Central Interceptor

Main Project Works Detailed Design

WATERCARE SERVICES LIMITED

Geotechnical Interpretative Report

PWCIN-DEL-REP-GT-J-100048 | 6

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Document history and status

Revision	Date	Description	By	Review	Approved
3	18 Dec 2015	Draft interim report for Watercare review	R Roberts	A Campbell	N Kay
4	17 June 2016	Issued for Watercare Use. Report incorporates peer review comments (February 2016) and Phase 2 (2016) site investigations	H McEwan	A Campbell	N Kay
6	8 February 2017	Draft for Watercare review. Revision includes interpretation of CIGI3 investigation results at Chamberlain Park and Keith Hay Park areas, update to site classification based on site specific analysis, inclusion of geochemical data graphs.	H McEwan	A Campbell	N Kay

Contents

This report is split into two volumes:

PWCIN-DEL-REP-GT-J-100048 Volume 1	Main text
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Executive Summary

The proposed Central Interceptor (CI) tunnel is a new 13 km long, 4.5 m diameter wastewater tunnel from Western Springs to Mangere Waste Water Treatment Plant in Auckland. It will lie between 21 and 107 m below ground level, and cross the Manukau Harbour at a depth of approximately 15 m below the seabed. There will be 10 shafts up to approximately 80 m deep on the main alignment including three large diameter working shafts, one of which will also serve as the pump station at Mangere WWTP.

The project also incorporates two link sewers (referred to as Link Sewers B and C) adding a further 4.5 km of smaller diameter tunnels and seven shafts to the project. Link Sewer A, Link Sewer D and three shafts were initially part of the project but have been eliminated as part of the preliminary design process.

Watercare has engaged Jacobs in association with AECOM (formerly URS) and McMillen Jacobs Associates as Principal Engineering Advisor responsible for undertaking investigations and preparing designs and construction documentation for CI project.

This Geotechnical Interpretive Report (GIR) details the interpretation of geological, hydrogeological, contamination and geotechnical conditions based on data available from concept design and the ground investigations undertaken during 2015 and 2016 to inform the preliminary design (as reported in the Geotechnical Factual report, Document No. PWCIN-DEL-GT-J-100047).

A geological framework for the project is presented together with geotechnical design information for the geotechnical units which are anticipated to be encountered.

The main geological units within the project area are:

1. East Coast Bays Formation
2. Parnell Volcaniclastic Conglomerate (Parnell Grit)
3. Kaawa Formation
4. Tauranga Group - Puketoka Formation
5. Auckland Volcanic Field basalt, tuff, ash and scoria
6. Recent alluvium
7. Made ground

The majority of the tunnel will be through the relatively well understood East Coast Bays Formation. Previous tunnelling projects including Project Hobson, Waterview and Rosedale have been successfully completed through this formation.

The key geotechnical risks identified are described in the table below.

Geotechnical Risk Name	Description	Proposed mitigation
Basalt within the tunnel horizon	Basalt may be encountered as a mixed face or full face condition. This will result in a change to the construction methodology, reduced productivity and higher machine tool wear, and potentially deflection of TBM resulting in alignment or grade issues.	It is expected that this will be addressed by specifying that the TBM will need to be able to bore through basalt for a minimum distance. To characterise this distance an intensive investigation in the zones with lowest cover has been undertaken, and will be further investigated in the second phase of the ground investigation to reduce the risk cost.
Parnell Volcaniclastic Conglomerate	Where Parnell Volcaniclastic Conglomerate (PVC, also known as Parnell Grit) has been encountered on previous projects it has been associated with hard boulders and high groundwater inflows. This material forms laterally discontinuous channels which makes it extremely hard to predict the location without an extremely intensive investigation of the whole alignment. The investigations to date have shown a relatively low quantity of PVC (approximately 2% of recovered rock) and have shown that the strength of the material is lower than would have been expected from previous experience.	The risk of encountering this material will be addressed by specifying that the TBM should be able to bore through a defined length of PVC up to the strengths encountered on previous similar projects. The exact quantity of PVC to be allowed shall be set in the GBR.
Variable and undulating contact between soil and rock units	Mixed face conditions comprising soil and rock units may result in face loss/pressure loss, tunnel face instability, ground surface movement, or deflection of TBM resulting in alignment or grade issues. These conditions are not significantly more onerous than those that have been encountered on other similar projects in Auckland.	This risk will be addressed by defining the likely scenarios in the GBR.
High groundwater head	The groundwater head is high for this diameter of tunnel and may result in tunnel inundation during construction or higher than acceptable seepage into tunnel.	This risk will be addressed by defining likely scenarios in GBR and specifying realistic seepage criteria for finished tunnel.
Basalt rock mass variability	Basalt may vary between intact basalt rock to rubbly gravel. This may result in tunnel or shaft instability, shaft excavation construction methodology changes, and additional loading on structure and support. Trenching may require additional support in rubbly gravel.	This risk will be addressed by defining the variability in the GBR.
Groundwater inflow in basalt	Basalt is typically vesicular and jointing allowing high water inflows. This may result in disruption to excavation work, pre-treatment such as grouting, or temporary pumping. Basalt is typically vesicular and jointing allowing high water inflows. This may result in disruption to excavation work, pre-treatment such as grouting, or temporary pumping.	This risk will be addressed by defining the variability in the GBR, Pre-defined mitigation measures ready to implement, and applying for groundwater consents in advance.

Geotechnical Risk Name	Description	Proposed mitigation
Wood fragments and logs within soils	Wood fragments and logs have been encountered within Tauranga Group and could also occur within Recent Alluvium and Kaawa Formation. Large wooden fragments or logs could impact constructability.	This risk will be addressed by defining the variability in the GBR.

List of Abbreviations

Abbreviations	Description
(Waterview)	Laboratory data from Waterview project
AC	Auckland Council
ACM	Asbestos Containing Material
AS	Associated shafts / Access shafts
AVF	Auckland Volcanic Field
AVFB	Auckland Volcanic Field Basalt
bgl	Below ground level
BH	Machine borehole
CH	Chainage
CI	Central Interceptor
CLSMP	Central Interceptor – Project Contaminated Land Site Management Plan
CoPC	Contaminants of potential concern
CPT	Cone Penetrometer Test
DDT	Dischlorodiphenyltrichloroethane
DPCIN	Drainage Pumping station Central Interceptor Number
DSCIN	Drainage Sewer Central Interceptor Number
DSLSB	Drainage Sewer Link Sewer B
DSLSC	Drainage Sewer Link Sewer C
DSLSD	Drainage Sewer Link Sewer D
DWG	Drawing
ECBF	East Coast Bays Formation
HA	Hand auger
k	Hydraulic conductivity
L1S	Link Sewer 1 / Link 1 Shaft
L2S	Link Sewer 2 / Link 2 Shaft
L3S	Link Sewer 3 / Link 3 Shaft
MfE	Ministry for the Environment
NC	No criteria
NES	National Environmental Standard
OCP	Organochlorine pesticide
PAH	Polycyclic aromatic hydrocarbon
PID	Photoionisation detector
PS	Pump Station

Abbreviations	Description
PSD	Particle size distribution
Q1a	Recent Alluvium
S	Storativity values
SPT	Standard Penetration Test
STP	Standpipe
SVOC / VOC	Semi-volatile organic compound / volatile organic compound
SY	Specific yield
T	Transmissivity
T&T	Tonkin & Taylor Limited
Tauranga Estuarine	Estuarine sediments, Tauranga Group
Tauranga Puketoka	Puketoka Formation, Tauranga Group
Tauranga Undif	Undifferentiated alluvium, Tauranga Group
TP	Test pit
TPH / BTEX	Total petroleum hydrocarbons / Benzene toluene Ethylbenzene and the Xylenes
UCS	Unconfined compressive strength
VW	Vibration Wire Piezometer
Waitemata MW-UW	Moderately weathered to unweathered ECBF
PVC	Parnell volcaniclastic conglomerate
Waitemata RW-HW Cl	Residually to highly weathered – cohesive soils
Waitemata RW-HW Gr	Residually to highly weathered – granular soils
WS	Work shaft
WWTP	Wastewater treatment plant
SWL	Standing water level
ISRM	International Society for Rock Mechanics
BRTS	Bamford Rock Testing Services
Coffey	Coffey Geotechnics NZ Limited
IANZ	International Accreditation New Zealand
SANAS	South African National Accreditation System
NATA	National Association of Testing Authorities, Australia
UU triaxial	Unconsolidated Undrained Triaxial
CU triaxial	Consolidated Undrained Triaxial
GIR	Geotechnical Interpretative Report
TBM	Tunnel Boring Machine

1. Introduction

1.1 Project overview

The proposed Central Interceptor (CI) tunnel is a new 13 km long, 4.5 m diameter wastewater tunnel. It will lie between 21 and 107 m below ground level, and cross the Manukau Harbour at a depth of approximately 15 m below the seabed. The 13 km long alignment investigated runs from Western Springs to Mangere Waste Water Treatment Plant in Auckland.

The project also incorporates two link sewers (referred to as Link Sewers B and C) adding a further 4.5 km of smaller diameter tunnels to the project, and 17 shafts up to approximately 78 m deep. Link Sewers A and D, together with three further shafts were initially part of the project but have been eliminated as part of the preliminary design process.

Watercare has engaged Jacobs in association with AECOM (formerly URS) and McMillen Jacobs Associates as Principal Engineering Advisor responsible for undertaking investigations and preparing designs and construction documentation for CI project.

1.2 Scope and purpose of this document

This Geotechnical Interpretive Report (GIR) details the interpretation of geological, hydrogeological, contamination and geotechnical conditions based on data available from concept design and the ground investigations undertaken during 2015 and 2016 to inform the preliminary design (as reported in Document No. PWCIN-DEL-GT-J-100047).

A geological framework for the project is presented together with geotechnical design information for the geotechnical units which are anticipated to be encountered.

Information provided in this GIR includes:

- A review of published geological information regarding Auckland geology relevant to the CI project.
- Review of site investigations undertaken for the project to-date, including the Matakite concept design team and those undertaken during 2015 and 2016.
- A geotechnical model of the main tunnel and each of the branch sewers with more detailed local models at each shaft location.
- Discussion of the range of geotechnical materials to be encountered and geotechnical risks.
- Recommended geotechnical design parameters for preliminary design.
- Analysis of aquifer testing and interpretation of groundwater information.
- Commentary of contamination assessments undertaken at shaft locations.

1.3 Structure of this report

This report is split into two volumes:

PWCIN-DEL-REP-GT-J-100048 Volume 1	Main text
PWCIN-DEL-REP-GT-J-100048 Volume 2	Appendices (including drawings)

1.4 Sources of factual information

Sources of factual information used to produce this report are presented in **Table 1-1**.

Table 1-1 Sources of factual information

Name	Author	Year	Document Number	Use for this report
Central Interceptor Main Project Works Detailed Design – Geotechnical Factual Report	Jacobs in association with AECOM and McMillen Jacobs Associates	2017	PWCIN-DEL-GT-J-100047	Site Investigation data used for geotechnical parameters, risks, geological and geotechnical models.
Central Interceptor and Associated Works: Phase 1 to Phase 4 Geotechnical Investigation	Matakite Central Interceptor Programme Team	2010 to 2011	N/A	Interpretation and site investigation data used for geotechnical parameters, risks, geological and geotechnical models.
Geotechnical Data Report – Waterview Connection NZTA	Aurecon New Zealand	2011	208955-AC-RPT-029-1/GET-GDR Rev 2.0	Laboratory data used for geotechnical parameters.
Rosedale Outfall – In situ Stress Testing Interpretative Report	CW-DC Limited	2008	24975-GEO-50008	
SH20 Mt Roskill Extension: Geotechnical Factual Report	AECOM New Zealand	2002	N/A	Site investigation data used for geological models
Western Interceptor Manukau Siphon Test Borings	Auckland Metropolitan Drainage Board	1955	See phase 1, volume 5 of Central Interceptor and Associated works geotechnical investigations (Matakite)	Site investigation data used for geological models

2. Geological setting and history

Understanding the geological history and mode of emplacement of the soils and rocks across the site is fundamental to comprehending the inherent variability and risks likely to be present. This section describes the geological history of the site in detail to aid this understanding.

2.1 Regional setting and evolution

Auckland is located within the Australian tectonic plate about 400 km northwest of the subducting Pacific plate. The temporal and geographical distribution of the main geological units encountered across the site is shown in Figure 2.1.

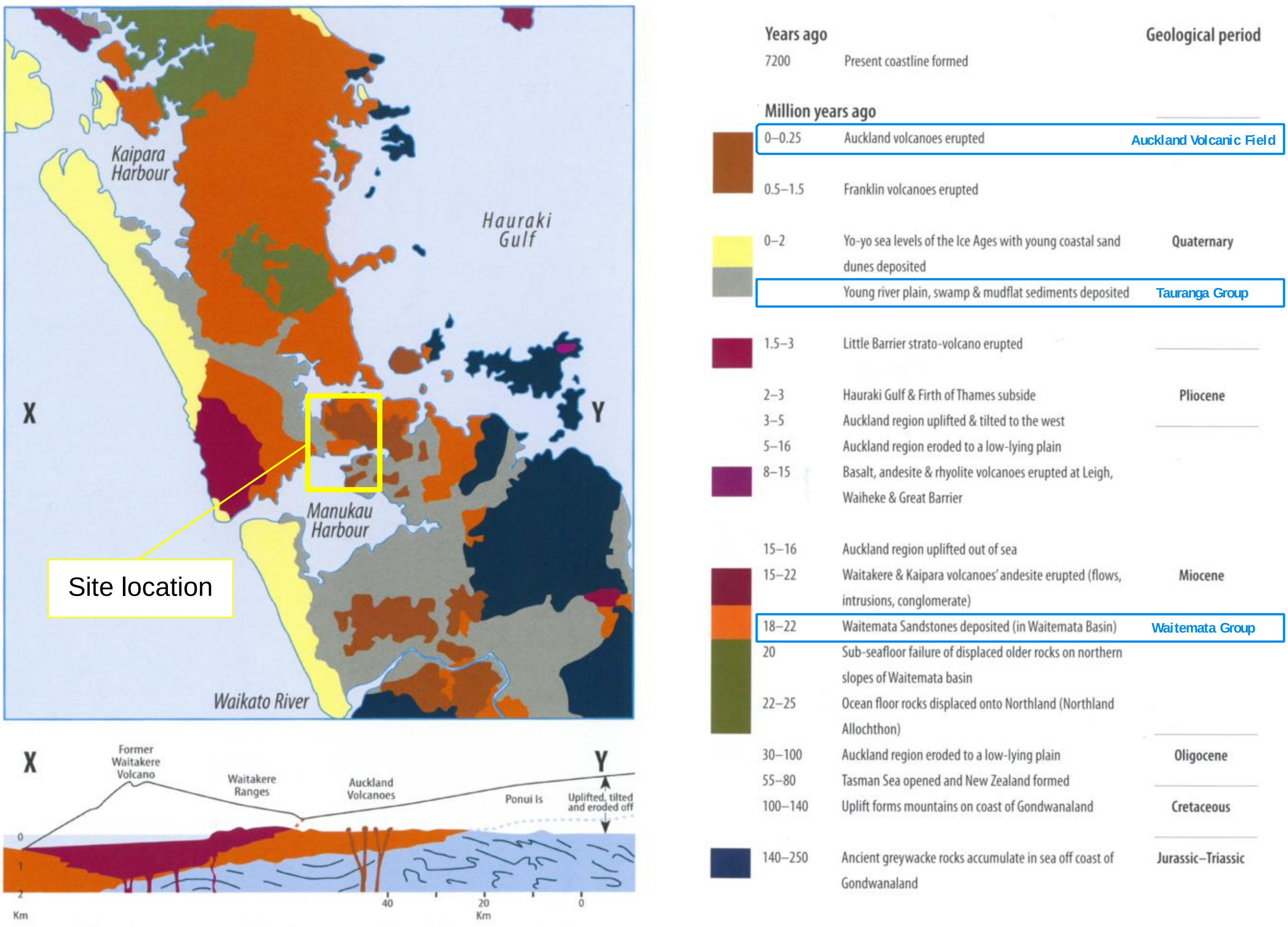


Figure 2.1 : Auckland simplified geology (Cameron, Hayward, & Murdoch, 2008) with the key geological units for the central interceptor project highlighted with blue boxes. Section shows the relative vertical positions of each unit.

2.2 Geological evolution

2.2.1 Jurassic/Triassic Basement Greywacke

250-100 million years ago: The basement rock – the oldest identified rock in the area - under Auckland is greywacke (low grade metamorphosed sandstone) of Triassic to early Cretaceous age. This hard greywacke accumulated on the sea floor as sand and mud off the coast of Gondwanaland on a boundary between two crustal plates. The leading edge of the ancient Pacific Ocean Plate was subducting beneath the Gondwanaland Plate, dragging down the sea floor to form a deep elongate ocean trench parallel to the coast.

As the Pacific Plate subducted, its pillow lavas and cherts became buried by mud from Gondwanaland. These layers of material were dragged down together, tilted and scraped off the top of the Pacific Plate to form a wedge of mixed source material that was partially metamorphosed into hardened greywacke rock.

In the project area these are many hundreds of metres deep and will not have any impact on the works, so they will not be discussed further in this report.

2.2.2 Cretaceous uplift

140-100 million years ago: Uplift formed mountains on coast of Gondwanaland and the basement greywackes were intensely deformed.

80-55 million years ago: Tasman Sea opened up and New Zealand separated from Gondwanaland

100-30 million years ago: Auckland Region was eroded to a low-lying plain.

2.2.3 Miocene Waitemata Group

25-18 million years ago: In the Miocene period, at around 20 million years ago, rapid subsidence caused the area now occupied by Auckland to form a marine basin with land masses to the east and north and the Waitakere / Manukau and Kaipara volcanoes to the west (Figure 2.2). Volcanic-poor and volcanic-rich sediments and occasional volcanic debris flows were deposited into the basin to form the Waitemata Group, a sedimentary sequence of sandstones and mudstones. The East Coast Bays Formation (ECBF) is the uppermost in the sequence, forming the bedrock of most of Auckland. It is this formation through which most of the tunnelling will take place.

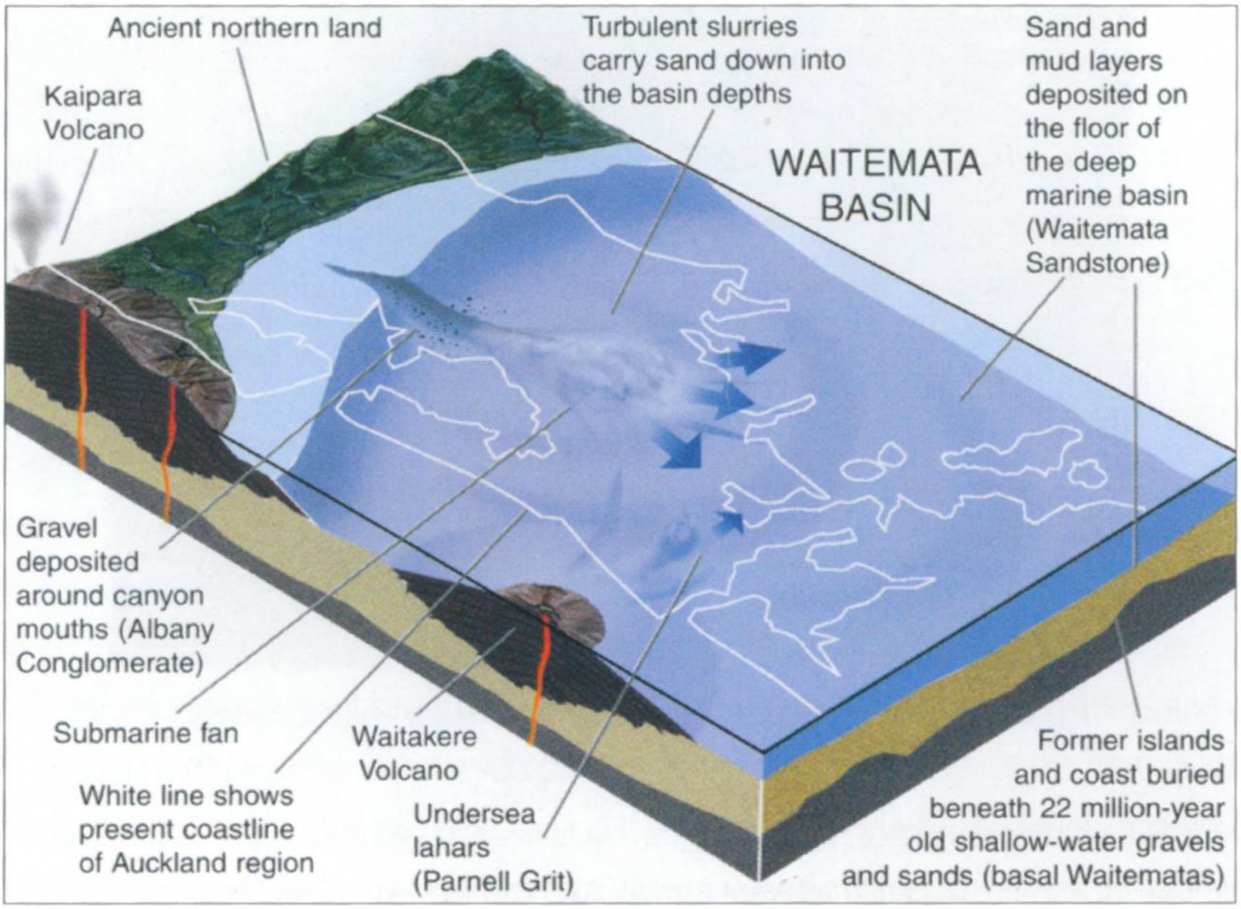


Figure 2.2 : Geography of the Auckland region in the early Miocene, 20 million years ago (Cameron, Hayward, & Murdoch, 2008)

2.2.4 Miocene uplift

15-5 million years ago: Tectonic compression uplifted the Waitemata basin and formed a series of block faults lifting some areas higher (such as the Hillsborough area) and dropping others (such as Manukau) in horst and graben structures. Once above sea level the Waitemata Group rocks were eroded by rivers and streams. The ancestral Waitemata River and associated streams cut valleys, probably following zones of weakness such as faults, which were created during the faulting and uplift. Auckland region was again eroded to a low-lying plain, with a surface eroded into a series of incised river channels and ridges. This plain is important for the interpretation of fault locations, as described later in this report. Faulting continued to change the elevation of areas of this planar surface.

5-3 million years ago: Uplift continued and the region tilted to the west.

2.2.5 Pliocene to Pleistocene Tauranga Group

5 million – 20,000 years ago: The Kaawa Formation and the Tauranga Group were deposited on top of the East Coast Bays Formation as lowland areas were progressively infilled with sand and silt deposits eroded by the ancient Waitemata River catchment from the surrounding land and then subsequently by rhyolitic and pumice rich deposits from central New Zealand volcanism. Interbedded within these deposits are layers of settlement-prone organic-rich clays, silts and occasional peats.

2 million years ago to present day: The sea level periodically rose and fell during ice ages. Some material deposited prior to the last ice age, in the Pleistocene, resisted erosion and remains as a soil above the bedrock. This includes shallow marine and estuarine shelly sands, pumiceous deposits from the Taupo region and coastal dune sands.

2.2.6 Pleistocene to Holocene Auckland Volcanic Field

140,000 years ago to present day: The volcanic rocks of the Auckland Volcanic Field were intruded up through the existing sediments in a series of approximately 50 volcanoes. Some volcanoes created a crater ringed by mainly tuff and some scoria as the lava blasted through to the surface. Others continued to build up a scoria cone and to produce magma after reaching the surface. Lava then flowed down pre-existing drainage channels. The lavas flooded out of the volcanoes covering a significant area and infilling many valleys, completely altering the topJ2Uography of Auckland.

Sediments continued to accumulate within estuaries and in present day stream beds and in intervening lowlands between lava flows where these dammed pre-existing valleys. These sediments consist of soft unconsolidated, peats, silts and sands with varying organic content.

20,000 years ago: During a period of glaciation, sea levels reached a low of about 100 m below current sea level resulting in rivers cutting deep valleys into the landscape including the Auckland harbours. Subsequent sea level rises ‘drowned’ and infilled many of the deep valleys with sediments. These valleys are infilled with deep, soft estuarine and alluvial sediments, often with terrace levels representing previous, higher sea levels or lower land levels.

Auckland’s climate became warm and wet, continuing the previous weathering rapidly and to significant depth. This weathering reduced rock strength and formed a thick soil mantle. It is typical for the weathering thickness to be greatest on ridges and thinnest in valley floors, where less weathered rock may be exposed in streams.

2.2.7 Holocene sea level changes

7200 years ago: Present coastline formed.

Present day active stream erosion and surface water runoff from rainfall continues to cause erosion and down cutting of the local topography, creating deepening valleys and steeper slopes.

3. Main geological units

For the purposes of this project the main geological units within the project area have been adopted as the geotechnical design units. These are:

1. Waitemata Group East Coast Bays Formation
2. Waitemata Group Parnell Volcaniclastic Conglomerate (Parnell Grit)
3. Residually to highly weathered East Coast Bays Formation
4. Kaawa Formation
5. Tauranga Group - Puketoka Formation
6. Auckland Volcanic Field basalt
7. Auckland Volcanic Field tuff, ash and scoria
8. Recent alluvium
9. Made ground

In general the geotechnical parameters within each of these units are consistent enough for the geological unit to serve as a general geotechnical unit for design across the project. This has an additional benefit of simplifying comparison with historical data from other projects. In specific locations where some variability from the norm was encountered these units have been further subdivided or site-specific parameters provided.

Table 3-1. Quantity of each geological unit encountered in the boreholes

Unit		Length drilled (m)	Length drilled as % of total
Made Ground		361	4
Recent Alluvium		160	2
Auckland Volcanic Field basalt		1859	19
Auckland Volcanic Field tuff, ash and scoria		94	1
Tauranga Group		1845	19
Kaawa Formation		302	3
East Coast Bays Formation		4280	44
Parnell Volcaniclastic Conglomerate		116	1
Residually to highly weathered ECBF	Cohesive	420	4
	Granular	295	3
Total		**9733**	**100**

Table 3-1 summarises the quantity of each unit encountered in the Matakite and 2015/2016 ground investigations. This should not be taken as directly representative of the quantum of each unit that will be encountered across the project. The data is skewed as a result of boreholes being vertical rather than horizontal, and because the ground investigation targeted specific zones of interest.

3.1 Waitemata Group

In early Miocene time the Auckland and Northland region experienced significant and complex changes. Arc related volcanoes erupted to the west, the Northland Allochthon was emplaced to the north of Auckland and the Waitemata Basin was created and subsided.

Submarine volcanic complexes (the Manukau and Kaipara volcanoes out to the west) together with eroding Northland Allochthon rock masses supplied large quantities of sediment to Waitemata Basin that built up the thick flysch[1] sequences of Waitemata Group (Edbrooke, 2001).

The Waitemata Group is divided into a number of geographically or stratigraphically distinct subgroups and formations. Conglomerate and shelly limestone accumulated close to the ancient coastlines of the slowly deepening Waitemata Basin, which eventually filled with a 2 km thick succession of sediments derived from the surrounding lands. Mud was the normal deposit on this deep seabed but strong turbid currents intermittently swept across the sea floor bringing copious quantities of coarser silt and sand from coastal areas and adjacent landmasses. The resulting sandstones were deposited by turbidity currents derived from erosion and mass movement on the growing andesite volcanoes to the west, the unstable advancing southern front of the Northland Allochthon as well as from erosion of other older argillaceous rocks of Northland and the eastern basement highs (Figure 2.2).

The lower part of the Waitemata Group is poor in volcanic detritus and the sandstones are dominated by sand-sized grains derived from older mudstones. Mixed volcanic-rich and volcanic-poor turbidites are present in the upper part of the formation and in the west. Close to the ancient volcanoes in the west the formation is almost wholly volcanic in origin. The material origin makes a significant difference to tunnelling projects as it has a very significant impact on strength, excavatability and spoil condition.

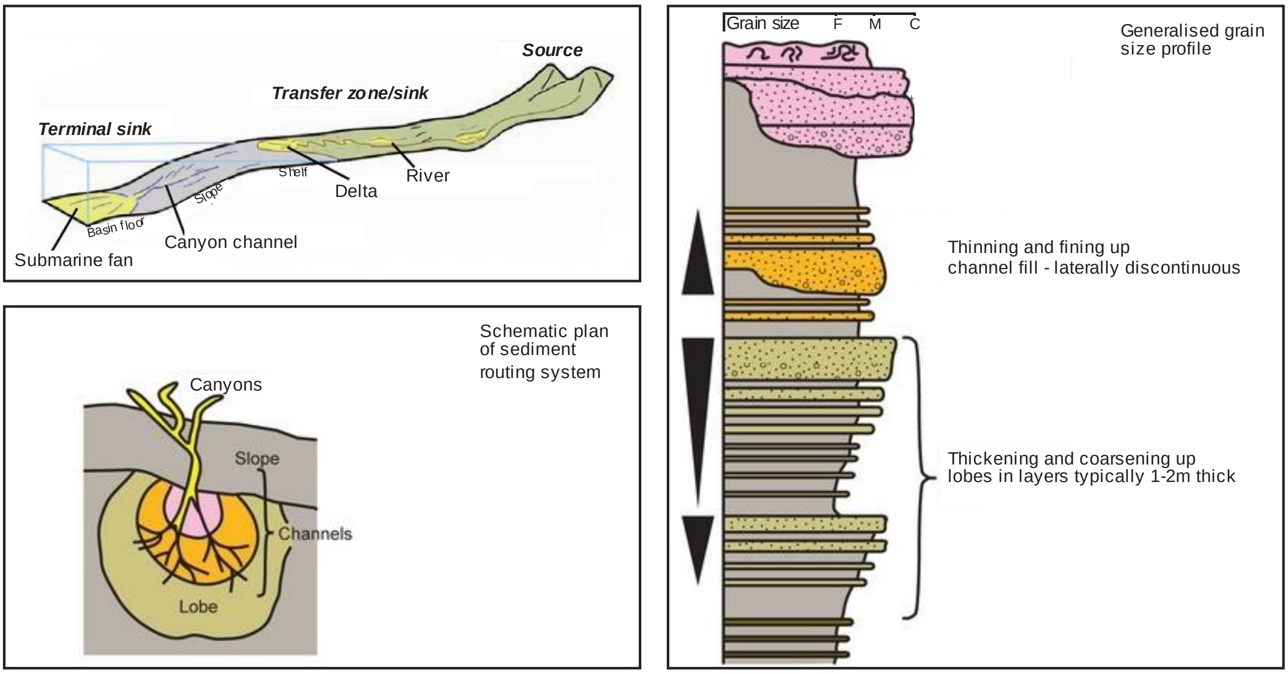


Figure 3.1 : Overview of the depositional environment of the Waitemata Group showing the relationships between mudstone (grey), fine sandstone (green), course volcaniclastic sandstone (orange) and Parnell Volcaniclastic Conglomerate (pink).

[1] Flysch consists of repeated sedimentary cycles with upwards fining of the sediments formed under deep marine circumstances, in a quiet and low-energetic depositional environment. At the bottom of each cycle are sometimes coarse conglomerates or sandstones, which gradually evolve upwards into mudstone. The coarser layers (which require higher energy) are disruptions in the low energy environment caused by flows of material transported from a nearby land mass or shallow sea. In many cases the mass transports are represented in the record by turbidites.

3.2 Waitemata Group - East Coast Bays Formation

The East Coast Bays Formation (ECBF) is the central flysch sequence of the Waitemata Group and is characterised by alternating, graded-bedded, decimetre-bedded, silty, muddy sandstones and laminated mudstones. The sandstones are grey to greenish grey and the mudstones are usually grey to light grey (Kermode, 1992).

The mudstones consist of a mixture of terrestrial detrital clay to silt-sized material and pelagic (non-terrestrial) material largely consisting of siliceous skeletons of organisms such as radiolaria and foraminifera. The complex sedimentary origin has resulted in variable tunnelling conditions which can be challenging to predict.

The ECBF forms the bedrock and underlies the rolling hill country of most of the Auckland region. The content of the formation reflects the sources of sediment from which it was derived and the way in which it was deposited. The volcanic chains were andesitic. Andesite volcanoes produce quantities of fine sandy ash and coarse debris which weather readily to clay, including the group of expansive clays known as smectite (or montmorillonite).

Similarly, the greywacke and argillite rock masses on the eastern side of the Waitemata Basin and the mudstones in the Northland Allochthon contain minerals such as quartz, feldspar and clays. The fine grained argillite rocks are composed of silty mud, rich in quartz and feldspar, with some clay. The Northland mudstones are clay-rich. Physical breakdown and weathering of all these rocks produces clayey detritus which was included in the original sediment entering the Waitemata Basin. Although the proportion of clays produced from these various non-volcanic rocks may be smaller and is probably more varied than that from the andesite rocks of the western volcanoes, the proportion is significant enough to potentially affect the physical behaviour of these rocks as engineering materials.

This is demonstrated by the variable cementation apparent within the sandstones of the ECBF. Areas low in clay content are susceptible to poor cementation and may be encountered as beds of sand commonly referred to as locked sand.

Figure 3.2 : View of cliffs at Hillsborough showing typical East Coast Bays Formation gently inclined planar mudstone and sandstone beds. This view is part of a larger dome or fold structure ranging approximately 400 m along the cliff. A defect can be seen extending from the lower left corner of the photograph to midway up the cliff face.

3.2.1 ECBF Sandstone

These beds vary in grain size from very fine to coarse and are matrix-rich, muddy sandstones. Approximately 60 to 70% of the ECBF is made up of grey to greenish grey, very poorly to moderately sorted, muddy sandstone (Kermode, 1992). The grains are reworked subrounded to well-rounded lithic grains of siltstone, mudstone, argillite, andesite and other igneous rocks with lesser amounts of feldspar and quartz. These grains are set in a clay-rich matrix of mainly smectite, which coats the grains and binds them together (Paterson and Prebble, 2004); hence the term "muddy sandstone". The detrital grains of various rock types (i.e. the "lithic grains") are weathered or altered to varying degrees, thus providing a further proportion of clay when the rock is ground up or disaggregated.

Deposition from turbidity currents is reflected in the variation in grain size, sorting, lamination, convolute lamination, matrix and the presence of mudstone rip-up clasts. Values for physical properties given in Kermode (1992) and Paterson and Prebble (2004) indicate that the sandstones are very weak to weak, approximately 1 to 3 MPa unconfined compressive strength, around 1900 to 2300 kg/cm' density, 15 to 30% natural moisture content and are slightly more resistant to slaking than the mudstones.

Strong hard concretions are found in the sandstone, for instance at the eastern side of St Heliers Bay where irregular, tube-shaped concretions several centimetres thick, several tens of centimetres across and up to a few metres long are found discontinuously along a sandstone bed. Concretions are noted elsewhere, such as at Musick Point (Kermode, 1992) but are generally not common in the ECBF.

3.2.2 ECBF Mudstone

The mudstone consists of silt-size quartz, feldspar and lithic grains in a clay matrix. There are also clay-size grains of quartz and other non-clay minerals, in addition to the large proportion of clay minerals, which makes up the matrix and amounts to approximately 50% of the rock. Mudstone is deposited as background sediment and thus occupies the intervals between the sandstone turbidites. The mudstone has similar physical properties to the sandstone but is a little stronger and less resistant to slaking.

3.3 Waitemata Group – Parnell Volcaniclastic Conglomerate

The Parnell Volcaniclastic Conglomerate (PVC, also known as the Parnell Grit) is a unit within the ECBF. Throughout the time of infilling of the Waitemata Basin, mass movement of volcanic detritus from the flanks of the western volcanoes sent debris flows of coarse material across the sea floor to form lenses and beds of conglomerates and grit up to 20 m thick. These are referred to as the Parnell Grit Member. They now form resistant headlands and reefs such as Bean Rock and West Bastion Reef, 4 km north east of Hobson Bay.

Figure 3.3 : View looking south west at White Bluff. Folded mudstone and sandstone beds of East Coast Bays Formation can be seen in the shore platform with massive sandy siltstone forming White Bluff. Discrete and localised beds of Parnell Volcaniclastic Conglomerate can be seen at the base of the cliffs.

The PVC is a series of volcanoclastic gravity flow deposits occurring at irregular intervals, and is thus vertically and laterally extremely variable making it difficult to predict the locations and extents.

The PVC comprises a pebble to boulder size conglomerate or a pebbly sandstone (depending on location), conspicuously dominated by clasts of basalt. Bedding thicknesses are typically 2.5 to 15 m, and the bed bases tend to be sharp boundaries either eroded into the surrounding Waitemata Group sediments or settled into the sediments, deforming them in the process.

The poorly sorted clasts consist of angular to sub-rounded fragments of andesite, scoria and argillite set in a compacted well-bound matrix of similar materials and clay. In contrast to the sandstones and mudstone, the Parnell Grit is moderately strong to strong with extremely variable fracture spacing.

Figure 3.4 : PVC in the face of the Davis Crescent tunnel. This rock was well cemented with calcite and recorded a uniaxial compressive strength in the order of 10-20 MPa. The largest grain (dark red grain near photo centre) is 14 mm across.

Figure 3.5 : Photomontage of clast types in the PVC (Shane, Strachan, & Smith, 2010). (a) Type 1, subangular basalt clast displaying columnar jointing and flow banding. (b) Type 2, sedimentary clasts. (c) Type 3, rounded mafic igneous clasts. (d) Type 4, pumice clasts within a sandstone; pumice clasts identifiable by crude bedding and light colour.

3.4 Residually to highly weathered East Coast Bays Formation

Residually to highly weathered East Coast Bays Formation is the weathered product of the ECBF that behaves in a 'soil like' manner. Further subdivisions into more refined weathering grades were not considered useful. Weathering grades in the ECBF are notoriously unreliable. Typically clay content is used as a method to determine the weathering profile of a rock, but this may be inappropriate. The detritus from weathering and erosion of the andesite volcanoes and ash from volcanic eruptions all contributed to the clay content of the ECBF. The detritus was clay-rich at source through chemical weathering and forms the particles in the original sediments prior to consolidation or cementing. It is therefore difficult and misleading to use clay content to determine the degree of weathering in the ECBF as would be applied in traditional weathering grade criteria for rocks such as basalt or granite.

For the purposes of this project this unit is divided into cohesive and granular soils. The majority of residual soils encountered are cohesive. Granular soils are less common due to the high clay content of mudstone and sandstone parent materials, as discussed in 3.2.1 and 3.2.2. With increased weathering sandstones become weaker (i.e. become soils), more clay-rich, less dense and with higher moisture content. Mudstone weathers to form highly plastic clay soils.

3.5 Kaihu Group and Tauranga Group

In the last 5 million years the tectonic plate boundary has migrated away from Auckland leading to reduced tectonic activity. Deep erosion and terrestrial deposition occurred in the Auckland area. From the end of the Miocene to the Holocene a diverse range of sediments were deposited. In the Auckland area, Kaawa Formation and Tauranga Group sediments unconformably overlie the Waitemata Group rocks.

3.5.1 Kaihu Group - Kaawa Formation

The Kaawa Formation accumulated in shallow marine and estuarine conditions and is reported to occur in beds made up of lenses up to 12 m thick. This is common across South Auckland, but on the Central Interceptor alignment is only present at the very southern end near the Mangere Waste Water Treatment Plant. The total thickness at the site is in the order of 10 m, although the total thickness increases south of the study area to approximately 250 m.

The Kaawa comprises pumiceous, fossiliferous fine to medium sand and weakly cemented sandstone with scattered pebbles and, in some local areas, organic inclusions. In Mangere these deposits are associated with shell beds and dark blue and green fine to medium grained poorly cemented sandstone.

The Kaawa Formation is highly permeable and an important aquifer in South Auckland. The layered structure, with its rather variable lithological composition, has an impact on its hydrogeological properties, namely a large variation in horizontal and vertical transmissivity (Viljevac et al. 2002). The permeability of this formation will have a significant impact on construction methodology.

3.5.2 Tauranga Group – Puketoka Formation

In general, the Tauranga Group comprises:

- Airborne and waterborne pumiceous deposits (the Puketoka Formation)
- Stream and coastal alluvium
- Hillslope and coastal colluvium
- Intertidal and beach deposits (Kermode, 1992).

Because these were often laterally impersistent, inter-fingered, and repetitive it has been the source of regular debate about source, age and correlations across the region (Haywood & Grenfell, 2010). For the purposes of this report these materials have not been differentiated as there have been no consistent horizons identified to split down the group into sub-units with different engineering properties.

The variability of this material means that region-wide geological or geotechnical descriptions can be inappropriate. Site specific descriptions, and in some cases site specific geotechnical parameters, are more appropriate.

3.6 Auckland Volcanic Field (AVF)

The AVF comprises numerous small volcanoes from which approximately 3 km^3 of material has been erupted periodically over the last 250,000 years, covering a total area of approximately 100 km^2 (Kermode, 1992). Most of the erupted material is olivine basalt, although there are significant deposits of associated material including scoria cones, ash and lapilli mantles, and tuff-ring deposits.

