ECBF.

2.7.5 Sector 9: Alan Wood Reserve

The geology of the Alan Wood Reserve area comprises Mount Albert basalt eruptives overlying Tauranga Group undifferentiated alluvium and ECBF interbedded sandstones and siltstones. The basalt lava flow is confined to two large alluvial paleovalleys where up to 15 m of unweathered basalt is encountered. The underlying Tauranga Group alluvium is up to 6.5 m thick between the basalt paleovalleys and thins significantly to <1 m directly beneath the basalt flows. Weathered ECBF sandstones and siltstones are up to 9 m thick. The weathered profile thins significantly to <3 m under the paleovalleys where weathered sediments have been partly removed as a result of erosion prior and during the deposition of Tauranga Group alluvial sediments.

The geology at the southern tunnel portal comprises Mount Albert basalt lava overlying a thin sequence (<4 m) of Tauranga Group alluvium and ECBF sandstones and siltstones. The basalt flow terminates at the southern end of Alan Wood Reserve, where it infills an alluvial paleovalley to depths of up to 15 m. The greatest thickness of basalt is in close proximity to the crown of the southern tunnel portal. There is typically a rapid transition from Tauranga Group alluvium into underlying unweathered ECBF rock, with only a limited thickness of weathered ECBF silt and sand (<4 m).

2.7.6 Sector 10: Maioro Street Interchange

The geology of the Maioro Street Interchange (pre-existing information) comprises up to 8 m of weathered ECBF overlying unweathered very weak to extremely weak sandstones and very weak siltstones. The ECBF in this area typically has a higher frequency of siltstone interbeds when compared to the rest of the alignment, although the strengths of these siltstone beds do not differ from those in other areas.

A basalt flow overlies Tauranga Group alluvium and ECBF in the southern area of the interchange. The flow is up to 4.6 m thick and is considered to have been erupted from the Mount Roskill volcano.

Further descriptions, interpretations and information relating to this Sector of the project are reported on by URS (2009).

3. Design Parameters

3.1 Introduction

This section presents the basis and rationale for selection of geotechnical design parameters. These parameters relate to design for the SH20 part of the whole project, based on data from the various investigations for the SH20 options. They may differ from those adopted for the SH16 part of the project, although data for that part have been reviewed and taken into account if appropriate. We consider that the scope and detail of this parameter derivation to be appropriate to the current level of design. However, further phases of design will require more detailed interpretation as is appropriate for a project of this magnitude.

The parameters have been derived in consideration of the primary geotechnical design objectives for this specific project, namely:

Design of retaining walls

Design of tunnels

Settlements and ground movements associated with retaining walls and tunnels

Design of bridge foundations.

The parameters are given for primary geological units that have an impact on the design, namely:

- Tauranga Group
- Waitemata Group
  - Weathered Waitemata Group
  - Parnell Grit (Volcanogenic Unit)
  - Waitemata Group rock (unweathered)
- Mount Albert Basalt (Auckland Volcanic Field)

Other materials have been logged that are not considered to have major influence on the preliminary design. These include fills, refuse, minor volcanics (tuff/ash) and recent alluvium confined to the stream beds.

The data upon which this assessment of parameters is based have been compiled from previously issued factual reports together with the data from the recent investigations. The factual reports for the “500-series” boreholes have yet to be finalised but all in situ and laboratory test results have been included in the data base. At the time of writing this report, only part of the 700: Part 1 series boreholes programme has been completed. The test results from this part (BH701-BH717 excluding BH715) are included in the data base. Further 700 series: Part 2 investigation boreholes are underway.

Summaries of data for each of the primary units and each parameter are presented in Appendix D (with sub-appendices for each unit) in the form of graphical plots or tables. The following sections discuss the significance of the plots and give recommended design ranges. Permeability design parameters area included in the separate groundwater report (Beca, 2010c).

The recommended design parameters are required to be “moderately conservative” in accordance with the design philosophy. This “moderately conservative” assumption has been taken as approximately at the 25th percentile of the available data set for cases where it would be conservative to underestimate the design value. In some cases (e.g. density of materials) it may be unconservative to assume lower bound values, and so a median or average value should then be adopted.

3.2 General Considerations

Parameters are required both for the design of the various engineering features (retaining walls, tunnels, bridges, etc) and for the assessment of effects; in geotechnical terms being the assessment of ground movements associated with construction and long term operation.

