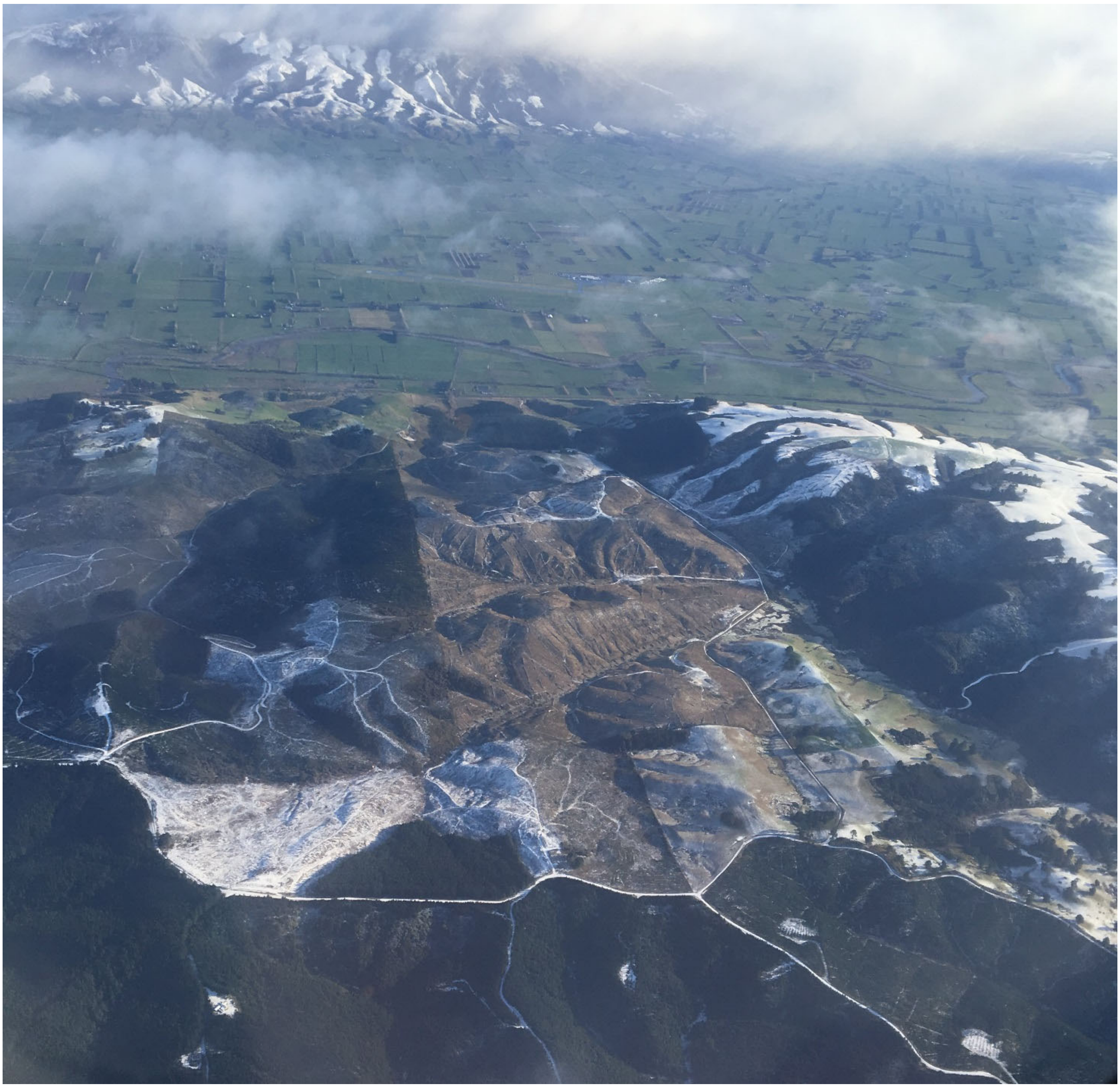
Appendix 5: Geotechnical Interpretive Report



Dunedin City Council

Waste Futures - Smooth Hill Landfill Consenting

Geotechnical Interpretive Report

DUNEDIN CITY COUNCIL | kaunihera a-rohe o Ōtepoti

August 2020 (updated May 2021)