Table 3-2 : Relative ages of the key volcanic centres (note that there is significant uncertainty in these dates)

Name	Best estimated age (years) (Lindsay & Leonard, 2009)	Description	Potential CI impact
Mangere Mt	22,000	Two large craters, and visible Maori pa site with many features	Overlies alignment
Mangere Lagoon	undated	A maar crater, filled with water, although there is a scoria cone in the middle of the lake. Older than Mangere Mountain as lagoon tuff overlain by Mountain basalt	Overlies alignment
Mt Eden	28,000	Likely to be slightly younger than Three Kings	May have flowed to Western Springs/Chamberlain Park
Three Kings	28,500	A complex series of cones, now a reservoir.	Some flow may overlie alignment
Mt St John	29,000	Te Kopuke or Tikikopuke to the Maori. It has the longest lava flow of any volcano in the Auckland volcanic field, creating the Meola Reef. Pre-dated Mt Eden and Three Kings.	Flowed through Western Springs close to tunnel crown level
Mt Roskill	30,000	Puketepapa, now contains an emergency reservoir. Older than Three Kings	Overlies alignment
Mt Albert	100,000	Called Owairaka by the Maori. It has been extensively quarried and is now a reservoir.	Overlies alignment

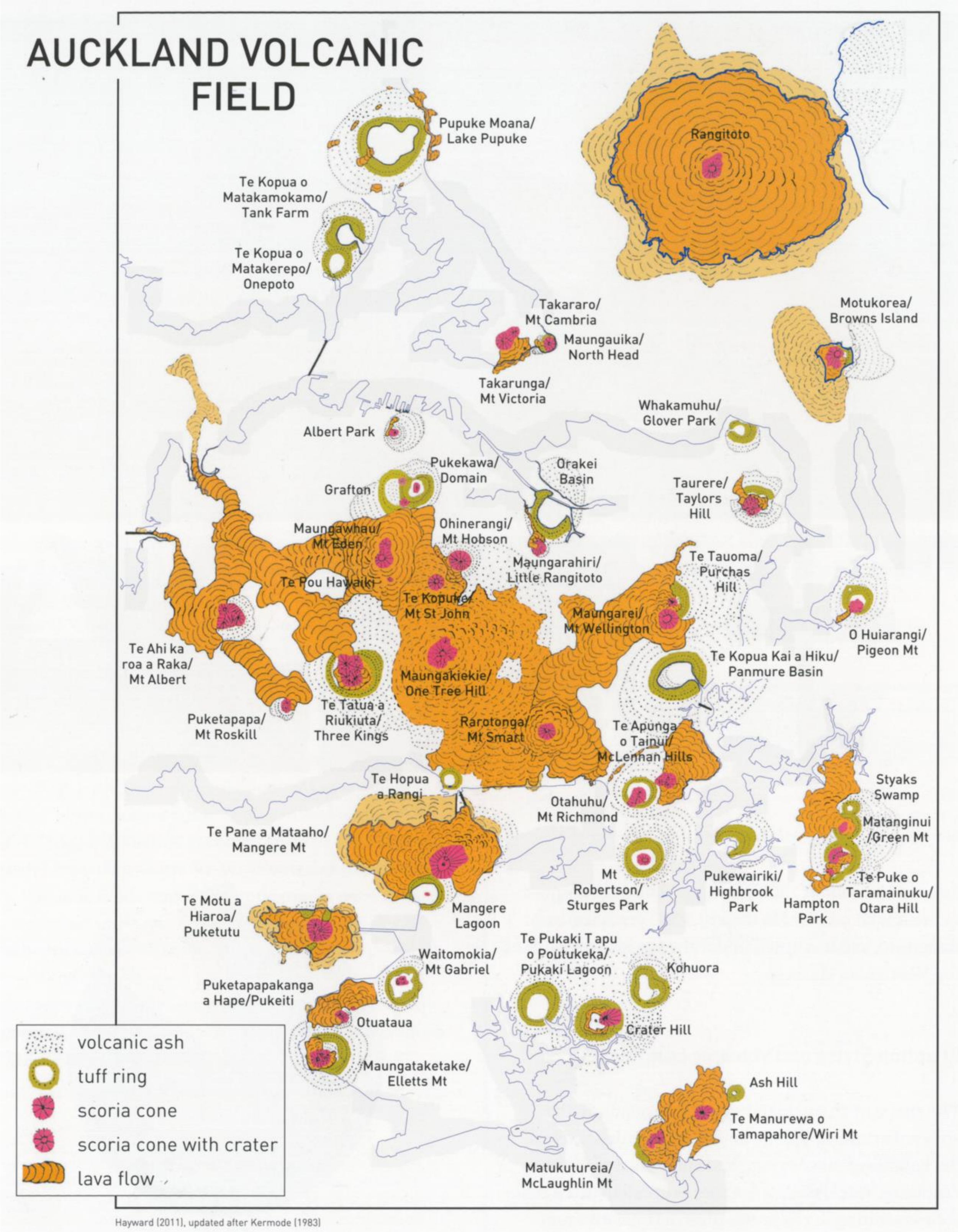


Figure 3.6 : Overview of the Auckland Volcanic Field (Hayward, Murdoch, Graeme, & Maitland, 2011). The main centres impacting on or adjacent to the Central Interceptor alignment are Mt Albert, Mt Roskill, Mt Eden, Mt St John, Three Kings, Mangere Mt, and Mangere Lagoon.

3.6.1 Mt Albert & Mt Roskill

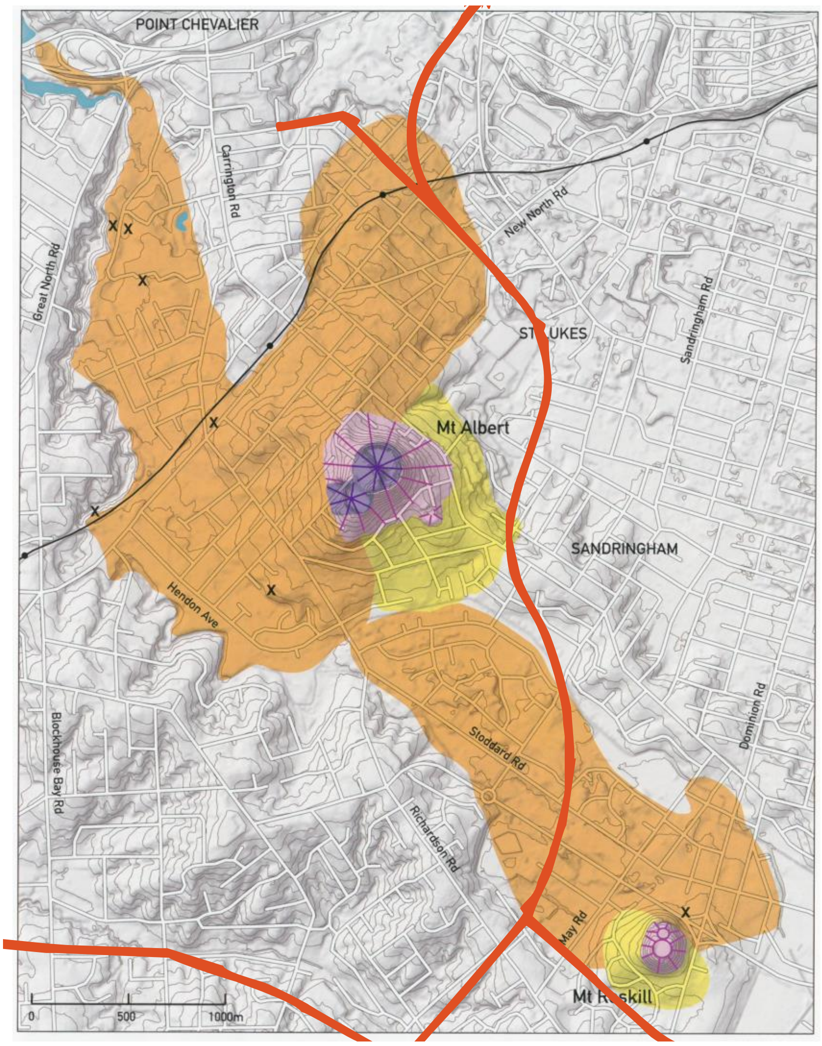


Figure 3.7 : Overview of the basalt extent from Mt Albert and Mt Roskill, with approximate CI alignment in red. After (Hayward, Murdoch, Graeme, & Maitland, 2011)

Mt Albert flow coincides with the project along Link Sewer B, Mt Albert Community Centre and at Linwood Avenue. It is believed that the flow stopped at this point. To the north of Linwood Avenue (into Chamberlain Park and Western Springs) the basalt originated from Mt St John and Three Kings. To the east of Mt Albert War Memorial there is a gap in the basalt cover before encountering the flow from Mt St John and Three Kings at Lyon Ave.

It is suspected that the basalt from Mt Albert pushed an existing watercourse to the north, leading to deeper erosion at the end of the flow through Chamberlain Park and creating the valley which was later infilled with the flow from Mt St John.

3.6.2 Mt St John

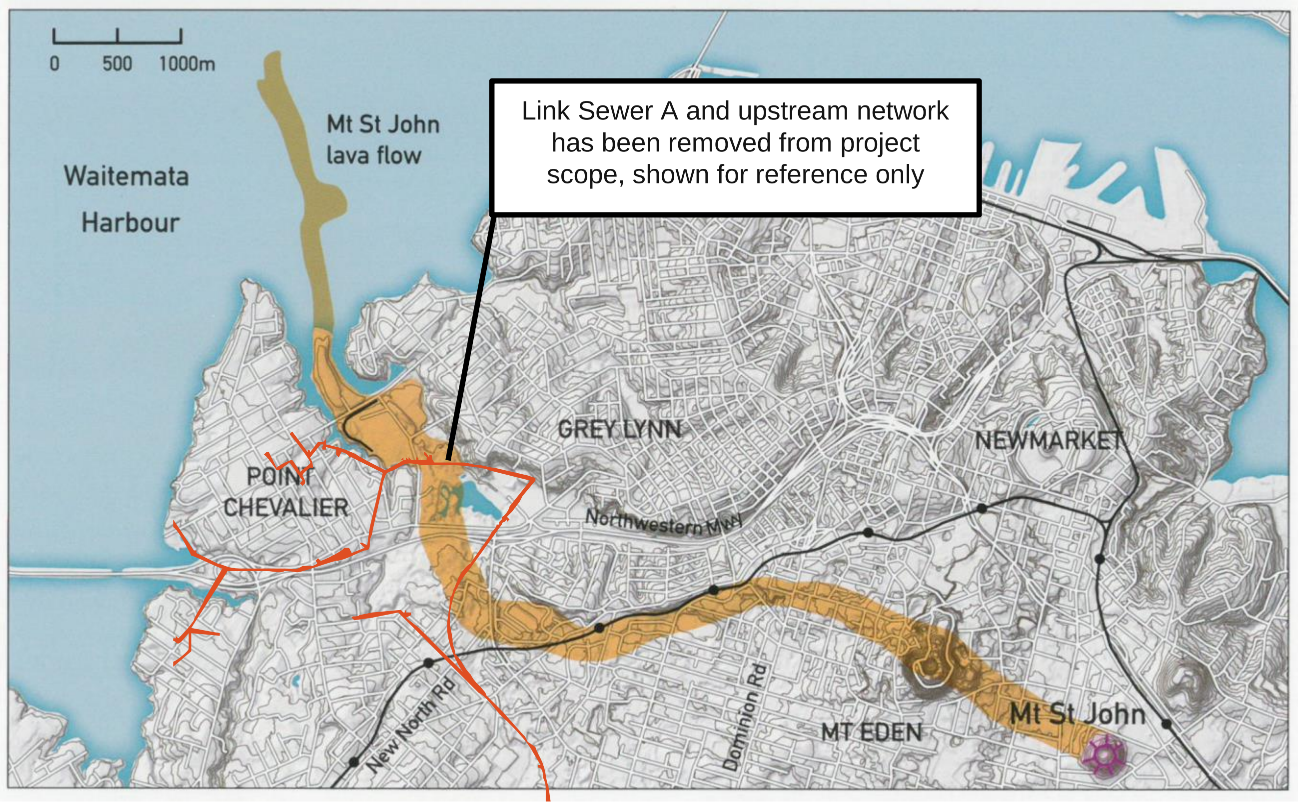


Figure 3.8 : Overview of the basalt extent from Mt St John, with approximate CI alignment in red. After (Hayward, Murdoch, Graeme, & Maitland, 2011)

The basalt from Mt St John flowed through a valley crossing the Central Interceptor alignment around Linwood Ave and Chamberlain Park where it would have come up against, and possibly ridden over the flow from Mt Albert. Further down the valley it crosses again at Link Sewer A before continuing downhill into the ancestral Waitemata Valley which would at this time have been well above sea level.

3.6.3 Three Kings

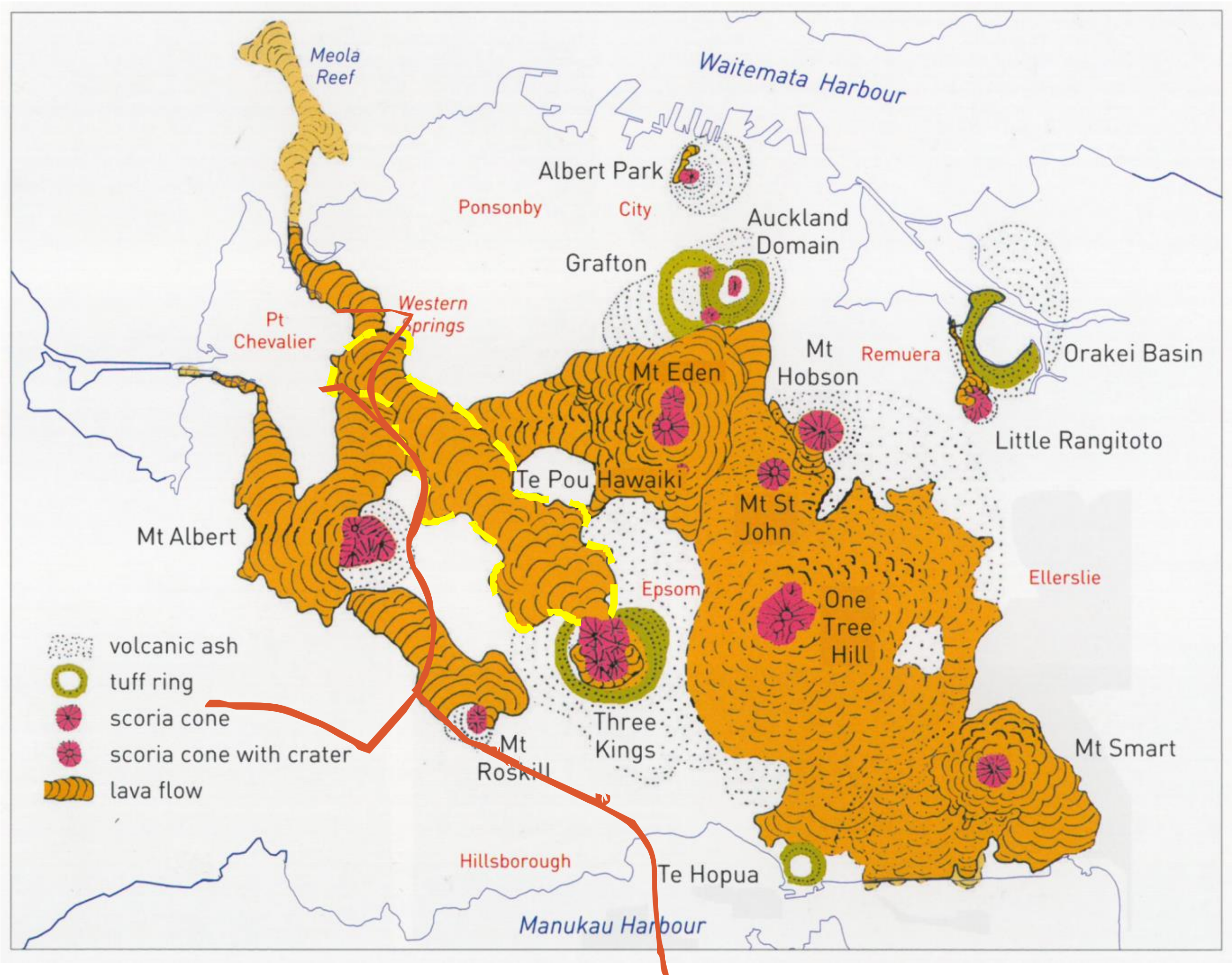


Figure 3.9 : Overview of the basalt extent from Three Kings, with approximate CI alignment in red. The Three Kings Basalt is outlined in yellow to differentiate from other flows. After (Hayward, Murdoch, Graeme, & Maitland, 2011)

Three Kings is believed to be slightly younger than Mt St John. The flow travelled north to overlie the alignment at Lyon Ave (at the very edge of the flow) and continued to Mt Albert War Memorial, which appears to be right on the edge of the flow. At this location it will overlie the older flow from Mt Albert. It also overlies the Mt St John flow through Chamberlain Park. North of Chamberlain Park the flow appears to have spilled over towards Western Springs Fields and Work Shaft 1.

3.6.4 Mangere Mountain and Lagoon

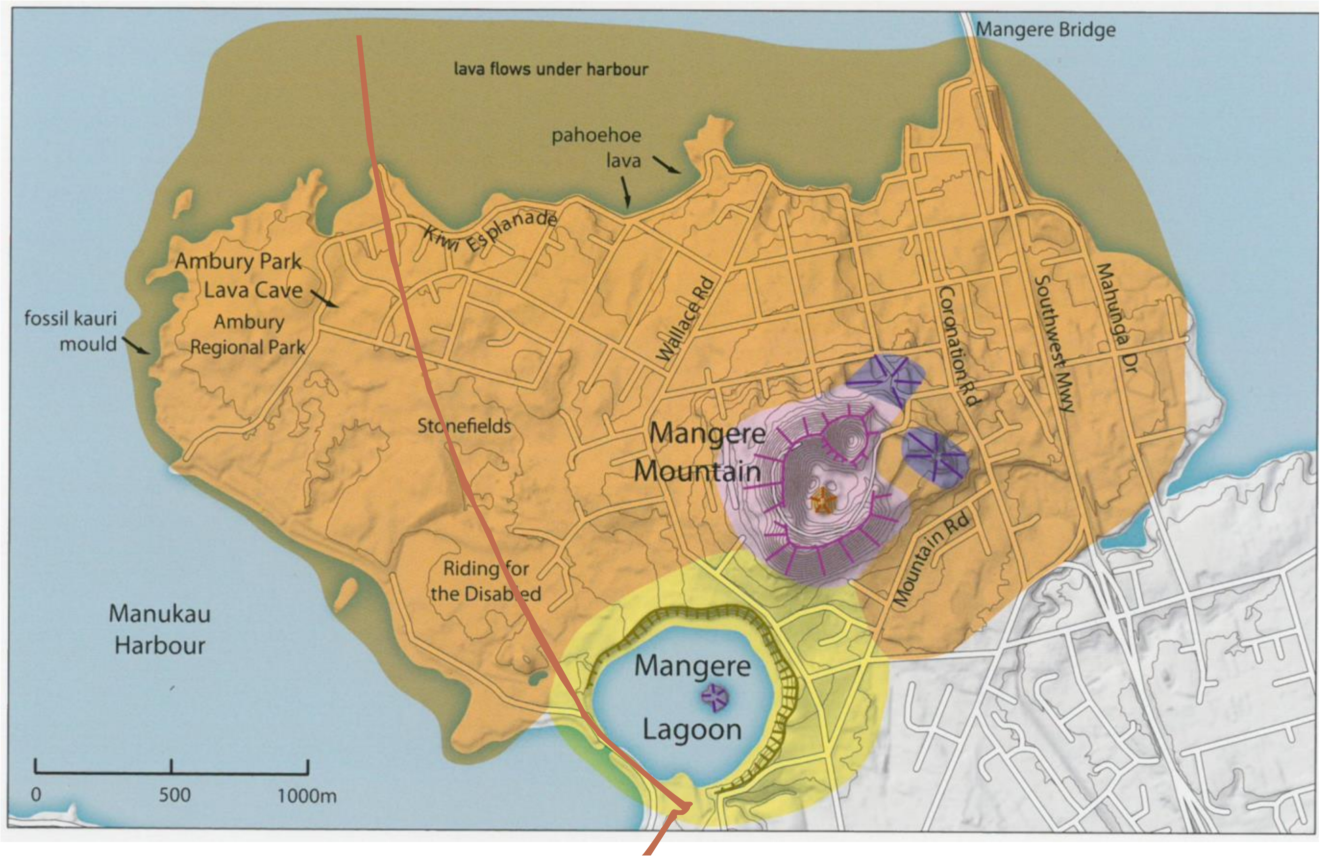


Figure 3.10 : Overview of the basalt extent from Mangere Mountain (also showing Mangere Lagoon), with approximate CI alignment in red. After (Hayward, Murdoch, Graeme, & Maitland, 2011)

Mangere Mountain and Lagoon are of very similar ages, but are very different in form. The Lagoon, believed to be the earlier of the two, was an explosive eruption which spread a layer of volcanic tuff around the immediate area including the waste water treatment plant.

Mangere Mountain was an effusive eruption which produced very large quantities of basalt which overlie the tuff. It should be expected that a layer of tuff from Mangere Lagoon exists under the basalt from Mangere Mountain.

3.7 Post-AVF Tauranga Group and Recent Alluvium

The Tauranga Group continued to be deposited as the Auckland Volcanic Field was formed. The material above the volcanic deposits is younger and less consolidated than the material below, so will tend to be weaker and more compressible. However, given the highly variable age of the volcanic field there is not a clear demarcation between pre-AVF ad post-AVF Tauranga Group deposits.

This Tauranga Group alluvium represents locally derived stream and coastal alluvium and minor fan deposits. It typically consists of up to 20m thick unconsolidated to very soft thinly to thickly bedded, yellow-grey to orange brown clay, silt, sand and gravel with local silty peat and pumiceous beds. The undifferentiated Tauranga Group is sometimes referred to as undifferentiated Pliocene to Pleistocene Alluvium.

Since the end of the last ice age (the Holocene) the most recent deposits in Auckland have been laid down. In lowland areas of Auckland these comprise floodplain, lacustrine and coastal alluvial deposits, while estuarine sand and silt occur in harbours and bays. The majority of the material along the proposed alignment in this category is found at the Manukau Harbour, although there are small pockets associated with other watercourses.

3.8 Made Ground

Made ground comprises engineered and non-engineered fill located in localised areas throughout the CI project area deposited during the last 150 years. Areas of reclamation are present at the Mangere WWTP and Kiwi Esplanade foreshores. At Western Springs a discrete area with 7m thick made ground is inferred to be an infilled quarry site.

Made ground is associated with risks of variable or poor ground conditions, unforeseen obstructions, items of heritage or cultural value, and elevated potential for contamination.

Details of fill types encountered are described within the relevant site specific sections later in this report.

4. Mineralogy

Mineralogy was identified in core samples of rock and soil using quantitative and qualitative X-Ray Diffraction (XRD) and petrographic analysis. Mineralogy observed in qualitative XRD analysis includes the following assemblages:

- Waitemata Group: Smectite, Illite, Kaolinite, Halloysite, Zeolite, Chlorite, Quartz, Calcite, Feldspars
- Tauranga Group: Smectite, Illite, Kaolinite, Quartz, Halloysite.

Quantitative XRD analysis identified Smectite as the dominant clay mineral in Parnell Volcaniclastic Conglomerate and East Coast Bays Formation Mudstones (Appendix U Table 12-50).

Summarised assemblage results are presented in Appendix U Table 12-51 and show clay and clay dominant rock fragments are a significant portion of the samples. There appears to be variation between analysis methods with XRD identifying a higher portion of quartz and lower portion of clays than petrographic analysis.

Summarised assemblage results for basalt are presented in Appendix U Table 12-52. Thin section analysis identified olivine and pyroxene phenocrysts with a groundmass of olivine, pyroxene, plagioclase, opaque and mesostasis.

5. Structural geology

5.1 Regional effects

By the end of the early Miocene, the uplift of the Waitemata Basin had begun with regional uplift and gentle westward tilting (Kermode, 1992). This was accompanied by extensional block-faulting, resulting in a series of steps in the terrain of Auckland (Figure 5.1 and Figure 5.2).

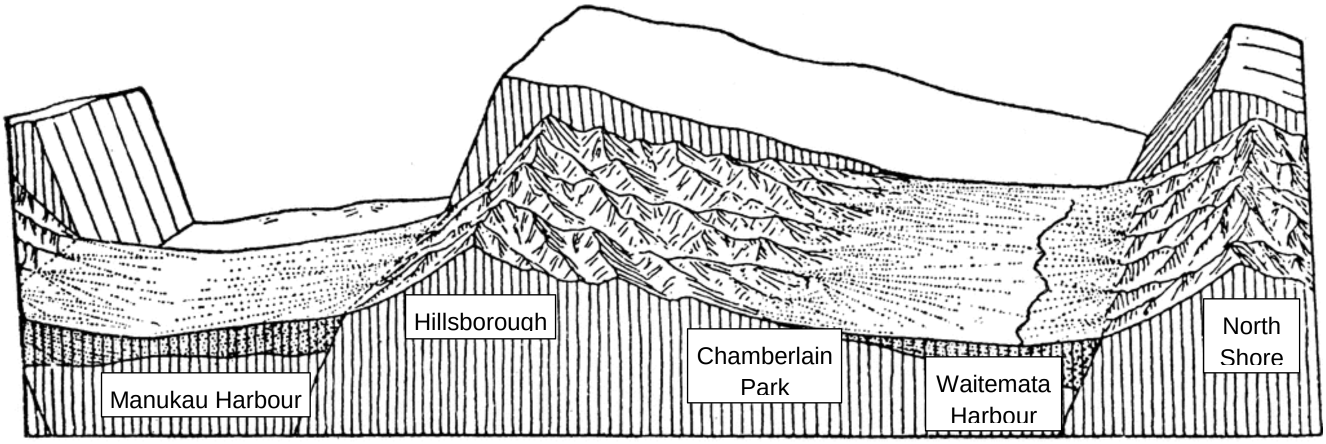


Figure 5.1 : Schematic representation of block faulting showing the immediate landform (rear) and the end-product land shape (front) (Grabau, 1920). Labels added to show how this landform may match with locations along the Central Interceptor alignment.

Erosion, since the formation and uplift of the Waitemata Group erosion surface, has reduced the height evenly across the whole region. Relatively erosion-resistant strata in the Waitemata Group have retained the form of the Waitemata Group erosion surface along ridge lines. Away from ridge lines, the less resistant strata have eroded in a regular and predictable pattern of streams and rivers, flowing either into the Waitemata Harbour or the Manukau Harbour. Across the project site the predominant flow direction is to the north, supported by the high cliffs on the Manukau Harbour Edge. The similarity between the landform across the site and the 'typical' block faulted landform shown schematically in Figure 5.1 is striking. This landform shape, along with the sudden reduction in the level of the top of Waitemata Group material (Figure 5.2 and Figure 5.3) is the main evidence used to postulate the presence of a Manukau Fault crossing the Central Interceptor alignment under the Manukau Harbour.

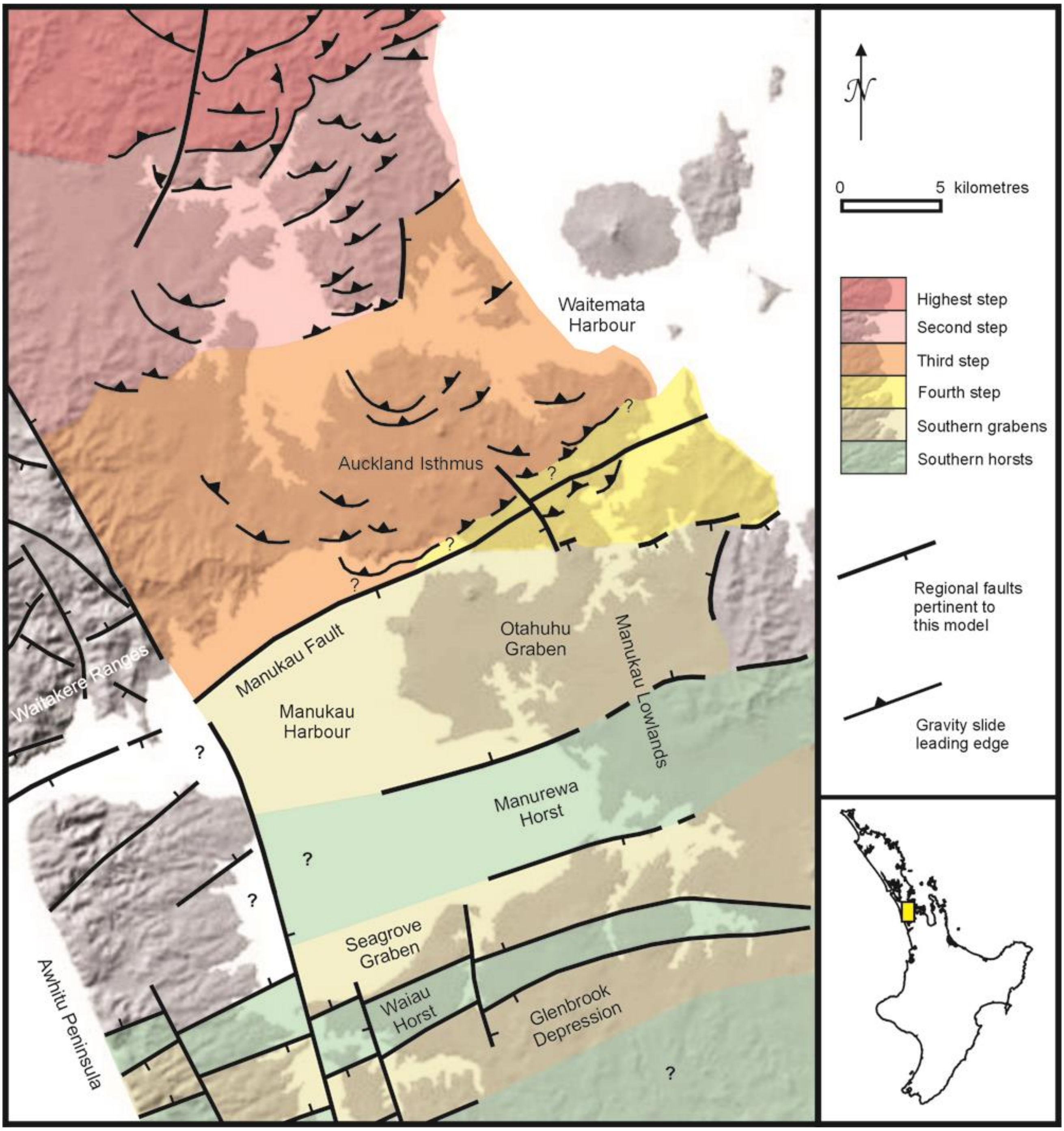


Figure 5.2 : Simplified map of the steps in the Waitemata Group erosion surface with only the most significant faults shown (Kenny, 2013). Note that the gravity slides shown on this figure are syn-depositional features. No current slope instability should be inferred from this figure.

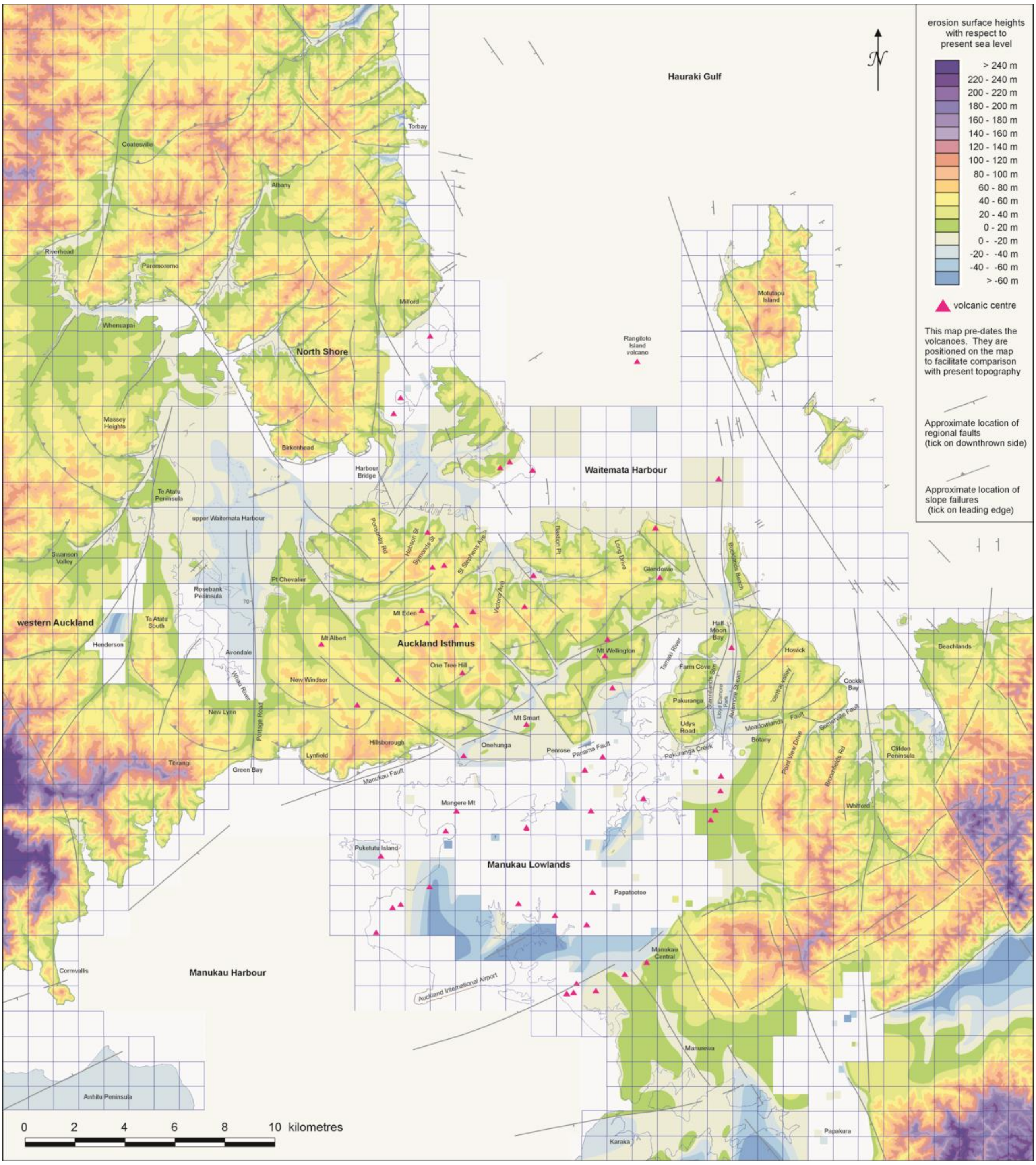


Figure 5.3 : Topography of the eroded surface of the Waitemata Group as it is today, including portions now concealed under younger sediments or volcanic material (Kenny, 2013). The green-grey interface corresponds to present sea level; purple colours are highest elevations; blues below present sea level darken in colour with increasing depth. Areas of white represent areas of unknown elevation. Note that more inferred faults are shown on this figure than on Figure 5.2.

The uplift process was accompanied by open folding of early Miocene strata on northeast and northwest axes (Edbrooke, 2001). The Waitemata Group rocks have been gently folded into broad open folds, which form simple structural domains and gentle bedding dips of around four to seven degrees (Figure 3.2). In some places this produced anticlinal ridge crests and synclinal stream valleys, such as in the Southern Landslide Zone

(Prebble, 1995 and 2001). More complex, tightly folded, complexly folded and closely fractured zones are found between the simple domains (Figure 5.4).

Figure 5.4 : View of cliffs at Hillsborough showing syndepositional deformation in East Coast Bays Formation including folded and contorted beds and a possible fault plane (outlined in yellow).

These complex structural domains are possibly the boundaries of large, syn-sedimentary slides and thrusts, which may have taken place during movement of the Allochthon beneath the still accumulating or recently deposited sediments that became the Waitemata Group. In some cases these deformations may have resulted from post-sedimentary fault movement causing 'drag' of the adjacent material (Figure 5.). Fault drag on normal fault movements such as those encountered in Auckland tend to result in relatively limited deformation; the severe distortions such as those in Figure 5.9 are more likely to have been formed by compression, in this case probably syn-depositional slumping.

In the Redoubt Road No.2 Inlet Tunnel (Wylie, 1989) three classes of fault were intersected in the Waitemata Group. These were normal, high angle reverse and low angle reverse. The normal and high angle reverse faults dip at angles of 45 to 90 degrees. The low angle reverse fault is virtually parallel to bedding.

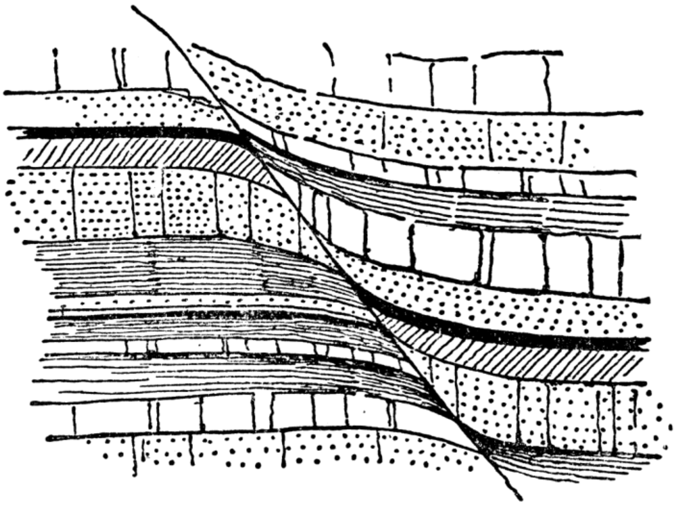


Figure 5.5 : Schematic representation of how normal fault movement (as expected in Auckland) can result in bedding deformation known as fault drag. Note that the amount of deformation is relatively small.

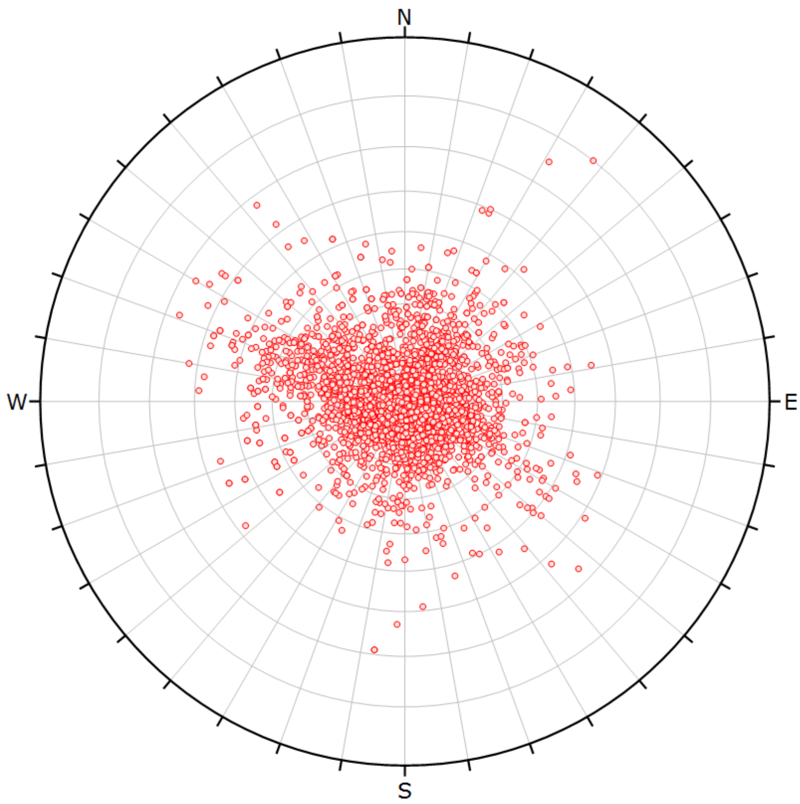
Figure 5.6 : Aerial view of shore platform at Hillsborough Bay. Folding and faulting, the result of syndepositional deformation, can be seen here at a larger scale than in cliff section photographs. Localised features which appear raised from the shore platform are likely Parnell Volcaniclastic Conglomerate.

5.2 Local effects within the Waitemata Group

The processes and deformation described in the preceding section gives rise to localised effects at each of the sites. Likely typical defect types based on previous experience and field observation are described here.

5.2.1 Bedding

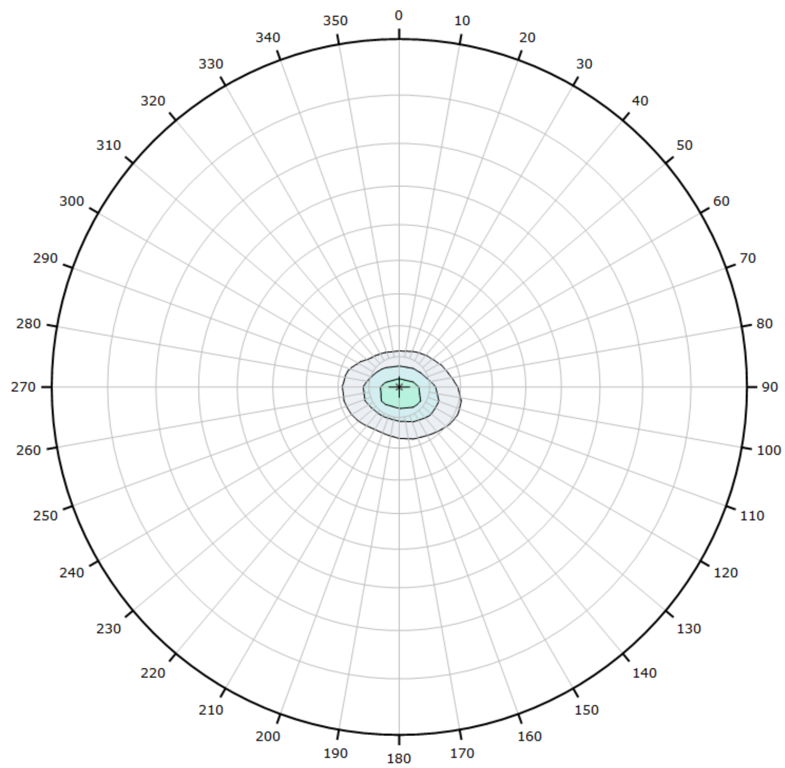
Bedding is generally sub-horizontal to gently inclined, Figure 5.7 and Figure 5.8, with localised significant variability associated with syn-depositional slumping or post-depositional faulting.



Symbol	TYPE	Quantity
o	Bedding Plane/Fracture	3342

Plot Mode	Pole Vectors
Vector Count	3342 (3342 Entries)
Hemisphere	Lower
Projection	Equal Angle

Figure 5.7 : Stereonet plot of all bedding planes and bedding fractures identified during televiewer logging.



Color	Density Concentrations
	0.00 - 5.00
	5.00 - 10.00
	10.00 - 15.00
	15.00 - 20.00
	20.00 - 25.00
	25.00 - 30.00
	30.00 - 35.00
	35.00 - 40.00
	40.00 - 45.00
	45.00 <
Maximum Density	18.05%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

Plot Mode	Pole Vectors
Vector Count	5670 (5670 Entries)
Hemisphere	Lower
Projection	Equal Angle
Too many entries for grid intersections	

Figure 5.8 : Contour plot of all bedding planes and bedding fractures identified during televiewer logging.

5.2.2 Joints

Vertical to very steep defects (joints) are found at closely spaced to widely spaced intervals throughout the sandstones in at least two orthogonal sets perpendicular to bedding (Paterson and Prebble 2004). Extremely closely spaced superficial desiccation fractures in the mudstones tend to obscure tectonic fractures in natural outcrops.

These defects tend to have low persistence (length typically less than 2 m), commonly terminating at bedding (Figure 5.9).

Figure 5.9 : View of cliffs at Blockhouse Bay showing an antiform fold in East Coast Bays Formation.