The quantification of ground movements has two primary components, namely:

- “Mechanical” effects: Movements associated with mechanical stress changes such as deflection of retaining walls and formation of the tunnel void.
- Groundwater effects: Settlements due to consolidation of soils resulting from groundwater drawdown.

This report is concerned with determining geotechnical parameters for both of these components. However, the assessments of groundwater effects are also based on various in situ field tests (e.g. falling head permeability tests and pumping tests) which together with associated interpreted permeability values are covered in a separate report (Beca, 2010c).

The design of engineering features and the assessment of the “Mechanical” ground movements depend on both material strength and stiffness. Strengths for soils are characterised in terms of both “undrained strength” and effective stress parameters. For rocks, strengths are given in terms of unconfined compressive strength (UCS) and, where appropriate, also the effective stress parameters. Stiffness is characterised by Young’s Modulus (E) and a Poisson’s ratio (υ). The latter has not been measured and is based on normal accepted values. The E-values also depend on whether the deformation process is short term (undrained) or long term (drained). In general the mechanical deformations are the former, and hence the undrained modulus (E_u) is given.

Note that because the rock compressibility is closer to the compressibility of water than for soils, the need to differentiate between “drained” and “undrained” stiffness values is not so important. These values of modulus may be taken as applying to both short term and long term conditions.

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 (mv) 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, plots are given for the ratios (Cc/Cr), $Cc/(1+ e_o)$ and $Cr/(1 + e_o)$, where e_o is initial void ratio.

3.3 Tauranga Group

Data plots for the Tauranga Group are given in Appendix D1.

3.3.1 General Distribution and Description

The Tauranga Group unit is present throughout much of the alignment, generally at higher level (shallow depth) but underlying the basalt where that is present. The soils are generally fine-grained cohesive silty clays and clayey silts. Amorphous peat and organic soils are present in localised lenses, generally immediately under the basalt. The soils of the Tauranga Group in this region are lightly over-consolidated, probably through historical desiccation, and are generally firm to very stiff. The nature of the soils appears to be the same throughout the alignment except for the very northern portion at the Great North Road Interchange, where softer soils are evident.

3.3.2 Density and Water Content

Separate figures show summary plots of bulk density and natural water content versus depth together with a summary plot of dry density and natural water content.

There is a trend for increasing bulk density and decreasing water content with depth. However, this is not significant in relation to design for which the following values would be appropriate.

Table 3.1: Tauranga Group Density and Water Content

	Bulk Density Mg/m³	Adopted Unit Weight kN/m³	Water Content %
Average	1.87	19	36
25th percentile	1.78	-	26
75th percentile	1.96	-	42

3.3.3 Classification Data

The Atterberg Limits are summarised on a Casagrande plasticity chart. The data plot as low plasticity (CL) to high plasticity (CH) clays. It is significant that no results plot below the A-line. The data give an average plasticity index (PI) of 32% which suggests an average su/p' ratio of 0.23 (Skempton, 1957).

3.3.4 Undrained Shear Strength

A separate figure shows a summary of undrained shear strength (su) plotted against depth. The scatter is considerable and there is no clear trend in variation with depth. The average of all tests in this unit (461 No.) computes to 83 kPa, with a lower 25-percentile at 52 kPa. Also shown on this figure is an approximate su/p' = 0.23 line. This appears to mark an approximate lower bound to the data which confirms the overconsolidated nature of the unit.

A separate figure presents a summary of SPT N-values. The data are again scattered but support an increase in undrained strength with depth.

3.3.5 Effective Stress Parameters

The effective stress parameters have been determined by triaxial tests (consolidated undrained with pore pressure measurement) on undisturbed samples of this unit. A summary of the data is presented in terms of p'-q plots.

The data have also been examined to assess variations in parameters in relation to nature of soil (predominantly frictional v. predominantly cohesive) and to assess consistency (soft/loose v. stiff/dense). Differences are not marked (see separate figures), amounting to about ± 2° for angle of internal friction (ϕ') and ± 5 kPa for effective cohesion (c'). For design purposes, it is proposed to adopt the average angle of friction for all data (28°) but reduce the assumed effective cohesion to about 50% of the test results. For preliminary design, even more conservative cohesions have been adopted (i.e. 3 to 7 kPa for design range).

3.3.6 Compressibility Parameters

Plots of all the consolidation test data for Tauranga Group are given in Appendix D1. The following points should be noted:

i) The plot for all mv-values against effective stress shows wide scatter but primarily due to three "outliers". These all relate to boreholes in the Great North Road Interchange area. With the outliers removed (see separate plot) the m_v-values are confined to a reasonably tight band with the following recommended design values.

Table 3.2: Tauranga Group Coefficient of Consolidation

In situ effective stress kPa	m_v m^2/MN		
	Lower bound	Mean	Upper bound
20	0.10	0.3	0.6
40	0.08	0.275	0.55
60	0.07	0.225	0.50
80	0.055	0.20	0.47
120	0.05	0.18	0.40
200	0.04	0.14	0.28
400	0.04	0.13	0.20
600	0.04	0.102	0.18
800	0.04	0.11	0.16

These values are compatible with those proposed by Aurecon for SH16.

i) The plots for preconsolidation pressure (pc) are difficult to interpret due to scatter. Also shown on the plot is an approximate in situ vertical effective stress line. Clearly there is significant over-consolidation in places. For simplicity, a constant value of 80 kPa for the preconsolidation range (i.e. difference between in situ vertical effective stress and preconsolidation pressure) would appear to be conservative.

ii) The remaining plots give information for the log-based settlement approach. The following design values appear appropriate:

Table 3.3: Tauranga Group Compressibility Parameters

Parameter	Lower bound	Mean	Upper bound
C_c	0.1	0.23	0.37
$C_c/(1+e_o)$	0.06	0.13	0.20

C_r	0.01	0.04	0.08
C_c/C_r	4	6	9
$C_r/(1+e_o)$	0.005	0.025	0.045
c_v (m²/year)	2	12	41

3.4 Weathered Waitemata Group

Data plots for the Weathered Waitemata Group unit are given in Appendix D2.

3.4.1 General Distribution and Description

The unit includes all materials described under Weathered Waitemata Group with SPT N-values less than 50, as an arbitrary boundary. There is no distinction between residual soils and other weathering grades. The unit includes the predominant weathered East Coast Bays Formation (ECBF) and the volcanogenic soils termed "weathered Parnell Grit". These latter soils are distinctive and generally softer than the general unit and so separate data plots are provided in Appendix D2.

The unit is present throughout the alignment in varying thickness, typically 1 to 5 m but reducing to negligible in some areas, and as much as 10 m in places. The unit obviously forms the top portion of the Waitemata Group, underlying the Tauranga Group and the Mount Albert Basalt.

The weathered Parnell Grit sub-unit appears confined to the area located beneath Phyllis Reserve/ Waterview Downs, in and around Oakley Creek. In this area, the Parnell Grit is interbedded with grey siltstone and is locally up to 20 m thick.

For engineering purposes, the soils are generally cohesive silty clays or clayey silts in the firm to stiff strength range. Due to the weathering history, there appears to be an "equivalent" over consolidation characteristic. The weathered Parnell Grit, however, appears to be in the soft to firm strength range.

3.4.2 Density and Water Content

Plots of bulk density and water content with depth are given in Appendix D2. There is a hint of increase in density and decrease in water content with depth (probably reflecting the changing weathering grade) but this is again not significant for design.

The statistics give the following values for the unit as a whole.

Table 3.4: Weathered Waitemata Group Density and Water Content

	Bulk Density	Adopted Unit Weight	Water Content

	Mg/m^3	kN/m^3	%
Average	1.80	18	39.5
25th percentile	1.74	-	35
75th percentile	1.86	-	43

Plots are given for the weathered Parnell Grit member but the number of data points is limited and the differences are not considered significant.

3.4.3 Classification Data

Casagrande plots are shown for both sub-units in Appendix D2. The soils plot predominantly as high plasticity clays (CH). The few data points for the weathered Parnell Grit indicate slightly lower plasticity but the difference is not significant. The data give an average plasticity index (PI) of 40 for the weathered ECBF and 31 for the weathered Parnell Grit.

3.4.4 Undrained Shear Strength

The plots of undrained shear strength show considerable scatter with no obvious trends with depth. The data tend to suggest slightly lower strengths for the weathered Parnell Grit, supporting the visual observations. The following values are given by the statistics:

Table 3.5: Weathered Waitemata Group Undrained Shear Strength

	Average	25th percentile
Weathered ECBF	88	57
Weathered Parnell Grit	72	49

Also shown in Appendix D2 are summary plots of SPT data. These suggest an increasing trend in shear strength with depth.