Joints normal to bedding can be seen in darker grey sandstone bed and appear stained and infilled. The joints often extend through a single bed only however several are seen with greater persistence. Brown staining parallel with laminations in the lighter grey mudstone is also visible. The small cave at the base of the cliff is taken to be associated with coastal weathering processes.

An indication of the scale of defects and likely resulting block sizes are given in Figure 5.9 and Figure 5.10 and are typically decimetre to meters in size.

For the purposes of block failure modelling it is recommended that 2 m persistence is taken as a typical value, with 5 m persistence as extreme. Bedding persistence is in excess of 10 m.

Figure 5.10 : View looking east at cliff near White Bluff showing surficial weathering profile and inclined beds of East Coast Bays Formation, including a bed of sandstone (outlined in yellow) terminating midway up the cliff with several joints normal to bedding.

Joints in the ECBF tend to be self-healing, in that they close up as a result of the very low strength of the rock. In general this results in relatively low permeability along these defects with the exception of defects in Parnell Volcaniclastic Conglomerate which are typically vertical and due to higher strength less likely to heal.

At borehole and site scale, trends in joint inclination and orientation can be seen and are discussed in Section 11. Stereonet plot for all joints recorded during televiewer analysis indicate high variability at the project scale Figure 5.11.

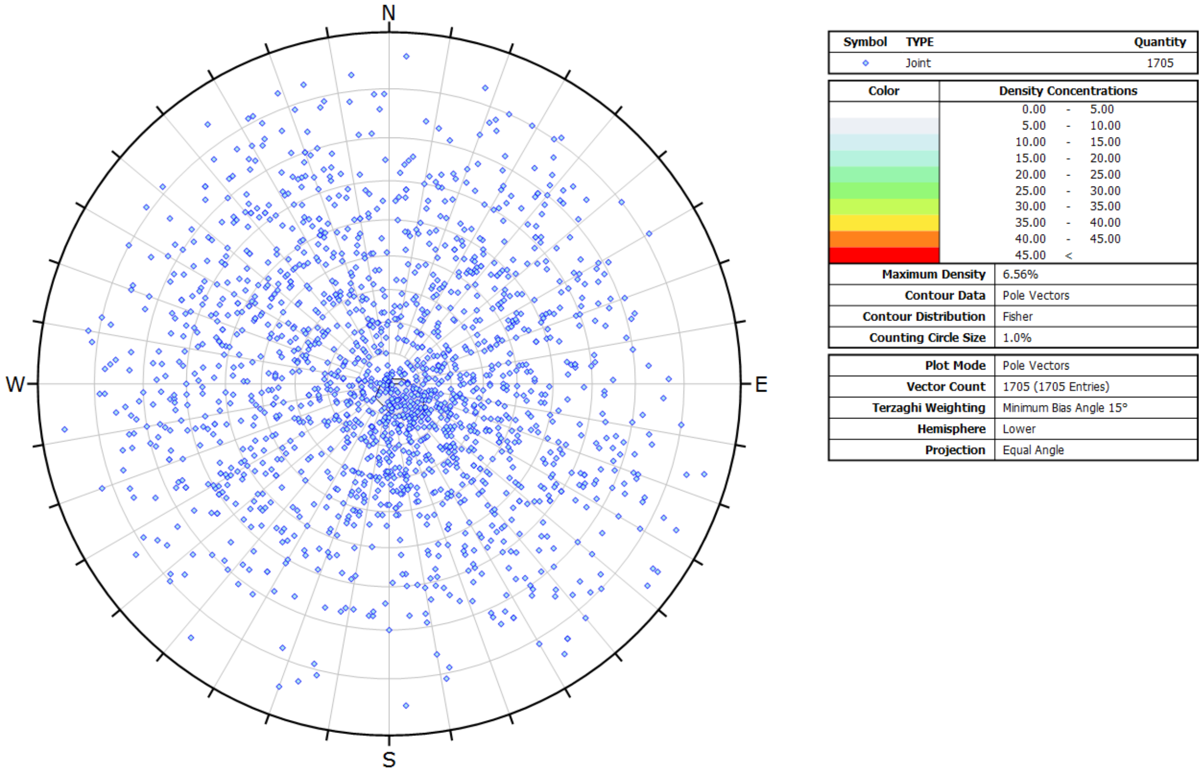


Figure 5.11: Stereonet plot of all joints recorded during televiewer logging (no contours shown as density too low)

5.2.3 Faults

Faults have been inferred along the project alignment, all of which have been identified from a combination of geological mapping and aerial photographic interpretation. No clearly defined fault planes were identified in boreholes. However, a number of fractured and poorly cemented zones were encountered across the project and may be associated with the effects of faulting.

Faults observed in cliffs within the ECBF tend to be tight, with no observed local fracture zone or other weakening. However, these are likely to be best case examples; where fracture zones or weak infill does exist, these are likely to be rapidly eroded and therefore not preserved in cliff sections.

Based on the data from boreholes it is expected that the fault zones inferred on the geological sections will be associated with zones of deeper weathering, a reduction in cementation, and a reduction in strength in a zone no more than a few metres wide.

5.2.4 Fault gouge

Fault gouge is the term used for material formed along a fault plane as a result of movement crushing the fault walls. It is commonly very finely grained, and unconsolidated on recent faults. No fault gouge was encountered in the boreholes drilled for the Central Interceptor project.

Wide zones of very closely fractured rock and clayey gouge are found in some faults around Auckland where they can form effective aquifers (Wylie, 1989). Local experience on SH16 causeway in West Auckland revealed a 1.5 m thick fault gouge of ‘toothpaste like’ consistency in a borehole drilled for a bridge pile (personal communication, Jill Kenny). This location was the postulated site of the fault between Point Chevalier and Green Bay which continues south and may run close to, and west of, Pump Station 25. No fault gouge was encountered at Pump Station 25, but an unusual weathering profile was encountered with moderately weathered rock overlying highly to completely weathered rock which is inferred to be the result of ‘sawtooth’ weathering induced increased by groundwater flow along the adjacent fault.

Because of the relatively large displacement anticipated on the Manukau Fault an extensive ground investigation was undertaken to attempt to identify the fault location and confirm the gouge thickness and material parameters. Gouge was not encountered during drilling, although numerous extremely weak rock and crush zones were found. The investigation was terminated when it was decided that:

- The thickness of relatively low permeability marine alluvium overlying the ECBF was sufficient that very large groundwater inflows through the gouge were unlikely.
- There was no evidence of a single large-displacement fault (multiple smaller faults were considered more likely).
- Tunnel Boring Machine will be operated in closed pressurised mode and therefore will not be materially affected by a fault.

5.2.5 Bedding parallel clay seams

Bedding parallel clay seams are well known in the Southern Landslide Zone of Auckland approximately 8 km east of the Central Interceptor alignment. To date no evidence of such features has been found along the proposed Central Interceptor alignment.

These bands of extreme continuity and extremely low strength form basal ruptures to large block slides in slopes. They are postulated to have formed as a result of shear movement between beds of relatively strong sandstone and weaker mudstone during initial uplift and folding.

In tunnels, clay seams can cause squeezing and inward movement of thicker beds that can trap a machine. In combination with steep defects in the roof or shaft walls they can also create block falls.

6. Geological hazards

6.1 Seismicity

6.1.1 Active faulting

There are no known active faults along the alignment. The nearest known active fault recorded in the GNS Active Faults Database (reviewed 9 Dec 2015) is the Waikopua Fault. This normal fault lies 21 km due east of Mangere WWTP.

6.1.2 Soil classification

Soil classifications were determined according to NZS1170.5:2004 and are presented in Table 6-1. Geotechnical units are described in Section 8.2 and detailed site specific descriptions in Section 11.For selected sites detailed analysis of shear velocities has been used to determine site classification and workings are presented in Appendix W.

Table 6-1 : Soil Classifications for the Individual Shaft Sites

Shaft Location	Reference BH	Soil Classification
DSCIN001 – Kiwi Esplanade	259, 260	C
DSCIN002 – PS23	256, 255	B
DSCIN003 – Keith Hay Park	250, 251, 249	C
DSCIN004 – May Road	247b, 246	D
DSCIN005 – Walmsley Park	232, 231	C
DSCIN006 – Haverstock Road	227-1, 228-1	C
DSCIN007 – Lyon Avenue	225, 224	C
DSCIN008 – Mt Albert War Memorial Reserve	221, 219, 220	C
DSCIN009 – Western Springs Shaft	205, 206b	C
DSLSB001 – Norgrove Avenue	218	D
DSLB001 – Rawalpindi Reserve	215, 216	C
DSLSC001 – Haycock Avenue	243, 244	C

Shaft Location	Reference BH	Soil Classification
DSLSC002 – Dundale Avenue	240	C
DSLSC003 – Whitney Street	239	C
DSLSC004 – Miranda Reserve	237, 238	C
DSLSC005 – PS25	CI-13	C
DPCIN – Mangere Pump Station	271, 273	C

6.1.3 Peak ground accelerations

The site investigations were carried out in two phases. In Phase One, Peak Ground Accelerations (PGA) were based on NZ1170.5:2004 and are presented in Table 6-2. The PGAs were weighted to an earthquake magnitude, M_w, of 7.5. According to NZ1170.0:2002, all wastewater treatment facilities are classified as importance level 3, and thus an annual probability of exceedance of 1/25 and 1/2500 years was selected for SLS and ULS events respectively for a design working live of 100 years and more.

In Phase Two, further CPT tests were carried out and tested for liquefaction risk. However, between the investigations in Phase One and Phase Two, the New Zealand Geotechnical Society (NZGS) and Ministry of Business Innovation & Employment (MBIE) released a new method in estimating the ground motion parameters which is considered more appropriate for liquefaction assessment (NZGS and MBIE, 2016). The new NZGS method was thus used in Phase Two. This resulted in an earthquake magnitude (M_w) change to 5.9 for the Auckland region. The new PGAs are also presented in Table 6-2. Sites which previously yielded a high or very high risk of liquefaction in Phase One were also retested using the new method for a sensitivity check.

Table 6-2 : PGAs for the SLS and ULS case for the Various Soil and Rock Classes

Soil Class	Return Period (years) NZ1170.0:2002	Limit State	PGA (g) (NZ1170.5:2004)	PGA (g) (NZGS and MBIE, 2016)
B Rock	1/25	SLS	0.033	0.029
	1/2500	ULS	0.0234	0.208
C Soil	1/25	SLS	0.043	0.038
	1/2500	ULS	0.311	0.276
D and E Soil	1/25	SLS	0.036	0.037
	1/2500	ULS	0.262	0.263

6.2 Liquefaction

CPT data was analysed using CPeT-IT version 1.70 and CLiq version 1.7 and results for the liquefaction analyses are presented in Appendix E. Analyses were carried out per shaft. No liquefaction is assumed beyond 20 m depth. No liquefaction was encountered under SLS conditions. A list of the shaft sites, related CPTs and the Liquefaction potential (LPI) are given in Table 6-3.

Table 6-3 : Summary of CPT analyses per shaft

Shaft Location	Reference CPT	Liquefaction ULS LPI	Max. Estimated Settlement, cm[2]
DSCIN001 – Kiwi Esplanade (Site AQ)	AS7-CPT258,	Low Risk Note 1	5
DSCIN002 – PS23 (Site AO)	-	-	
DSCIN003 – Keith Hay Park (Site AI)	AS5-CPT01, AS5-CPT02, AS5-CPT03, AS5-CPT04	Low	2
DSCIN004 – May Road (Site AF.1)	WS2-CPT01, WS2-CPT03D, WS2-CPT04, WS2-CPT08,	Low	7
DSCIN005 – Walmsley Park (Site V)	AS4-CPT01, AS4-CPT02, AS4-CPT03	Low – High Risk	3
DSCIN006 – Haverstock Road (Site S)	AS3-CPT02, AS3-CPT03, AS3-CPT06, AS3-CPT07	Low Risk	1
DSCIN007 – Lyon Avenue (Site Q)	-	-	
DSCIN008 – Mt Albert War Memorial Reserve (Site N)	AS1-CPT220	Low Risk	3
DSCIN009 – Western Springs Shaft (Site E)	WS1-CPT03 WS1-CPT04A, WS1-CPT04, WS1-CPT05, WS1-CPT11, WS1-CPT12, WS1-CPT13, WS1-CPT14, WS1-CPT16, WS1-CPT17, WS1-CPT18	Low Risk	2
DSLSB001 – Norgrove Avenue (Site M.2)	L2S2-CPT02	Low Risk	3
DSLB001 – Rawalpindi Reserve (Site L)	L2S1-CPT02, L2S1-CPT03, L2S1-CPT04, L2S1-CPT05	Low Risk	2
DSLSC001 – Haycock Avenue (Site AE.1 & AE.2)	L3S5-CPT01, L3S5-CPT02, L3S5-CPT03, L3S5-CPT04	Low - High Risk	6
DSLSC002 – Dundale Avenue (Site AC)	L3S4-CPT01, L3S4-CPT02, L3S4-CPT03, L3S4-CPT04, L3S4-CPT06, L3S4-CPT07	Low - High Risk	3
DSLSC003 – Whitney Street (Site AB)	L3S3-CPT01	Low Risk	1
DSLSC004 – Miranda Reserve (Site AA.1)	L3S2-CPT02, L3S2-CPT03	Low – High Risk	7

[2] Settlement values are taken from liquefaction analysis using CLiq following Idriss and Boulanger (2008). Maximum value for each site rounded up to the nearest centimetre.

Shaft Location	Reference CPT	Liquefaction ULS LPI	Max. Estimated Settlement, cm^2
DSLSC005 – PS25 (Site Z.1)	L3S1-CPT01, L3S1-CPT02, L3S1-CPT03, L3S1-CPT05, L3S1-CPT10	Low – High Risk	10
DPCIN – Mangere Pump Station (Site AU)	WS3-CPT01, WS3-CPT02	Low Risk	1

Note 1: CPT data from 18 m onwards, below the basalt.

6.3 Landslides

No active landslides were observed.

It is suspected that the land west of PS23, encompassing Hoskins Avenue and White Bluff, may be a relict block slide. There is no evidence of recent activity and the risk of this having an impact on the project is considered low.

6.4 Pre-existing volcanic conduits

Each volcanic vent will have a conduit feeding the basalt to the surface. These tubes of basalt would be expected to be vertical or sub-vertical and directly beneath the centre of each eruptive vent. The Central Interceptor alignment avoids travelling under these and so the risk of intersecting one during tunnelling is considered low. The closest feature is likely to be Mangere Lagoon.

6.5 Future volcanic activity

An eruption within the AVF could occur at any time: The return period between past events has ranged from tens to thousands of years. The most recent eruption occurred some 550 years ago. A future eruption may therefore occur at any time in the future. The site of future eruption cannot be predicted: The geologic record indicates that the AVF is a monogenetic volcanic field (typically only one eruption episode occurs from each vent, although some eruption episodes have involved more than one vent). The monogenetic nature of Auckland's volcanoes means that a future eruption will most probably involve a new volcano being formed, rather than renewed activity from an existing volcano.

There may be a relatively short pre-eruption period (possibly only a couple of days): The AVF volcanoes are characterised by low viscosity basaltic magma, which rises quickly to the crust (at speeds of between 0.1 and 2.2 km/hr). This means that the warning period for any pending eruption (from the early stages of detection to the commencement of volcanic activity) is likely to be short, in the order of 1 day to a few weeks.

The initial phase of activity is likely to be the most catastrophic: Most of the past eruptions have started with an explosive phreatomagmatic eruption producing base surges and resulting in the formation of a tuff ring. The abundance of sea water and groundwater in and around Auckland means a future eruption is also likely to start this way. The volcanic event may continue for a long time: Unlike many other natural hazards in New Zealand, a volcanic event will occur over a long time frame, over a period of months up to a year or more;

Given the relatively extended return period, the inability to predict the location and the inherent difficulty in designing for such an event no further consideration of this risk is recommended.

7. Groundwater

7.1 Introduction

The hydrogeology throughout the project consists of a highly stratified geological sequence including faulting and fracturing, which compartmentalise parts of the groundwater system. Aquifer studies that have been undertaken for preliminary design are outlined in Figure 7.1 and Table 1-1 Sources of factual information.

The interpreted results of the groundwater monitoring are presented on the drawings in Appendix C and Appendix D

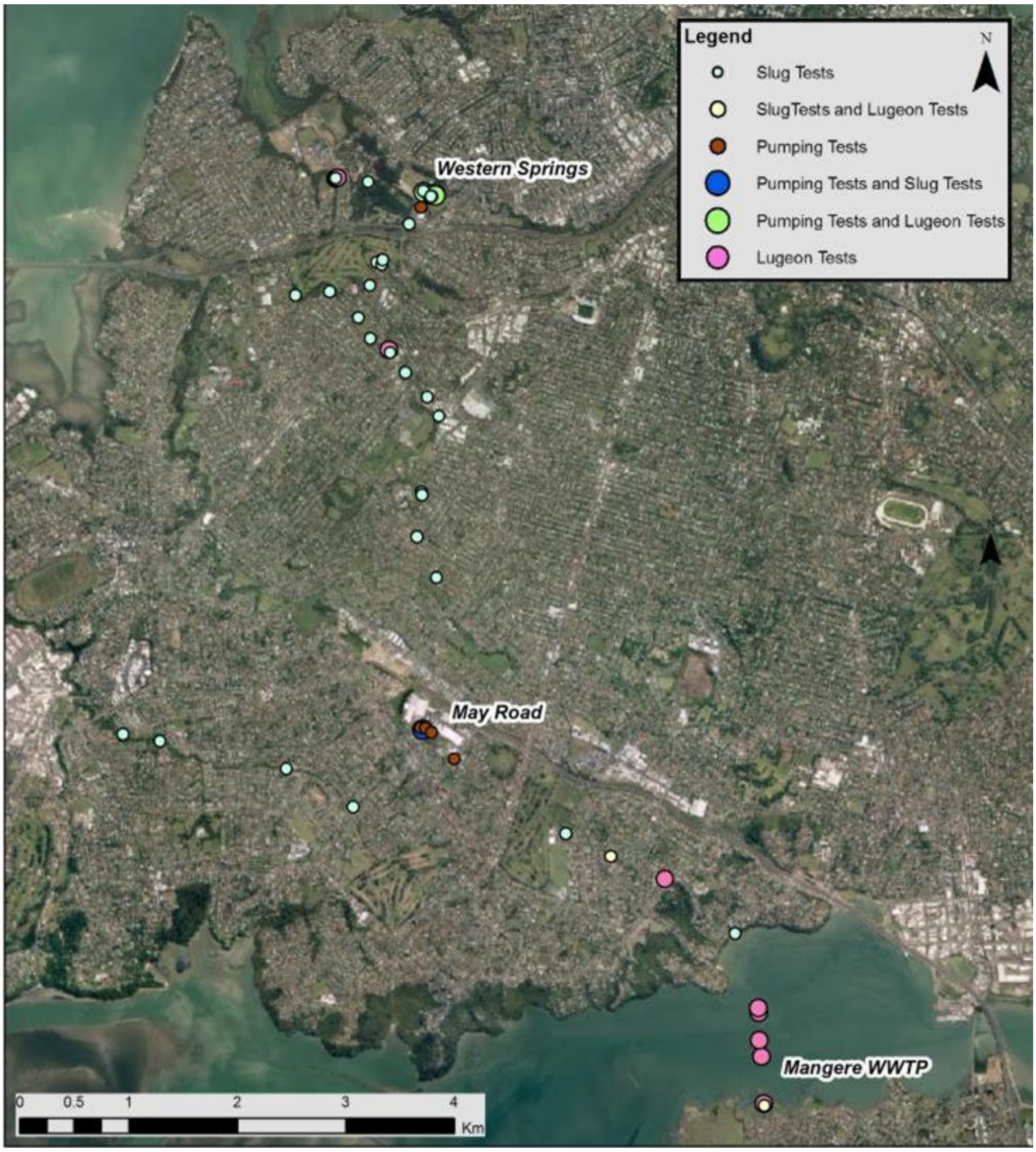


Figure 7.1 : Central Interceptor sites at which hydraulic tests have been carried out (excluding Manukau harbour crossing)

Table 7-1 : Summary of groundwater testing and monitoring for preliminary design

Test / monitoring type	Number undertaken
Vibrating wire piezometers (number of tips installed)	74
Standpipe piezometers (number of screens installed)	52
Slug tests	44
Lugeon tests	18
Pumping tests (number of sites tested)	3

7.2 Hydrogeology

The primary geological units within the project area are classified below, in terms of their hydrogeological properties.

7.2.1 Auckland Volcanic Field Basalts (AVFB)

This unit sometimes carries important quantities of groundwater through fractures in the rock mass, and plays an important role recharging both the shallow sediments units (Puketoka and Kaawa Formations) and the underlying Waitemata Group rocks (Institute of Geological and Nuclear Sciences, 2001). This basalt unit may inter-finger the Puketoka Formation, sometimes overlaying it and often times occurring under it. Shallow basalt aquifers have a near surface unconfined setting with water tables that are often close to the ground surface. Basalt rock is often described as having a high permeability but a relatively low dual porosity, with transmissivity values that can vary widely (Viljevac 2002).

7.2.2 Tauranga Group

In the Auckland and Manukau areas, this formation comprises a mixture of laterally discontinuous sands, silts and clays with various amounts of pumiceous and organic material. Consequently, groundwater yields from this formation can vary depending on location, heterogeneity and permeability of the aquifer. Generally the Tauranga group are considered to be a regional aquitard confining the Kaawa sediments.

7.2.3 Kaawa Formation

The Kaawa formation includes mudstones, muddy and shelly sandstones, bioclastic conglomerates, and lithic conglomerates (andesite and basalt pebbles) (Institute of Geological and Nuclear Sciences, 2001). This formation is present beneath much of the Manukau Lowland area, and it is an important aquifer system towards the South of the Manukau Harbour, due to the presence of porous shell beds and sand deposits with fractured sandstone. The layer is often considered to be confined by the Tauranga group and has a variable lithological composition, which can impact on its horizontal and vertical transmissivity. Higher groundwater flows are likely to be related to preferred flow paths in certain shell beds (Viljevac 2002).

7.2.4 Waitemata Group - ECBF

This group forms the hydrogeological basement formation in the Auckland area, and has influences on groundwater flows in the Kaawa Formation. Generally, the permeability is considered to be low to very low (averaging 2.7×10^{-2} m/d) (Viljevac 2002). Groundwater movement is likely to be through more permeable beds or distinct fractured zones (such as higher porosity fractured sandstone).

7.2.5 Waitemata Group - Parnell Volcaniclastic Conglomerate

Parnell Volcaniclastic Conglomerate is coarse sandstone to conglomerate found as lenses within the ECBF. Due to the unit's strength and lower clay content joints can remain open and have a greater persistence than ECBF allowing localised pathways for groundwater flow.

7.3 Aquifer properties

Recommended aquifer properties, adopted for design, are presented in Table 7-2 for each geotechnical unit. These properties have been estimated/established using results from pumping tests, slug tests and Lugeon testing undertaken during the investigations and previous investigations, and are based on best engineering practice and experience. Site specific parameters are presented for pumping test sites in Table 7-5. The analysis for the hydraulic testing is presented in Section 7.4, Appendix F, Appendix G and Appendix H.

Comparison of aquifer properties to previous studies for Central Interceptor Project and other Auckland tunnelling projects has been made by Coffey (2014). Table 7-3 and Table 7-4 compare horizontal hydraulic conductivity and anisotropy values of the hydrogeological units adopted from previous groundwater studies along the Central Interceptor Alignment and other Auckland tunnelling projects.

Table 7-2 : Recommended aquifer properties for preliminary design.

Stratigraphic/Geological Unit	Geotechnical Units		Transmissivity (T) (m2/d)	Hydraulic Conductivity (k) (m/s)	Storativity (S), Specific Yield (SY)
Made Ground	Made Ground	Engineered Fill	N/A	1×10^{-8} to 1×10^{-6} (1)	N/A
		Non-Engineered Fill			
Post AVF Tauranga Group alluvium and marine sediments	Recent Alluvium		4×10^{0} to 6×10^{0}	1×10^{-7}	SY: 8×10^{-1}
Tauranga group including Puketoka Fmn., estuarine, undifferentiated, colluvium	Undifferentiated Tauranga Group	Cohesive	1×10^{2} to 2×10^{2}	4×10^{-5} to 2×10^{-4}	S: 6×10^{-2} to 6×10^{-1}
		Granular			
Kaawa Formation	Kaawa Formation		1×10^{2} to 3×10^{2}	4×10^{-5} to 1×10^{-4}	S :4×10^{-2} to 4×10^{-1}
Auckland Volcanic Field	Tuff/Ash/Scoria		N/A	1×10^{-7} to 1×10^{-3} (2)	N/A
	Basalt		6×10^{-1} to 1×10^{1}	1×10^{-7} to 1×10^{-3}	SY:2×10^{-3} to 6×10^{-2}
East Coast Bays Formation	Residually to highly weathered cohesive soils		3×10^{0} to 4×10^{0}	1×10^{-7} to 1×10^{-6}	S: 4×10^{-3}
	Residually to highly weathered granular soils		N/A	N/A	N/A
	Moderately weathered to unweathered ECBF		1×10^{0} to 1×10^{1}	2×10^{-8} to 2×10^{-5}	S:4×10^{-5} to 4×10^{-3}
Parnell Volcaniclastic Conglomerate	Parnell Volcaniclastic Conglomerate		N/A	5×10^{-7} to 1×10^{-3} (1)	N/A

(1) Values adopted from Matakite

(2) Values adopted from Tonkin and Taylor (2012)

Table 7-3 : Horizontal hydraulic conductivity values adopted for Preliminary Design, Matakite and other Auckland tunnelling projects (after (Coffey Geotechnics, 2014)).

Hydrogeological Unit	Horizontal Hydraulic Conductivity (m/s)	Data Source[(1)]	Relevance[(2)]
Fill	$1x10^{-08}$ to $1x10^{-06}$	Matakite	High
Tauranga Group Alluvium	$1x10^{-08}$ to $1x10^{-05}$	Matakite	High
	$1x10^{-09}$ to $1x10^{-06}$	Tonkin and Taylor (2012)	High
Tauranga Group Estuarine Sediments	$1x10^{-08}$ to $1x10^{-05}$	Matakite	High
	$1x10^{-09}$ to $2x10^{-07}$	Tonkin and Taylor (2012)	High
Puketoka Formation	Sands: $1x10^{-07}$ to 1x10-04	Matakite	High
	Silts: $1x10^{-09}$ to $1x10^{-07}$	Matakite	High
	Clays: $1x10^{-11}$ to $1x10^{-08}$	Matakite	High
	Fine grained: $2x10^{-08}$ to $2x10^{-06}$	Tonkin and Taylor (2012)	High
	Coarse grained: $2x10^{-07}$ to $2x10^{-05}$	Tonkin and Taylor (2012)	High
	Fine grained: $1x10^{-07}$ to $2x10^{-07}$	Waterview Connection project, T&T	Moderate
	Fine grained: $2x10^{-07}$	Vic Park Tunnel project, T&T	Low
	Fine grained: $3x10^{-07}$	New Lynn project, T&T	Low
	Fine grained: $4x10^{-09}$ to $2x10^{-07}$	Rosedale Tunnel project, T&T	Very low
Kaawa Formation	$1x10^{-07}$ to $1x10^{-05}$	Matakite	High
	$1x10^{-07}$ to $1x10^{-04}$	Tonkin and Taylor (2012)	High
AVF Basalt	$1x10^{-06}$ to $1x10^{-01}$	Matakite	High
	$2x10^{-05}$ to $2x10^{-03}$	Auckland Council (2014)	High
	$1x10^{-05}$ to $1x10^{-01}$	Strayton et al (2005)	High
	$1x10^{-06}$ to $1x10^{-04}$	Tonkin and Taylor (2012)	High
	$1x10^{-05}$	Waterview Connection project, T&T	Moderate
	$1x10^{-08}$ to $2x10^{-04}$	Vector Tunnel project, T&T	Low
	2x10-4	Three Kings Quarry project, T&T	High
AVF Tuff	Tuff: $1x10^{-05}$ to $1x10^{-03}$	Matakite	High
	Ash: $1x10^{-08}$ to $1x10^{-05}$	Matakite	High
	$1x10^{-07}$ to $1x10^{-03}$	Tonkin and Taylor (2012)	High
ECBF	$1x10^{-08}$ to $1x10^{-06}$	Matakite	High
	$2x10^{-08}$ to $2x10^{-06}$	Tonkin and Taylor (2012)	High
	$3x10^{-07}$ to $5x10^{-07}$	Waterview Connection project,	Moderate

Hydrogeological Unit	Horizontal Hydraulic Conductivity (m/s)	Data Source[1]	Relevance[2]
		T&T	
	1x10^{-07} to 4x10^{-07}	Vic Park Tunnel project, T&T	Low
	1x10^{-07}	New Lynn Rail project, T&T	Low
	5x10^{-08}	Rosedale Tunnel project, T&T	Very low
	5x10^{-08}	Hobson Bay Tunnel project, T&T	Low
	2x10^{-07} to 4x10^{-08}	Vector Tunnel project, T&T	Low
	1x10^{-08}	Three Kings Quarry project, T&T	High
ECBF (weathered)	1x10^{-08} to 1x10^{-03}	Matakite	High
	2x10^{-08} to 2x10^{-06}	Tonkin and Taylor (2012)	High
	2x10^{-07}	Waterview Connection project, T&T	Moderate
	2x10^{-07}	Vic Park Tunnel project, T&T	Low
	3x10^{-07}	New Lynn Rail project, T&T	Low
Parnell Grit Member	1x10^{-07} to 1x10^{-03}	Matakite	High

(1) Matakite refers to the Matakite Part D1, Phase I Geotechnical Investigation Report, Volume 1 of 5; T&T refers to Tonkin and Taylor (2012).

(2) Relevance to CI project based on proximity to CI alignment.

Table 7-4 : Hydraulic Conductivity Anisotropy values adopted for Preliminary Design, Matakite and other Auckland tunnelling projects (after (Coffey Geotechnics, 2014)).

Hydrogeological Unit	Ratio of Vertical to Horizontal Hydraulic Conductivity (k_v/k_h)	Data Source	Relevance
Fill	1	Matakite	High
Tauranga Group Alluvium	1	Matakite	High
Tauranga Group Estuarine Sediments	1	Matakite	High
Puketoka Formation	1 Fine-grained: 0.1 Fine-grained: 0.2	Matakite Victoria Park Tunnel project, T&T New Lynn project, T&T	High Low Low
Kaawa Formation	1	Matakite	High
AVF Basalt	1 1 to 4 1	Matakite Waterview Connection project, T&T Three Kings Quarry project, T&T	High Moderate High

Hydrogeological Unit	Ratio of Vertical to Horizontal Hydraulic Conductivity (k_v/k_h)	Data Source	Relevance
AVF Tuff	1	Matakite	High
ECBF	0.1 0.2 to 1 0.1 0.2 0.075 0.075	Matakite Waterview Connection project, T&T Victoria Park Tunnel project, T&T New Lynn Rail project, T&T Rosedale Tunnel project, T&T Hobson Bay Tunnel project, T&T	High Moderate Low Low Very low Low
ECBF (weathered)	0.1 0.1 0.2	Waterview Connection project, T&T Victoria Park Tunnel project, T&T New Lynn Rail project, T&T	Moderate Low Low

7.4 Design aquifer studies

Aquifer tests were carried out at the three primary sites: Western Springs, May Rd, and Mangere WWTP; and along the tunnel alignment and at other shaft sites (Figure 7.1). The hydraulic tests carried out at these sites are a combination of Constant Discharge Pumping Tests, Lugeon Tests, and Slug Tests (Rising and Falling Head Tests) and were used to derive hydraulic parameters (hydraulic conductivity, transmissivity, and storativity).

A summary of the results and discussion are presented in this section. Site descriptions, methodology, results and analysis are presented in Appendix F. Slug test calculations are presented in Appendix G and Pumping test calculations in Appendix H.

7.4.1 Summary

Average results of pumping tests are presented in Table 7-5 and give hydraulic parameters for transmissivity, hydraulic conductivity and storativity. The results presented are representative of the formations hydrogeological properties derived from pumping and observation bores, with the pumping test well presented as an indication of location.

Table 7-5 Hydraulic parameters derived from pumping test results

Location	Pumping well	Unit Tested	Transmissivity (T) (m^2/d)	Hydraulic Conductivity (k) (m/s)	Storativity (S), Specific Yield (SY)
Western Springs	BH206b	AVF - Basalt	1.11	3.2×10^{-6}	$3.3x10^{-2}$ (SY)
Western Springs	205	ECBF	4.04	3.2×10^{-6}	$2.1x10^{-4}$ (S)
May Road	BH247a	AVF - Basalt	0.29	1.7×10^{-7}	$1.3x10^{-2}$ (SY)
May Road	BH247b	ECBF	2.16	4.9×10^{-7}	$2.0x10^{-3}$ (S)

Location	Pumping well	Unit Tested	Transmissivity (T) (m^2/d)	Hydraulic Conductivity (k) (m/s)	Storativity (S), Specific Yield (SY)
Mangere WWTP	BH273	Tauranga Group - Puketoka Fmn	145	$8.2x10^{-5}$	$2.4x10^{-1}$
Mangere WWTP	BH273	Kaawa Formation	182	$8.6x10^{-5}$	$1.9x10^{-1}$

According to the results of tests in the Western Springs area, the hydraulic conductivity (k) for the East Coast Bay Formation (ECBF) is in the 10^{-6} m/s order of magnitude with a Storativity (S) of $2.1x10^{-4}$. The Auckland Volcanic Field Basalts (AVFB) unit has a similar k value to the one calculated for the ECBF in this area (10^{-6} m/s order of magnitude), but with a higher S ($3.31x10^{-2}$).

In the May Road area, hydraulic tests indicate a lower k for the ECBF (10^{-7} m/s order of magnitude) in this area than in Western Springs, but a higher S values ($1.97x10^{-3}$). Similar k results were obtained for the AVFB in this area (10^{-7} m/s) but slightly lower S values ($1.25x10^{-2}$).

Hydraulic tests at the Mangere WWTP indicate a k value in the 10^{-7} m/s order of magnitude for the ECBF and about $8x10^{-5}$ m/s for both the Kaawa Formation and the Puketoka Formation. In addition, the S value for these last two units is 0.19 and 0.24 respectively. The Kaawa Formation exhibits a slightly lower k value at the Mangere Lagoon Isthmus (10^{-6} m/s order of magnitude). The Puketoka Formation hydraulic properties calculated with these tests characterise an unconfined aquifer system consisting mainly of sands.

Average hydraulic conductivity results derived from Lugeon tests are presented in Table 7-6 and from slug tests in Table **7-7**. Lugeon and Slug tests along other areas of the alignment suggest k values for the ECBF in the 10-7 m/s order of magnitude, for bores to the south of Western Springs, south of May Road, and north of Mangere (next to Kiwi Esplanade). Tests in the AVFB and the Puketoka Formation suggest k values in the same order of magnitude (10-7 m/s) for these units away from main shaft areas.

Results from this study are in accordance with the hydraulic properties of similar materials in other basins (Fetter, 1988). In general, the calculated k and S values are in agreement with previous studies carried out near the Britomart area (PDP, 2014) and for the Waterview Tunnel (Tuhono Consortium, 2011). The only exception is the hydraulic conductivity at the Western Springs site which seems to be about 1 order of magnitude higher than the one estimated in previous studies.

Table 7-6 Hydraulic conductivity values (k) (m/s) derived from Lugeon results

Hydraulic Conductivity (k) (m/s)	ECBF Average k	ECBF Max k	AVF - Basalt
Number of results	17	17	1
Minimum	$8.4 x10^{-8}$	$1.5 x10^{-7}$	

Hydraulic Conductivity (k) (m/s)	ECBF Average k	ECBF Max k	AVF - Basalt
25th percentile	2.5×10^{-7}	4.8×10^{-7}	
Median	6.3×10^{-7}	1.4×10^{-6}	1.6×10^{-7}
75th percentile	1.7×10^{-6}	6.1×10^{-6}	
90th percentile	4.0×10^{-6}	9.2×10^{-6}	
Maximum	1.5×10^{-5}	2.9×10^{-5}	

Table 7-7 Hydraulic conductivity values (k) (m/s) derived from slug test results

Hydraulic Conductivity (k) (m/s)	Tauranga Group - Puketoka Fmn	Kaawa Fmn	AFV - Basalt	Parnell Volcaniclastic Conglomerate	ECBF
Number of results[1]	8	1	8	1	22
Minimum	1.3×10^{-7}		1.2×10^{-7}		5.2×10^{-9}
25th Percentile	2.2×10^{-7}		5.5×10^{-7}		2.0×10^{-7}
Median	4.5×10^{-7}	3.1×10^{-6}	1.1×10^{-6}	1.1×10^{-7}	4.0×10^{-7}
75th Percentile	6.3×10^{-7}		2.2×10^{-6}		9.0×10^{-7}
90th Percentile	1.0×10^{-6}		2.7×10^{-6}		2.2×10^{-6}
Maximum	2.0×10^{-6}		3.2×10^{-6}		3.5×10^{-6}

[1]Note: Four slug tests results were not included in the table because the test section crossed over two geological units.

7.4.2 Discussion

7.4.2.1 Western Springs

Lugeon tests conducted in the ECBF (Waitemata Group) within the Western Springs area suggest an average hydraulic conductivity value of 7×10^{-6} m/s ($3. \times 10^{-1}$ m/day) for this unit. Similarly, the average hydraulic conductivity calculated with Slug Tests is about 1.1×10^{-6} m/s ($1. \times 10^{-1}$ m/day) for the ECBF in this area. The pumping test in the basalt (BH206b) was conducted over 1 day, with a 1 day recovery. Only one observation bore (BH206) was utilised in the assessment of aquifer characteristics. The ECBF pumping test (BH205) was conducted over 6 days, with a 1 day recovery. Three observation bores (BH204, BH206, and BH207) were used for aquifer characterisation. Average pumping test results are presented in Table 7-8**.**

In summary**,** the hydraulic conductivity (k) values for the ECBF from the various tests, yield results in the same order of magnitude (e.g. 10^{-6} m/s). Sandstone and mudstone sedimentary rocks are known to have hydraulic conductivity values of this order of magnitude (Freeze A, 1979) so these aquifer test results are consistent with literature values.

A previous study by PDP (2014) suggests that localised anomalies are evident and characteristic of the ECBF, and there could be a high degree of variability in the hydraulic properties of this unit. For example, the PDP report adopts low (10^{-7} m/s) and very low (10^{-8} m/s) k values for the ECBF and a Storage coefficient of $1x10^{-5}$ m^{-1}. Investigations for the Waterview tunnel (Tuhono Consortium, 2011), located at about 2–2.5km to the West of the Central Interceptor alignment between Western Springs and May Rd, show similar results. The value of the hydraulic conductivity in this study was estimated as $2.3x10^{-7}$ m/s ($1.9x10^{-2}$ m/day) and a storage coefficient of $9x10^{-6}$ m^{-1} for the ECBF. On the other hand, estimates for the present study suggest that k for the ECBF is in the 10^{-6} m/s order of magnitude and the calculated Storage coefficient is 1.44 $x10^{-5}$ m^{-1} (aquifer thickness = 14.6m).

Therefore, the Storage coefficient for Western Springs in the present study is in the same order of magnitude than the value previously adopted by PDP in the Britomart area. However, the k value calculated in the present study is at least one order of magnitude higher than previously calculated values by PDP (2014) and Tuhono Consortium (2011).

Table 7-8. Results from Western Spring Pumping Tests

Formation	T (m^2/d)	k (m/s)	S & SY
AVF Basalt (BH206b)	1.11	3.2×10^{-6}	3.31×10^{-2} (SY)
ECBF (BH205)	4.04	3.19×10^{-6}	2.10×10^{-4} (S)
Notes: T = Transmissivity k = Hydraulic Conductivity Sy = Specific Yield S = Storativity (dimensionless)			

7.4.2.2 May Road

There were no Lugeon tests carried out at the May Rd site. However, a test in the vicinity of this site (BH252) resulted in a hydraulic conductivity value of 1.18×10^{-7} m/s for the Waitemata Group (ECBF). Slug tests carried out at May Rd resulted in the following average hydraulic conductivity values:

- k = 9.82×10^{-7} m/s (ECBF)
- k = 1.02×10^{-7} m/s (Puketoka/ECBF)
- k = 1.27×10^{-7} m/day (Puketoka)

The Pumping Test in the basalt was conducted over 1 day, with a 1 day recovery period. Only one VW piezometer (BH246 at 4m) was utilised in the assessment of aquifer characteristics of the AVFB unit because this was the only borehole screened within the basalt formation. The ECBF pumping test was conducted over 6 days, with an 8 day recovery. Two observation bores were used for aquifer characterisation. Average results are presented in Table 7-9**.**

All the test types employed in this investigation yielded hydraulic conductivity results in the same order of magnitude for tests in the ECBF (e.g. 10^{-7} m/s) in the May Rd area and its vicinity. These values are also within the order of magnitude of k values for sandstones and limestones, observed in other basins (Freeze A, 1979), and in the same order of magnitude of k values calculated in the PDP (2014) and the Waterview (2011) studies. The Storage coefficient for the ECBF in this area (Ss = 1.5×10^{-5} m^{-1} considering an aquifer thickness =14m) is comparable to values adopted in previous studies (e.g. Ss =1×10^{-5} m^{-1} in the Britomart area (PDP, 2014) and Ss= 9×10^{-6} m^{-1} for the Waterview tunnel (Tuhono Consortium, 2011)). The specific yield calculated for this unit is also comparable (albeit one order of magnitude lower) than values adopted in the PDP (2014) study. However, this last value is only from one test in the May Road area so it is not possible to extend this result to other areas without further testing.

Table 7-9. Results from May Road Pumping Tests

Formation	T (m^2/d)	k (m/s)	S & SY*
AFVB (BH247a)	0.29	1.71×10^{-7}	1.25×10^{-2} (SY)
ECBF (BH247b)	2.16	4.9×10^{-7}	1.97×10^{-3} (S)

*Specific yield or storativity

7.4.2.3 Mangere WWTP

Two Lugeon tests, targeting the ECBF at 2 depth intervals, were carried out in BH271 at the Mangere WWTP. The average hydraulic conductivity results for these units are summarised are follows:

- From 30.8 to 33.7 mBGL, k = 5.92×10^{-7} m/s
- From 45 to 48 mBGL, k = 6.27×10^{-7} m/s

In addition, Lugeon tests in the Manukau Harbour, north of Mangere, targeted the ECBF between 15mBGL and 45.3mBGL. The average hydraulic conductivity for this unit at this location was 2.7×10^{-6} m/s.

The only Slug Tests at the Mangere WWTP were carried out in BH268 as it was being drilled and in BH273 after the Pumping Test, when this bore was completed at a deeper unit targeting the ECBF. The hydraulic conductivity values resulting from these tests are:

- k = 1.10×10^{-7} m/s for the ECBF at the Mangere WWTP (BH273)
- k = 3.10×10^{-6} m/s for the Kaawa Formation on the Mangere Lagoon isthmus (BH268)

Lugeon and Slug tests suggest that the average hydraulic conductivity of the ECBF at the Mangere WWTP site in the order of 10^{-7} m/s, which is consistent with what is expected for sandstones and mudstones (Freeze A, 1979). The Kaawa Formation at the Mangere Lagoon Isthmus (e.g. BH268) presents a relatively high hydraulic conductivity equivalent to the lower end of a silty sand material. This is not surprising because this formation consists of shelly silty sand material with good hydraulic properties.

The Mangere Pumping Test has resulted in estimates of hydraulic properties for the Puketoka Formation and the Kaawa Formation equivalent, as summarised in Table 7-10. The Puketoka Formation inter-fingers with lava and tuff of the South Auckland Volcanic Field in the Manukau Lowland area (Greig, 1989). The logs for bores in the Mangere study area show terrestrial sediments (sands, silts and clays), volcanic field basalts, and tuff.

Table 7-10. Aquifer properties for Mangere, calculated from pumping tests

	T (m^2/d)	k (m/s)	S
Puketoka Formation	145	8.2×10^{-5}	2.37×10^{-1}
Kaawa Formation (BH273)	182	8.6×10^{-5}	1.91×10^{-1}

The Puketoka Formation hydraulic properties calculated with this pumping test analysis characterise an unconfined aquifer system consisting mainly of sands. Nearing the base of the Puketoka Formation, silty clay materials are evident in these bores, which have good confining properties. In the “Mangere Pump Station” area the Puketoka Formation has an average Transmissivity value of 145 m^2/d (maximum 210 m^2/d) and a Storativity of about 2.5x10^{-1}. This value is within the range (0.02 - 0.3) of known storativity values for unconfined aquifers (Fetter, 1988).

The Kaawa Formation (e.g. shell beds) has an average Transmissivity of 181 m^2/d, taking into account all bores, and a maximum value of 285 m^2/d calculated from the pumping bore. However, it is noted that the response of vibrating wire piezometers, in observation bores at 30m–92m from the pumping bore, is not necessarily a direct response to the pumping itself as these bores could be subject to tidal variations, barometric pressure changes, and recharge variations. Nevertheless, the equivalent hydraulic conductivity value for the Kaawa at this location (BH273) is significantly higher than the one for BH268 (Mangere Isthmus). Again, this is not surprising because the bore log for BH273 shows clean sand material with a lower proportion of fine sediment.

The average Storativity for the Kaawa has been estimated at about 1.91x10^{-1}. The average Transmissivity value for the Kaawa is of similar magnitude than average values derived from recharge investigations in South Auckland (Viljevac Z, 2002). However, the Storativity values derived from the South Auckland study were about two to four orders of magnitude lower than the ones derived in the present investigation. Also, the 2002 values came from analyses of pumping test data from bores located to the south of Papakura, in the Franklin area. In this area, the Kaawa Formation is known to be an important aquifer, used extensively, but presenting significant variation in hydraulic properties. Towards the Manukau Harbour, the Kaawa thins out and is not extensively used; its confining units also tend to thin out, so the Kaawa may behave as a leaky aquifer in this area. Consequently, the Kaawa Formation has higher storativity values in the Mangere Pump Station area. Crowcroft and Smaill (2001) have reported a lower bound value (10^{-2}) for the Kaawa storativity, which is only 1 order of magnitude higher than the value calculated with the Mangere Pumping Test.

7.4.2.4 Other areas along the Tunnel alignment

Lugeon tests carried out along the tunnel alignment (e.g. not within the main shaft areas) focused on estimating the hydraulic conductivity of the ECBF. One Lugeon test was conducted in BH219 immediately south of the Western Springs site and another one in BH252 was carried out to the west of May Road. An additional test was carried out in BH253 but these test results were considered inaccurate due to data inadequacies. The average hydraulic conductivity results for these units are summarised are follows:

- k = 8.4 x10^{-8} m/s for the ECBF in BH219 near the Western Springs Site
- k = 1.2 x10^{-7} m/s for the ECBF in BH252 to the West of May Road

Lugeon tests in BH258 and BH259 (next to Kiwi Esplanade, about 2.5km north of the main testing site) targeted the AFVB and the ECBF respectively.

- k = 1.3 x10^{-7} m/s for the AVFB at about 2.5km north of the Mangere WWTP site (BH258)
- k = 1.7 x10^{-6} m/s [28-38m depths], 1. 2 x10^{-7} m/s [35-38m depths], and 2.5 x10^{-7} m/s [55.5-63m depths] for the ECBF in BH259, located at about 2.5km north of the Mangere WWTP site

In general, slug tests for selected bores outside of the main shaft areas showed an ECBF average hydraulic conductivity of 3.0 x10^{-7} m/s for bores to the south of Western Springs, 1.2 x10^{-7} m/s for bores to the south of May Road, and 7.5 x10^{-7} m/s for a bore north of Mangere (next to Kiwi Esplanade). For bores south of Western Springs, slug tests showed an average k of 3.7 x10^{-7} m/s for screened intervals targeting the Puketoka Formation. No Pumping tests were carried out along the alignment, outside the main shaft areas. Given the proximity to the Mangere WWTP and similar order of magnitudes for ECBF k values calculated with slug tests, parameters for other units in the Kiwi Esplanade Shaft can be taken from the Mangere WWTP shaft.

8. Geotechnical parameter development

8.1 Introduction

The following gives a detailed description of the division of the geological materials into geotechnical units and the methodology for deriving the geotechnical design parameters for each of the geotechnical units for rock (Section 8.3.4) and soil (Section 8.5). The data collection contains results from field and laboratory data collected from boreholes (BH), hand auger (HA) holes and test pits (TP). Sources of data include:

- Central Interceptor Main Project Works Detailed Design - Geotechnical Factual Report (2015/2016)
- Central Interceptor and Associated Works: Phase 1 to Phase 4 Geotechnical Investigations (Matakite) (2010/2011)
- Geotechnical Data Report - Waterview Connect NZTA (2011)

Measured and derived parameters have been tabulated for 0th (minimum), 10th, 25th, 50th (median), 75th and 100th (maximum) percentile values – using a normal probability distribution – and results are summarised in Appendix I to Appendix S.

Samples were tested by laboratories accredited to international standard ISO 10725 by organisations such as IANZ, NATA or SANAS. The following accredited laboratories were engaged for this project:

- Coffey Geotechnics NZ Limited (Coffey) – East Tamaki Laboratory, Auckland, New Zealand – IANZ Accredited Laboratory
- Bamford Rock Testing Services (BRTS) – North Melbourne, Victoria, Australia – NATA Accredited laboratory
- Rocklab, Division of Soillab, Part of the SMEC Group – Pretoria, South Africa – SANAS Accredited Laboratory

Testing methods from historic data (Waterview) were given where available.

8.2 Division of geological materials into geotechnical units for design

Geotechnical units are based on the geological units outlined in Section 3 and are outlined in Table 8-1.

Table 8-1 Geotechnical units adopted for design parameters and relationship with geological unit

<table>
<tr><th>Stratigraphic/Geological Unit</th><th colspan="2">Geotechnical Units</th><th>Lithology</th><th>Material Type</th></tr>
<tr><td rowspan="2">Made Ground</td><td rowspan="2">Made Ground</td><td>Engineered Fill</td><td rowspan="2">Clay, silt, sand and gravel</td><td rowspan="7">Soil</td></tr>
<tr><td>Non-Engineered Fill</td></tr>
<tr><td>Post AVF Tauranga Group alluvium and marine sediments</td><td colspan="2">Recent Alluvium</td><td>Silt and sand</td></tr>
<tr><td rowspan="2">Tauranga group including Puketoka Fmn., estuarine, undifferentiated, colluvium</td><td rowspan="2">Undifferentiated Tauranga Group</td><td>Cohesive</td><td>Clay and silt</td></tr>
<tr><td>Granular</td><td>Sand</td></tr>
<tr><td>Kaawa Formation</td><td colspan="2">Kaawa Formation</td><td>Shelly with silt and sand</td></tr>
<tr><td rowspan="2">Auckland Volcanic Field</td><td colspan="2">Tuff/Ash/Scoria</td><td>Silt, sand and gravel, can be intermixed with clay</td></tr>
<tr><td colspan="2">Basalt</td><td>Intact, jointed, vesicular and rubbly</td><td>Rock</td></tr>
<tr><td rowspan="3">East Coast Bays Formation</td><td colspan="2">Residually to highly weathered cohesive soils</td><td>Silt and clay</td><td rowspan="2">Soil</td></tr>
<tr><td colspan="2">Residually to highly weathered granular soils</td><td>Sand</td></tr>
<tr><td colspan="2">Moderately weathered to unweathered ECBF</td><td>Mudstone and muddy sandstone</td><td rowspan="2">Rock</td></tr>
<tr><td>Parnell Volcaniclastic Conglomerate</td><td colspan="2">Parnell Volcaniclastic Conglomerate</td><td>Course sandstone to conglomerate</td></tr>
</table>

8.2.1 Made ground

For the purposes of parameter development Made Ground is split into engineered and non-engineered fill. However, in geological sections and geotechnical descriptions no such division was made.

8.2.2 Recent Alluvium

Recent alluvium comprises sediments which are part of active waterways identifiable during investigations. Marine sediments observed in the Manukau Harbour have been grouped with recent alluvium for parameter description. Recent Alluvium tends to be weaker and more compressible than Undifferentiated Tauranga Group discussed in Section 8.2.3.

8.2.3 Undifferentiated Tauranga Group

Tauranga Group deposits have been grouped into two geotechnical units: cohesive and granular soils. However, in geological sections and geotechnical descriptions no such division was made as the materials often are inter-fingered and laterally discontinuous. Wood fragments and logs have been encountered during drilling within this material.

8.2.4 Kaawa Formation

Materials identified as Kaawa Formation during investigations are typically shelly and comprise medium dense to very dense sands and sandy silts. In borehole logging, the Kaawa Formation has been distinguished from Tauranga group by the occurrence of abundant shell remains. The Kaawa Formation is identified at Mangere Pumping Station and extends north for approximately 1 km.

8.2.5 Tuff/Ash/Scoria

Tuff, ash and scoria deposits of the Auckland Volcanic Field are grouped into one geotechnical unit. These deposits are often inter-fingered and not identified as sub-units in project geological sections or descriptions.

8.2.6 Basalt

Basalt includes all lava identified within the project area and can comprise a range of rock masses including intact basalt, columnar jointed, vesicular and rubbly basalt.

8.2.7 Residually to highly weathered cohesive soils

Residually to highly weathered cohesive soils are those weathered from ECBF.

8.2.8 Residually to highly weathered granular soils

Granular soils weathered from ECBF are uncommon and are only identified at Rawalpindi Reserve.

8.2.9 Moderately to Unweathered ECBF

Moderately to unweathered ECBF comprises mudstone and sandstone but excludes PVC. While possible to differentiate siltstone beds from sandstone beds, at the scale of this project both will be encountered across the full face of excavations and therefore composite behaviour should be expected. Only one set of parameters is given.

8.2.10 Parnell Volcaniclastic Conglomerate

Parnell Volcaniclastic Conglomerate is identified as coarse sandstone to conglomerate with clasts of volcanic material and is described in Section 3.3.

8.3 General parameters

8.3.1 Bulk density / unit weight and dry density / unit weight

The bulk and dry density of the Geotechnical Units was determined using the following methods:

- Laboratory testing on rock samples as part of UCS testing (see Section 8.4.1 below);
- Laboratory testing on soil samples for in-situ soil density (NZS 4402:1986 Test 6.2.1 Part 4.2c)
- Laboratory testing on soil samples as part of one point compaction testing (NZS 4402:1986 Test 4.1.1)
- Laboratory testing on soil samples as part of one dimensional consolidation testing (NZS 4402:1986 Test 7.1)

Results of the bulk density / bulk unit weight and dry density / dry unit weight tests are presented in Appendix I

The recommended bulk and dry unit weight values are summarised in Table 9-1 and are based on laboratory test results, where available.

8.3.2 Moisture content

Sample were analysed for moisture content following:

- NZS 4402:1986 Test 2.1 (Coffey),
- AS 4133.1.1.1 – 2005 (BRTS),
- and taken from PSD tests, rock porosity density, UCS, consolidation and soil density tests.

Results of the moisture content tests are presented in Appendix J

8.3.3 Mineralogy

Mineralogy was identified in core samples of rock and soil using quantitative and qualitative X-Ray Diffraction (XRD) and petrographic analysis. The results are reported in Appendix U. The quantum and location of the tests undertaken is summarised in Table 8-2: Summary of mineralogical testing. The individual analyses are presented in the factual report.

Table 8-2: Summary of mineralogical testing data

Laboratory	Number of petrographic determinations	Number of Quantitative XRD tests	Number of Qualitative XRD tests
Auckland University	0	0	18
BRTS (Melbourne)	8	7	0
Rocklab (South Africa)	36	0	9

8.3.4 Corrosivity/Durability

Chemical tests were undertaken on selected soil and rock samples and include pH, Sulphate and Chloride. The results are summarised in the tables below and presented in graphs in Appendix V.

The results indicate potential durability issues for permanent concrete cast against natural ground, particularly in the Tauranga Group materials. Where required, chemical exposure classifications in accordance with NZS3101 should be adopted unless imported backfill or waterproofing membranes are placed between natural ground and the permanent structures.

Table 8-3 pH tests

pH										
Geological Unit	Minimum	10th Percentile	25th Percentile	Median	75th Percentile	90th Percentile	Maximum	Mean	Standard deviation	Count
All results in this investigation	2.6	5.1	6.2	8.0	9.5	10	10	8	2	179
Made Ground / Fill	6	7	7	8	8	8	10	8	1	8
Recent Alluvium / Q1a	6	6	7	8	8	8	9	7	1	4
TAURANGA GROUP										
Undifferentiated Tauranga Group	5	5	6	8	9	10	10	7	2	10

pH										
Geological Unit	Minimum	10^{th} Percentile	25^{th} Percentile	Median	75^{th} Percentile	90^{th} Percentile	Maximum	Mean	Standard deviation	Count
Tauranga Estuarine	4	5	6	7	9	9	10	7	3	2
Tauranga Puketoka	2.6	5.2	6.3	8.1	9.5	10.0	10.0	7.9	1.9	113
Kaawa Formation	6.7	6.8	7.0	7.2	7.5	7.6	7.7	7.2	0.5	2
AUCKLAND VOLCANIC FIELD (PLEISTOCENE)										
Tuff / Ash/ Scoria	7	7	7	7	7	7	7	7	0.0	1
WAITEMATA GROUP (MIOCENE)										
Residual to Highly Weathered Residual Soils - Cohesive Soils	3.5	4.7	5.3	8.0	9.0	10.0	10.0	7.5	2.1	26
Residual to Highly Weathered Residual Soils - Granular Soils	4.9	5.4	6.2	7.5	8.7	9.5	10.0	7.5	2.6	2
Parnell Volcaniclastic Conglomerate	8.5	8.5	8.6	8.6	8.7	9	9	8.6	0.1	2
Moderately Weathered to Unweathered ECBF	4.2	4.5	6.2	8.0	8.4	9.6	10	7.4	1.9	9

Table 8-4 Chloride tests

Chloride (mg/Kg)										
Geological Unit	Minimum	10^{th} Percentile	25^{th} Percentile	Median	75^{th} Percentile	90^{th} Percentile	Maximum	Mean	Standard deviation	Count
All results in this investigation	3.1	7.4	13.0	26.0	66.5	164	7700	289	1165	87
Made Ground / Fill	32	34	38	44	49	53	55	44	12	2
Recent Alluvium / Q1a	180	180	180	180	180	180	180	180	0	1
TAURANGA GROUP										
Undifferentiated Tauranga Group	7	9	13	20	25	27	29	19	9	3
Tauranga Estuarine	18	18	18	18	18	18	18	18	0	1
Tauranga Puketoka	3.5	7.7	14.0	29.0	71.8	164	7700	338	1282	54
Kaawa Formation	22.0	35.8	56.5	91.0	125	146	160	91.0	69.0	2
AUCKLAND VOLCANIC FIELD (PLEISTOCENE)										
Tuff / Ash/ Scoria	50	50	50	50	50	50	50	50	0.0	1
WAITEMATA GROUP (MIOCENE)										
Residual to Highly Weathered Residual Soils - Cohesive Soils	3.1	5.2	11.7	28.0	45.0	67.5	5500	483	1512	12
Residual to Highly Weathered Residual Soils - Granular Soils	3.9	3.9	3.9	3.9	3.9	3.9	3.9	3.9	0.0	1
Parnell Volcaniclastic Conglomerate	15.0	16.6	19.0	23.0	27.0	29	31	23.0	8.0	2
Moderately Weathered to Unweathered ECBF	7.7	8.7	10.5	14.0	17.3	116	340	54.1	108	8

Table 8-5 Soil Sulphate tests

Soil Sulphate (KCL Extraction) (mg/Kg)										
Geological Unit	Minimum	10th Percentile	25th Percentile	Median	75th Percentile	90th Percentile	Maximum	Mean	Standard deviation	Count
All results in this investigation	3.1	10.8	26.0	130.0	455.0	1340	4700	469	859	99
Made Ground / Fill	7	9	11	62	98	111	120	60	45	5
Recent Alluvium / Q1a	44	400	933	1822	2711	3244	3600	1822	1778	2
TAURANGA GROUP										
Undifferentiated Tauranga Group	3	3	6	127	248	775	1300	302	459	6
Tauranga Estuarine	380	380	380	380	380	380	380	380	0	1
Tauranga Puketoka	4.6	10.7	21.3	70.0	275	832	4700	398	897	58
Kaawa Formation	1000	1110	1275	1550	1825	1990	2100	1550	550	2
AUCKLAND VOLCANIC FIELD (PLEISTOCENE)										
Basalt	-	-	-	-	-	-	-	-	-	0
Basalt (Waterview)	-	-	-	-	-	-	-	-	-	0
Tuff / Ash/ Scoria	110	110	110	110	110	110	110	110	0.0	1
Tuff / Ash/ Scoria (Waterview)	-	-	-	-	-	-	-	-	-	0
WAITEMATA GROUP (MIOCENE)										
Residual to Highly Weathered Residual Soils - Cohesive Soils	6.3	22.2	28.0	520	760	1550	1800	578	590	13
Residual to Highly Weathered Residual Soils - Granular Soils	340	340	340	340	340	340	340	340	0.0	1
Parnell Volcaniclastic Conglomerate	100	119	147	195	242	271	290	195	95.0	2
Moderately Weathered to Unweathered ECBF	200	207	247	290	697	1830	2600	722	817	8

8.4 Rock

8.4.1 Unconfined compressive strength

Uniaxial Compressive Strength (UCS) tests were carried out on selected rock core samples in accordance with test method NZS 4402:1986 Test 6.3.1 (Coffey) and ISRM's specification Part II:1979:9.1 (Rocklab). Results from the Waterview project were based on NZS 4402:1986, ASTM D2938:95, ISRM 1978 and ISRM modified carried out by various laboratories.

The distribution of UCS test results for each Geotechnical Unit is shown on the UCS versus depth plot provided in Appendix L.

In addition to the laboratory controlled UCS tests, rock strength was estimated in the field in accordance with the New Zealand Geotechnical Society Inc. "Field Description of Soil and Rock" (see Table 8-6).

The characteristic UCS design parameters are specified from testing data from Central Interceptor, similar local projects (Waterview) and best engineering practise and experience. Upper bound UCS parameters should be considered when assessing excavatability and plant performance.

UCS results for ECBF cohesive soil are indicative of the upper strength values for that material.

Table 8-6 : Rock Strength (NZGS, 2005)

Term	UCS Range (MPa)	Point Load Strength, $I_{S(50)}$ (MPa)
Extremely Weak (EW)	< 1	< 1
Very Weak (VW)	1 – 5	< 1
Weak (W)	5 – 20	< 1
Moderately Strong (MS)	20 – 50	1 – 2
Strong (S)	50 – 100	2 – 5
Very Strong (VS)	100 – 250	5 – 10
Extremely Strong (ES)	> 250	> 10

8.4.2 Young's modulus

Modulus of elasticity for intact rock samples was measured in laboratory during UCS testing. Modulus is often correlated with UCS and values of modulus ratio (MR) have been reported in Appendix M. Where available, data from Pressuremeter Tests where incorporated.

8.4.3 Tensile strength

The tensile strength of rock is measured indirectly by conducting the Brazilian test on selected rock core samples. Brazilian tests were undertaken in accordance with test method ISRM Part II:1978:12.2 (Rocklab).

The distribution of tensile strength with depth is presented in Appendix O.

Upper bound tensile strength parameters should be considered when assessing excavatability and plant performance.

8.4.4 Poisson's ratio

Poisson's ratio measures the ratio of lateral strain to axial strain in the linearly-elastic zone. Static Poisson's ratio was recorded during UCS testing and inferred from correlations to Full Waveform Sonic wireline logging. Testing for Poisson's ratio in soil was not undertaken, so typical values were specified for soil units (Look, 2007).

8.4.5 Hoek Brown parameters

The generalized Hoek-Brown failure criterion (Hoek, et al 2002) estimates failure stresses for rock mass in terms of material constants m, s and a. The criterion is expressed as:

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_b \frac{\sigma_3'}{\sigma_{ci}} + s\right)^a$$

Where m_b, s and a are given by

$$m_b = m_i \exp\left(\frac{GSI - 100}{28 - 14D}\right)$$
$$s = \exp\left(\frac{GSI - 100}{9 - 3D}\right)$$
$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right)$$

The input parameters required to determine Hoek-Brown failure criterion are:

- Geological Strength Index, GSI, dependent on rock mass structure and surface conditions of defects;
- Material constant, m_i, dependent on material type;
- Intact uniaxial compressive strength, σ_{ci}; and
- Disturbance factor, D, dependent on construction method, quality control material type, structure, and distance from the excavation face.

It is recommended to adopt lower-bound design input parameters for GSI, m_i and σ_{ci}, and make a conservative assessment of D.

Upper bound parameters should be considered when assessing excavatability and plant performance.

8.4.6 Rock mass modulus

The rock mass modulus, E, is estimated using Hoek and Diederichs (2006) simplified equation

$$E_{rm} = 100{,}000\left(\frac{1 - D/2}{1 + e^{\frac{75+25D-GSI}{11}}}\right)$$

and generalised equation

$$E_{rm} = E_i\left(0.02 + \frac{1 - D/2}{1 + e^{\frac{60+15D-GSI}{11}}}\right)$$

The lesser of the two was used and is presented in Table 9-1 together with Pressuremeter tests where available.

8.4.7 Horizontal to vertical stress ratio

8.4.7.1 ECBF and Parnell Volcanoclastic Conglomerate

Assessment of in-situ stress ratio for rock units comprising ECBF and Parnell Grit have been based on 14 hydraulic fracture tests conducted during investigations for Central Interceptor and for the Rosedale Outfall Tunnel. Stress has been resolved into a vertical (Sig V), major horizontal (Sig H) and minor horizontal (Sig h).

The Central Interceptor tests were undertaken in the bottom of six boreholes (BH206, 219, 252, 253 259 and 271) at nominal tunnel horizon. The test intervals were logged with a televiewer then sealed using a single packer and pressurised to induce new fractures followed by cycles of pressurisation to enable determination of shut-in pressure; a televiewer was lowered into the hole after testing to record induced fractures. Of the six tests three are considered reliable (BH206, BH219 and BH259), two are doubtful (BH253 and BH271) and one is rejected as a failed test (BH252).

Hydraulic fracturing was undertaken for the Rosedale Outfall project in 2008. A total of 18 tests were undertaken in four boreholes of which 11 tests are considered reliable and have been used in the assessment.

The results are plotted as stress vs. depth (Figure 8.1) and stress ratio vs. depth (Figure 8.2). The data indicates a stress ratio of 0.8 – 2.1 for Sig h / Sig V and 0.7 – 3.1 for Sig H / Sig V. The major horizontal stress direction observed in the Rosedale data is towards the southeast and "corresponds well with the dominant strike of the NNW-SSE fault system." A similar orientation is evident in the limited CI test results.

For the purposes of design, a range of K = 0.8 – 1.5 is considered representative with an expected value of 1.2 however it is recommended that tunnel lining is checked for a stress ratio in the range 0.5 – 2.0. The stress directions should be checked for the most and least favourable potential orientation in relation to vertical shafts and horizontal tunnels.

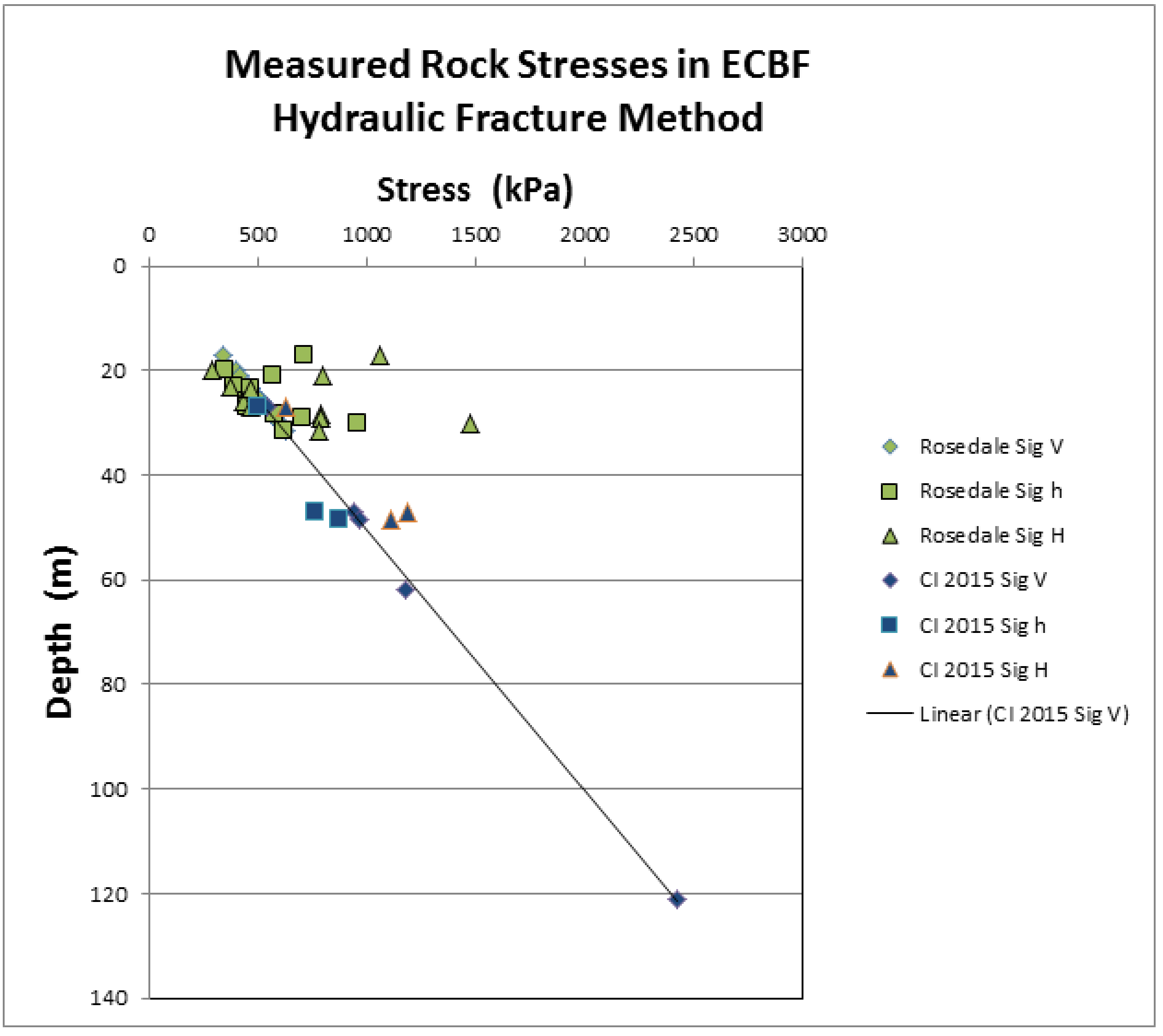


Figure 8.1: Measured Rock Stresses in ECBF (Hydraulic Fracture Method)

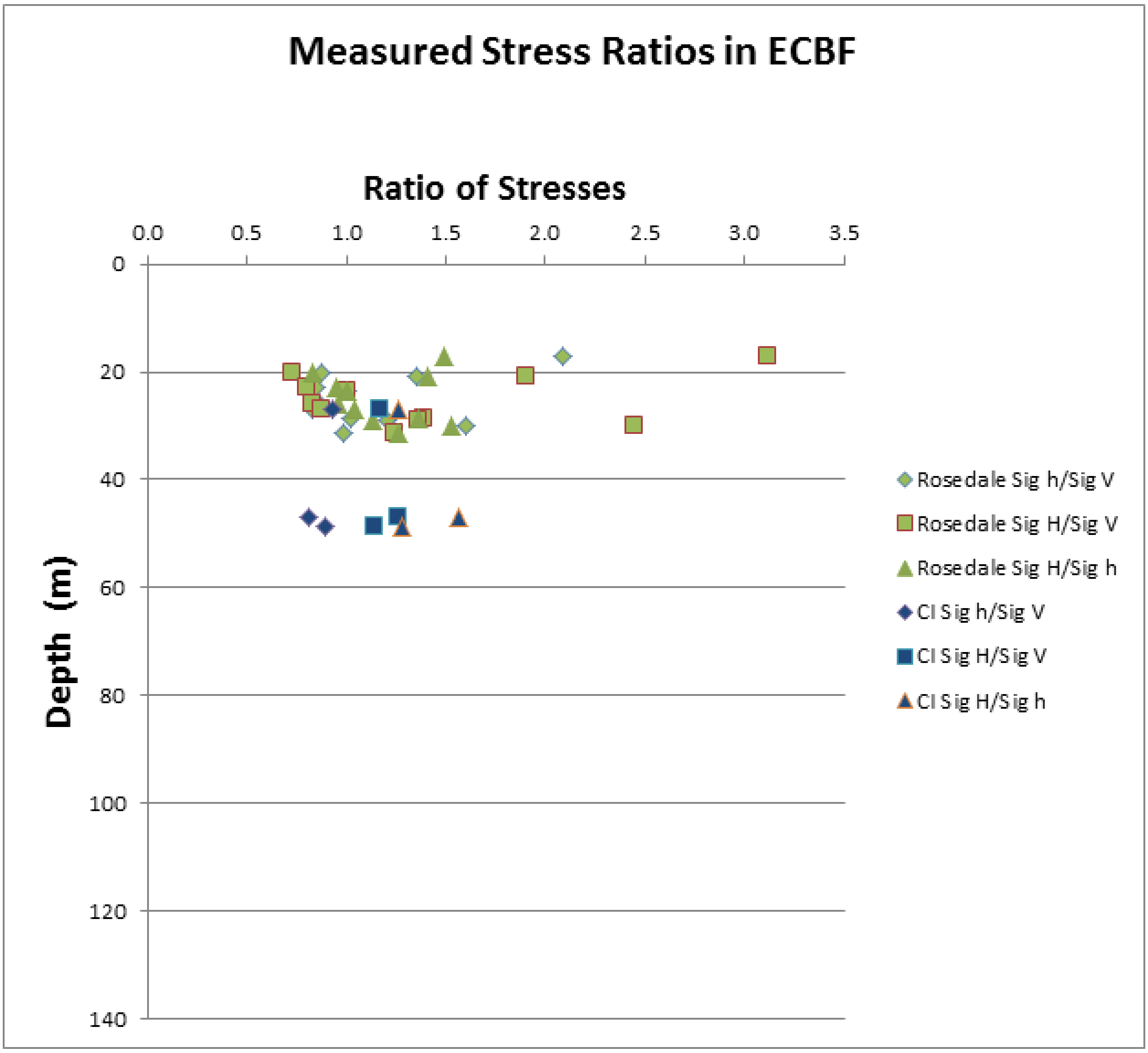


Figure 8.2: Measured Stress Ratios in ECBF (Hydraulic Fracture Method)

8.4.7.2 Basalt

No stress measurements have been undertaken in the rock units of the Auckland Volcanic Field. It is recommended that a range of 0.1 – 1.5 be adopted. Columnar jointed rock is likely to have a low horizontal in situ stress however pressure on a structure needs to consider toppling and sliding of wedges. Rubbly material is expected to behave as a granular material and therefore should be assessed empirically using Jaky. A minimum post-excavation lateral stress of $\sigma h = 0.1\sigma v$ should be assumed for all excavated surfaces.

8.4.8 Slake durability

Slake Durability test were carried out by Coffey and Rocklab following AS4133.3.4. Results presented in Appendix Q represent the Second Cycle slake durability index.

8.4.9 CERCHAR abrasivity

CERCHAR abrasivity was determined by BRTS and presented in Appendix T.

8.4.10 Shore hardness

Hardness was determined by BRTS using a SKLEROGRAF Model D. The instrument was then used to convert SKL D values to the equivalent values of Shore Hardness. Analysis is presented in Appendix T.

8.4.11 Soil abrasion test

Soil abrasion tests were undertaken by BRTS and results are presented in the Geotechnical Factual Report.

8.5 Soil

8.5.1 Atterberg limits and particle size distribution

Atterberg Limit values (Liquid Limit (LL), Plastic Limit (PL) and Plasticity Index (PI = LL – PL)), and Particle size distribution (PSD), were undertaken in accordance with the following test methods:

- LL (NZS 4402:1986 Test 2.2)
- PL (NZS 4402:1986 Test 2.3)
- PI (NZS 4402:1986 Test 2.4)

Results of the Atterberg Limit tests and PSD results are provided in Appendix K. Results for the Atterberg Limit Tests are presented project wide and for the following individual sites:

- E – Western Springs / Waterbores (WS1)
- M.2 – Norgrove (L2S2)
- N – Mt. Albert Community (AS1)
- S – Havestock P&F (AS3)
- V – Walmsley Park (AS4)
- Z.1 – PS25 (L3S1)
- AA.1 – Miranda Playground / Reserve (L3S2)
- AF.1 – May Road (WS2)
- AQ – Kiwi Esplanade (AS7)
- AU – Mangere WWTP (WS3)
- No Atterberg results are available for site L, Q, AB, AC, AE.1, AE.2, AI and AO

Atterberg Limit values provided in the Geotechnical Design Parameters table are based on:

- Lower bound value: 10^{th} percentile value;
- Upper bound value: 90^{th} percentile value ; and
- Characteristic value: not applicable for this parameter.

8.5.2 Undrained shear strength

The undrained shear strength was determined in-situ using hand held shear vanes and in the laboratory with unconsolidated undrained (UU) triaxial tests according to BS 1377-7:1990:8 (Geotechnics) and AS 1289.6.4.1 – 1998 (Coffey). Samples were consolidated from 13 kPa to 200 kPa prior to multistage testing (Coffey). Results are summaries in Table 9-1.

8.5.3 Mohr Coulomb strength parameters

The Mohr Coulomb strength parameters are used to describe the strength of the material to resist deformation due to shear stress. Geological materials resist shear stress by two internal mechanisms i.e., cohesion and internal friction. Cohesion is a measure of internal bonding of the rock material. Internal friction is caused by contact between particles, and is defined by the internal friction angle, φ.

Mohr Coulomb strength parameters for soil were taken from consolidated undrained (CU) triaxial test (BS 1377-8:1990:7 (Geotechnics) and AS 1289.6.4.1 (Coffey)) and direct shear tests (AS 1289.6.2.2, Coffey) using both undisturbed and compacted to supplied MWD (Heavy Compaction).

Rock mass parameters were estimated from the Hoek-Brown failure criterion, as discussed in Section 8.4.5. The Rocscience software RocLab version 1.033 was used to determine the rock mass effective cohesion and friction angle, considering the normal stress range in a depth range of 30 m to 80 m for tunnels (ECBF and PVC) and 15 m for slopes (Basalt).

The Hoek-Brown failure criterion inputs were as discussed in Section 8.4.5. A disturbance factor D of 0.0 for tunnels and 0.7 for slopes was adopted for the general assessment of parameters. A disturbance factor of 0.7 for slopes is considered conservative.

Upper bound Mohr-Coulomb strength parameters should be considered when assessing excavatability and plant performance.

8.5.4 Soil mass modulus

Soil mass modulus, E, was derived using soil consistency from published data (Look, 2007).

8.5.5 Horizontal to vertical stress ratio

Horizontal stress values of earth pressure at-rest (k_0) have been determined based on empirical relationships and effective friction angle. It is assumed that the horizontal stress is uniform in the horizontal plane.

9. Recommended geotechnical parameters

Measured and derived parameters have been expressed as a range of likely values, and a characteristic design value. Unless stated otherwise, the range of geotechnical design parameters are based on:

- Lower bound value: typically the 10th percentile value – estimated likelihood that actual property will be less than this value is approximately 10%.
- Upper bound: typically the 90th percentile value – estimated likelihood that actual property will be less than this value is approximately 90%.
- Characteristic design value: cautious estimate of the value affecting a critical state. For most parameters, this is typically taken as the 25th percentile value unless noted.

Designers should assess which value is most appropriate for specific cases. Upper bound intact and rock mass strength parameters should be considered when assessing excavatability and plant performance. Values have been calculated from a cumulative distribution frequency plot where sufficient data exists.

Project-wide parameters have been assigned to geological units and are presented in Table 9-1. Where there is local variation from the site wide parameters at specific locations these have been detailed in Section 11.

Table 9-1 : Recommended Geotechnical Parameters

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

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

10. Contamination

The purpose of this section of the report is to allow contractors to view the levels of soil contamination, to inform the future contractor with information to assess health and safety requirements for the protection of workers handling potentially contaminated soil as well as off-site disposal options.

A summary table showing the off-site spoil disposal options is located at the end of this section. Precautions should not detract from the soil management practices outlined in the CI Site Management Plan detailed below.

10.1 Scope

The scope of work fulfils the requirements of conditions 1.1 to 1.34 and 8.1 to 8.23 of Auckland Council (AC) resource consents R/LUC/2012/2846/1, PRC 40963 (NES for Assessing and Managing Contaminants in Soil to Protect Human Health) and 40843 (Contaminated sites).

Based on information presented in the following two reports below submitted as part of the resource consent application for the project, the resource consents require that fourteen of the nineteen shaft and construction sites be investigated prior to construction.

1. Desk Study and Ground Contamination Assessment - Main Works Central Interceptor Project, Tonkin & Taylor Limited (T&T), July 2012, referred to hereafter as the "Desk Study".

2. Central Interceptor - Project Contaminated Land Site Management Plan (CLSMP), T&T, December 2012 (Rev 1).

This report addresses all nineteen sites (i.e. not just the fourteen required by the resource consent). Refer to the table below for a summary of the sites. This assessment includes sites eliminated during the preliminary design process.

It is noted that intrusive investigations were carried out by T&T at four sites (Mangere WWTP, May Road, Western Springs and Motions Road) as part of the above desk top investigation. The results of the T&T investigations have not been repeated in this report.

This report describes for each of the nineteen sites:

1. A brief description of the proposed works.

2. A brief summary of the field observations recorded during the intrusive sampling investigation.

A tabulated summary of soil test results and comparison to appropriate guidelines for the protection of human health and the environment as well as guidance on landfill disposal options.

Table 10-1 : Summary of site details

Site Details			Auckland Background Soil [1]	
			Volcanic	Non-volcanic
Link Sewer 1	L1S1	Motions Road	✓	
	L1S2	Western Springs Depot		✓
Link Sewer 2	L2S1	Rawalpindi Reserve		✓
	L2S2	Norgrove Avenue		✓
Link Sewer 3	L3S1	Pump Station 25		✓
	L3S2	Miranda Reserve		✓
	L3S3	Whitney Street		✓
	L3S4	Dundale Avenue		✓
	L3S5	Haycock Avenue		✓
Main Tunnel	WS1	Western Springs		✓
	AS1	Mt Albert War Memorial		✓
	AS2	Lyon Ave	✓	
	AS3	Haverstock Road	✓	
	AS4	Walmsley Park	✓	
	WS2	May Road	✓	
	AS5	*Keith Hay Park	✓	
	AS6	Pump Station 23		✓
	AS7	Kiwi Esplanade	✓	
	WS3	Mangere Pumping Station	✓	

Note: [1] The Auckland background soils can be split into two groups, from a soil contamination perspective: volcanic soils and non-volcanic soils. Volcanic soils typically have higher concentrations of inorganic parameters/contaminants. This is important when assessing the nineteen site test results as discussed in Section 11 below. The Auckland regional geological map has been used to assess whether the nineteen sites are located in a volcanic or non-volcanic area.

10.2 Methodology

The assessment was undertaken in general accordance with published Ministry for the Environment (MfE) guidelines, which detail recommended methods and considerations when undertaking an assessment on potentially contaminated sites. These guidelines include:

- Contaminated Land Management Guideline No. 1 – Reporting on Contaminated Sites in New Zealand (Revised 2011).
- Contaminated Land Management Guideline No. 2 – Hierarchy and Application in New Zealand of Environmental Guideline Values (Revised 2011).
- Contaminated Land Management Guideline No. 5 – Site Investigation and Analysis of Soils (Revised 2011).
- Guidelines for Assessing and Managing Petroleum Hydrocarbon Contaminated Sites in New Zealand, (revised 2011)

The full details of the site investigation works are provided in factual report. This report only details the interpretation of the investigation results.

10.3 Soil acceptance criteria

Soil screening criteria for the assessment have been adopted in accordance with the hierarchy defined by MfE Contaminated Land Management Guidelines No.2. Acceptance criteria for a commercial / industrial land use scenario have been adopted for contaminants of potential concern (CoPC) related to the sites.

Table 10-2 : Soil Acceptance Criteria

Adopted Acceptance Criteria	Guideline/Regulation	
Soil	NES SCS	Resource Management Act (National Environmental Standard for Assessing and Managing Contaminants in Soil to Protect Human Health) Regulations, 2011. NES soil contaminant standards for a commercial land use scenario.
	Auckland Background Concentrations	Auckland Regional Council, 2001. Background Concentrations of Inorganic Elements in Soil from the Auckland Region. Background ranges for metals in volcanic and non-volcanic range soils based on the location of the site.
	Schedule 10 Permitted Activity Criteria	Auckland Council, 2012. Auckland Council Regional Plan: Air, Land, and Water.
	MfE Guidelines	Ministry for the Environment, 1999 (rev 2012). Guidelines for Assessing and Managing Petroleum Hydrocarbon Contaminated Sites in New Zealand. All pathways soil acceptance criteria for a commercial / industrial land use scenario.