3.4.5 Effective Stress Parameters

Effective stress parameters are based on the p'-q plots given in Appendix D2. Separate plots are given for the two sub-units.

Whilst the data for the weathered Parnell Grit are limited and include a significant "outlier", the computed parameters are considered appropriate because visual observations have indicated that these soils are noticeably softer than the main unit.

In accordance with the "moderately conservative" design objective, it is considered appropriate to adopt the effective friction angle given by the data but to reduce the effective cohesion by 50%.

The selected effective stress parameters are summarises as follows:

Table 3.6: Weathered Waitemata Group Effective Stress

		Data	Design
Weathered ECBF	φ'	30°	30°
	c´ (kPa)	15	8
Weathered Parnell Grit	φ'	26°	26°
	c (kPa)	10	5

3.4.6 Settlement Parameters

Various plots summarising the data from consolidation tests are given in Appendix D2 for both sub-units. The soils show characteristics equivalent to slightly over consolidated materials. The following points should be noted:

3.4.6.1 Weathered ECBF

i) The m_v-values are contained in a reasonably tight band with the following recommended design criteria:

Table 3.7: Weathered Waitemata Group Coefficient of Consolidation

In situ effective stress kPa	m_v m^2/MN		
	Lower bound	Mean	Upper bound
20	0.10	0.3	0.60
40	0.8	0.28	0.52
60	0.075	0.25	0.46
80	0.07	0.22	0.40
120	0.06	0.18	0.34
200	0.04	0.14	0.27

400	0.04	0.12	0.20
600	0.04	0.11	0.19
800	0.04	0.10	0.18

ii) For the log-based approach, the following design values appear appropriate:

Table 3.8: Weathered Waitemata Group Compressibility Parameters

Parameter	Lower bound	Mean	Upper bound
C_c	0.2	0.32	0.40
$C_c/(1+e_o)$	0.11	0.15	0.20
C_r	0.02	0.03	0.05
$C_{c/}\ C_r$	4	9	14
$C_r/(1+e_o)$	0.010	0.015	0.025
c_v(m²/year)	2	35	69

These statistics are based on all data except for parameter c_v for which two extreme outliers have been excluded.

The pre-consolidation pressures show a significant increase with depth. A constant difference between in situ effective vertical stress and pre-consolidation pressure of 110 kPa would be conservative.

3.4.6.2 Weathered Parnell Grit

Five consolidation tests were carried out on this sub-unit, of which one (BH124) was considerably less weathered than the others. This test has not been considered as an outlier due to the small sample size.

The following points should be noted:

i) The m_v-values all plot within the same envelope as the weathered ECBF given above.

ii) For the log-based approach, the following design values appear appropriate:

Table 3.9: Weathered Parnell Grit Compressibility Parameters

Parameter	Lower bound	Mean	Upper bound
C_c	0.10	0.28	0.42

$C_c/(1+e_o)$	0.05	0.13	0.18
C_r	0.012	0.033	0.053
$C_{c/}\ C_r$	7.08	8.32	9.90
$C_r/(1+e_o)$	0.007	0.015	0.023
$c_V(m^2/year)$	6	32	69

The "equivalent" pre-consolidation pressures show a clear increase with depth. Again, a constant difference between in situ vertical effective stress and pre-consolidation pressure of 110 kPa would be conservative. The differences between the compressibility parameters for the Weathered ECBF and Weathered Parnell Grit are minor. For the purpose of design and settlement analyses one set of parameters has been adopted, based on the most conservative values.

3.5 Waitemata Group Rock

Data plots to support the selection of parameters for the unweathered Waitemata Group rock are given in Appendix D3. These weak rocks have been tested both in the laboratory and in situ using a pressuremeter and a dilatometer (see Section 1.4). The principles are the same for these instruments and, for the purpose of discussion in this report, the term "pressuremeter" is used generically for both instruments.

The pressuremeter tests have been interpreted using the methods set out by Hughes et al. (1977). The tests provide a measure of both strength and stiffness.

3.5.1 General Distribution and Description

This unit forms the bedrock for the entire alignment. The top of this unit is defined as SPT N=50 or greater. The materials vary in nature, both in relation to grain size (siltstone / sandstone) and strength.

This unit will govern the design of the tunnel support and will have significant influence on the "mechanical" settlements above the tunnel. For this reason, an attempt has been made to differentiate between the different rock types in terms of grain size for the design parameters.