10.4 Site assessments

Individual site assessments are incorporated in Section 11.

10.5 Summary of off-site spoil disposal options

Table 10-3 : Summary of Off-Site Disposal Options for the Nineteen Sites Investigated in this Report [1]

Site Details			Spoil Disposal Option [2, 3]		
			Cleanfill	Managed Fill	Solid Waste Landfill
Link Sewer 1	L1S1	Motions Road		✓	
	L1S2	Western Springs Depot [4]			
Link Sewer 2	L2S1	Rawalpindi Reserve	✓	✓	
	L2S2	Norgrove Avenue		✓	
Link Sewer 3	L3S1	Pump Station 25			
	L3S2	Miranda Reserve	✓		
	L3S3	Whitney Street		✓	
	L3S4	Dundale Avenue		✓	
	L3S5	Haycock Avenue	✓		
Main Tunnel	WS1	Western Springs		✓	
	AS1	Mt Albert War Memorial		✓	
	AS2	Lyon Ave [5]			✓
	AS3	Haverstock Road	✓	✓	
	AS4	Walmsley Park		✓	
	WS2	May Road	✓		
	AS5	*Keith Hay Park		✓	
	AS6	Pump Station 23		✓	
	AS7	Kiwi Esplanade [5] Ambury Regional Park			✓ ✓
	WS3	Mangere Pumping Station Mangere Waste Water Treatment Plan (Rising Main)	 ✓	 ✓	✓

Notes:

[1] It is noted that intrusive investigations were carried out by T&T at four sites (Mangere WWTP, May Road, Western Springs and Motions Road). The results of the T&T investigations have not been repeated in this report, however, they can be found in the references.

[2] For all disposal options: the contractor should contact the landfill operator prior to site works starting and check with the landfill operator that they can accept the material based on the test results presented in this report.

[3] Further inspections and/or testing should be undertaken by the contractor where significant volumes of fill material are encountered during the works.

[4] No samples were able to be collected at the site due to the gravelly nature of the near surface material and the equipment used (a geotechnical handauger). Further sampling and testing should be undertaken by the contractor prior to site work starting onsite.

[5] Soil-asbestos was detected. Further testing of soil is required to determine the concentration of asbestos fibres in soil prior to commencing works. The testing will also be required to confirm disposal requirements. Currently, the material will potentially require disposal to a licensed landfill facility authorised to accept Asbestos Containing Material (ACM).

11. Location specific details

For any locations where the project wide parameters are not appropriate, specific parameters are provided in this section.

11.1 DSCIN Main Tunnel

11.1.1 Site description

DSCIN Main Tunnel extends north from DPCIN Mangere Pumping Station (CH10000) to DSCIN009 – Western Springs (CH23067) and includes 10 shaft sites which are described in Sections 11.6 to 11.15.

11.1.2 Geotechnical ground model

The geotechnical ground model is presented in Main Tunnel Long Sections – Sheets 1 to 10 (DWG No. 20150610.016 – 20150610.25). A summary of the anticipated ground conditions along the main tunnel are outlined in Table 11-1. The references of chainages are approximate only.

The quantity of geotechnical units anticipated to be encountered along the mainline tunnel is presented in Table 11-2. Geotechnical units have been measured horizontally at tunnel crown off the interpreted geological sections presented in Appendix C. Parnell Volcaniclastic Conglomerate has only been identified on the geological sections when greater than approximately 500mm due to readability and this unit can occur anywhere within ECBF rock. For comparison with the interpreted sections and with Table 11-2, PVC as a percentage of ECBF rock measured from logged drillcore from all relevant investigations within the project area is presented in Table 11-3.

Table 11-1. Summary of main tunnel ground model

Chainage / location	
CH10000 to CH11300	The tunnel is anticipated to encounter Kaawa Formation soils and undifferentiated Tauranga Group soils.
CH11300 to CH12600	The tunnel transitions between Undifferentiated Tauranga Group, Residually to Highly Weathered Waitemata Group soils and Moderately to Unweathered ECFB rock. It is anticipated mixed face conditions will be present for much of this section. Parnell Volcaniclastic Conglomerate is observed in BH263 in thickness greater than the tunnel diameter. The lateral extent of the conglomerate is unknown but was not observed in adjacent investigations.
CH12600 to DSCIN002 PS23	The first half of the marine section of the tunnel (up to BH302) is similar to that described above; Transitioning in and out of Residually to Highly Weathered Waitemata Group soils and Moderately to Unweathered ECFB rock. The tunnel is then anticipated to be solely in Moderately to Unweathered ECFB rock.
DSCIN002 PS23 to DSCIN007 Lyon Avenue	The tunnel is expected to be driven through in Moderately to Unweathered ECFB rock.
DSCIN007 to DSCIN008 Mt Albert Memorial Reserve	Undifferentiated Tauranga Group is observed in BH223 to -9mRL and within approximately 2m of the tunnel. It is possible that the paleo-surface of the ECBF is incised and this unit may extend into the tunnel near this location.

Chainage / location	
DSCIN008 Mt Albert Memorial Reserve to DSCIN009 Western Springs	Generally the tunnel is in Moderately to Unweathered ECFB rock. Through Chamberlain Park, from CH22000 to CH22400, the tunnel will transition partly and possibly completely into Residually to Highly Weathered Waitemata Group and Undifferentiated Tauranga Group. Also through this area, basalt was observed () at around crown level in BH510, BH511 and could be encountered elsewhere over this interval. The tunnel may transition partly into Undifferentiated Tauranga Group between BH208 and DSCIN009 Western Springs. Basalt is observed in drill holes within 5m of the tunnel horizon and extensive drilling within this area has been undertaken to identify deep basalt and it is possible localised zones of basalt may occur at tunnel horizon between CH22000 to 22400.

Table 11-2. Quantity of each geotechnical unit anticipated along mainline tunnel crown measured horizontally off geological sections

Geotechnical Units		Length anticipated measured horizontally off sections at crown (m)	Length anticipated as % of total
Made Ground	Engineered Fill	0	0
	Non-Engineered Fill	0	0
Recent Alluvium		0	0
Undifferentiated Tauranga Group	Cohesive	1076	8
	Granular		
Kaawa Formation		615	5
Tuff/Ash/Scoria		0	0
Basalt		10	0.1
Residually to highly weathered cohesive and granular soils		1224	9
Moderately weathered to unweathered ECBF		9932	76
Parnell Volcaniclastic Conglomerate		210	2
Total		**13067**	**100**

Table 11-3. Drillcore logged as PVC as a percentage of East Coast Bays Formation rock (MW-UW ECBF + PVC)

Geotechnical Units	Geotechnical unit length logged from Drillcore (m)	Length anticipated as % of total
Moderately weathered to unweathered ECBF	5936	98
Parnell Volcaniclastic Conglomerate	121	2
Total	**6037**	**100**

11.1.3 Groundwater model

The piezometric surface at tunnel level is above the tunnel for the entire alignment and is presented in Main tunnel Long Sections – Sheets 1 to 10. The groundwater model is summarised below. The references of chainages are approximate only.

- CH10000 to CH11300: The tunnel is anticipated to encounter aquifers within the Kaawa Formation and Tauranga Group. The Kaawa aquifer is a regionally significant aquifer and higher groundwater flows are expected. Saturated and potentially soft/loose soils are likely to be encountered through this section.
- CH11300 to DSCIN002 PS23: The piezometric surface remains at a relatively consistent level however variability in groundwater flows may occur as the tunnel will encounter a range of full face and mixed face conditions comprising Tauranga Group, Residual soil ECBF and moderately to unweathered ECBF.
- DSCIN002 PS23 to DSCIN007 Lyon Avenue: Pressure head is likely to increase from PS23 northward as the piezometric surface follows the higher elevation of the Auckland Isthmus. The tunnel at this section is expected to be fully in ECBF.
- DPCIN007 Lyon Avenue to DSCIN009: A paleo-channel comprising Tauranga Group and Basalt may be encountered by the tunnel resulting in variable groundwater conditions. Jointing and vesicles in basalt can provide high groundwater flows.

11.1.4 Geotechnical risks

Table 11-4. Geotechnical risks

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
Basalt within tunnel horizon	Basalt may be encountered as a mixed face or full face condition	Excavation requires change to construction methodology Reduced productivity and higher machine tool wear Deflection of TBM resulting in alignment or grade issues High groundwater inflows to be managed during construction	Basalt	DSCIN008 Mt Albert Memorial Reserve to DSCIN009 Western Springs
Parnell Volcaniclastic Conglomerate	Conglomerate may be encountered as a mixed face or full face condition	Excavation requires change to construction methodology Reduced productivity and higher machine tool wear Deflection of TBM resulting in alignment or grade issues	Parnell Volcaniclastic Conglomerate, Moderately to Unweathered ECBF	Anywhere within Moderately to Unweathered ECBF Around BH263
Variable and undulating contact between soil and rock units	Mixed face conditions comprising soil and rock units	Face loss/pressure loss Tunnel face instability Ground surface movement Deflection of TBM resulting in alignment or grade issues	All	CH10000 to CH11300 CH11300 to CH12600 CH12600to DSCIN002 PS23 DSCIN007 to DSCIN008 Mt Albert Memorial Reserve DSCIN008 Mt Albert Memorial Reserve to DSCIN009 Western Springs
High groundwater pressures	Groundwater pressure at tunnel level may be significant	High pressures to be accommodated/management during construction Change in pressure could be sudden if PVC encountered Designed for in tunnel lining	Parnell Volcaniclastic Conglomerate, Moderately to Unweathered ECBF	CH12600 to DSCIN004 May Road

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
Soft / Loose soil and water bearing sediments	Soft or loose saturated soil is likely to be encountered	Difficult conditions for starting the TBM Tunnel face instability	Tauranga Group, Kaawa Formation	CH10000 to CH11300
Wood fragments and logs	Wood fragments and logs may be encountered	Tunnel face instability Jamming of TBM and need for intervention to remove blockage Deflection of TBM resulting in alignment or grade issues	Tauranga Group	CH10000 to CH11300

11.2 DSLSA Drainage Sewer Link Sewer A

Downgraded to combined storm/sewer overflow and connected into Link Sewer B

11.3 DSLSB Drainage Sewer Link Sewer B

11.3.1 Site description

Link Sewer B joins the main tunnel near DSCIN008 Mt Albert and extends north and then west to shaft DSLSB002 Rawalpindi Reserve and is approximately 1.2km long. The sewer includes two shaft sites which are discussed in Section 11.18 and Section 11.19.

11.3.2 Geotechnical ground model

The geotechnical ground model is presented in Link Sewer B Long Section (DWG No. 2011952.001). An anticipated ground conditions along the sewer are expected to be generally in moderately weathered to unweathered ECBF. At approximate chainages CH300 to CH450 Undifferentiated Tauranga Group could extend near the tunnel crown or below it producing mixed face conditions.

The quantity of geotechnical units anticipated to be encountered along the Link Sewer B is presented in Table 11-5. Geotechnical units have been measured horizontally at tunnel crown off the interpreted geological sections presented in Appendix C. As discussed in section 3.3, Parnell Volcaniclastic Conglomerate can occur within ECBF rock as lenses and beds and while not shown on the geological sections this unit should be expected to be encountered.

Table 11-5. Quantity of each geotechnical unit anticipated along Link Sewer B crown measured horizontally off geological sections

Geotechnical Units		Length anticipated measured horizontally off sections at crown (m)	Length anticipated as % of total
Made Ground	Engineered Fill	0	0
	Non-Engineered Fill	0	0
Recent Alluvium		0	0
Undifferentiated Tauranga Group	Cohesive	55	5
	Granular		
Kaawa Formation		0	0
Tuff/Ash/Scoria		0	0
Basalt		0	0
Residually to highly weathered cohesive and granular soils		0	0
Moderately weathered to unweathered ECBF		1078	95
Parnell Volcaniclastic Conglomerate		0	0
Total		**1133**	**100**

11.3.3 Groundwater model

The piezometric surface at tunnel level is expected to decrease away from the main tunnel and is presented on the long section. The potential for mixed face conditions with Tauranga Group described in Section 11.3.2 could produce variable groundwater flows.

11.3.4 Geotechnical risks

Table 11-6 Geotechnical risks

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
Parnell Volcaniclastic Conglomerate	Conglomerate may be encountered as a mixed face or full face condition	Excavation requires change to construction methodology Reduced productivity and higher machine tool wear Deflection of TBM resulting in alignment or grade issues	Parnell Volcaniclastic Conglomerate, Moderately to Unweathered ECBF	Anywhere within Moderately to Unweathered ECBF
Variable and undulating contact between soil and rock units	Mixed face conditions comprising soil and rock units	Face loss/pressure loss Tunnel face instability Ground surface movement Deflection of TBM resulting in alignment or grade issues	All	CH300 to CH450
Wood fragments and logs	Wood fragments and logs may be encountered	Tunnel face instability Jamming of TBM and need for intervention to remove blockage Deflection of TBM resulting in alignment or grade issues	Tauranga Group	CH300 to CH450

11.4 DSLSC Drainage Sewer Link Sewer C

11.4.1 Site description

DSLSC Drainage Sewer Link Sewer C (Link Sewer C) joins DSCIN Main Tunnel near DSCIN004 - May Road and extends westwards for approximately 3300m length to shaft DSLSC005 – PS25. The sewer includes five shafts which are discussed in Sections 11.20 to 11.24.

11.4.2 Geotechnical ground model

The geotechnical ground model is presented in Link Sewer C Long Sections – Sheets 1 to 3 (DWG No. 2012004.001, 2012004.002 & 2012004.003). A summary of the anticipated ground conditions along the sewer are outlined below.

- From DSCIN004 – May Road to CH3000 the tunnel will pass through moderately weathered to unweathered East Coast Bays Formation comprising interbedded mudstone and sandstone. Investigations along the sewer have encountered Parnell Volcaniclastic Conglomerate up to 1m thick. As discussed in Section 3.3, volcaniclastic conglomerate will be encountered along the tunnel in localised areas.
- From CH3000 to DSLC005 – PS25 residually to highly weathered Waitemata group soils may be encountered as full face conditions, or mixed face with the residually to highly weathered soils underlying Tauranga group soils or overlying ECBF rock.

The quantity of geotechnical units anticipated to be encountered along the Link Sewer C is presented in Table 11-7. Geotechnical units have been measured horizontally at tunnel crown off the interpreted geological sections presented in Appendix C. As discussed in Section 3.3, Parnell Volcaniclastic Conglomerate can occur within ECBF rock as lenses and beds and while not shown on the geological sections this unit should be expected to be encountered.

Table 11-7. Quantity of each geotechnical unit anticipated along Link Sewer C crown measured horizontally off geological sections

<table>
<tr><th colspan="2">Geotechnical Units</th><th>Length anticipated measured horizontally off sections at crown (m)</th><th>Length anticipated as % of total</th></tr>
<tr><td rowspan="2">Made Ground</td><td>Engineered Fill</td><td>0</td><td>0</td></tr>
<tr><td>Non-Engineered Fill</td><td>0</td><td>0</td></tr>
<tr><td colspan="2">Recent Alluvium</td><td>0</td><td>0</td></tr>
<tr><td rowspan="2">Undifferentiated Tauranga Group</td><td>Cohesive</td><td rowspan="2">70</td><td rowspan="2">2</td></tr>
<tr><td>Granular</td></tr>
<tr><td colspan="2">Kaawa Formation</td><td>0</td><td>0</td></tr>
<tr><td colspan="2">Tuff/Ash/Scoria</td><td>0</td><td>0</td></tr>
<tr><td colspan="2">Basalt</td><td>0</td><td>0</td></tr>
<tr><td colspan="2">Residually to highly weathered cohesive and granular soils</td><td>150</td><td>5</td></tr>
<tr><td colspan="2">Moderately weathered to unweathered ECBF</td><td>3095</td><td>93</td></tr>
<tr><td colspan="2">Parnell Volcaniclastic Conglomerate</td><td>0</td><td>0</td></tr>
<tr><td colspan="2">Total</td><td>3315</td><td>100</td></tr>
</table>

11.4.3 Groundwater model

Link Sewer C is anticipated to be below groundwater for the entire length of the tunnel. Groundwater monitoring results, interpreted piezometric surface and installed piezometers on the Link Sewer C Long Sections – Sheets 1 to 3.

Groundwater appears to be unconfined and typically within 5m depth of ground level. It is anticipated between CH200 and CH500 piezometric head is greatest along the sewer at approximately 50 m from tunnel invert. This high head may cause issues for some tunnelling methodologies. Under normal conditions the flow from the ECBF would be expected to be relatively low as a result of low transmissivity despite the high head. However, there is a risk of defect controlled flow in localised zones with higher transmissivity.

11.4.4 Geotechnical risks

Table 11-8 Geotechnical risks

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
Parnell Volcaniclastic Conglomerate	Conglomerate may be encountered as a mixed face or full face condition	Excavation requires change to construction methodology Reduced productivity and higher machine tool wear Deflection of TBM resulting in alignment or grade issues	Parnell Volcaniclastic Conglomerate, Moderately to Unweathered ECBF	Anywhere within Moderately to Unweathered ECBF From DSCIN004 – May Road to CH3000
Variable and undulating contact between soil and rock units	Mixed face conditions comprising soil and rock units	Face loss/pressure loss Tunnel face instability Ground surface movement Deflection of TBM resulting in alignment or grade issues	All	CH3000 to DSLC005 – PS25
High groundwater head	The groundwater head is high for this diameter of tunnel; high pressures to be anticipated.	Tunnel inundation Higher than acceptable seepage into tunnel	ECBF	Richardson Road ridge
Wood fragments and logs	Wood fragments and logs may be encountered	Tunnel face instability Jamming of TBM and need for intervention to remove blockage Deflection of TBM resulting in alignment or grade issues	Tauranga Group	CH3000 to DSLC005 – PS25

11.5 DSLSD Drainage Sewer Link Sewer D

11.5.1 Site description

DSLSD Drainage Sewer Link Sewer D (Link Sewer D) has been eliminated from the scheme during preliminary design. It had been planned to connect with DSCIN Main Tunnel near DSCIN001 – Kiwi Esplanade and extend approximately 690m in length, cutting southeast across a reserve to the top of Yorkton Rise then follow the road corridor to Witla Court, terminating 40m to the east.

11.5.2 Geotechnical ground model

A thin layer of made ground overlies basalt rock along the sewer alignment. The made ground is generally <1m thick comprising clays, silts and gravels. Basalt is described in drill hole CI-19 to -13mRL and is vesicular and generally intact with zones of very closely spaced joints. Underlying basalt is Tauranga Group sand and clayey silt to -24.5mRL where moderately to unweathered ECBF sandstone and mudstone rock is encountered.

11.5.3 Groundwater model

Groundwater is likely to be approximately 1.5m below ground level based on piezometers installed in Tauranga group soils and ECBF rock in CI-19. Nested piezometers nearby which measure Tauranga ground, ECBF and overlying basalt (BH261, BH262) have similar readings suggesting the basalt is hydrologically connected with the underlying units.

11.5.4 Geotechnical risks

Table 11-9. Geotechnical risks

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
Basalt within tunnel horizon	Basalt may be encountered as a mixed face or full face condition	Excavation requires change to construction methodology Reduced productivity and higher machine tool wear Deflection of TBM resulting in alignment or grade issues	Basalt	Whole Alignment
Variable and undulating contact between soil and rock units	Depth of fill above basalt and the thickness of basalt may vary	Excavation requires change to construction methodology Reduced productivity and higher machine tool wear Tunnel Instability	Basalt, Tauranga group, Fill	Whole Alignment
Basalt rock mass variability	Basalt may vary between intact basalt rock to rubbly gravel	Tunnel instability Excavation requires change to construction methodology Trenching may require additional support in rubbly gravel	Basalt	Whole Alignment
Groundwater inflow	Basalt is typically vesicular and jointing allowing high water inflows	Disruption to excavation work and may require pumping	Basalt	Whole Alignment

Investigations along Link Sewer D are typically shallow terminating at the top of basalt. There is one drill hole, CI-19 which has extended below the basalt and there are no investigations along the sewer alignment past CH380. There is a risk both the depth of fill above basalt and the thickness of basalt may vary which may impact constructability.

The quality of basalt may vary along Link Sewer D between intact basalt rock and rubbly gravel. CI-19 and nearby investigations (BH261, CI-05, CI-17) generally encountered intact vesicular basalt with some jointing however CI-05 encountered zones of highly fractured rubbly basalt typically 100mm thick but up to 0.5m thick.

11.6 DPCIN Mangere Pumping Station

11.6.1 Site description

The Mangere Pumping Station site is separated into four areas based on the structure type and ground conditions; Shaft, Pumping Station and Switchroom; Valve Chamber; Rising Mains to Confluence Chamber; Emergency Pressure Relief Chamber.

11.6.2 Geotechnical ground model

The geotechnical ground model is presented in geotechnical drawings DPCIN – Mangere Pumping Station Shaft Geology (DWG No. 2012043.001, 2012043.002, 2012043.003).

The geotechnical ground model for the shaft, pumping station and switch room is presented in Table 11-10 below.

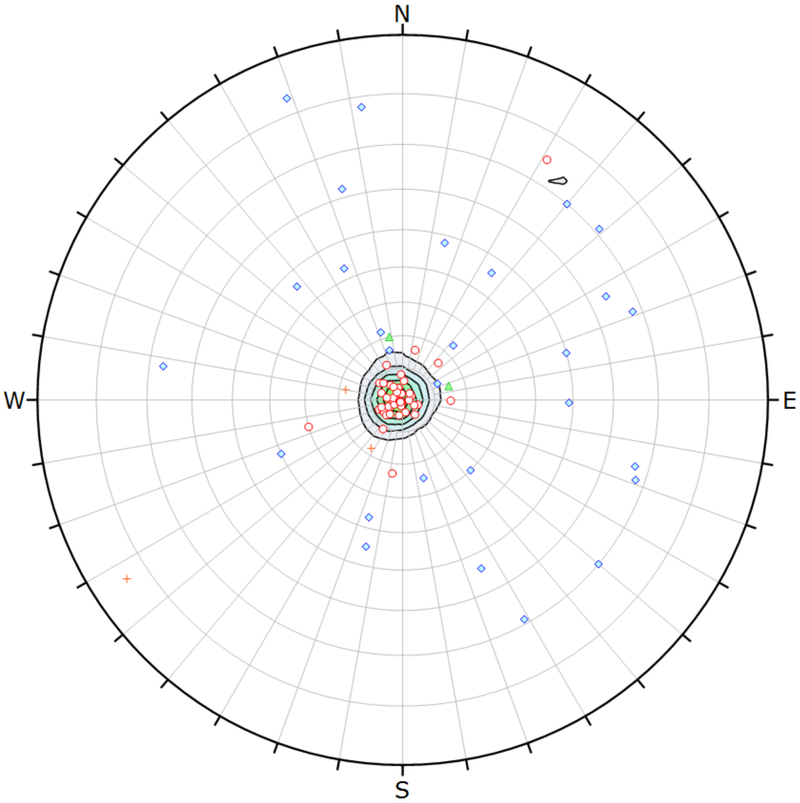
Table 11-10 Geotechnical ground model for DPCIN Mangere Pumping Station Shaft

From (mRL)	To (mRL)	Geotechnical Unit
Surface (3.5mRL)	3	Made Ground
3	-4	Tuff/Scoria
-4	-10 to -15	Undifferentiated Tauranga Group
-10 to -15	-34 to -36	Kaawa Formation
-34 to -36	-47	Parnell Volcaniclastic Conglomerate
-47	Not Determined	Moderately Weathered to Unweathered ECBF

The shaft excavation will encounter saturated and potentially soft / loose soil of the undifferentiated Tauranga Group and Kaawa Formation. Parnell Volcanoclastic Conglomerate has been observed underlying the Kaawa Formation and can have persistent vertical joints, Figure 11.1.

The ground conditions at the valve chamber are likely to include 2-3m of fill overlying Tuff/Scoria and Undifferentiated Tauranga Group. Tuff/scoria is present to -2mRL and may not extend across the entire site. Undifferentiated Tauranga Group is anticipated to extend to -10mRL depth. Undifferentiated Tauranga Group is expected to range from soft to firm cohesive soil and loose to medium dense granular soil. Design of the valve chamber should consider both material types to accommodate expected variability.

Discontinuities for BH271 show bedding is typically sub-horizontal to gently inclined (0°-10°) dipping in a range of directions, Figure 11.1. Jointing is steeply inclined and should be expected to have variable dip direction.



Symbol	TYPE	Quantity
○	Bedding Plane/Fracture	47
△	Crush/Shear	2
◇	Joint	28
+	Unidentified	3

Color	Density Concentrations
	0.00 - 5.00
	5.00 - 10.00
	10.00 - 15.00
	15.00 - 20.00
	20.00 - 25.00
	25.00 - 30.00
	30.00 - 35.00
	35.00 - 40.00
	40.00 - 45.00
	45.00 <
Maximum Density	29.59%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

Plot Mode	Pole Vectors
Vector Count	80 (80 Entries)
Terzaghi Weighting	Minimum Bias Angle 15°
Hemisphere	Lower
Projection	Equal Angle

Figure 11.1 Stereonet plot of discontinuities observed in BH271

The two rising mains leading to confluence chamber will likely pass through Undifferentiated Tauranga Group and Made Ground. The made ground could be of variable thickness, is observed in trial pits to 3m bgl and associated with works from the waste water treatment plant.

The emergency pressure relief chamber is anticipated to cross Tuff/Scoria into variable amounts of Made Ground overlying Undifferentiated Tauranga Group. Made Ground could be deeper in localised areas associated with works from the waste water treatment plant, Figure 11.2. Where the Tuff/scoria is present it's lower boundary is approximately -3mRL. Undifferentiated Tauranga Group is anticipated to extend to at least -8mRL.

Figure 11.2 Aerial Photograph of Mangere WWTP circa 2001. Ponds appear to be being filled where the proposed emergency pressure relief chamber and rising mains will pass.

11.6.3 Groundwater model

Based piezometers installed in BH271, BH272 and BH273 piezometric surface at tunnel level is approximately 1.5mRL. Groundwater is expected to be encountered near the ground surface at approximately 3mRL. Fluctuations up to 100mm are observed in BH271 and are comparable with tidal cycles however other hydrological factors appear to dominate results, Figure 11.3.

Discussion of the aquifer studies is presented in Section 7.4.2.3 and further detail in Appendix F.

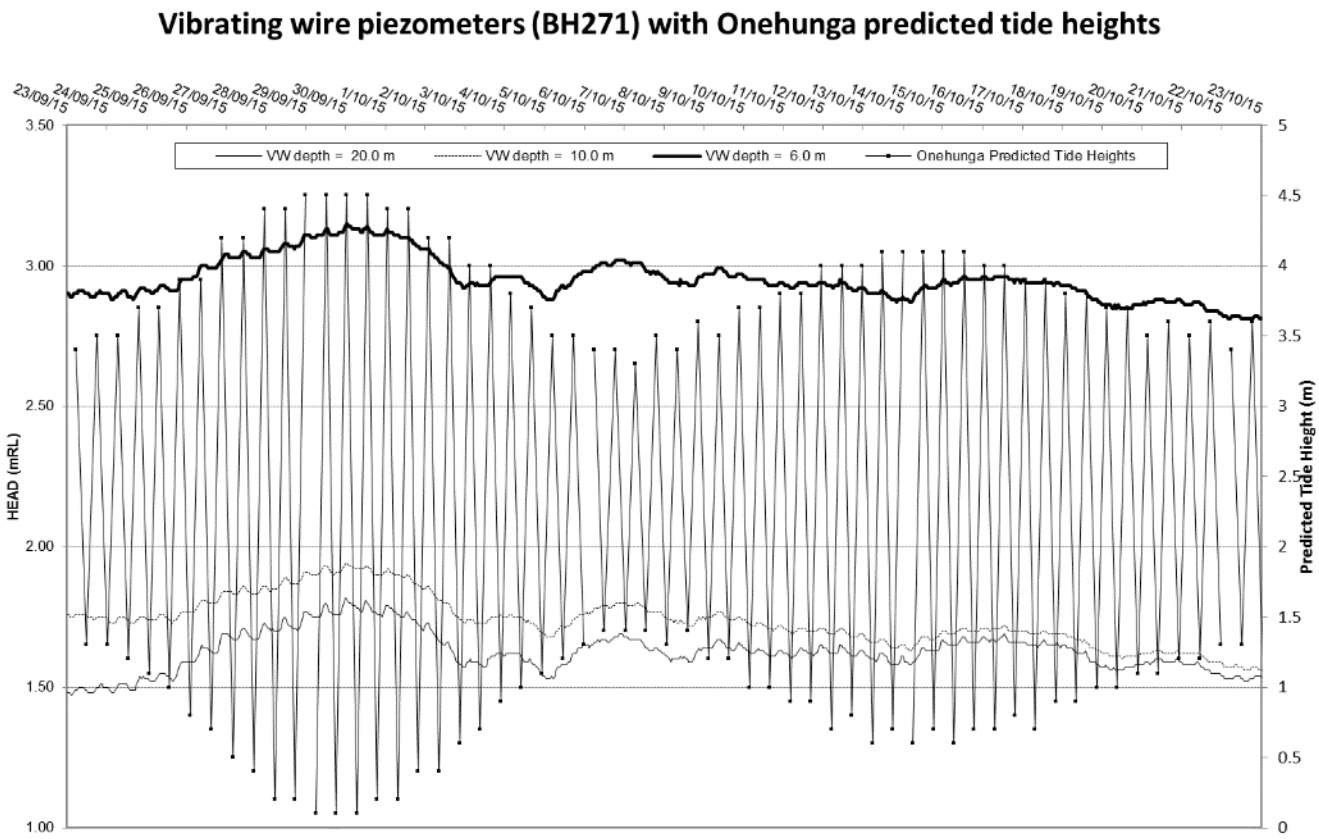


Figure 11.3 Vibrating wire piezometers in BH271 are compared with predicted tide heights for Onehunga. While fluctuations up to 100mm are observed it appears the overall groundwater trends are dominated by other factors.

11.6.4 Contaminated land assessment (pump station)

Previous desk top investigations indicated the site has been subject to historic filling on the construction site could comprise construction fill. CoPC for the site include heavy metals.

The following observations were noted during the intrusive sampling investigation:

- Fill material was observed from surface to 2 m bgl.
- No dumping or fly tipping as observed.
- PID readings in soil ranged from 0.0 to 0.6 ppm.
- Groundwater was encountered in TP04 at a depth 1.9 m.
- Asbestos Containing Material (ACM) was not observed.

The following table presents a summary of a comparison of soil sampling results to the adopted acceptance criteria. A full results table can be viewed as appended.

Table 11-11 : Comparison of soil sampling results to the adopted acceptance criteria (Mangere pump station)

Contaminant of Potential Concern	Assessment Criteria		
	Schedule 10 Permitted Activity Criteria	Auckland Background Concentrations (volcanic)	NES SCS
Heavy Metals	Exceedances detected for chromium, copper, nickel, and zinc.	Exceedances detected for arsenic cadmium, chromium, copper, mercury, nickel, and zinc.	All below or NC.

NC = No Criteria.

- A comparison to the adopted acceptance criteria indicates that there are no exceedances of NES SCS for protection of human health.
- Further inspection and/or testing should be undertaken at this site prior to works commencement.
- The average concentrations of cadmium, chromium, lead and mercury were above the Auckland Background Concentrations.
- The average concentrations of copper and nickel were above the PA discharge criteria.
- Further inspection and/or testing should be undertaken where significant volumes of fill material are encountered.

Soil materials disposed from this site would likely be suitable for disposal at a licensed landfill facility.

11.6.5 Contaminated land assessment (rising main)

The Mangere Waste Water Treatment Plant (WWTP) Raising Main forms part of the main WWTP and the same contamination profile as detailed for the WWTP.

Previous desk top investigations indicated the site has been subject to historic filling on the construction site could comprise construction fill. CoPC for the site include heavy metals.

The following observations were noted during the intrusive sampling investigation:

- Fill material was observed from surface to depths up to 3.6 m bgl.
- No evidence of dumping of fly tipping was observed.
- PID readings in soil samples did not exceed 0.0 ppm.
- Groundwater was not encountered during sampling.
- ACM was not observed.

The following table presents a summary of a comparison of soil sampling results to the adopted acceptance criteria. A full results table can be viewed as appended.

Table 11-12 : Comparison of soil sampling results to the adopted acceptance criteria (Mangere rising main)

Contaminant of Potential Concern	Assessment Criteria		
	Schedule 10 Permitted Activity Criteria	Auckland Background Concentrations (volcanic)	NES SCS
Heavy Metals	All below	Exceedance of arsenic detected	All below.

NC = No Criteria.

- A comparison to the adopted acceptance criteria indicates that there are no exceedances of NES SCS for protection of human health.
- A single sample yield concentration of arsenic above Auckland Background Concentrations. The average concentrations of arsenic are below the respective values.

While these concentrations of CoPc present no immediate risk to human or the environment, the soil is likely suitable for disposal at a cleanfill or managed fill facility. A comparison to landfill criteria should be undertaken at the time of disposal.

11.6.6 Geotechnical risks

Table 11-13 : Geotechnical risks (Mangere)

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
Variable of Ground Conditions	Unanticipated ground conditions due to wide spacing between investigation locations away from pump station	Different construction methodologies required during construction Design may not be appropriate for ground conditions	All	Emergency pressure relief chamber Rising mains leading to confluence chamber Valve chamber
Highly permeable ground	The Kaawa group is a known aquifer and is probably hydraulically connected to the adjacent bay	Risk of high groundwater inflow. Risk of liquefaction during earthquake.	Kaawa	Main pump station
Soft / Loose soil and water bearing	Soft or loose saturated soil is likely to be	Difficult conditions for excavation	Tauranga Group, Kaawa Formation	Shaft and pumping station

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
sediments	encountered			
Wood fragments and logs	Wood fragments and logs may be encountered	Difficult conditions for slurry walling / excavation / constructability	Tauranga Group, Kaawa Formation	All locations

11.7 DSCIN001 Kiwi Esplanade

11.7.1 Geotechnical ground model

The geotechnical ground model is presented in geotechnical drawings DSCIN001 – Kiwi Esplanade Shaft Geology (DWG No. 2011879.001 to 2011879.004). As part of the preliminary design process this shaft has been eliminated.

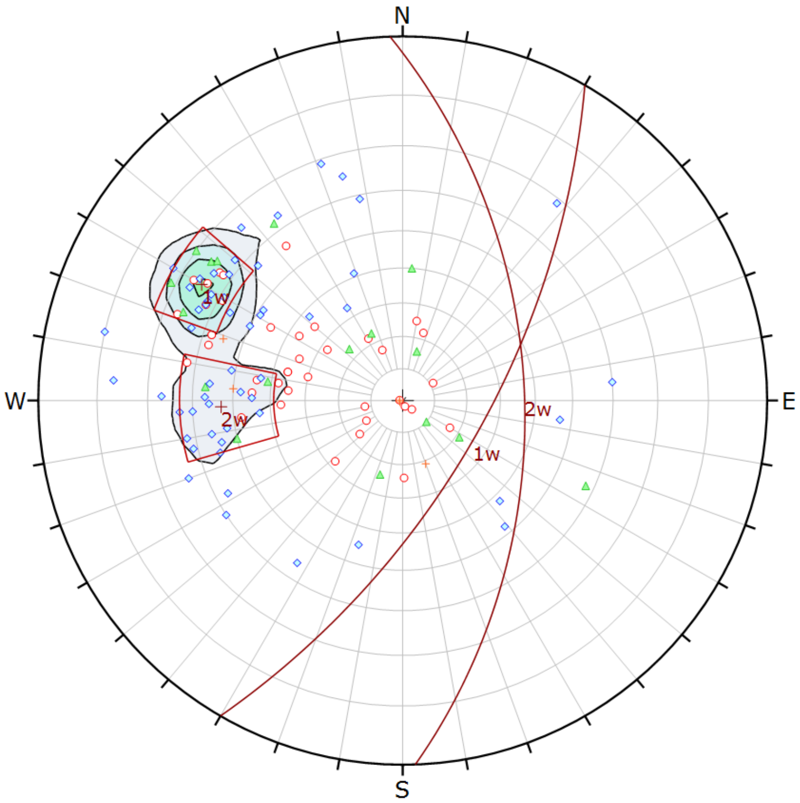
The geotechnical ground model for the shaft site is presented in Table 11-14. Undifferentiated Tauranga Group is expected to range from soft to firm cohesive soil and loose to medium dense granular soil. Design of shaft structures should consider both material types to accommodate expected variability.

Table 11-14 Geotechnical ground model for DSCIN001 – Kiwi Esplanade Shaft.

From (mRL)	To (mRL)	Geotechnical Unit
Surface (3.7mRL)	3	Made Ground
3	-12	Basalt
-12	-19	Undifferentiated Tauranga Group
-19	Not Determined	Moderately Weathered to Unweathered ECBF

Bedding in ECBF is typically 45-60° dipping to the east and should also be expected to dip to the north and south, Figure 11.4. Two broad sets of joints and shear zones were observed dipping on average 60/120 and 53/188. Discontinuities at variable orientations were also observed.

Joints, crush and shear zones in basalt are presented in Figure 11.5. Bedding is recorded in the televiewer logs within basalt rock these structures are interpreted as foliation or fabric associated with solidification of lava rather than separate flows; no evidence of defects or partings is evident in drillcore and therefore these have been excluded from stereonet analysis. Observed defects are typically steeply to very steeply inclined dipping in a range of orientations.



Symbol	TYPE	Quantity
○	Bedding Plane/Fracture	40
△	Crush/Shear	17
◇	Joint	54
+	Unidentified	5

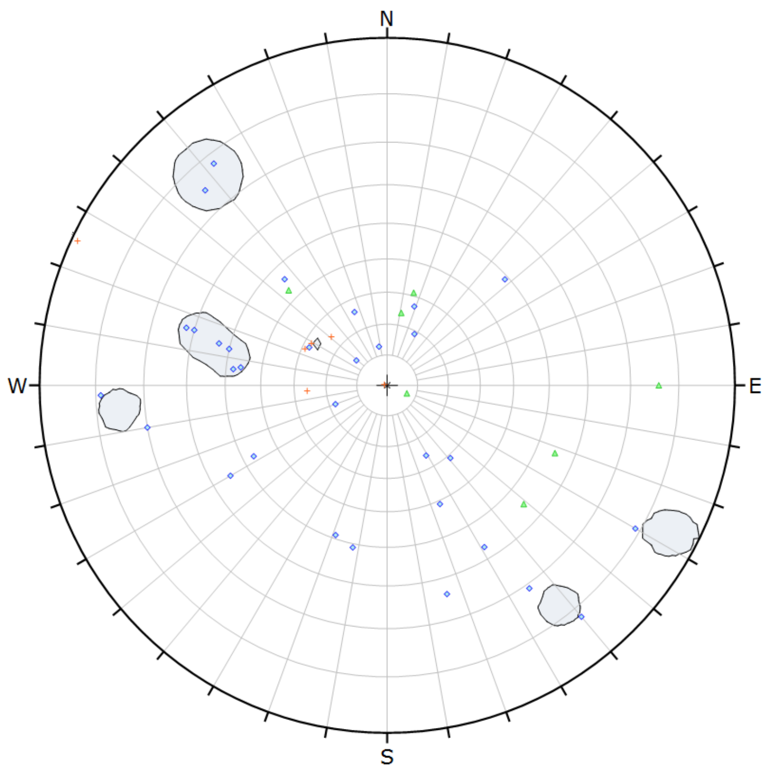
Color	Density Concentrations
	0.00 - 5.00
	5.00 - 10.00
	10.00 - 15.00
	15.00 - 20.00
	20.00 - 25.00
	25.00 - 30.00
	30.00 - 35.00
	35.00 - 40.00
	40.00 - 45.00
	45.00 <

Maximum Density	21.54%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

	Color	Dip	Dip Direction	Label
Mean Set Planes				
1w		65	120	
2w		53	88	

Plot Mode	Pole Vectors
Vector Count	116 (116 Entries)
Terzaghi Weighting	Minimum Bias Angle 15°
Hemisphere	Lower
Projection	Equal Angle

Figure 11.4 Discontinuities in ECBF only for BH259



Symbol	TYPE	Quantity
△	Crush/Shear	7
◇	Joint	31
+	Unidentified	6

Color	Density Concentrations
	0.00 - 5.00
	5.00 - 10.00
	10.00 - 15.00
	15.00 - 20.00
	20.00 - 25.00
	25.00 - 30.00
	30.00 - 35.00
	35.00 - 40.00
	40.00 - 45.00
	45.00 <

Maximum Density	9.03%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

Plot Mode	Pole Vectors
Vector Count	44 (44 Entries)
Terzaghi Weighting	Minimum Bias Angle 15°
Hemisphere	Lower
Projection	Equal Angle

Figure 11.5 Discontinuities in basalt only for BH259

11.7.2 Groundwater model

Standing groundwater is expected to be encountered and the piezometric surface at tunnel level is 1.5mRL based on piezometers in BH258, BH259 and BH260.

Piezometer results in BH260 appear to fluctuate in cycles related to tidal changes of the Manukau Harbour, Figure 11.6. High and low peaks are greater in the piezometer installed at 14mbgl within basalt than in the piezometer at 26mbgl, within Waitemata Group rock, and the piezometer at 22mbgl within a thin silty sand bed surrounded by silty clay beds (Tauranga Group). It is likely lower permeability's at 22mbgl and BH26mbgl impede tidal cycle fluctuations.

The fluctuations observed in piezometers do not peak at high tide, rather at low tide indicating there may be a lag between tides and observations. The variation in head within the basalt suggests a high degree of connection to the sea and therefore potential for significant inflows to open excavations.