Included in this unit is the unweathered Parnell Grit. It is expected to be encountered in the tunnel alignment over a length of about 200 m and at other intermittent locations along the alignment. Visual logging suggests that the Parnell Grit is significantly stronger than the ECBF rock. However, apart from the UCS tests, there has been no differentiation between these rock types in the data interpretation.

At an early stage in the investigations, the significance of the “uncemented sands” within the ECBF was recognised. Sampling and testing were targeted at these zones and consequently there is a bias towards the weaker materials.

3.5.2 Density and Water Content

Plots of bulk density against depth, water content against depth and dry density against water content are given in Appendix D3.

The following design values are given by the following statistics.

Table 3.10: Waitemata Group Density and Water Content

	Bulk Density Mg/m³	Adopted Unit Weight kN/m³	Water Content %
Average	2.09	21	20.6
25th percentile	2.02	-	18.1
75th percentile	2.12	-	23.1

The average solid density computes to 2.70 from 24 tests, with very little variation (2.68 – 2.71).

3.5.3 Strength

The descriptive strengths of this unit vary considerably, from “uncemented” to “weak”. Strengths, therefore, have been characterised by both unconfined compressive strength (UCS) and effective stress parameters. UCS tests tend to be biased towards the higher strength due to selection of more cemented intact samples. The UCS results are summarised on a plot against depth, differential between ECBF and Parnell Grit rock types. Also shown on the plot are the 25th, 50th and 75th percentiles of all the tabulated data. The statistics give the following values:

Table 3.11: Waitemata Group Strength

	All Data	ECBF	Parnell Grit
UCS Median (MPa)	2.62	2.22	7.77
UCS 25th percentile	1.04	0.94	5.61
UCS 75th percentile	5.56	4.17	10.43

A frequency plot shows the rock to be predominantly “very weak”, with a mode strength (highest frequency) in the 1 to 1.5 MPa range, a median (50th percentile) strength of 2.2 MPa and a 25th percentile of 1.0 MPa. These are slightly different to the above values as they are based on a fitted log-normal distribution as opposed to the tabulated values. Also shown on the same plot is a frequency plot for the equivalent strength determined from the pressuremeter tests. These results suggest much higher strengths tested in situ, with an average of about 9.5 MPa.

The effective stress parameters have been determined from both triaxial tests and pressuremeter tests. For the latter, it is assumed that the test is drained and that the vertical effective stress is the minor principal effective stress (σ_3), from which a value of p’= ½(σ_1+σ_3) can be determined. Various plots are given in Appendix D3 and the following points should be noted:

i) p’-q plot showing all triaxial and pressuremeter data: The data for the two types of test appear compatible but strengths at higher confining pressures (p’ > 3000 kPa) are higher than expected. Proposed design criteria for the Class 1 to Class 3 rock are superimposed and show that design assumptions are conservative.

ii) p’-q plot for triaxial tests, differentiated by fines content: The differentiation appears to be significant only at low confining pressures where effective cohesion (c′) is negligible for sandstones (FC < 20%: uncemented sand), increasing to more than 300 kPa for the siltstone (FC > 40%).

iii) Stress-strain curves for triaxial tests, differentiated by fines content: Again, the differentiation is not significant but the majority of materials exhibit loss of strength with strain (brittleness), with peaks generally occurring at strains less than 2.5%.

iv) p’-q plot for triaxial data showing both peak strengths and values at 3% strain: This illustrates how the stress paths trend towards the origin with decreasing cohesion. Also shown on this plot are the proposed design criteria for Class 1 to Class 3 rock, together with a proposed residual strength relationship of ϕ’ = 40° and zero effective cohesion.

v) p’-q plot for triaxial data at 3% strain, differentiated by fines content: The sandier soils (FC < 20%) tested in triaxial cell tend to give a lower bound.

vi) Plot of pressuremeter stress-strain curves, differentiated by depth range: As expected by theory, the data suggest a trend of lower strengths for depths down to 30 m. However, thereafter strengths are generally higher and there is no clear trend with depth.

vii) Frequency plot for effective cohesion based on triaxial data, assuming a uniform effective friction angle ϕ’ = 40°: Because of the very few results below the zero cohesion intercept; the assumption of ϕ = 40° appears appropriate for both peak and residual strengths criteria.