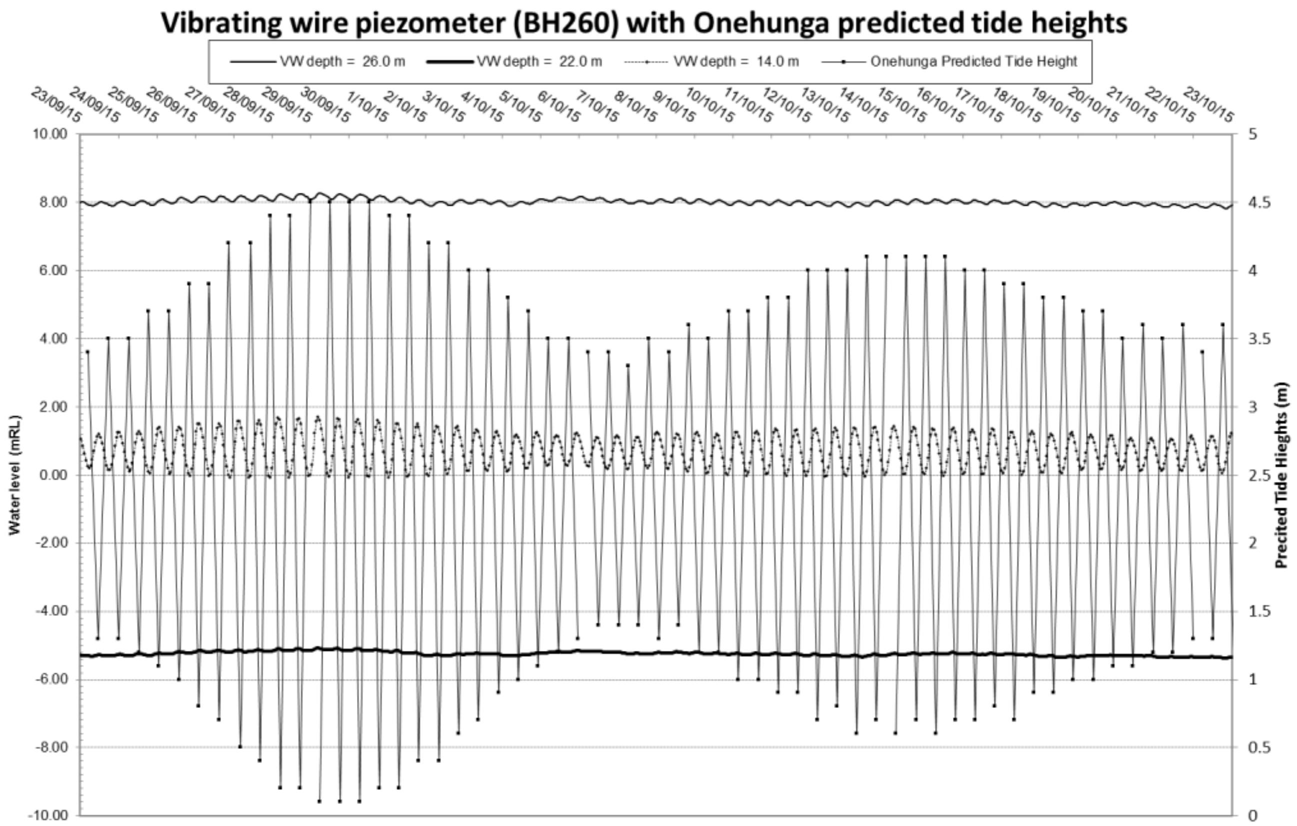


Figure 11.6 Vibrating wire piezometer results from BH260 for 30 days with predicted tide heights for Onehunga. Daily fluctuations in piezometer results appear comparable to predicted tide heights for Onehunga

Fluctuations are greater during spring tides than neap tides and within the basalt (piezometer at 14mbgl) the fluctuations of 1.7m within one 6 hour cycle were observed, Figure 11.7.

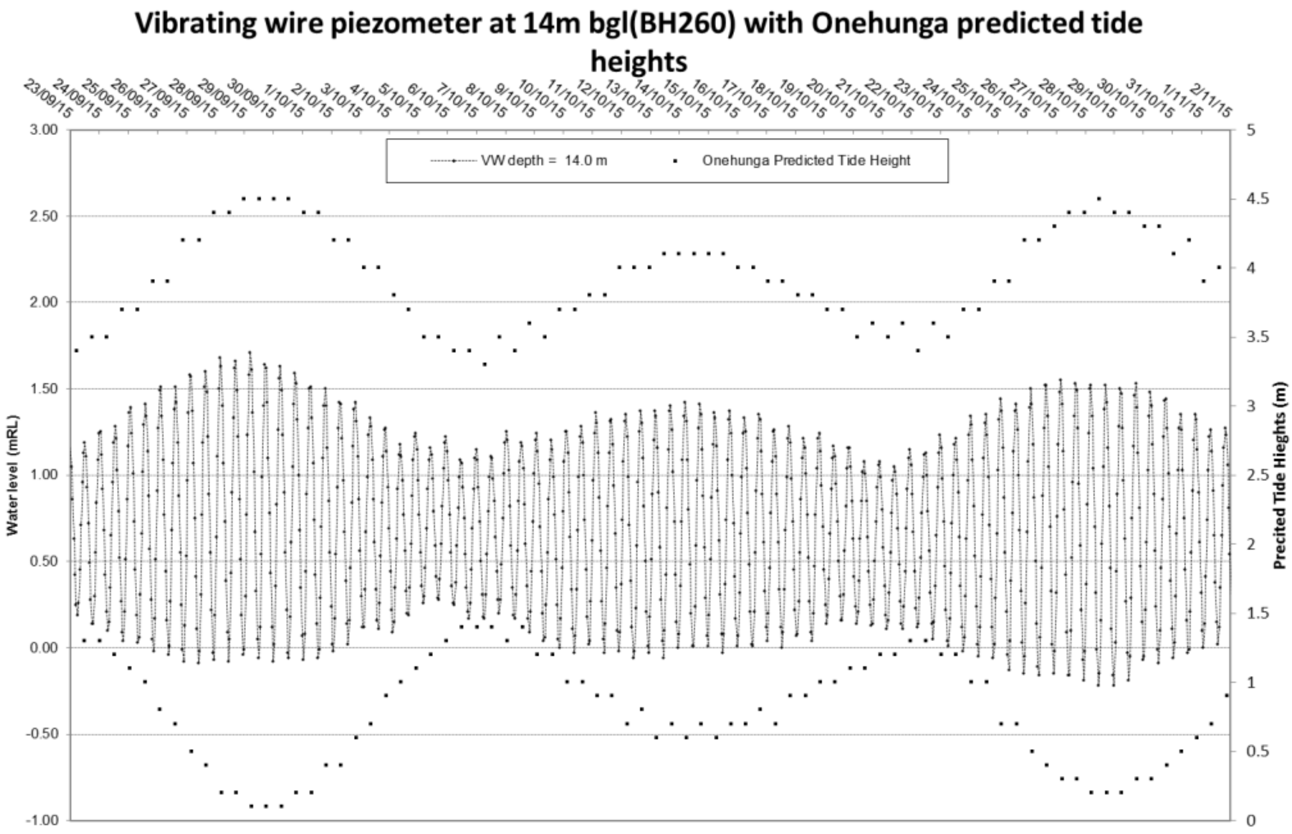


Figure 11.7 Results from vibrating wire piezometer installed at 14mbgl from BH260 for 30 days with predicted tide heights for Onehunga. 6 hourly fluctuations in piezometer results up to 1.7m are observed with spring tides showing greater variation than neap tides

11.7.3 Contaminated land assessment

The proposed works for the construction of Access Shaft 7 on the Main Tunnel comprise site works for laydown areas and pavements, excavation of 5 m and 9 m diameter deep shafts and excavations for a below-ground air treatment facility, manhole and trenched connections to network sewers.

Previous desk top investigations indicated the site has been subject to historical filled but the source of the fill is unknown. CoPC for the site include heavy metals, OCP compounds and ACM.

The following observations were recorded during the intrusive sampling investigation:

- Fill material (gravels) was observed from surface to depths of 0.95 to 1.2 m bgl.
- No evidence of dumping or fly tipping was observed.
- PID readings in soil ranged from 0.0 to 1.2 ppm.
- Groundwater was not encountered during sampling.
- Possible ACM (hardy board) was uncovered at a depth of 1.5 m in TP05. Two additional samples were collected for analysis, including one soil sample and one material sample.

The following table presents a summary of a comparison of soil sampling results to the adopted acceptance criteria. A full results table can be viewed as appended.

Table 11-15 : Comparison of soil sampling results to the adopted acceptance criteria (Kiwi Esplanada))

Contaminant of Potential Concern	Assessment Criteria		
	Schedule 10 Permitted Activity Criteria	Auckland Background Concentrations (volcanic)	NES SCS
Heavy Metals	All below or NC.	All below or NC.	All below or NC.
Other organic compounds including DDT	All below or NC.	All below or NC.	All below or NC.
ACM	Chrysotile and Amosite detected asbestos fibres detected.		

NC = No Criteria.

- A comparison to the adopted acceptance criteria indicates that there are no exceedances of NES SCS for protection of human health.

Detection of asbestos fibres in soil suggests a potential risk to human health. Further testing of soil is required to determine the concentration of asbestos fibres in soil prior to commencing works. Further testing will also be required to confirm disposal requirements. Currently, the material will potentially require disposal to a licensed landfill facility authorised to accept ACM.

11.7.4 Geotechnical risks

Table 11-16 : Geotechnical risks (Kiwi Esplanade)

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
Variable basalt rock mass	Basalt has a high or low quality rock mass and can be dependent on fracture zones, vesicularity or be rubbly.	Additional loading on structure and support Basalt may be easier to excavate	Basalt	Within basalt
Groundwater inflow	Basalt is typically vesicular and jointing allowing high water inflows	Disruption to excavation work and may require pumping	Basalt	Within basalt

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
Dewatering induced settlement	Tauranga Group soils may be susceptible to dewatering inducted settlement during excavation	Settlement of overlying basalt may occur causing damage to shaft and excavation support Ground settlement	Tauranga Group and possibly overlying basalt	Shaft and surrounding ground.
Wood fragments and logs	Wood fragments and logs may be encountered	Difficult conditions for excavation / constructability	Tauranga Group	Anywhere within Tauranga Group

11.8 DSCIN002 PS23

11.8.1 Geotechnical ground model

The geotechnical ground model is presented in geotechnical drawings DSCIN002 – PS23 Shaft Geology (DWG No. 2011880.001 to 2011880.004).

The geotechnical ground model for the shaft site is presented in Table 11-17 below.

Table 11-17 Geotechnical ground model for DSCIN002 – PS23 Shaft Geology.

From (mRL)	To (mRL)	Geotechnical Unit
Surface (3.9mRL)	1	Residually to Highly Weathered Cohesive Soils of ECBF
1	Not Determined	Moderately Weathered to Unweathered ECBF

11.8.2 Groundwater model

The piezometric surface inferred at tunnel level is 2mRL and it is expected standing groundwater level will be encountered at the same level based on piezometers in BH255 and BH256. Water levels observed in BH255 appear to fluctuate and are comparable with tidal effects due to proximity to Manukau Harbour. These fluctuations are observed to be approximately 0.1m, peaking each high tide and peaks are greater during spring tides than neap tides, Figure 11.8. It is likely other hydrological factors are also affecting the water levels observed.

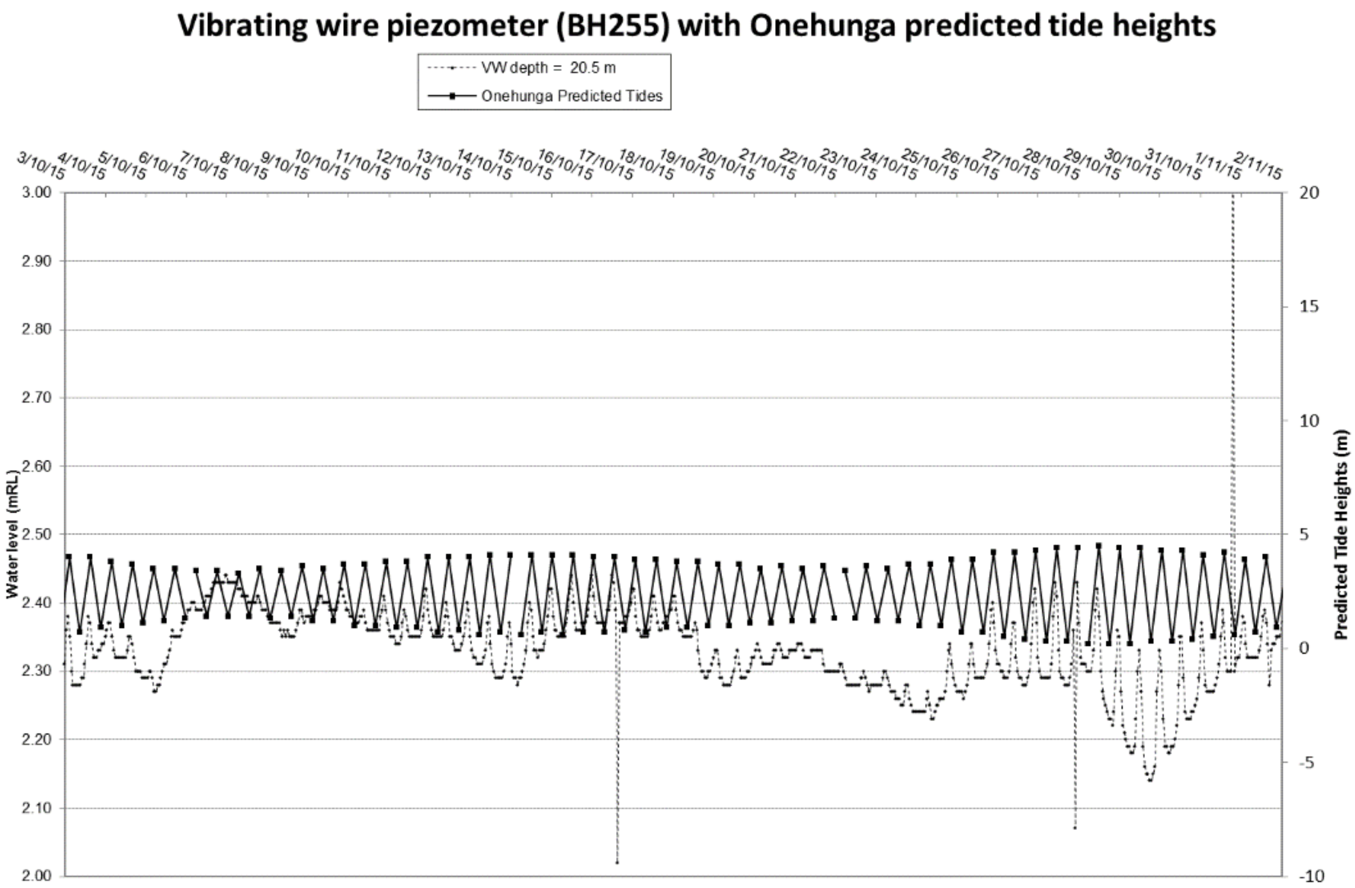


Figure 11.8 Vibrating wire piezometer results from BH255 for 30 days with predicted tide heights for Onehunga. Daily fluctuations in piezometer results appear comparable predicted tide heights for Onehunga. Notwithstanding, the tidal effects are very minor in relation

11.8.3 Contaminated land assessment

The proposed works for the construction of Access Shaft 6 on the Main Tunnel comprise demolition of the existing pump station and site works comprising realignment of the existing road, a new sea-wall, laydown areas and pavements. Excavation is also required for 7m and 9m diameter deep shafts.

Previous desk top investigations indicated part of the site has been reclaimed and the source of the fill is unknown. CoPC for the site include heavy metals, hydrocarbons, and nitrogen compounds.

The following observations were noted during the intrusive sampling investigation:

- Fill material was observed from surface to a depth of 0.5 m bgl.
- No evidence of dumping or fly tipping was observed.
- PID readings in soil samples did not exceed 0.0 ppm.
- Groundwater was not encountered during sampling.
- ACM was not observed.

The following table presents a summary of a comparison of soil sampling results to the adopted acceptance criteria. A full results table can be viewed as appended.

Table 11-18 : Comparison of soil sampling results to the adopted acceptance criteria (PS23)

Contaminant of Potential Concern	Assessment Criteria			
	Schedule 10 Permitted Activity Criteria	Auckland Background Concentrations (non-volcanic)	MfE Guidelines	NES SCS
Heavy Metals	Exceedances detected for lead.	Exceedances detected for arsenic and lead.	NC.	All below.
SVOC / VOC Compounds	All below or NC.	All below or NC.	All below or NC.	All below or NC.
TPH / BTEX Compounds	All below or NC.	All below or NC.	All below or NC.	All below or NC.
OCP Compounds	All below or NC.	All below or NC.	NC.	All below or NC.
General Testing including Nitrogen	All below or NC.	All below or NC.	NC.	All below or NC.

NC = No Criteria.

- A comparison to respective guidelines and criteria indicates that there are no exceedances of NES SCS for protection of human health.
- Select samples yielded concentrations of arsenic and lead concentrations above the Auckland Background Concentrations.
  - The average concentration of arsenic is below the respective Auckland Background Concentration.
  - The average with concentration of lead is above the Auckland Background Concentration.
- A single sample exceeded the Schedule 10 Permitted Activity Criteria for lead.
- While these concentrations of CoPC present no immediate risk to human health, the soil is likely suitable for disposal at a managed fill facility. A comparison to managed fill criteria should be undertaken at the time of disposal.

Further inspection and/or testing should be undertaken where significant volumes of fill material are encountered.

11.8.4 Geotechnical risks

Table 11-19 : Geotechnical risks (PS23)

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
Extremely weak rock	Extremely weak rock can occur at any place within ECBF	Unstable excavation Shaft design not suitable for ground conditions	Moderately weathered to unweathered ECBF	Anywhere within ECBF
Parnell Volcaniclastic Conglomerate	Conglomerate may be encountered during excavation	Excavation requires change to construction methodology Reduced productivity and higher machine tool wear	Parnell Volcaniclastic Conglomerate, Moderately to Unweathered ECBF	Anywhere within Moderately to Unweathered ECBF

11.9 DSCIN003 Keith Hay Park

11.9.1 Shaft geotechnical ground model

The geotechnical ground model is presented in geotechnical drawings DSCIN003 – Keith Hay Park Shaft Geology (DWG No. 2011881.001 to 2011881.004).

The geotechnical ground model for the shaft site is presented in Table 11-20 below. The shaft site is on the edge of a paleovalley filled with Undifferentiated Tauranga Group soil. Undifferentiated Tauranga Group soil is observed to thin towards the east, wedging out completely approximately 25m east of the shaft with the unit boundary oriented north/south. Undifferentiated Tauranga Group is expected to range from soft to firm cohesive soil and loose to medium dense granular soil. Design of shaft structures should consider both material types to accommodate expected variability.

Table 11-20 Geotechnical ground model for DSCIN003 – Keith Hay Park Shaft Geology.

From (mRL)	To (mRL)	Geotechnical Unit
Surface (57.4mRL)	57	Made Ground
57	52 to 50	Undifferentiated Tauranga Group
52 to 50	47.5	Residually to Highly Weathered Cohesive soil of ECBF
47.5	Not determined	Moderately Weathered to Unweathered ECBF

Discontinuities observed in BH250 are presented in Figure 11.9 and are generally gently inclined. Bedding orientation is variable and likely reflects undulating bedding or measurement variability of gentle bedding in televiewer images. The discontinuities have been separated into two sets. One set with sub horizontal dip at variable dip direction and another dipping 10° to the northwest.

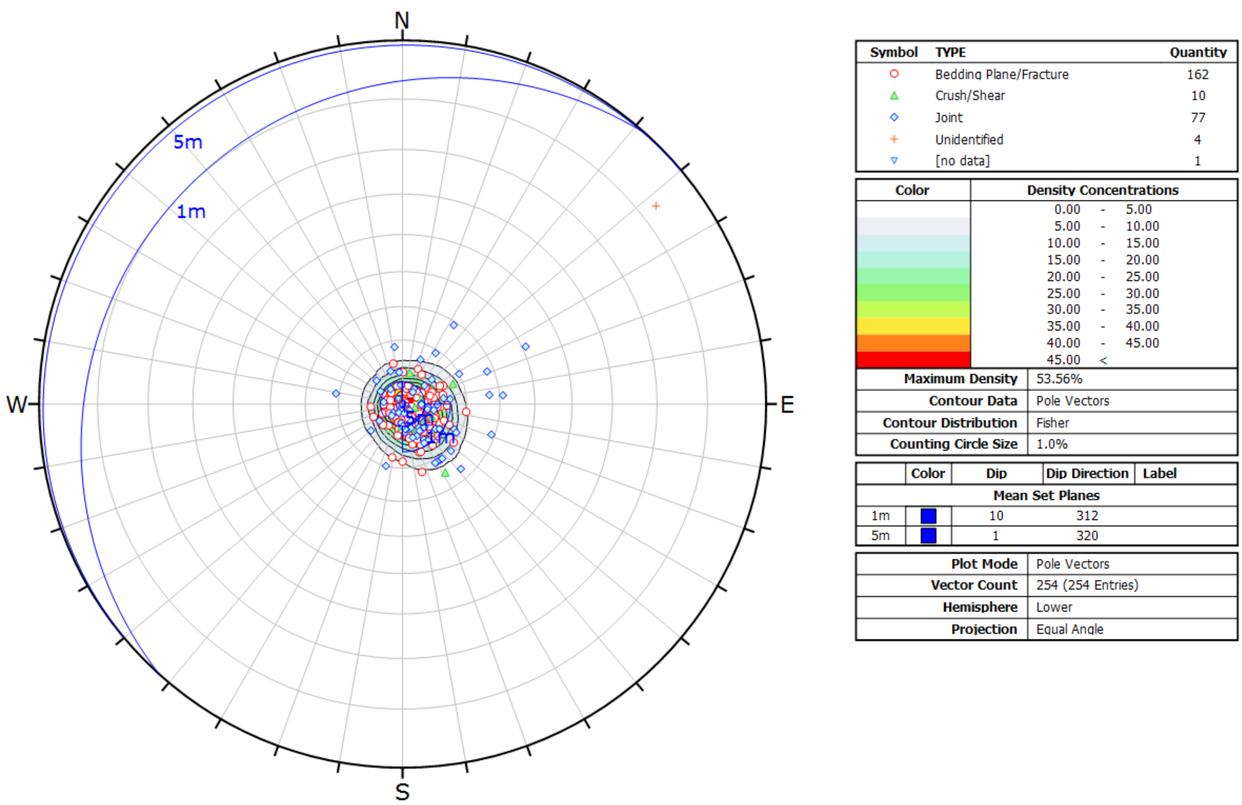


Figure 11.9 Stereonet of discontinuities for BH250. Discontinuities dip between 0° to 20°.

11.9.2 KHP Branch 9B ground model

The geotechnical ground model is presented in geotechnical drawings DSCIN003 – Keith Hay Park Shaft Geology – KHP Branch 9B (DWG 2011881.005).

Made ground extends across the site to approximately 54mRL above Undifferentiated Tauranga Group. Undifferentiated Tauranga Group becomes deeper to the northeast away from DSCIN003 shaft from 47mRL to at least 42mRL (The base of the Tauranga Group was not observed in BH404). A thin bed, up to 2m thick, of residually to highly weathered cohesive soil (ECBF soil) underlies Tauranga group. Moderately weathered to Unweathered ECBF (Rock) is observed from 47mRL near DSCIN003 and from CH300, varies between 39mRL to 42mRL, however the base is not observed past CH600 and is deeper than 42mRL

11.9.3 Groundwater model

The piezometric surface at tunnel level is inferred at 55mRL, approximately 3m below existing ground surface based on the deep vibrating wire piezometer in BH250. Standing groundwater is likely to be encountered at the ground surface based on piezometers installed in BH249 and BH250.

11.9.4 Contaminated land assessment

The proposed works for the construction of Access Shaft 5 on the Main Tunnel comprises a main work site with demolition of an existing house, site works to establish laydown areas and pavements, excavation of 7 m and 9 m diameter deep shafts and a 3.5 m shaft for micro-tunnelling. Excavations are also required for a connection and stop-log chamber, manhole and trenched connections to branch sewer 9. Three minor sites require excavation of 3.5 m shafts for micro-tunnelling.

Previous desk top investigations indicated the fill on the western part of the construction site (within the park) comprises silt and clay. CoPC for the site include heavy metals, hydrocarbons, and nitrogen compounds.

The following observations were made during the intrusive sampling investigation:

- Clean fill was observed to a depth of 1.5 m bgl.
- No evidence of dumping of fly tipping was observed.
- PID readings in soil samples did not exceed 0.0 ppm.
- Groundwater was not encountered during sampling.
- ACM was not observed.

The following table presents a summary of a comparison of soil sampling results to the adopted acceptance criteria. A full results table can be viewed as appended.

Table 11-21 : Comparison of soil sampling results to the adopted acceptance criteria (Keith Hay Park)

Contaminant of Potential Concern	Assessment Criteria			
	Schedule 10 Permitted Activity Criteria	Auckland Background Concentrations (non- volcanic)	MfE Guidelines	NES SCS
Heavy Metals	All below.	Exceedance detected for arsenic and nickel.	NC.	All below.
SVOC / VOC Compounds	All below or NC.	All below or NC.	All below or NC.	All below or NC.
Other organic compounds including DDT	All below or NC.	All below or NC.	All below or NC.	All below or NC

NC = No Criteria.

- A comparison to the adopted acceptance criteria indicates that there are no exceedances of NES SCS for protection of human health.
- The average with concentration of nickel was above the Auckland Background Concentrations.

While these concentrations of CoPC present no immediate risk to human, the soil is likely suitable for disposal at a managed fill facility. Soils may also be accepted by clean fill sites. A comparison to landfill criteria should be undertaken at the time of disposal.

11.9.5 Geotechnical risks

Table 11-22 : Geotechnical risks (Keith Hay Park)

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
Extremely weak rock	Extremely weak rock can occur at any place within ECBF	Unstable excavation Shaft design not suitable for ground conditions	Moderately weathered to unweathered ECBF	Anywhere within ECBF
Parnell Volcaniclastic Conglomerate	Conglomerate may be encountered during excavation	Excavation requires change to construction methodology Reduced productivity and higher machine tool wear	Parnell Volcaniclastic Conglomerate, Moderately to Unweathered ECBF	Anywhere within Moderately to Unweathered ECBF
Variable and undulating contact between soil and rock units	Mixed face conditions comprising soil and rock units	Face loss/pressure loss Tunnel face instability Ground surface movement Deflection of TBM resulting in alignment or grade issues	Tauranga Group Moderately to unweathered ECBF Residual Soil to highly weathered ECBF	KHP Branch 9B
Wood fragments and logs	Wood fragments and logs may be encountered	Difficult conditions for excavation / constructability	Tauranga Group	Anywhere within Tauranga Group

11.10 DSCIN004 May Road

11.10.1 Geotechnical ground model

The geotechnical ground model is presented in geotechnical drawings DSCIN004 – May Road Shaft Geology (DWG No. 2011882.001 to 2011882.005).

The geotechnical ground model for the shaft sites is presented in Table 11-23 and Table 11-24 below. Basalt occurs near surface extends entirely across DSCIN004 shaft and DSCIN004A construction shaft. The basalt appears to deepen to the north and its thickness may vary across both shafts. Undifferentiated alluvium underlies basalt and overlies a 10m thick horizon of residually to highly weathered ECBF. Moderately weathered to unweathered ECBF rock occurs at approximately 12mRL.

Undifferentiated Tauranga Group is expected to range from soft to firm cohesive soil and loose to medium dense granular soil. Design of shaft structures should consider both material types to accommodate expected variability.

Table 11-23 Geotechnical ground model for DSCIN004 – May Road Shaft Geology

From (mRL)	To (mRL)	Geotechnical Unit
Surface (47.8mRL)	47	Made Ground
47	45	Basalt
47	32 to 33	Undifferentiated Tauranga Group
32 to 33	15 to 13	Residually to Highly Weathered Cohesive soil of ECBF
15 to 13	Not determined	Moderately Weathered to Unweathered ECBF

Table 11-24 Geotechnical ground model for DSCIN004A – May Road Construction Shaft Geology

From (mRL)	To (mRL)	Geotechnical Unit
Surface (47.8mRL)	47	Made Ground
47	44 to 40	Basalt
44 to 40	27	Undifferentiated Tauranga Group
27	12 to 15	Residually to Highly Weathered Cohesive soil of ECBF
12 to 15	Not determined	Moderately Weathered to Unweathered ECBF

Discontinuities for BH246 are presented in Figure 11.10 and include data from ECBF only. There is a limited amount of data as Tauranga Group alluvium is present in drill hole to 21.5m bgl and televiewer logging was undertaken to 27m bgl. Due to the limited information inferences on typical discontinuities have not been made.

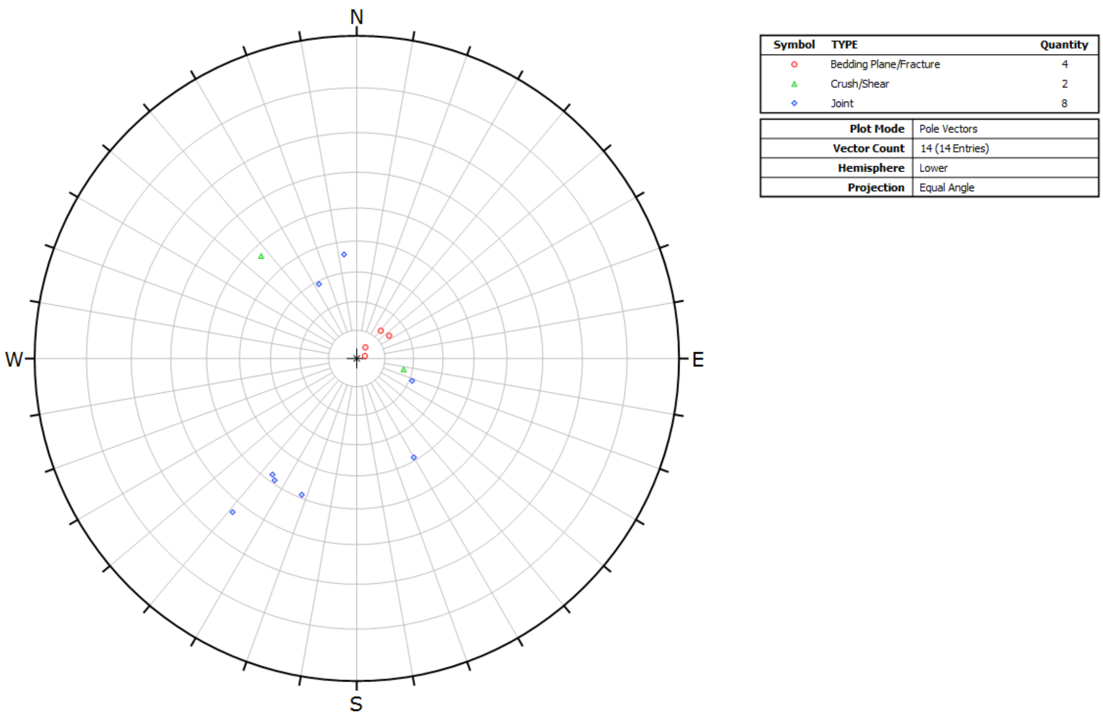


Figure 11.10 Stereonet plot of discontinuities for BH246

11.10.2 Groundwater model

Based on piezometers from BH245, BH246 and BH247 groundwater is likely to be encountered at 48mRL. The piezometric surface at tunnel level is 47mRL based on deep vibrating wire piezometer installed in BH245.

Discussion of the aquifer studies undertaken at this site are presented in Section 7.4.2.2 and in further detail in Appendix F

11.10.3 Contaminated land assessment

Previous desk top investigations identified that the site has been subject to historic filling and has had multiple pollution incidents on or near the site. Because the property has been unoccupied, the risk of uncontrolled filling is high. CoPC for the site include heavy metals, hydrocarbons, and ACM.

The following observations were noted during the intrusive sampling investigation.

- Fill material was observed to depths of 0.7 to 1.5 m bgl.
- Fill material dumping has occurred over the site, no evidence of fly tipping was observed.
- PID readings in soils ranged from 0.0 to 0.3 ppm.
- Groundwater was not encountered during sampling.
- ACM was not observed.

The following table presents a summary of a comparison of soil sampling results to the adopted acceptance criteria. A full results table can be viewed as appended.

Table 11-25 : Comparison of soil sampling results to the adopted acceptance criteria (May Road)

Contaminant of Potential Concern	Assessment Criteria			
	Schedule 10 Permitted Activity Criteria	Auckland Background Concentrations (volcanic)	MfE Guidelines	NES SCS
Heavy Metals	All below.	All below.	NC.	All below.
SVOC / VOC Compounds	All below or NC	All below or NC	All below or NC.	All below or NC
PAH Compounds	All below or NC.	All below or NC.	All below or NC.	All below or NC
Other organic compounds including DDT	All below or NC.	All below or NC.	NC.	All below or NC

NC = No Criteria.

- A comparison to the adopted acceptance criteria indicates that there are no exceedances of NES SCS for protection of human health.

The soil is likely suitable for disposal at a cleanfill facility. A comparison to landfill criteria should be undertaken at the time of disposal.

11.10.4 Geotechnical risks

Table 11-26 : Geotechncial risks (May Road)

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
Extremely weak rock	Extremely weak rock can occur at any place within ECBF	Unstable excavation Shaft design not suitable for ground conditions	Moderately weathered to unweathered ECBF	Anywhere within ECBF
Parnell Volcaniclastic Conglomerate	Conglomerate may be encountered during excavation	Excavation requires change to construction methodology Reduced productivity and higher machine tool wear	Parnell Volcaniclastic Conglomerate, Moderately to Unweathered ECBF	Anywhere within Moderately to Unweathered ECBF
Variable basalt rock mass	Basalt has a high or low quality rock mass and can be dependent on fracture zones, vesicularity or be rubbly.	Additional loading on structure and support Basalt may be easier to excavate	Basalt	Within basalt
Groundwater inflow	Basalt is typically vesicular and jointing allowing high water inflows	Disruption to excavation work and may require pumping	Basalt	Within basalt
Wood fragments and logs	Wood fragments and logs may be encountered	Difficult conditions for excavation / constructability	Tauranga Group	Anywhere within Tauranga Group

11.11 DSCIN005 Walmsley Park

11.11.1 Geotechnical ground model

The geotechnical ground model is presented in geotechnical drawings DSCIN005 – Walmsley Park Shaft Geology (DWG No. 2011883.001 to 2011883.004).

The geotechnical ground model for the shaft site is presented in Table 11-27 below. Basalt is observed with undifferentiated Tauranga Group overlying and underlying the lava which is taken to be part of the Mt Roskill lava flow. Within ECBF, Parnell Volcaniclastic Conglomerate is present in lenses up to 5m thick between 19mRL to 5mRL and below -13.5mRL.

Undifferentiated Tauranga Group is expected to range from soft to firm cohesive soil and loose to medium dense granular soil. Design of shaft structures should consider both material types to accommodate expected variability.

Table 11-27 Geotechnical ground model for DSCIN005 – Walmsley Park Shaft Geology.

From (mRL)	To (mRL)	Geotechnical Unit
Surface (47mRL)	46	Made Ground
46	40	Undifferentiated Tauranga Group
40	38	Basalt
38	36.5	Undifferentiated Tauranga Group
36.5	31	Residually to Highly Weathered Cohesive soil of ECBF
31	Not determined	Moderately Weathered to Unweathered ECBF with lenses of PVC up to 5m thick

Discontinuities for BH232 are presented in Figure 11.11 and include those in ECBF only. Discontinuities generally steeply inclined, dipping east to south. The structural logs for BH232 observed a zone of high angle shears between 19m to 24m bgl and several zones of closely spaced joints. Two sets of discontinuities have been identified; a cluster of bedding observed dipping 28° to the south and a set of joints and bedding planes dipping 43° to the south east. A broad group of steeply inclined joints and shear zones is observed dipping to the northwest.

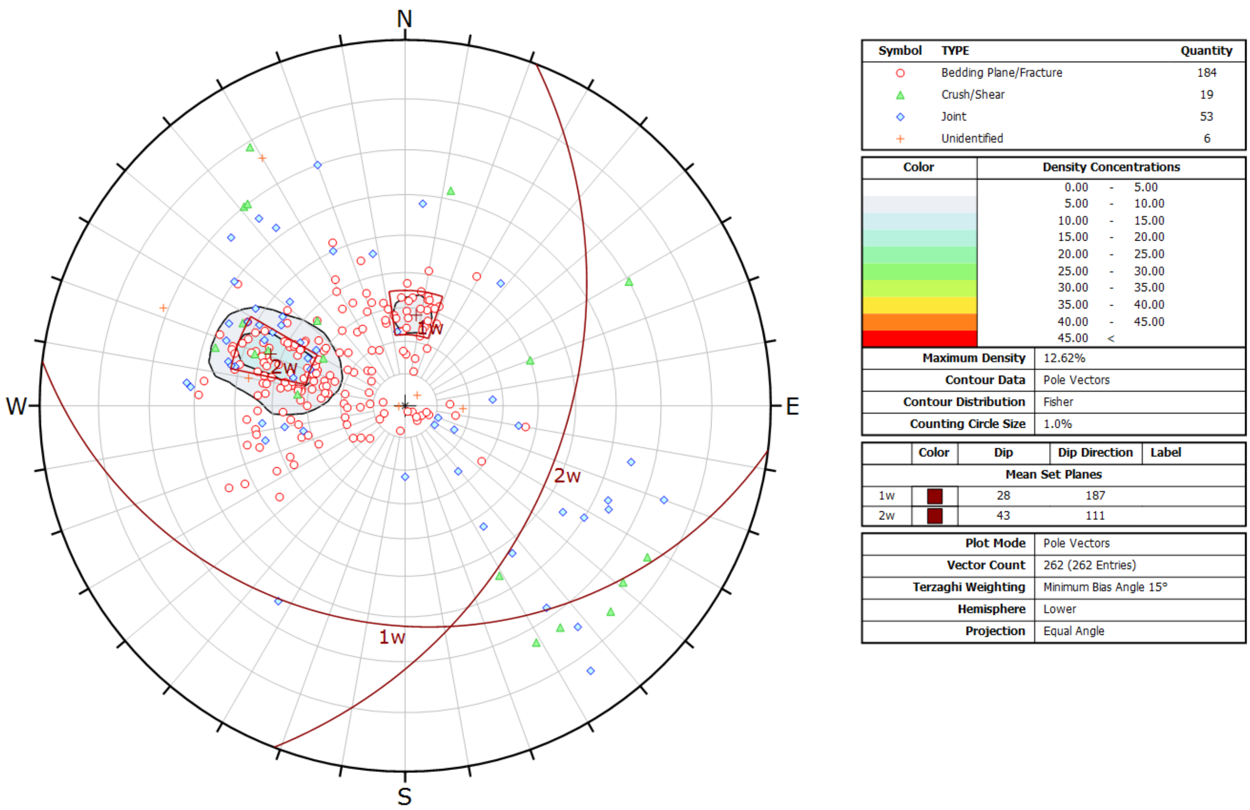


Figure 11.11 Stereonet plot of discontinuities for BH232

11.11.2 Groundwater model

Based on shallow piezometers from BH231 and BH231 groundwater table is 46.5mRL. There are no deep piezometers at this site however based on BH234a and BH230 the piezometric surface at tunnel level is likely to be approximately 47mRL.

11.11.3 Contaminated land assessment

The proposed works for the construction of Access Shaft 4 on the Main Tunnel comprises site works for lay-down areas and pavements, excavation of two deep shafts, excavations for chambers and trenched connections to the branch 9 sewer system.

Previous desk top investigations indicated potential contaminated fill at the site. CoPC for the site include heavy metals, hydrocarbons, and ACM.

The following observations were recorded during the intrusive sampling investigation:

- Fill material (including topsoil) was observed at depths of 1.3 to 1.6 m bgl.
- No evidence of dumping or fly tipping observed.
- PID readings in soil ranged between 0.3 to 1 ppm.
- Groundwater was not encountered during sampling.
- ACM was not observed.

The following table presents a summary of a comparison of soil sampling results to the adopted acceptance criteria. A full results table can be viewed as appended.

Table 11-28 : Comparison of soil sampling results to the adopted acceptance criteria (Walmsley Park)

Contaminant of Potential Concern	Assessment Criteria			
	Schedule 10 Permitted Activity Criteria	Auckland Background Concentrations (volcanic)	MfE Guidelines	NES SCS
Heavy Metals	All below.	Exceedances detected for arsenic, copper, and lead.	NC.	All below.
SVOC / VOC	All below or NC.	All below or NC.	All below or NC.	All below or NC.
PAH Compounds	All below or NC.	All below or NC.	All below or NC.	All below or NC.
OCP Compounds	All below or NC.	All below or NC.	NC.	All below or NC.
ACM	Not detected.			

NC = No Criteria.

- A comparison to the adopted acceptance criteria indicates that there are no exceedances of NES SCS for protection of human health.
- A single sample yielded concentrations of arsenic, copper and lead above the Auckland Background Concentrations. The average concentration of these metals across the site was below the adopted acceptance criteria.
- Concentrations of select PAH compounds are present in shallow soil. The adopted acceptance criteria were not exceeded.
- While these concentrations of CoPC present no immediate risk to human health or the environment, the soil is likely suitable for disposal at a managed fill facility. A comparison to landfill criteria should be undertaken at the time of disposal.

Further inspection and/or testing should be undertaken where significant volumes of fill material are encountered during the works.

11.11.4 Geotechnical risks

Table 11-29 : Geotechnical risks (Walmsley Park)

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
Extremely weak rock	Extremely weak rock can occur at any place within ECBF	Unstable excavation Shaft design not suitable for ground conditions	Moderately weathered to unweathered ECBF	Anywhere within ECBF
Parnell Volcaniclastic Conglomerate	Conglomerate may be encountered during excavation	Excavation requires change to construction methodology Reduced productivity and higher machine tool wear	Parnell Volcaniclastic Conglomerate, Moderately to Unweathered ECBF	Anywhere within Moderately to Unweathered ECBF
Variable basalt rock mass	Basalt may have a high or low quality rock mass and can be dependent on fracture zones, vesicularity or be rubbly.	Additional loading on structure and support Basalt may be easier to excavate	Basalt	Within basalt
Groundwater inflow	Basalt is typically vesicular and jointed allowing high water inflows	Disruption to excavation work and may require pumping	Basalt	Within basalt
Wood fragments and logs	Wood fragments and logs may be encountered	Difficult conditions for excavation / constructability	Tauranga Group	Anywhere within Tauranga Group

11.12 DSCIN006 Haverstock Road

11.12.1 Geotechnical ground model

The geotechnical ground model is presented in geotechnical drawings DSCIN006 Haverstock Road Shaft Geology (DWG No. 2011884.001 to 2011884.004).

The geotechnical ground model for the shaft site is presented in

Table 11-30 below. Two thick to very thick lenses of Parnell Volcaniclastic Conglomerate were observed in BH227-1 between -7mRL and -16mRL.

Undifferentiated Tauranga Group is expected to range from soft to firm cohesive soil and loose to medium dense granular soil. Design of shaft structures should consider both material types to accommodate expected variability.

Table 11-30 Geotechnical ground model for DSCIN006 Haverstock Road Shaft Geology.

From (mRL)	To (mRL)	Geotechnical Unit
Surface (31.2mRL)	30	Made Ground
30	21	Undifferentiated Tauranga Group
21	20.5	Residually to Highly Weathered Cohesive soil of ECBF
20.5	> - 30	Moderately Weathered to Unweathered ECBF with lenses of PVC up to 5m thick.

Discontinuities in ECBF only for BH227 are presented in Figure 11.12. The majority of discontinuities observed in televiewer analysis were moderately inclined, dipping to the north east. Several very steeply inclined to sub-vertical joints were encountered at 28m to 30m bgl.

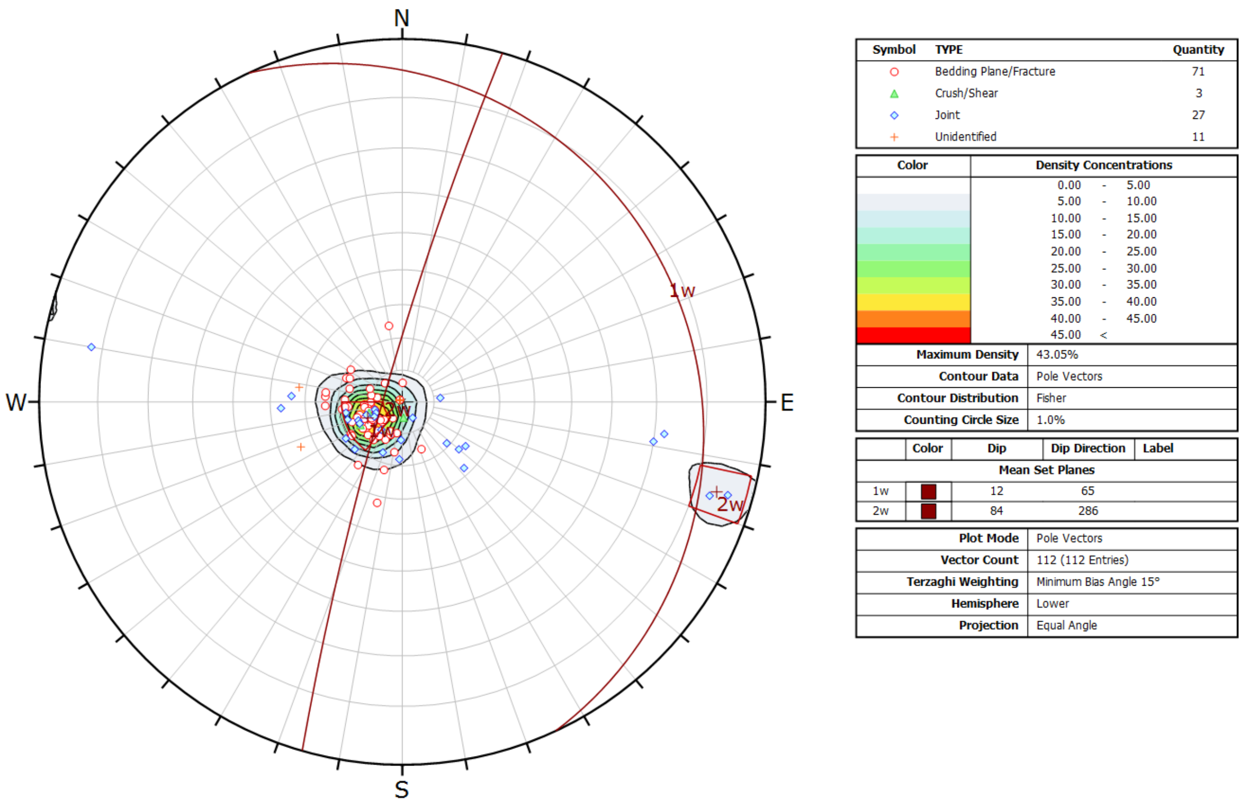


Figure 11.12 Stereonet plot of discontinuities in BH227

11.12.2 Groundwater model

Based on shallow piezometers in BH227-1 and BH228-1 groundwater is likely to be encountered at or near the surface. Vibrating wire piezometers in BH227-2 indicate the piezometric surface at tunnel level is 38.5mRL. Water seepage, likely the result of upward groundwater pressure through the piezometer installation was observed in BH228.

11.12.3 Contaminated land assessment

The proposed works for the construction of Access Shaft 3 on the main tunnel comprises site works for laydown areas and pavements, excavation of two 9 m diameter deep shafts as well as excavations for a control chamber, manholes and associated trench connections.

Previous desk top investigations indicated potential for hotspots of contamination from former chemical storage areas, although it is unclear where within the site these storage areas were. CoPC for the site include heavy metals (including mercury) and OCP compounds.

The following observations were recorded during the intrusive sampling investigation:

- Fill including terracotta pieces, weed matting and gravels were observed from surface under 0.3 to 0.5 m of topsoil.
- No evidence of recent dumping or fly tipping was observed.
- PID readings in soil ranged between 0.0 to 0.5 ppm.
- Groundwater was encountered at a depth of 1.2 m during sampling.
- ACM was not observed.

The following table presents a summary of a comparison of soil sampling results to the adopted acceptance criteria. A full results table can be viewed as appended.

Table 11-31 : Comparison of soil sampling results to the adopted acceptance criteria (Haverstock Road)

Contaminant of Potential Concern	Assessment Criteria		
	Schedule 10 Permitted Activity Criteria	Auckland Background Concentrations (volcanic)	NES SCS
Heavy Metals	All below.	Exceedance of mercury detected.	All below.
Other organic compounds including DDT	All below or NC	All below or NC	All below or NC

NC = No Criteria.

- A comparison to the adopted acceptance criteria indicates that there are no exceedances of NES SCS for protection of human health.
- A single sample yielded concentrations of mercury above the Auckland Background Concentration. The average concentration of mercury is below the adopted acceptance criteria.

While these concentrations of CoPC present no immediate risk to human or the environment, the soil is likely suitable for disposal at a cleanfill or managed fill facility. A comparison to landfill criteria should be undertaken at the time of disposal.

11.12.4 Geotechnical risks

Table 11-32 : Geotaechnical risks (Haverstock Road)

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
Extremely weak rock	Extremely weak rock can occur at any place within ECBF	Unstable excavation Shaft design not suitable for ground conditions	Moderately weathered to unweathered ECBF	Anywhere within ECBF
Parnell Volcaniclastic Conglomerate	Conglomerate may be encountered during excavation	Excavation requires change to construction methodology Reduced productivity and higher machine tool wear	Parnell Volcaniclastic Conglomerate, Moderately to Unweathered ECBF	Anywhere within Moderately to Unweathered ECBF
Wood fragments and logs	Wood fragments and logs may be encountered	Difficult conditions for excavation / constructability	Tauranga Group	Anywhere within Tauranga Group

11.13 DSCIN007 Lyon Avenue

11.13.1 Geotechnical ground model

The geotechnical ground model is presented in geotechnical drawings DSCIN007 – Lyon Avenue Shaft Geology (DWG No. 2011885.001 to 2011885.004).

The geotechnical ground model for the shaft site is presented in Table 11-33 below. Geotechnical investigations are approximately 5-10m from the shaft due to accessibility restrictions and the site appears to have particularly variable ground conditions. Basalt may extend across the eastern side of the shaft and is observed outcropping to the east of the shaft. Elsewhere Undifferentiated Tauranga Group is encountered beneath a layer of Made Ground. Made ground thickness may vary across the shaft site associated with the construction of the stormwater culvert and carpark area. Thick (up to 1.5m thick) beds of Parnell Volcaniclastic Conglomerate were observed in BH225.

Similar to the shaft, the proposed retaining wall may extend across variable amounts of Made Ground, Basalt and Undifferentiated Tauranga Group.

The proposed plant room, box culverts and bifurcation chamber will likely encounter Made Ground associated with construction of the existing car park and existing overflow culvert, Basalt and Undifferentiated Tauranga Group.

Undifferentiated Tauranga Group is expected to range from soft to firm cohesive soil and loose to medium dense granular soil. Design of shaft structures should consider both material types to accommodate expected variability.

Table 11-33 Geotechnical ground model for DSCIN007 – Lyon Avenue Shaft Geology

From (mRL)	To (mRL)	Geotechnical Unit
Surface (26.4mRL sloping)	26.5	Made Ground
26.5	23	Basalt – eastern side of shaft only
23 if basalt, otherwise 26.5	2 to 0	Undifferentiated Tauranga Group
2 to 0	0 to -1	Residually to Highly Weathered Cohesive soil of ECBF
0 to -1	> -25	Moderately Weathered to Unweathered ECBF

Discontinuities in ECBF for BH224 are presented in Figure 11.13. Bedding is typically gently inclined with variable dip. A set of joints have been identified dipping to the west at an average of 30°. Steeply inclined joints at a range of dip directions are observed also. Jointing in basalt is predominantly moderately to very steeply inclined with variable dip direction, Figure 11.14.

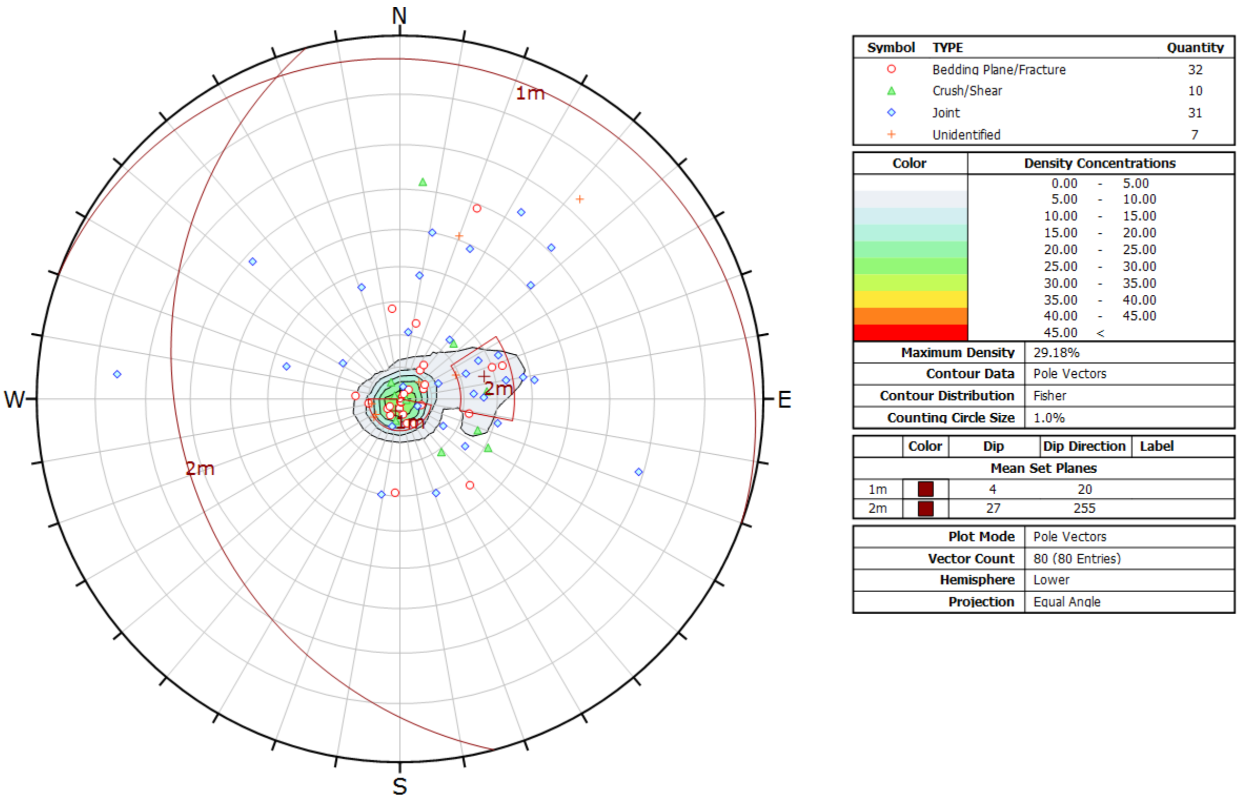


Figure 11.13 Stereonet plot of discontinuities in ECBF for BH224

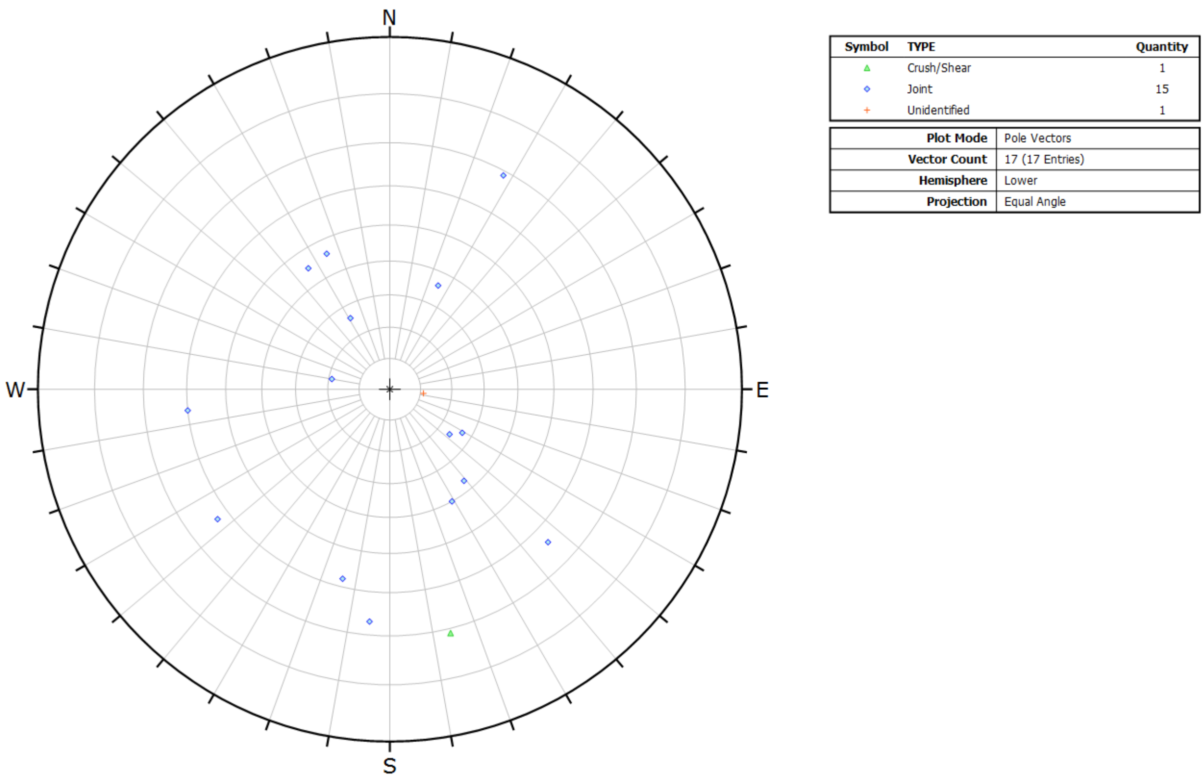


Figure 11.14 Stereonet of discontinuities in basalt for BH224

11.13.2 Groundwater model

Based on piezometers installed in BH225 and BH224 groundwater level is inferred to be at 25.5mRL. At tunnel level the piezometric surface is inferred to be at 27mRL based on the vibrating wire piezometer installed in BH225.

11.13.3 Contaminated land assessment

The proposed works for the construction of Access Shaft 2 on the main tunnel comprises site works for laydown areas and pavements, excavation of two 9 m shafts as well as excavations for a diversion chamber, trenched connections and outfall.

Previous desk top investigations indicated that on the northern part of the construction site there is up to 4.5 m of fill material. The fill material comprises gravelly silt and demolition material. Part of the site was used for industrial activities. A former underground storage tank was located at the portion of the site related to 15 Lyon Ave. CoPC for the site include heavy metals (including mercury), hydrocarbons, and ACM.

The following observations were recorded during the intrusive sampling investigation:

- Fill material was observed from surface below 0.1 m of topsoil.
- No dumping or fly tipping observed.
- PID readings in soil samples did not exceed 0.0 ppm.
- No groundwater was encountered during sampling.
- Potential asbestos observed in soil.

The following table presents a summary of a comparison of soil sampling results to the adopted acceptance criteria. A full results table can be viewed as appended.

Table 11-34 : Comparison of soil sampling results to the adopted acceptance criteria (Lyon Avenue)

Contaminant of Potential Concern	Assessment Criteria			
	Schedule 10 Permitted Activity Criteria	Auckland Background Concentrations (volcanic)	MfE Guidelines	NES SCS
Heavy Metals	All below.	Exceedances detected for arsenic and lead.	NC.	All below.
SVOC / VOC Compounds	All below or NC.	All below or NC.	All below or NC.	All below or NC
PAH Compounds	All below or NC.	All below or NC.	All below or NC.	All below or NC
ACM	Chrysotile (white asbestos) was detected in soil.			

NC = No Criteria.

- A comparison to the adopted acceptance criteria indicates that there are no exceedances of NES SCS for protection of human health.
- While below respective human health and environmental criteria, it is noted that a concentrations of PAH compounds are present in shallow soil. Further inspection and/or testing should be undertaken where significant volumes of fill material are encountered during the works.
  - Select samples yielded concentrations of arsenic and lead above Auckland Background Concentrations.
  - The average concentration of arsenic was recorded below the adopted acceptance criteria.
  - The average concentration of lead in soil across the site was above the adopted acceptance criteria.

Detection of asbestos fibres in soil suggests a potential risk to human health. Further testing of soil is required to determine the concentration of asbestos fibres in soil prior to commencing works. The testing will also be required to confirm disposal requirements. Currently, the material will potentially require disposal to a licensed landfill facility authorised to accept ACM.

11.13.4 Geotechnical risks

Table 11-35 : Geotechnical risks (Lyon Avenue)

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
Extremely weak rock	Extremely weak rock can occur at any place within ECBF	Unstable excavation Shaft design not suitable for ground conditions	Moderately weathered to unweathered ECBF	Anywhere within ECBF
Parnell Volcaniclastic Conglomerate	Conglomerate may be encountered during excavation	Excavation requires change to construction methodology Reduced productivity and higher machine tool wear	Parnell Volcaniclastic Conglomerate, Moderately to Unweathered ECBF	Anywhere within Moderately to Unweathered ECBF
Variable basalt rock mass	Basalt has a high or low quality rock mass and can be dependent on fracture zones, vesicularity or be rubbly.	Additional loading on structure and support Basalt may be easier to excavate	Basalt	Within basalt
Groundwater inflow	Basalt is typically vesicular and jointing allowing high water inflows	Disruption to excavation work and may require pumping	Basalt	Within basalt
Wood fragments and logs	Wood fragments and logs may be encountered	Difficult conditions for excavation / constructability	Tauranga Group	Anywhere within Tauranga Group

11.14 DSCIN008 Mt Albert Memorial Reserve

11.14.1 Geotechnical ground model

The geotechnical ground model is presented in geotechnical drawings DSCIN008 – Mt Albert Memorial Reserve Shaft Geology (DWG No. 2011886.001 to 2011886.004).

The geotechnical ground model for the shaft site is presented in Table 11-36 below. Within the basalt, a 0.5m thick bed of silt was encountered in drill holes BH220 and BH219. In the summary below this has been excluded and the basalt is taken as 5m thick.

Undifferentiated Tauranga Group is expected to range from soft to firm cohesive soil and loose to medium dense granular soil. Design of shaft structures should consider both material types to accommodate expected variability.

Table 11-36 Geotechnical ground model for DSCIN008 – Mt Albert Memorial Reserve Shaft Geology.

From (mRL)	To (mRL)	Geotechnical Unit
Surface (20.8mRL)	20	Made Ground
20	15	Basalt
15	-4	Undifferentiated Tauranga Group
-4	-5.5	Residually to Highly Weathered Cohesive soil of ECBF
-5.5	> -29	Moderately Weathered to Unweathered ECBF

Discontinuities in ECBF for BH219 are presented in Figure 11.15 and are steeply inclined (average dip 43°) dipping to the southwest. The fluid in the drillhole was too turbid for optical televiewer and only joints were discerned in televiewer analysis. Drillhole core logging indicates bedding is present and is moderately to steeply inclined.

The drillhole was cased to 22.6m bgl which concealed basalt present at the top part of the hole, thus it was not observed by the televiewer.

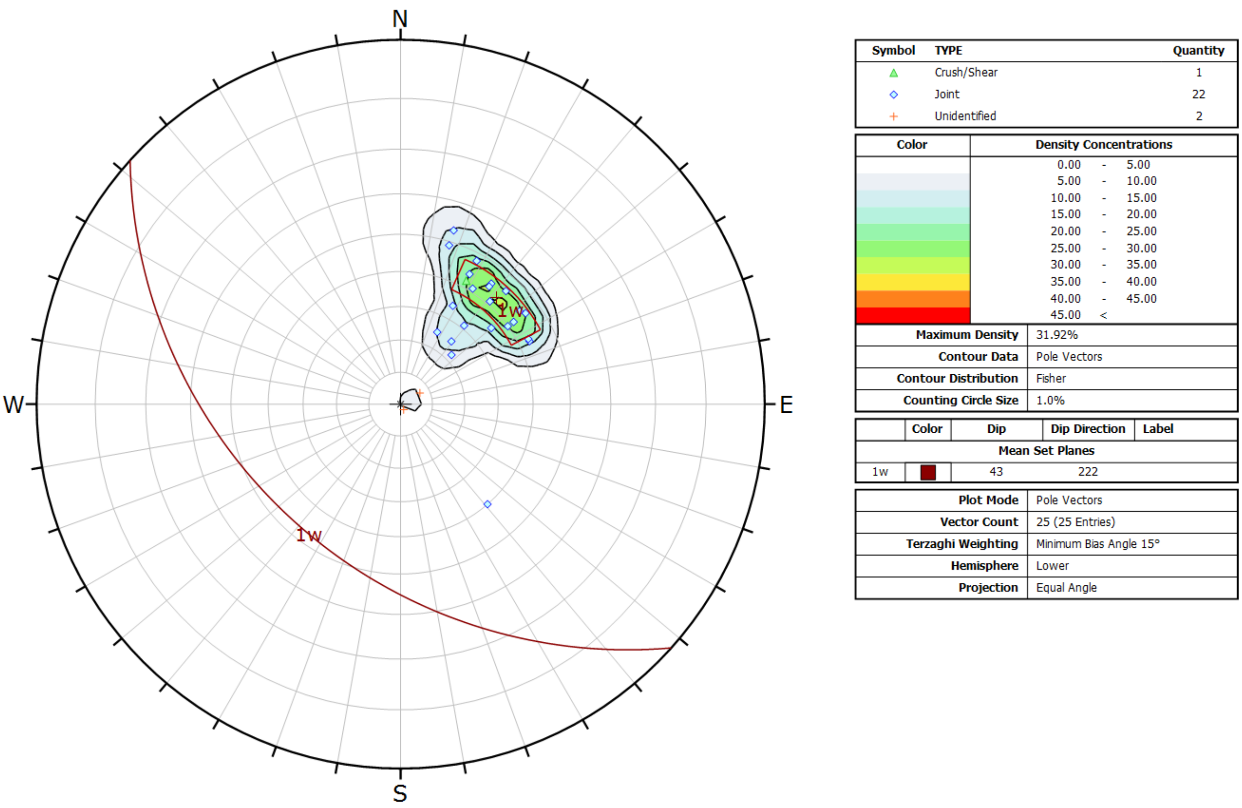


Figure 11.15 Stereonet plot of discontinuities in ECBF for BH219

11.14.2 Groundwater model

The piezometric surface at tunnel level and standing groundwater is expected to be at approximately 20mRL based on piezometers in BH221 and BH219. Higher groundwater flows are expected within basalt which occurs near the surface.

11.14.3 Contaminated land assessment

The proposed works for the construction of Access Shaft 1 on the main tunnel comprises site works for laydown areas and pavements as well as excavation of twin 9 m diameter shafts and a single 8.5 m shaft as well as excavations for a control chamber, manholes and trenched connections.

The previous desk top investigation indicated likely presence of fill material on the site as well as a former depot and workshop east of the construction site which housed an underground storage tank. CoPC identified for the site included heavy metals hydrocarbons and ACM.

The following observations were recorded during the intrusive sampling investigation:

- Fill from surface to a depth of 0.5 m was observed.
- No dumping or fly tipping was observed.
- PID soil readings ranged between 0.0 to 0.3 ppm.
- No groundwater was encountered during sampling.

The following table presents a summary of a comparison of soil sampling results to the adopted acceptance criteria. A full results table can be viewed as appended.

Table 11-37 : Comparison of soil sampling results to the adopted acceptance criteria (Mt Albert Memorial Reserve)

Contaminant of Potential Concern	Assessment Criteria			
	Schedule 10 Permitted Activity Criteria	Auckland Background Concentrations (non-volcanic)	MfE Guidelines	NES SCS
Heavy Metals	Exceedance detected for nickel.	All below.	NC.	All below.
SVOC / VOC Compounds	All below or NC.	All below or NC.	All below or NC.	All below or NC.
PAH Compounds	All below or NC.	All below or NC.	All below or NC.	All below or NC.
ACM	Not detected.			

NC = No Criteria.

- A comparison to the adopted acceptance criteria indicates that there are no exceedances of NES SCS for protection of human health.
- A single concentration of nickel was above the PA discharge criteria. The average concentration of nickel across the site was below the respective criteria.

- While these concentrations of CoPC present no immediate risk to human or the environment, the soil is likely suitable for disposal at a managed fill facility. A comparison to landfill criteria should be undertaken at the time of disposal.

Further inspection and/or testing should be undertaken where significant volumes of fill materials are encountered.

11.14.4 Geotechnical risks

Table 11-38 : Geotechnical risk (Mt Albert Memorial Reserve)

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
Extremely weak rock	Extremely weak rock can occur at any place within ECBF	Unstable excavation Shaft design not suitable for ground conditions	Moderately weathered to unweathered ECBF	Anywhere within ECBF
Parnell Volcaniclastic Conglomerate	Conglomerate may be encountered during excavation	Excavation requires change to construction methodology Reduced productivity and higher machine tool wear	Parnell Volcaniclastic Conglomerate, Moderately to Unweathered ECBF	Anywhere within Moderately to Unweathered ECBF
Variable basalt rock mass	Basalt has a high or low quality rock mass and can be dependent on fracture zones, vesicularity or be rubbly.	Additional loading on structure and support Basalt may be easier to excavate	Basalt	Within basalt
Groundwater inflow	Basalt is typically vesicular and jointing allowing high water inflows	Disruption to excavation work and may require pumping	Basalt	Within basalt
Wood fragments and logs	Wood fragments and logs may be encountered	Difficult conditions for excavation / constructability	Tauranga Group	Anywhere within Tauranga Group

11.15 DSCIN009 Western Springs

11.15.1 Geotechnical ground model

The geotechnical ground model is presented in geotechnical drawings DSCIN009 – Western Springs Shaft Geology (Dwg No. 2011887.001 to 2011887.004).

The geotechnical ground model for the shaft site is presented in Table 11-39 below and should be read in conjunction with the geological drawings due to the complex and variable ground conditions at this site.

There are variable thicknesses of made ground and basalt observed in investigations at the Western Springs site and the site is inferred to be the location of a historic quarry since infilled. The shaft location appears to cross over the margin of the historic quarry and also a paleo-channel of alluvium and tuff/ash between basalt lava flows. Backfill is assumed to be non-engineered. Undifferentiated Tauranga Group is beneath basalt and made ground, and is underlain by Waitemata group. The boundary between the two groups appears to dip to the south west.

Undifferentiated Tauranga Group is expected to range from soft to firm cohesive soil and loose to medium dense granular soil. Design of shaft structures should consider both material types to accommodate expected variability.

The geology is likely to be variable at other structures at the Western Springs site. To the east of the shaft, undifferentiated Tauranga Group thins and the extent of the Tauranga Group has been estimated in the geological plan (Dwg no. 2011887.001). Based on test pit WS1-TP06 the surrounding CPTs (WS1-CPT06, -CPT07, -CPT08) are assumed to have refused in ECBF rather than basalt.

The extent of the historic quarry site has been estimated on the plan and there is little evidence to delineate the quarry boundary.

Table 11-39 Geotechnical ground model for DSCIN009 – Western Springs Shaft Geology

From (mRL)	To (mRL)	Geotechnical Unit
Surface (12mRL)	11 to 7	Made Ground
7	3	Possibly localised Basalt
11 (3 where basalt occurs)	3 to 0	Undifferentiated Tauranga Group
3 to 0	1 to -1.5	Residually to Highly Weathered Cohesive soil of ECBF
5 to 1	> -30	Moderately Weathered to Unweathered ECBF

Discontinuities for BH205 and BH206 for ECBF are presented in Figure 11.16 and show bedding is gently inclined dipping to the north east. A set of joints is identified very steeply dipping to the east. Many of these joints were observed in BH205 between 17m to 19.5m bgl. Joints were also observed dipping in a range of inclinations to the east and west.

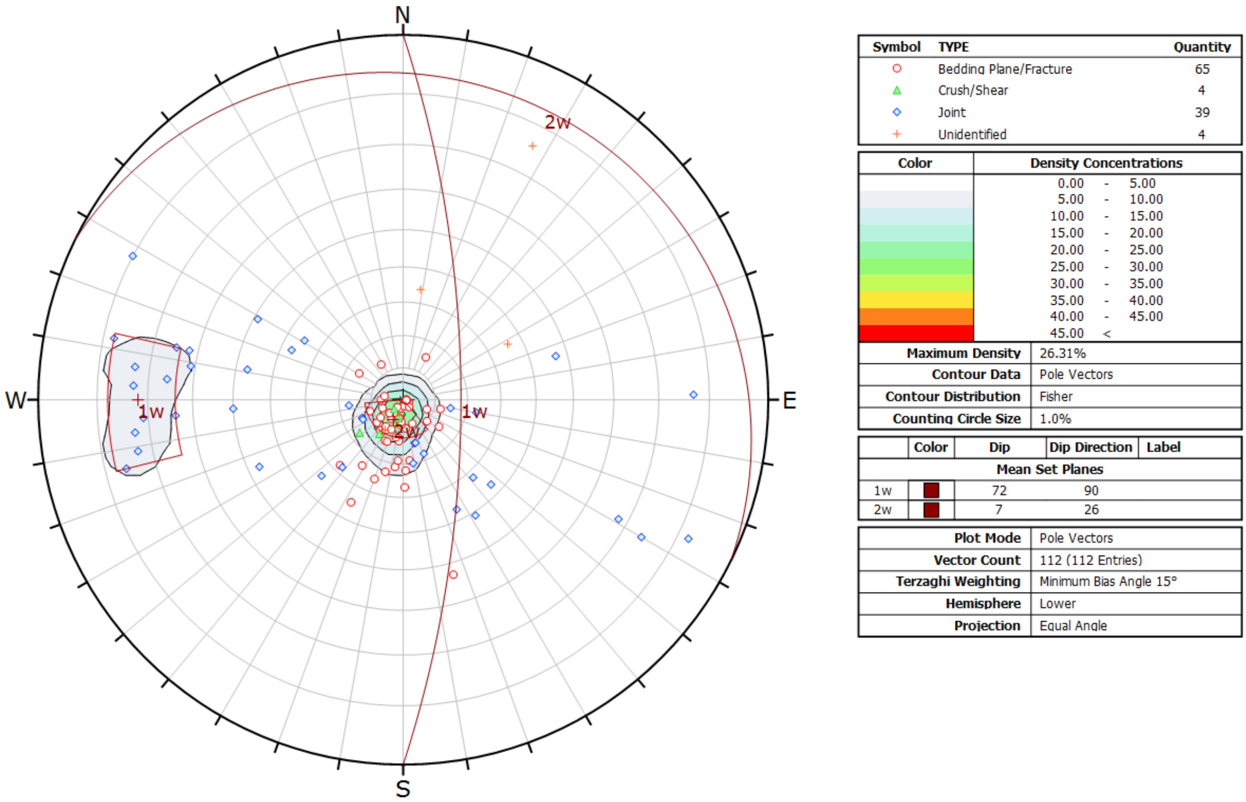


Figure 11.16 Stereonet plot of discontinuities in ECBF for BH205 and BH206

11.15.2 Groundwater model

Based on piezometers installed in BH206, BH205 and BH206b the inferred piezometric surface at tunnel level is approximately 12mRL. The groundwater table is likely to be encountered at or near the surface however it is noted during excavation of test pit WS1-TP05 water seepage was encountered at 7mRL (4.8m below ground level).

Discussion of the aquifer tests undertaken in this area are presented in Section 7.4.2.1 and in further detail in Appendix F.

11.15.3 Contaminated land assessment

The proposed works at the Western Springs include construction of Shaft DCSIN009 including construction of lay-down areas, pavement areas and excavation of a deep working shaft. Soil sampling undertaken as part of the assessment generally reflects the location of these works.

Previous desk top investigations indicated the presence of a former council depot and workshop bordering the east of the construction site with an underground storage tank with potential for some fill on the site. Potential contaminants of concern identified for the site included heavy metals, hydrocarbons and ACM.

The following observations were recorded during the intrusive sampling investigation:

- Fill material was observed from surface in the form of coarse gravel.
- A large pile of sand material had been dumped in the western corner of site.
- No evidence of fly tipping was observed.
- PID readings from surface soil were recorded at 0.0 ppm.

- Groundwater was not encountered during hand-auger advancement.
- No evidence of ACM was noted.

The following table presents a summary of a comparison of soil sampling results to the adopted acceptance criteria. A full results table can be viewed as appended.

Table 11-40 : Summary of a comparison of soil sampling result to the adopted acceptance criteria (Western Springs)

Contaminant of Potential Concern	Assessment Criteria			
	Schedule 10 Permitted Activity Criteria	Auckland Background Concentrations (non-volcanic)	MfE Guidelines	NES SCS
Heavy Metals	All below.	Exceedances detected for chromium, copper, lead, and nickel.	NC.	All below.
SVOC / VOC Compounds	All below or NC.	All below or NC.	All below or NC.	All below or NC
PAH Compounds	All below or NC.	All below or NC.	All below or NC.	All below or NC
Other organic compounds including DDT	All below or NC.	All below or NC.	NC.	All below or NC

NC = No Criteria.

- A comparison to the adopted acceptance criteria indicates that there are no exceedances of NES SCS for protection of human health.
- Select samples yielded concentrations of chromium, copper, lead and nickel above Auckland Background Concentrations. The average concentration of these heavy metals was below the adopted acceptance criteria.
- While these concentrations of CoPC present no immediate risk to human health or the environment, the soil is likely suitable for disposal at a managed fill facility. Comparison to landfill criteria should be undertaken at the time of disposal.

Further inspection and/or testing should be undertaken where significant volumes of fill material are encountered.

11.15.4 Geotechnical risks

Table 11-41 : Geotechncial risks (Western Springs)

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
Extremely weak rock	Extremely weak rock can occur at any place within ECBF	Unstable excavation Shaft design not suitable for ground conditions	Moderately weathered to unweathered ECBF	Anywhere within ECBF
Possible existing landfill	Appears an infilled quarry may exist on site and could be potential for contamination or obstructions	Obstructions encountered during construction Additional costs to dispose of contamination material	Made ground	Upper 10m of site
Parnell Volcaniclastic Conglomerate	Conglomerate may be encountered during excavation	Excavation requires change to construction methodology Reduced productivity and higher machine tool wear	Parnell Volcaniclastic Conglomerate, Moderately to Unweathered ECBF	Anywhere within Moderately to Unweathered ECBF
Variable basalt rock mass	Basalt has a high or low quality rock mass and can be dependent on fracture zones, vesicularity or be rubbly.	Additional loading on structure and support Basalt may be easier to excavate	Basalt	Within basalt
Groundwater inflow	Basalt is typically vesicular and jointing allowing high water inflows	Disruption to excavation work and may require pumping	Basalt	Within basalt
Variability of Ground Conditions	Unanticipated ground conditions due to complex geology at site	Different design construction methodologies by required during construction Design may not be appropriate for ground conditions	All	Entire site
Wood fragments and logs	Wood fragments and logs may be encountered	Difficult conditions for excavation / constructability	Tauranga Group	Anywhere within Tauranga Group

11.16 DSLSA001 Western Springs Depot

11.16.1 Contaminated land assessment

The proposed works at the Western Springs Park Depot have been eliminated as part of the preliminary design process. Soil sampling was undertaken prior to the scope change and is assessed below.

Previous desk top investigation studies indicated the presence of a former council depot and workshop bordering the east of the construction site with an underground storage tank. The desk top investigation also indicated some fill on the site. Potential contaminants of concern identified for the site included heavy metals, hydrocarbons and ACM.

Due to the presence of coarse gravel at shallow depths (less than 0.05 m) the hand auger boreholes were unable to be advanced to depth. No samples were collected at this site.

The following observations were recorded during the intrusive sampling investigation:

- Fill material was observed from surface in the form of coarse gravel.
- A large pile of sand material had been dumped in the western corner of site.
- No evidence of fly tipping was observed.
- PID readings from surface soil were recorded at 0.0 ppm.
- Groundwater was not encountered during hand-auger advancement.
- No evidence of ACM was noted.

Further inspection and/or testing should be undertaken where significant volumes of fill material are encountered during the ground disturbance works.

11.17 DSLSA002 Motions Road

11.17.1 Contaminated land assessment

The proposed works at the Motions Road have been eliminated as part of the preliminary design process. Soil sampling was undertaken prior to the scope change and is assessed below.

Previous desk top investigations indicated former use of the site as a landfill with CoPC identified for the site including heavy metals, hydrocarbons and asbestos containing materials (ACM).

The following observations were recorded during the intrusive sampling investigation.

- Waste / refuse material was encountered between 0.25 and 0.3 m. These materials comprised of glass, metal and gravel.
- PID headspace readings were recorded between 0.0 and 0.8 ppm (parts per million).
- No groundwater was encountered during sampling.
- No evidence of ACM was noted.

The following table presents a summary of a comparison of soil sampling results against the adopted acceptance criteria. A full results table can be viewed as appended.

Table 11-42 : Comparison of soil sampling results to the adopted acceptance criteria (Motions Road)

Contaminant of Potential Concern	Assessment Criteria			
	Schedule 10 Permitted Activity Criteria	Auckland Background Concentrations (volcanic)	MfE Guidelines	NES SCS
Heavy Metals	Exceedances detected for nickel.	Exceedances detected for arsenic and lead.	NC.	All below.
SVOC / VOC Compounds	All below or NC.	All below or NC.	All below or NC.	All below or NC.
PAH Compounds	All below or NC.	All below or NC.	All below or NC.	All below or NC.
Other organic compounds including DDT	All below or NC.	All below or NC.	NC.	All below or NC.

NC = No Criteria Available.

- A comparison to the adopted acceptance criteria indicates that there are no exceedances of NES SCS for protection of human health.
- Select samples yielded concentrations of arsenic, lead, and nickel above either the Schedule 10 Criteria or Auckland Background Concentrations.
  - The average concentrations of arsenic and nickel were below the adopted acceptance criteria.
  - The average concentration of lead was above the Auckland Background Concentrations.
- While below the adopted acceptance criteria, it is noted that a concentrations of PAH compounds are present in shallow soil.
- While these concentrations of CoPC present no immediate risk to human health or the environment, the soil is likely suitable for disposal at a managed fill facility. Comparison to landfill criteria should be undertaken at the time of disposal.
- Further inspection and/or testing should be undertaken where significant volumes of fill material are encountered during the works.

11.18 DSLSB001 Norgrove Avenue

11.18.1 Geotechnical ground model

The geotechnical ground model is presented in geotechnical drawings DSLSB001 – Norgrove Avenue Shaft Geology (DWG No. 2011953.001 to 2011953.004).

The geotechnical ground model for the shaft site is presented in Table 11-43. Undifferentiated Tauranga Group is observed to -4mRL and overlies a layer of residually to highly weathered ECBF. Moderately weathered to unweathered ECBF is encountered at -5.6mRL. Undifferentiated Tauranga Group is expected to range from soft to firm cohesive soil and loose to medium dense granular soil. Design of shaft structures should consider both material types to accommodate expected variability.

Table 11-43 Geotechnical ground model for DSLSB001 – Norgrove Avenue Shaft Geology

From (mRL)	To (mRL)	Geotechnical Unit
Surface: (14.5mRL)	12.9	Made Ground
12.9	-4.0	Undifferentiated Tauranga Group
-4.0	-5.6	Residually to highly weathered ECBF
-5.6	> -15	Moderately Weathered to Unweathered ECBF

Discontinuities for BH217 for ECBF are presented in Figure 11.17. Relatively few discontinuities were observed during televiewer logging and drill core logging. Due to turbid drillhole conditions optical televiewer was not undertaken. Contours have not been fitted due to sparsity of the data. In drillhole core logging rock was either massive or had indistinct bedding with few defects recorded.

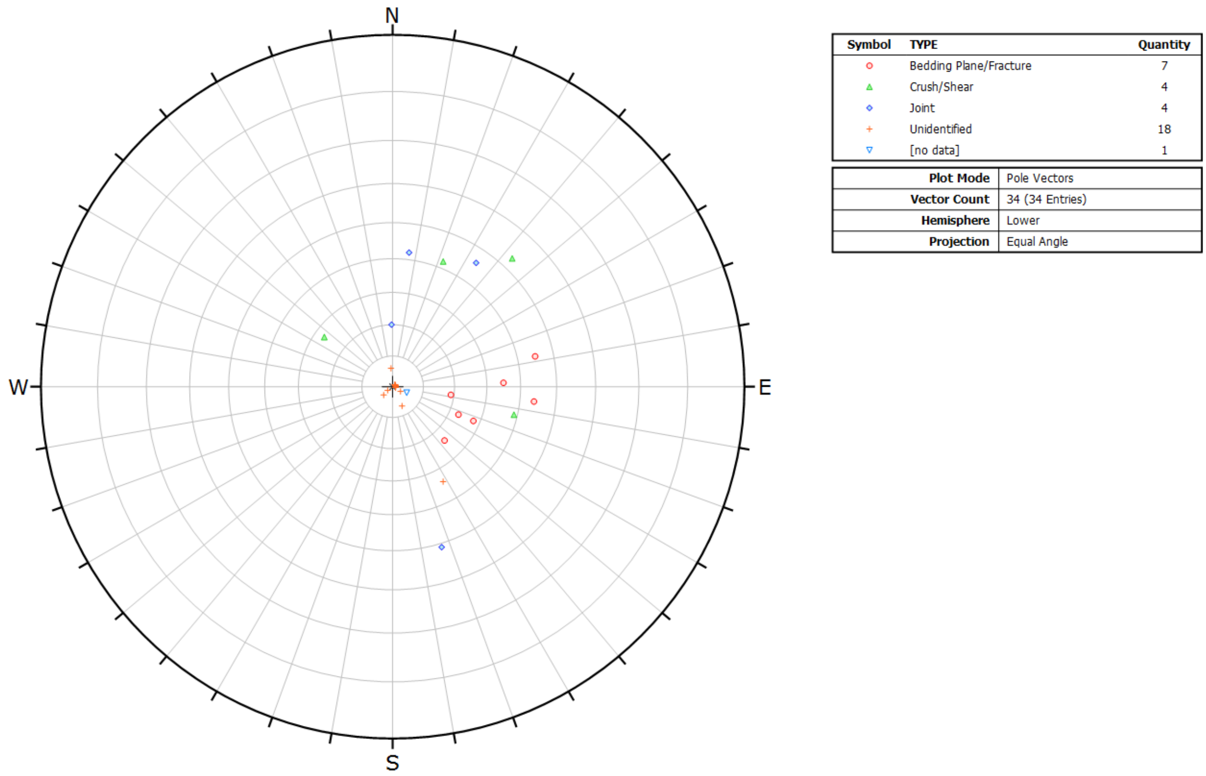


Figure 11.17 Stereonet of discontinuities in ECBF for BH217

11.18.2 Groundwater model

Based on piezometers in BH217 and BH218, the piezometric surface at tunnel level and expected groundwater level is 12.5mRL.