3.5.4 Stiffness

Stiffness for this unit is characterised by two values of Young’s Modulus as follows:

- E_i : Initial, low-strain modulus as first loading
- $E_{u/r}$: Unload / Reload modulus.

These values have been determined from both triaxial tests and pressuremeter tests.

Other stiffness modulus values can be deduced with an assumption of Poisson's ratio. This has not been measured and may be based on normal accepted values for weak sedimentary rock ($\nu = 0.25$).

Initial rock modulus (E_i) against the unload/reload modulus ($E_{u/r}$) shows a consistent trend of $E_{u/r} = 2E_i$.

A summary of both E_i and $E_{u/r}$ for triaxial and pressuremeter data is given plotted against strength, taken as q-peak. As is normal, there is a trend for increasing stiffness with strength. The following relationships appear reasonable for the data:

$$E_i = 100\ q \text{ and } E_{u/r} = 200\ q.$$

The settlement analyses have assumed E-values independent of strength. This is clearly conservative and settlements for Class 1 and Class 2 rock would likely be significantly less than computed.

Various authors have suggested that the unload/reload modulus would be more appropriate for real situations (e.g. Haberfield & Johnston, 1993 and Ervin et al., 1980). However, for preliminary assessment of settlements, the initial modulus (E_i) has been adopted and hence should be conservative.

3.6 Auckland Volcanic Field Basalt

A number of core samples were tested for bulk density. The results ranged from 2.56Mg/m^3 to 2.93Mg/m^3, with an average of 2.77 Mg/m^3.

Strengths of basalt were not measured. Intact basalt can be assumed to be in the very strong to extremely strong range. Rock defects will generally control the rock mass strength.

Modulus of the basalt rock mass has been taken as 10,000MPa, based on normally accepted values.

4. Summary of Parameters

Summaries of the recommended design parameters are given in Tables 4.1 and 4.2. In general, these are based on the test data, as discussed above. For the preliminary settlement and ground movement analyses, some of the parameters adopted differ slightly from the values given in the tables. This is due to the need to commence analyses before all the test data were available. In all cases, the adopted parameters were either more conservative than the recommended values or the differences were insignificant.

Notes are given with each table to indicate where adopted values differ from the recommended values.

Table 4.1: Classification and Strength Parameters

Geological Unit	Classification Parameters				Strength Parameters					
	Bulk Density Mg/m^3	Water Content %	Atterberg Limits		S.. kPa	UCS MPa	c’ (peak) kPa	**F’** (peak) °	c’ (residual) kPa	**F’** (residual) °
			PI	LL						
Tauranga Group	1.86	36	32	60	52(1)	-	3	28°	-	-
Waitemata Group (Weathered)										
Weathered ECBF	1.78	39.5	40	70	57(1)	-	8	30°		
Weathered Parnell Grit	1.78	39.5	31	60	49(1)	-	5	26°		
Waitemata Group (unweathered)										
ECBF Rock (General)	2.08	20.6	-	-	-	2.2	50(2)	40°		
Parnell Grit (General)	2.08	20.6	-	-	-	7.8	50(2)	40°		
Waitemata Group (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°	-	-

Notes

(1) Undrained strengths at 25th percentile

(2) Constant cohesion adopted for all rock classes in retaining wall design

Table 4.2: Compressibility and Stiffness Parameters

Geological Unit	Compressibility Parameters[4]							Stiffness Parameters (Youngs Modulus and Poissons Ratio)			
	c_v	p_c kPa	C_c	C_r	C_c/C_r	$C_c/(1+e_0)$	$C_r/(1+e_0)$	E_i MPa	$E_{u/r}$ MPa	n_u	n
Tauranga Group	12	s_v'+ 80	0.23	0.04	6	0.13	0.024	10[3]	-	0.49	0.35
Waitemata Group (Weathered)											
Weathered ECBF	35	s_v'+ 110	0.32	0.03	9	0.15	0.017	20[3]		0.49	0.30
Weathered Parnell Grit	32	s_v'+ 110	0.28	0.033	8.5	0.13	0.015	20[3]		0.49	0.30
Waitemata Group (unweathered)											
ECBF Rock (General)								150[5]	300	0.25	0.25
Parnell Grit (General)								150[5]	300	0.25	0.25
Waitemata Group (Tunnel Design)											
ECBF Rock Class 1								1050[6]	2100	0.25	0.25
ECBF Rock Class 2								450[6]	900	0.25	0.25
ECBF Rock Class 3								150[6]	300	0.25	0.25

Basalt Rock								10000		0.15	0.15

Notes

(3) Undrained modulus values
(4) All compressibilities taken at median values
(5) Constant rock mass modulus taken for all rock classes
(6) Values assumed proportional to rock class strength.