11.18.3 Contaminated land assessment

The proposed works for the construction of Shaft 2 on Link Sewer 2 comprise site works for laydown areas and pavements and excavation of a deep shaft respective control chamber as well as trenched connections.

Previous desk top investigations indicated no potential contamination of the road or reserve portion of the site. Therefore, only two locations related to areas of disturbance; the scheduled geotechnical borehole at the shaft and an additional hand-auger at the control chamber. CoPC identified for the site included heavy metals and pesticides.

The following site observations were recorded during the intrusive sampling investigation:

- No fill material was encountered at site.
- No evidence of dumping or fly tipping was observed.
- The PID background reading was 0.7 ppm, with soil readings between 0.2 and 0.6 ppm.
- No groundwater was encountered during sampling.
- ACM was not observed.

The following table presents a summary of a comparison of soil sampling results to human health and environmental screening criteria and guidelines. A full results table can be viewed as appended.

Table 11-44 : Comparison of soil sampling results to the adopted acceptance criteria (Norgrove Road)

Contaminant of Potential Concern	Assessment Criteria		
	Schedule 10 Permitted Activity Criteria	Auckland Background Concentrations (non-volcanic)	NES SCS
Heavy Metals	All below.	Exceedances detected for lead.	All below.
OCP Compounds	All below or NC.	All below or NC.	All below or NC.

NC = No Criteria.

- A comparison to adopted acceptance criteria indicates that there are no exceedances of NES SCS for protection of human health.
- The average concentration of lead across the site is above Auckland Background Concentrations.
- While these concentrations of CoPC present no immediate risk to human or the environment, the soil is likely suitable for disposal at a managed fill facility. Comparison to managed fill criteria should be undertaken at the time of disposal.

Further inspection and/or testing should be undertaken where significant volumes of fill material are encountered.

11.18.4 Geotechnical risks

Table 11-45 : Geotechnical risks (Norgrove Road)

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
Extremely weak rock	Extremely weak rock can occur at any place within ECBF	Unstable excavation Shaft design not suitable for ground conditions	Moderately weathered to unweathered ECBF	Anywhere within ECBF
Parnell Volcaniclastic Conglomerate	Conglomerate may be encountered during excavation	Excavation requires change to construction methodology Reduced productivity and higher machine tool wear	Parnell Volcaniclastic Conglomerate, Moderately to Unweathered ECBF	Anywhere within Moderately to Unweathered ECBF
Wood fragments and logs	Wood fragments and logs may be encountered	Difficult conditions for excavation / constructability	Tauranga Group	Anywhere within Tauranga Group

11.19 DSLSB002 Rawalpindi Reserve

11.19.1 Geotechnical ground model

The geotechnical ground model is presented in geotechnical drawings DSLSB002 – Rawalpindi Reserve Shaft Geology (DWG No. 2011954.001 to 2011954.004).

The geotechnical ground model for the shaft site is presented in Table 11-46. Made ground is expected from the surface (12.87mRL) to approximately 11mRL, and could vary across the site. Residual soil to highly weathered ECBF underlies made ground and extends to approximately 3.2mRL where moderately weathered to unweathered ECBF is encountered. To the east of the shaft, alluvium is expected to be encountered overlying residual ECBF soil.

Residual soil to highly weathered ECBF is expected to range from soft to hard cohesive soil and loose to very dense granular soil. Design of shaft structures should consider both material types to accommodate expected variability.

Table 11-46 Geotechnical ground model for DSLSB002 – Rawalpindi Reserve Shaft Geology

From (mRL)	To (mRL)	Geotechnical Unit
Surface (12.9mRL)	11	Made Ground
10.9	3.2	Residually to Highly Weathered Cohesive soil of ECBF
3.2	> -17	Moderately Weathered to Unweathered ECBF

Discontinuities for BH215 are presented in Figure 11.18 and bedding is moderately inclined dipping to the northwest. Several very steeply inclined joints dipping to the east have also been observed in televiewer and core logging.

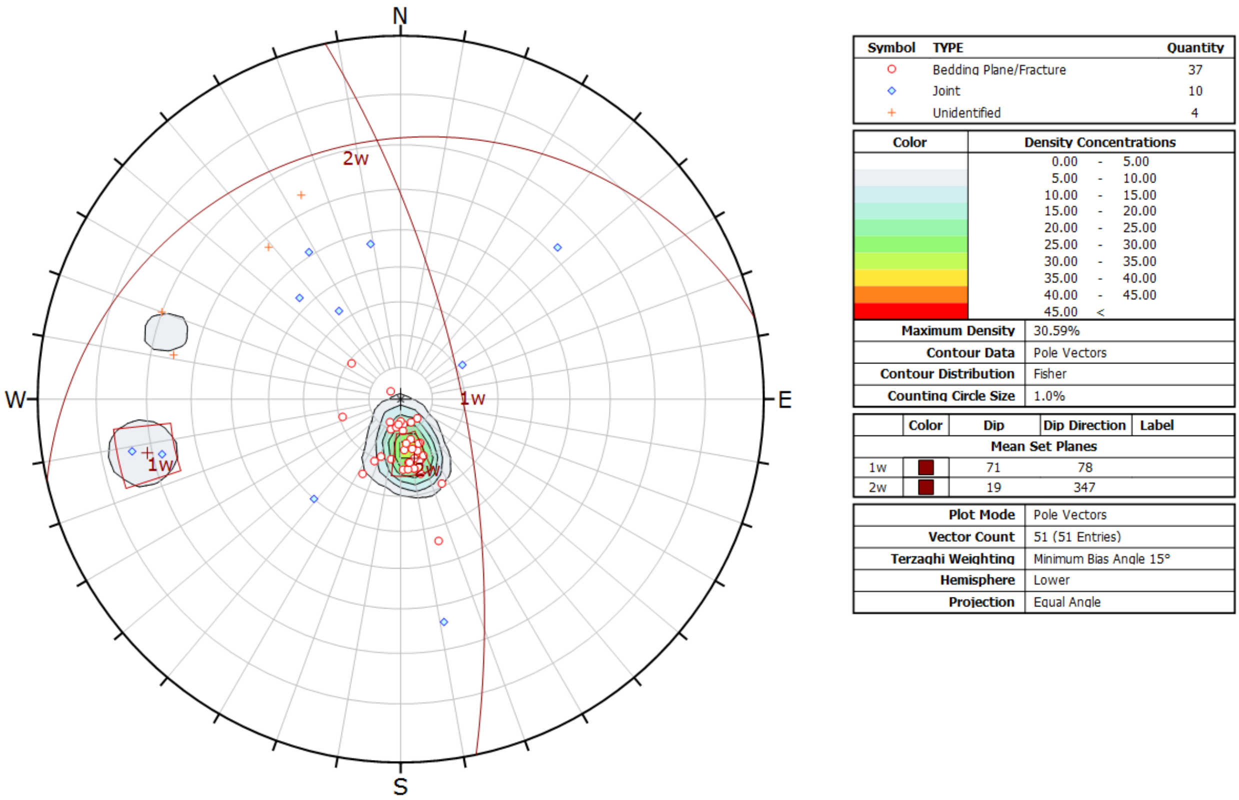


Figure 11.18 Stereonet of discontinuities in ECBF for BH215

11.19.2 Groundwater model

Ground water level is based on piezometers in BH216 and BH215 is 10mRL. There are no piezometers installed at tunnel level however the piezometric surface at tunnel level taken to be 10mRL also.

11.19.3 Contaminated land assessment

The proposed works for the construction of Shaft 1 on Link Sewer 2 comprise site works for lay-down areas and pavements, excavation of a deep shaft as well as diversion and control chambers plus trenched connections to Branch Sewer 8.

Previous desk top investigations indicated that the site has been part of a reserve since 1940 with no significant visible change over time. Potential contaminants of concern identified for the site included heavy metals and nitrogen compounds.

The following observations were recorded during the intrusive sampling investigation:

- Disturbed fill material observed from surface to depths between 3.6 and 5 m.
- No dumping or fly tipping observed.
- PID soil readings ranged between 0.0 and 0.7 ppm.

- No groundwater was encountered during sampling.
- ACM was not observed.

The following table presents a summary of a comparison of soil sampling results against the adopted acceptance criteria. A full results table can be viewed as appended.

Table 11-47 : Comparison of soil sampling results to the adopted acceptance criteria (Rawalpindi Reserve)

Contaminant of Potential Concern	Assessment Criteria		
	Schedule 10 Permitted Activity Criteria	Auckland Background Concentrations (non-volcanic)	NES SCS
Heavy Metals	All below.	Exceedances detected for nickel.	All below.

NC = No Criteria.

- A comparison to the adopted acceptance criteria indicates that there are no exceedances of NES SCS for protection of human health.
- A single concentration of nickel was recorded above the Auckland Background Concentrations. The average concentration of nickel across the site was below the adopted acceptance criteria.

While these concentrations of CoPC present no immediate risk to human, the soil is likely suitable for disposal at a cleanfill or managed fill facility. A comparison to landfill criteria should be undertaken at the time of disposal.

11.19.4 Geotechnical risks

Table 11-48 : Geotechnical risks (Rawalpidini Reserve)

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
Extremely weak rock	Extremely weak rock can occur at any place within ECBF	Unstable excavation Shaft design not suitable for ground conditions	Moderately weathered to unweathered ECBF	Anywhere within ECBF
Parnell Volcaniclastic Conglomerate	Conglomerate may be encountered during excavation	Excavation requires change to construction methodology Reduced productivity and higher machine tool wear	Parnell Volcaniclastic Conglomerate, Moderately to Unweathered ECBF	Anywhere within Moderately to Unweathered ECBF

11.20 DSLSC001 Haycock Avenue

11.20.1 Geotechnical ground model

The geotechnical ground model is presented in geotechnical drawings DSLSC001 – Haycock Avenue Shaft Geology (DWG No. 2012006.001 to 2012006.004).

The geotechnical ground model for the shaft site is presented in Table 11-49. Beds of Parnell Volcaniclastic Conglomerate up to 1m thick are observed in BH242 at 11.5mRL and 1mRL.

Undifferentiated Tauranga Group is expected to range from soft to firm cohesive soil and loose to medium dense granular soil. Design of shaft structures should consider both material types to accommodate expected variability.

Table 11-49 Geotechnical ground model for DSLSC001 – Haycock Avenue Shaft Geology

From (mRL)	To (mRL)	Geotechnical Unit
Surface (24.8mRL)	23.5	Made Ground
23.5	15	Undifferentiated Tauranga Group
15	14	Residually to Highly Weathered Cohesive soil of ECBF
14	> -19	Moderately Weathered to Unweathered ECBF

Discontinuities for BH244 are presented in Figure 11.19. A set comprising the majority of discontinuities is identified gently inclined (average dip 12°) dipping to the west and two sets of joints at 72/125 and 47/059. Jointing is also observed dipping to the northwest between 40 to 60°.

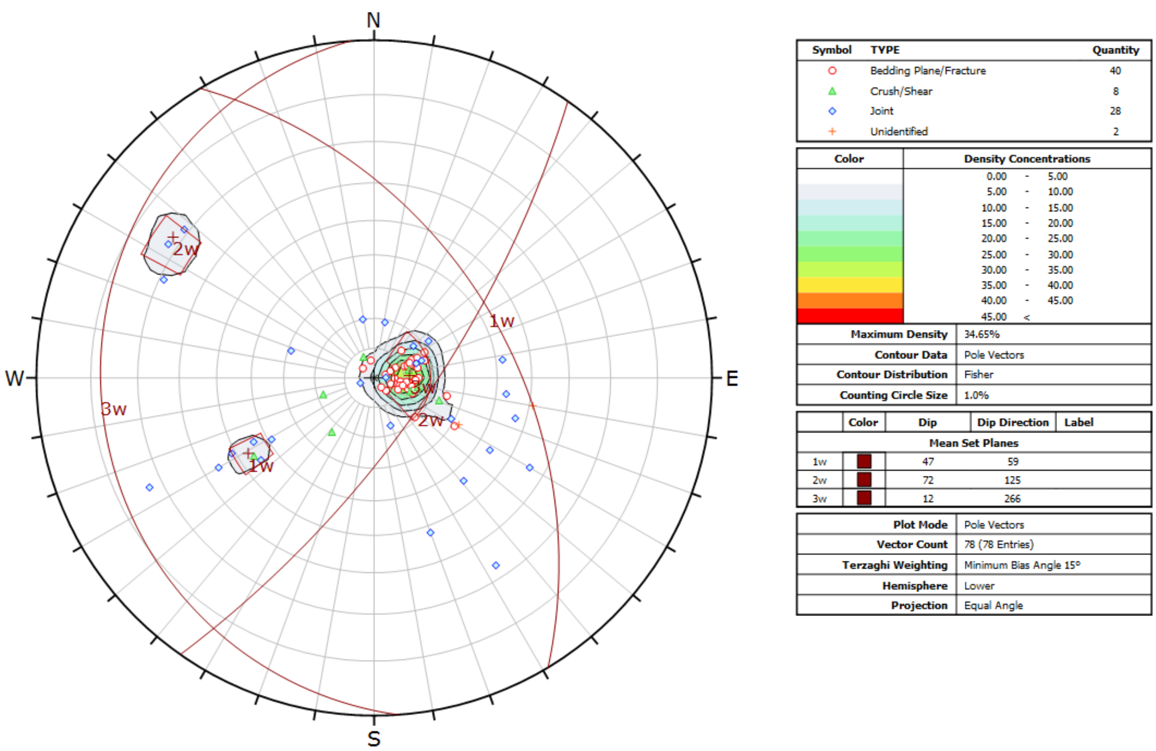


Figure 11.19 Stereonet of discontinuities in ECBF for BH244

11.20.2 Groundwater model

Based on piezometers in BH244 the piezometric surface at tunnel level is 28.5mRL and piezometric head increases with depth. Groundwater is likely to be encountered within 1m depth of ground surface based on piezometers in BH243 and hand auger hole L3S5 – HA01.

11.20.3 Contaminated land assessment

The proposed works for the construction of Shaft 5 on Link Sewer 3 comprises demolition of the existing house, site works for laydown areas and pavements, excavation of an 8.5 m diameter deep shaft. Excavations are also required for connection chambers, manholes and trenched connections.

Previous desk top study found no information indicating potential contamination at the site.

The following observations were recorded during the intrusive sampling investigation:

- Fill material were observed to 1.1 m bgl.
- No evidence of dumping or fly tipping was observed.
- PID readings in soil samples did not exceed 0.0 ppm.
- Groundwater was not encountered during sampling.
- ACM was not observed.

The following table presents a summary of a comparison of soil sampling results to the adopted acceptance criteria. A full results table can be viewed as appended.

Table 11-50 : Comparison of soil sampling results to the adopted acceptance criteria (Haycock Avenue))

Contaminant of Potential Concern	Assessment Criteria		
	Schedule 10 Permitted Activity Criteria	Auckland Background Concentrations (non-volcanic)	NES SCS
Heavy Metals	All below.	All below.	All below.
Other organic compounds including DDT	All below or NC.	All below or NC.	All below or NC
ACM	Not present.		

NC = No Criteria.

- A comparison to the adopted acceptance criteria indicates that there are no exceedances of NES SCS for protection of human health.

The soil is likely suitable for disposal at a cleanfill facility. A comparison to landfill criteria should be undertaken at the time of disposal.

11.20.4 Geotechnical risks

Table 11-51 : Geotechnical risks (Haycock Avenue)

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
Extremely weak rock	Extremely weak rock can occur at any place within ECBF	Unstable excavation Shaft design not suitable for ground conditions	Moderately weathered to unweathered ECBF	Anywhere within ECBF
Parnell Volcaniclastic Conglomerate	Conglomerate may be encountered during excavation	Excavation requires change to construction methodology Reduced productivity and higher machine tool wear	Parnell Volcaniclastic Conglomerate, Moderately to Unweathered ECBF	Anywhere within Moderately to Unweathered ECBF
Wood fragments and logs	Wood fragments and logs may be encountered	Difficult conditions for excavation / constructability	Tauranga Group	Anywhere within Tauranga Group

11.21 DSLSC002 Dundale Avenue

11.21.1 Geotechnical ground model

The geotechnical ground model is presented in geotechnical drawings DSLSC002 – Dundale Avenue Shaft Geology (DWG No. 2012007.001 to 2012007.004).

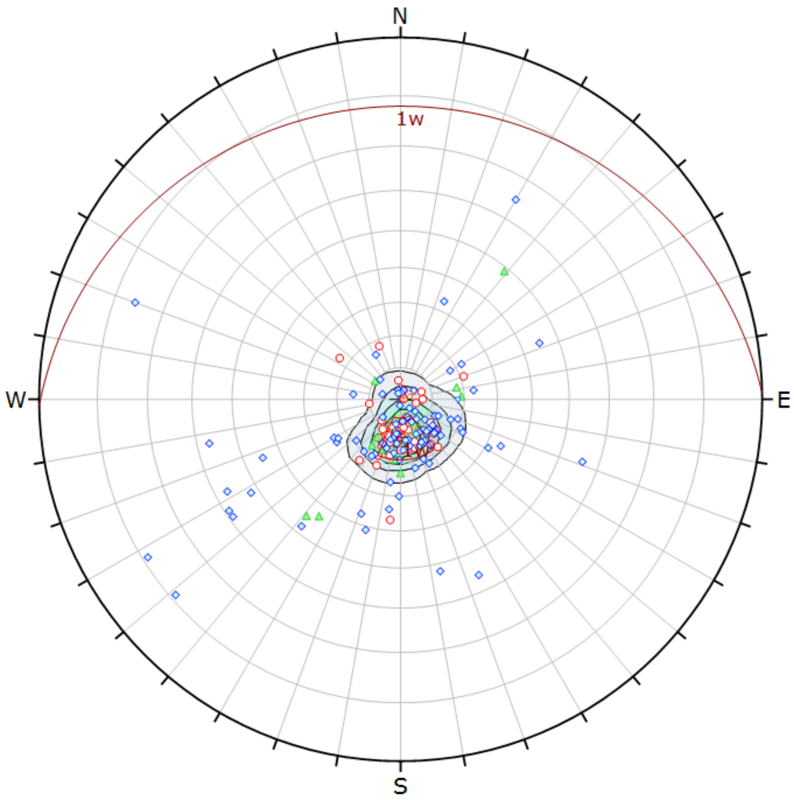
The geotechnical ground model for the shaft site is presented in Table 11-52. Undifferentiated Tauranga Group underlies made ground and extends to 15.8mRL. Residually to highly weathered cohesive soil of ECBF underlies Tauranga Group and overlies moderately weathered to unweathered ECBF at to 11.3mRL.

Undifferentiated Tauranga Group is expected to range from soft to firm cohesive soil and loose to medium dense granular soil. Design of shaft structures should consider both material types to accommodate expected variability.

Table 11-52 Geotechnical ground model for DSLSC002 – Dundale Avenue Shaft Geology

From (mRL)	To (mRL)	Geotechnical Unit
Surface (20.2mRL)	17	Made Ground
17	15.8	Undifferentiated Tauranga Group
15.8	11.3	Residually to Highly Weathered Cohesive soil of ECBF
11.3	> 29	Moderately Weathered to Unweathered ECBF

Discontinuities for BH240 are presented in Figure 11.20 and are typically gently inclined dipping to the north (12/359). Steeply inclined joints up to 80° dip have been observed in televiewer and core logging with a range of dip directions.



Symbol	TYPE	Quantity
○	Bedding Plane/Fracture	46
△	Crush/Shear	13
◇	Joint	89
+	Unidentified	2
▽	[no data]	1

Color	Density Concentrations
	0.00 - 5.00
	5.00 - 10.00
	10.00 - 15.00
	15.00 - 20.00
	20.00 - 25.00
	25.00 - 30.00
	30.00 - 35.00
	35.00 - 40.00
	40.00 - 45.00
	45.00 <
Maximum Density	28.74%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

	Color	Dip	Dip Direction	Label
Mean Set Planes				
1w		12	359	

Plot Mode	Pole Vectors
Vector Count	151 (151 Entries)
Terzaghi Weighting	Minimum Bias Angle 15°
Hemisphere	Lower
Projection	Equal Angle

Figure 11.20 Stereonet of discontinuities in ECBF for BH240

11.21.2 Groundwater model

The groundwater level expected from surface and the piezometric surface at tunnel level are both 21mRL based on piezometers installed in BH240 and BH241.

11.21.3 Contaminated land assessment

The proposed works for the construction of Shaft 4 on Link Sewer 3 comprises site works to construct laydown areas and pavements and excavation of a 10m diameter deep shaft.

The Desk Study found no information indicating potential contamination at the site.

The following observations were made during the intrusive sampling investigation:

- Clean fill observed to a depth of 1.2 m bgl.
- No evidence of dumping or fly tipping was observed.
- PID readings in soils ranged from 0.2 to 0.5 ppm.
- Groundwater was not encountered during sampling.
- ACM was not observed.

The following table presents a summary of a comparison of soil sampling results to the adopted acceptance criteria. A full results table can be viewed as appended.

Table 11-53 : Comparison of soil sampling results to the adopted acceptance criteria (Dundale Avenue))

Contaminant of Potential Concern	Assessment Criteria		
	Schedule 10 Permitted Activity Criteria	Auckland Background Concentrations (non-volcanic)	NES SCS
Heavy Metals	All below.	Exceedance detected for arsenic and lead.	All below.
Other organic compounds including DDT	All below or NC.	All below or NC.	All below or NC.

NC = No Criteria.

- A comparison to the adopted acceptance criteria indicates that there are no exceedances of NES SCS for protection of human health
- Select samples yielded concentrations of arsenic and lead above the Auckland Background Concentration.
  - The average concentration of arsenic is below the adopted acceptance criteria.
  - The average concentration of lead was above the adopted acceptance criteria.

While these concentrations of CoPC present no immediate risk to human or human health, the soil is likely suitable for disposal at a managed fill facility. A comparison to landfill criteria should be undertaken at the time of disposal.

11.21.4 Geotechnical risks

Table 11-54 : Geotechncial risks (Dundale Avenue)

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
Extremely weak rock	Extremely weak rock can occur at any place within ECBF	Unstable excavation Shaft design not suitable for ground conditions	Moderately weathered to unweathered ECBF	Anywhere within ECBF
Parnell Volcaniclastic Conglomerate	Conglomerate may be encountered during excavation	Excavation requires change to construction methodology Reduced productivity and higher machine tool wear	Parnell Volcaniclastic Conglomerate, Moderately to Unweathered ECBF	Anywhere within Moderately to Unweathered ECBF
Wood fragments and logs	Wood fragments and logs may be encountered	Difficult conditions for excavation / constructability	Tauranga Group	Anywhere within Tauranga Group

11.22 DSLSC003 Whitney Avenue

11.22.1 Geotechnical ground model

The geotechnical ground model is presented in geotechnical drawings DSLSC003 – Whitney Street Shaft Geology (DWG No. 2012008.001 to 2012008.004).

The geotechnical ground model for the shaft site is presented in Table 11-55. The ground level slopes at this site and it is inferred made ground is approximately 1m thick from ground level. Undifferentiated Tauranga Group is observed to thin towards the north east and the boundary between it and underlying ECBF is anticipated to be higher in elevation in the north east half of the shaft.

Undifferentiated Tauranga Group is expected to range from soft to firm cohesive soil and loose to medium dense granular soil. Design of shaft structures should consider both material types to accommodate expected variability.

Table 11-55 Geotechnical ground model for DSLSC003 – Whitney Street Shaft Geology

From (mRL)	To (mRL)	Geotechnical Unit
Surface (27mRL)	25.5 to 26	Made Ground
25.5 to 26	23 to 24	Undifferentiated Tauranga Group
23 to 24	20.5 to 19.5	Residually to Highly Weathered Cohesive soil of ECBF
20.5 to 19.5	> -42	Moderately Weathered to Unweathered ECBF

Discontinuities for BH239 are presented in Figure 11.21 and are moderately inclined (dip 20 to 30°) dipping to the northwest. Only few steeply inclined joints were observed and these had variable dipping direction.

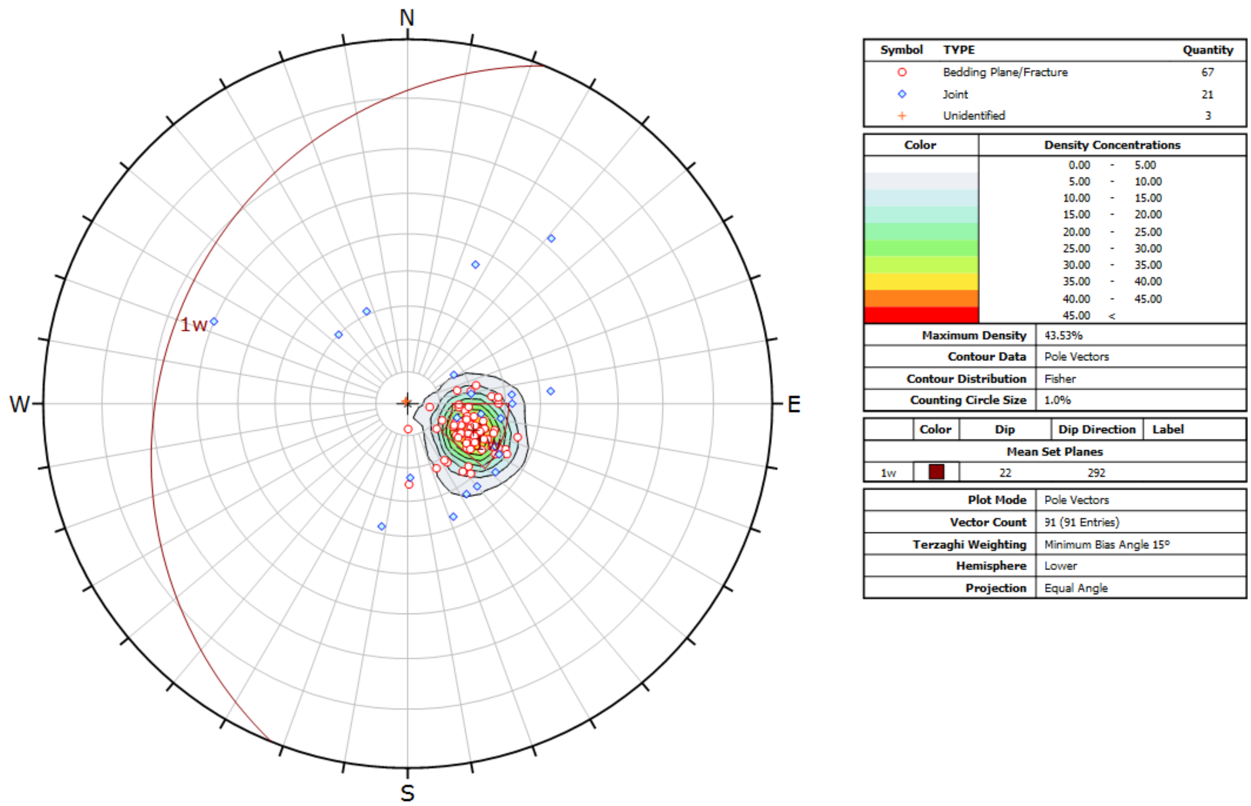


Figure 11.21 Stereonet of discontinuities in ECBF for BH239

11.22.2 Groundwater model

Vibrating wire piezometers were installed in CI-12 (A Matakite investigation) and observations from monitoring in 2010 indicate piezometric head is approximately 25mRL. CI-12 is approximately 50m down the valley from the shaft site and the groundwater model for the site has been inferred from this data. The piezometric head at tunnel level and groundwater expected from surface is 27mRL.

11.22.3 Contaminated land assessment

The proposed works for the construction of Shaft 3 on Link Sewer 3 comprises site works in the road reserve to construct lay-down areas, excavation of a single deep shaft. Excavations are also required for a manhole and trenched connections to the Avondale Branch Diversion Sewer.

The following observations were noted during the intrusive sampling investigation:

- No fill was observed to a depth of 0.2 m bgl.
- No evidence of dumping or fly tipping was observed.
- PID readings in soils ranged from 0.0 to 0.2 ppm.
- Groundwater was not encountered during sampling.
- ACM was not observed.

The following table presents a summary of a comparison of soil sampling results to the adopted acceptance criteria. A full results table can be viewed as appended.

Table 11-56 : Comparison of soil sampling results to the adopted acceptance criteria (Whitney Avenue))

Contaminant of Potential Concern	Assessment Criteria			
	Schedule 10 Permitted Activity Criteria	Auckland Background Concentrations (non- volcanic)	MfE Guidelines	NES SCS
Heavy Metals	All below.	Exceedance detected for lead.	NC.	All below.
SVOC / VOC Compounds	All below or NC.	All below or NC.	All below or NC.	All below or NC
PAH Compounds	All below or NC.	All below or NC.	All below or NC.	All below or NC
Other organic compounds including DDT	All below or NC.	All below or NC.	NC.	All below or NC

NC = No Criteria.

- A comparison to the adopted acceptance criteria indicates that there are no exceedances of NES SCS for protection of human health.
- The single soil sample analysed from the site exceeded the Auckland Background Concentration for lead.
- While below respective human health and environmental criteria, it is noted that a concentrations of select PAH compounds are present in shallow soil.

Further inspection and/or testing should be undertaken where significant volumes of fill material are encountered during the works. Without further testing the soil is likely suitable for disposal at a managed fill facility. A comparison to landfill criteria should be undertaken at the time of disposal.

11.22.4 Geotechnical risks

Table 11-57 : Geotechnical risks (Whitney Avenue)

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
Extremely weak rock	Extremely weak rock can occur at any place within ECBF	Unstable excavation Shaft design not suitable for ground conditions	Moderately weathered to unweathered ECBF	Anywhere within ECBF
Parnell Volcaniclastic Conglomerate	Conglomerate may be encountered during excavation	Excavation requires change to construction methodology Reduced productivity and higher machine tool wear	Parnell Volcaniclastic Conglomerate, Moderately to Unweathered ECBF	Anywhere within Moderately to Unweathered ECBF

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
Wood fragments and logs	Wood fragments and logs may be encountered	Difficult conditions for excavation / constructability	Tauranga Group	Anywhere within Tauranga Group

11.23 DSLSC004 Miranda Avenue

11.23.1 Geotechnical ground model

The geotechnical ground model is presented in geotechnical drawings DSLSC004 – Miranda Reserve Shaft Geology (DWG No. 2012009.001 to 2012009.004).

The geotechnical ground model for the shaft site is presented in Table 11-58. Observed in drillholes BH237 and BH238, it is anticipated Residually to Highly Weathered Granular Soil will vary between 1.5m to 4m thick, increasing to the west.

Undifferentiated Tauranga Group is expected to range from soft to firm cohesive soil and loose to medium dense granular soil. Design of shaft structures should consider both material types to accommodate expected variability.

Table 11-58 Geotechnical ground model for DSLSC004 – Miranda Reserve Shaft Geology

From (mRL)	To (mRL)	Geotechnical Unit
Surface (11.6mRL)	11.5	Made Ground
11.5	4	Undifferentiated Tauranga Group
4	2.5 to 0	Residually to Highly Weathered Granular Soil of ECBF
2.5 to 0	> -28	Moderately Weathered to Unweathered ECBF

No televiewer logging was undertaken near DSLSC004. Drill core logging in BH237 and BH238 observed gently to moderately inclined bedding and steeply to very steeply inclined joints. Joints are closely spaced in widely spaced zones.

11.23.2 Groundwater model

Vibrating wire piezometers are installed in BH238 and results indicate the piezometric surface at tunnel level is 10.8mRL. Based on piezometers in BH238 and BH237 groundwater is expected to be encountered at 11mRL, approximately 1m depth from ground surface.

11.23.3 Contaminated land assessment

The proposed works for the construction of Shaft 2 on Link Sewer 3 comprises site works to construct laydown areas and permanent pavements and excavation of a 10 m diameter deep shaft.

Previous desk top investigations indicated the site has been part of a reserve since 1940. CoPC for the site include heavy metals and pesticides.

The following observations were noted during the intrusive sampling investigation:

- Clean fill was noted to a depth of 1.2 to 2.8 m bgl.
- No evidence of dumping or fly tipping was observed.
- PID readings in soil samples did not exceed 0.0 ppm.
- Groundwater was not encountered during sampling.
- ACM was not observed.

The following table presents a summary of a comparison of soil sampling results to the adopted acceptance criteria. A full results table can be viewed as appended.

Table 11-59 : Comparison of soil sampling results to the adopted acceptance criteria (Miranda Avenue))

Contaminant of Potential Concern	Assessment Criteria		
	Schedule 10 Permitted Activity Criteria	Auckland Background Concentrations (non-volcanic)	NES SCS
Heavy Metals	All below.	All below.	All below.
OCP Compounds	All below or NC.	All below or NC.	All below or NC.

NC = No Criteria.

- A comparison to the adopted acceptance criteria indicates that there are no exceedances of NES SCS for protection of human health.

The soil is likely suitable for disposal at cleanfill facility. A comparison to landfill criteria should be undertaken at the time of disposal.

11.23.4 Geotechnical risks

Table 11-60 : Geotechnical risks (Miranda Avenue)

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
Extremely weak rock	Extremely weak rock can occur at any place within ECBF	Unstable excavation Shaft design not suitable for ground conditions	Moderately weathered to unweathered ECBF	Anywhere within ECBF
Parnell Volcaniclastic Conglomerate	Conglomerate may be encountered during excavation	Excavation requires change to construction methodology Reduced productivity and higher machine tool wear	Parnell Volcaniclastic Conglomerate, Moderately to Unweathered ECBF	Anywhere within Moderately to Unweathered ECBF
Wood fragments and logs	Wood fragments and logs may be	Difficult conditions for excavation / constructability	Tauranga Group	Anywhere within Tauranga Group

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
	encountered			

11.24 DSLSC005 PS25

11.24.1 Geotechnical ground model

The geotechnical ground model is presented in geotechnical drawings DSLSC005 – PS25 Shaft Geology (DWG No. 2012010.001 to 2012010.004).

The geotechnical ground model for the shaft site is presented in Table 11-61. Undifferentiated Tauranga Group is expected to range from soft to firm cohesive soil and loose to medium dense granular soil. Design of shaft structures should consider both material types to accommodate expected variability. Beneath Tauranga Group, Residually to highly weathered cohesive soil extends from 0mRL to -5mRL and overlies ECBF rock.

Table 11-61 Geotechnical ground model for DSLSC005 – PS25 Shaft Geology

From (mRL)	To (mRL)	Geotechnical Unit
Surface (11mRL)	8.5	Made Ground
8.5	0	Undifferentiated Tauranga Group
0	-5	Residually to Highly weathered cohesive soil
-5	> -42	Moderately Weathered to Unweathered ECBF

Discontinuities for BH236 are presented in Figure 11.22. Bedding is gently to moderately inclined dipping broadly to the north. Joints are moderately to steeply inclined with a set identified dipping to the north (30/001). Many other joints are observed dipping to the northeast to northwest or southeast to southwest.

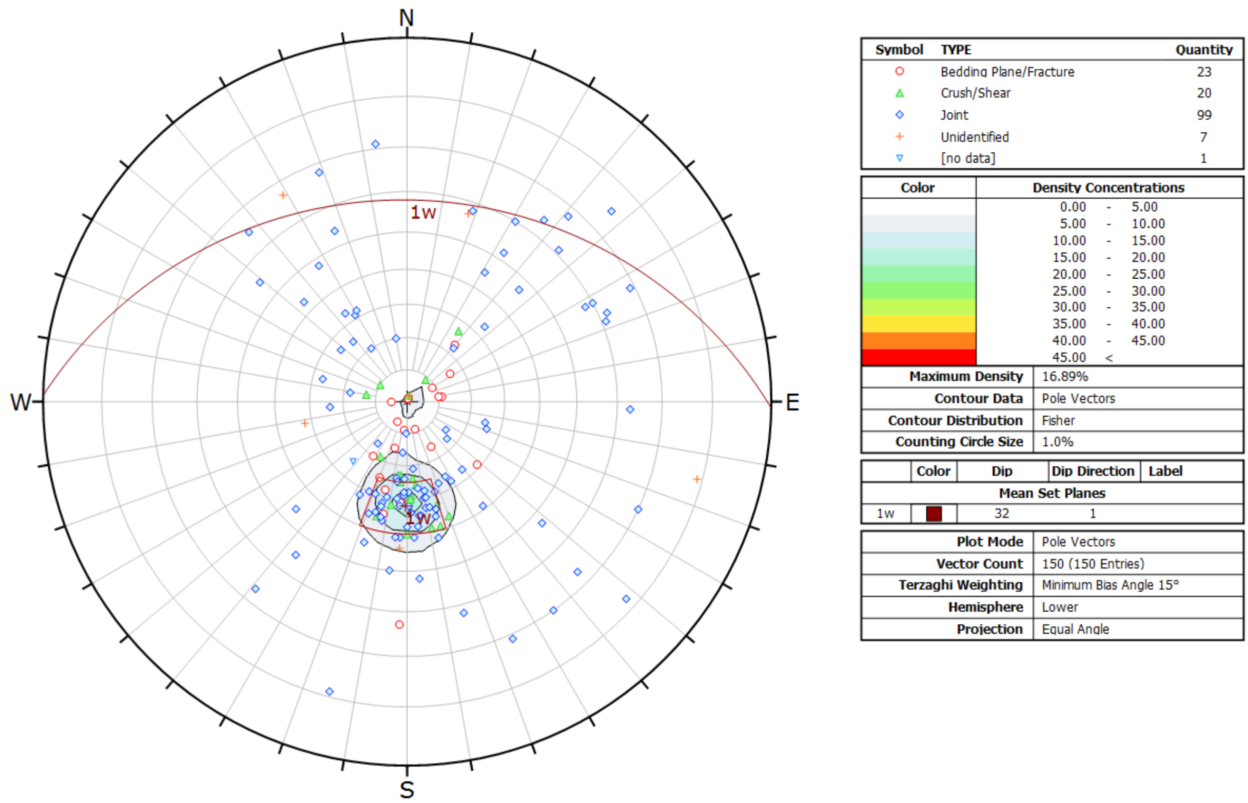


Figure 11.22 Stereonet plot of discontinuities in ECBF for BH236

11.24.2 Groundwater model

Based on piezometers in CI-13 and BH235 groundwater is expected to be encountered at 9.5mRL. At tunnel level, the piezometric surface is 9mRL based on piezometers in CI-13 (A Matakite investigation).

11.24.3 Contaminated land assessment

The proposed works for the construction of Link Sewer 3 comprises demolishing the existing pump station, valve chamber and sewer. Site works will be carried out to establish a working platform, retaining walls and pavements followed by excavation of two deep shafts as well as excavations for grit and control chambers and trenched connections.

Previous desk top investigations indicated that site has been part of a reserve since 1940, with buildings established after 1959. CoPC for the site include heavy metals, hydrocarbons, and nitrate compounds.

The following observations were made during the intrusive sampling investigation:

- Cleanfill material was observed to a depth of 1.3 to 2.7 m bgl.
- No evidence of dumping or fly tipping was observed.
- PID readings in soil samples did not exceed 0.0 ppm.
- Groundwater was not encountered during sampling.
- ACM was not observed.

The following table presents a summary of a comparison of soil sampling results to the adopted acceptance criteria. A full results table can be viewed as appended.

Table 11-62 : Comparison of soil sampling results to the adopted acceptance criteria (PS25)

Contaminant of Potential Concern	Assessment Criteria			
	Schedule 10 Permitted Activity Criteria	Auckland Background Concentrations (non-volcanic)	MfE Guidelines	NES SCS
Heavy Metals	All below.	Exceedances detected for nickel.	NC.	All below.
SVOC / VOC Compounds	All below or NC.	All below or NC.	All below or NC.	All below or NC.
PAH Compounds	All below or NC.	All below or NC.	All below or NC.	All below or NC.
OCP Compounds	All below or NC.	All below or NC.	NC.	All below or NC.

NC = No Criteria.

- A comparison to the adopted acceptance criteria indicates that there are no exceedances of NES SCS for protection of human health.
- Select samples yielded concentrations of nickel Auckland Background Concentration. The average concentration of nickel is below the respective background value.
- While these concentrations of CoPC present no immediate risk to human or human health, the soil is likely suitable for disposal at a cleanfill or managed fill facility.   A comparison to landfill criteria should be undertaken at the time of disposal.

Further inspection and/or testing should be undertaken where significant volumes of fill material are encountered during the works.

11.24.4 Geotechnical risks

Table 11-63 : Geotechncial risks (PS25)

Geotechnical Risk Name	Description	Impact	Relevant Geotechnical Unit	Location
Extremely weak rock	Extremely weak rock can occur at any place within ECBF	Unstable excavation Shaft design not suitable for ground conditions	Moderately weathered to unweathered ECBF	Anywhere within ECBF
Parnell Volcaniclastic Conglomerate	Conglomerate may be encountered during excavation	Excavation requires change to construction methodology Reduced productivity and higher machine tool wear	Parnell Volcaniclastic Conglomerate, Moderately to Unweathered ECBF	Anywhere within Moderately to Unweathered ECBF
Wood fragments and logs	Wood fragments and logs may be encountered	Difficult conditions for excavation / constructability	Tauranga Group	Anywhere within Tauranga Group

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