5. References

Auckland City Council (ACC) (2009) Closed Landfills asset management plan 2009/2010 prepared by ACC Environmental and Utility Management August 2009.

Beca (2010a) SH20 Waterview Connection CST Option – Geotechnical Factual Report – BH500 Series (May 2010 Rev. B)

Beca (2010b) SH20 Waterview Connection CST Option – Geotechnical Factual Report – BH700 Series: Part 1 (May 2010 Rev. B)

Beca (2010c) Waterview Connection Project SH16/ SH20 – Groundwater Assessment Report (In-Press – May 2010)

Beca (2010d) Waterview Connection Contamination Assessment – Interpretative Report (In-Press – May 2010)

Brothers, R.N. (1953) The Relationship of Waitemata Formation and the Manukau Breccia, Auckland, New Zealand. Transactions of the Royal Society of New Zealand, 81, pp521-538.

Connell Wagner (2007) Geotechnical Data Report - SH20 Waterview Connection Driven Tunnel Concept. April 2007 Reference 22977.001.03 Rev. 2 prepared for Transit NZ.

Edbrooke, S.W. (Complier) (2001) Geology of the Auckland area. 1:250,000, Geological Map 3. Lower Hutt, New Zealand. Institute of Geological and Nuclear Sciences Limited.

Ervin, M.C., Burman, B.C. and Hughes, J.M.D. (1980). The use of a high capacity pressuremeter for design of foundations in medium strength rock. Int. Conf. On Struct. Found. On Rock, Sydney.

Haberfield, C.M. and Johnston, I.W. (1993). Factors influencing the interpretation of pressuremeter tests in rock. Geotechnical Engineering of Hard Soils – Soft Rocks. Anagnostopoulos et al. (eds.), Balkema, Rotterdam.

Hughes, J.M.O., Wroth, C.P. and Windle, D. (1977). Pressuremeter tests in sands, Geotechnique 27.

Institute of Geological and Nuclear Sciences, Active Faults Database, web address, http://maps.gns.cri.nz/website/af/index.html Wellington, New Zealand.

Isaac, M.J., Herzer, R.H., Brook, F.J., Hayward, B.W. (1994) Cretaceous and Cenozoic Sedimentary Basins of Northland, New Zealand. Institute of Geological & Nuclear Sciences Monograph 8, 203p and 4 sheets.

Kermode, L.O., (1992) Geology of the Auckland urban area. Scale 1:50,000, Geological Map 2. Lower Hutt, New Zealand. Institute of Geological and Nuclear Sciences Limited.

Keunen, P.H (1950) Slump structures in the Waitemata Beds around Auckland. Transactions of the Royal Society of New Zealand, 81, pp467-475.

New Zealand Geotechnical Society (NZGS) (2005) Field Description of Soil and Rock. Guideline for the field classification and description of soil and rock for engineering purposes. New Zealand Geotechnical Society Incorporated.

Searle, E.J. revised by Mayhill, R.D. (1981) City of Volcanoes: A geology of Auckland. Longman Paul.

Skempton, A.W. (1957). Discussion. Proc. Inst. Civ. Eng. 7 and in Bishop & Henkel: The Measurement of Soil Properties in the Triaxial Test, 2nd Ed. Edward Arnold.

Stirling, M., McVerry, G., Berryman, K., McGinty, P., Villamor, P., Van Dissen, R., Dowrick, D., Cousins, J., Sutherland, R. (2000) Probabilistic Seismic Hazard Assessment of New Zealand: New Active Fault Data Seismicity Data Attenuation Relationships and Methods. Institute of Geological & Nuclear Sciences Client Report 2000/53 Project Number 41984B.10.

Williams, A. L., Olsen, J., Prebble, W. M., Locke, C. A., Cassidy, J., Shane, P., and Coleman, J., (2006) Active Faulting in the Auckland Region, Earthquakes and Urban Development, New Zealand Geotechnical Society 2006 Symposium, Nelson. The Institute of Professional Engineers, Wellington.

URS (2009) Western Ring Route – Maioro St Interchange – Geotechnical Interpretative Report. Final, December 2009. Document Reference No.20.1.11-R-J-1301.