Table of contents

Table index

Figure index

Appendices

1. Introduction

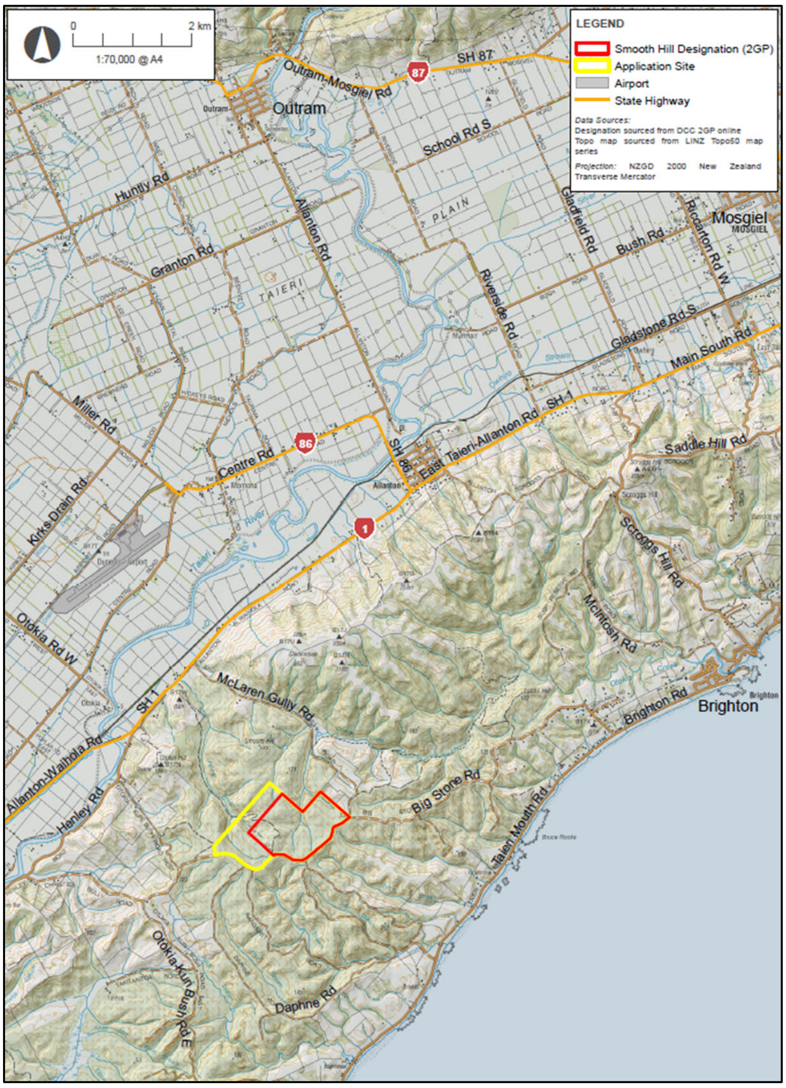
1.1 Background

The Dunedin City Council (Council) collects residential waste and manages the disposal of both residential and most commercial waste generated from the Dunedin City area and environs.

The Council has embarked on the Waste Futures Project to develop an improved comprehensive waste management and diverted material system for Dunedin, including future kerbside collection and waste disposal options. As part of the project, the Council has confirmed the need to develop a new landfill to replace the Council's current Green Island Landfill, which is envisaged to reach full capacity in the next few years. Final closure could be around 2028 depending on the closure strategy adopted by the Council. ~~likely to come to the end of its functional life sometime between 2023 and 2028.~~.

The Council commenced siting~~a~~ studies ~~search~~ for a new landfill location in the late 1980's and early 1990's and selected the Smooth Hill site in south-west Dunedin, Figure 1 below, as the preferred location. At that time, the site was designated in the Dunedin District Plan, signalling and enabling its future use as a landfill site. The Council also secured an agreement with the then landowner, Fulton Hogan Ltd, to purchase the land and the Council took ownership of the land in September 2020. Since the 1990's the Council extended the life of Green Island Landfill and further development of the Smooth Hill site has been on hold

Figure 1 Site Location (Updated May 2021)



As part of the Waste Future's Project, the Council has reconfirmed the technical suitability of Smooth Hill for the disposal of waste.. The Council has proceeded to develop a concept design for the landfill, site access via existing rural roads, and associated road upgrades. The concept design (the subject of this report) for the landfill has been developed by GHD with technical input from Boffa Miskell, and represents contemporary good practice landfill design that meets adopted New Zealand landfill design standards.

The Council lodged applications for resource consents for Smooth Hill landfill with both the Otago Regional Council and Dunedin City Council in August 2020. The applications included an earlier version of this report. This report has now been updated to reflect both the changes in the design and in response to specific s92 questions.

While being similar in many ways to the previous design, the key changes are summarised as follows:

- The landfill size has been reduced. The revised landfill lies within the footprint of Stage 1 and Stage 2 of the original design, with the western Stages 3, 4 and 5 no longer included (for comparison see Drawings C102 and C104). In overall terms:
  - the footprint of the landfill is reduced from 44.5 ha to 18.6 ha
  - landfill (gross) capacity is reduced from approximately 7.9-million m3 to 3.3-million m3

net waste capacity is reduced from 6.2-million m3 to 2.9-million m3

  – the predicted landfill life has reduced from 55-years to 40-years

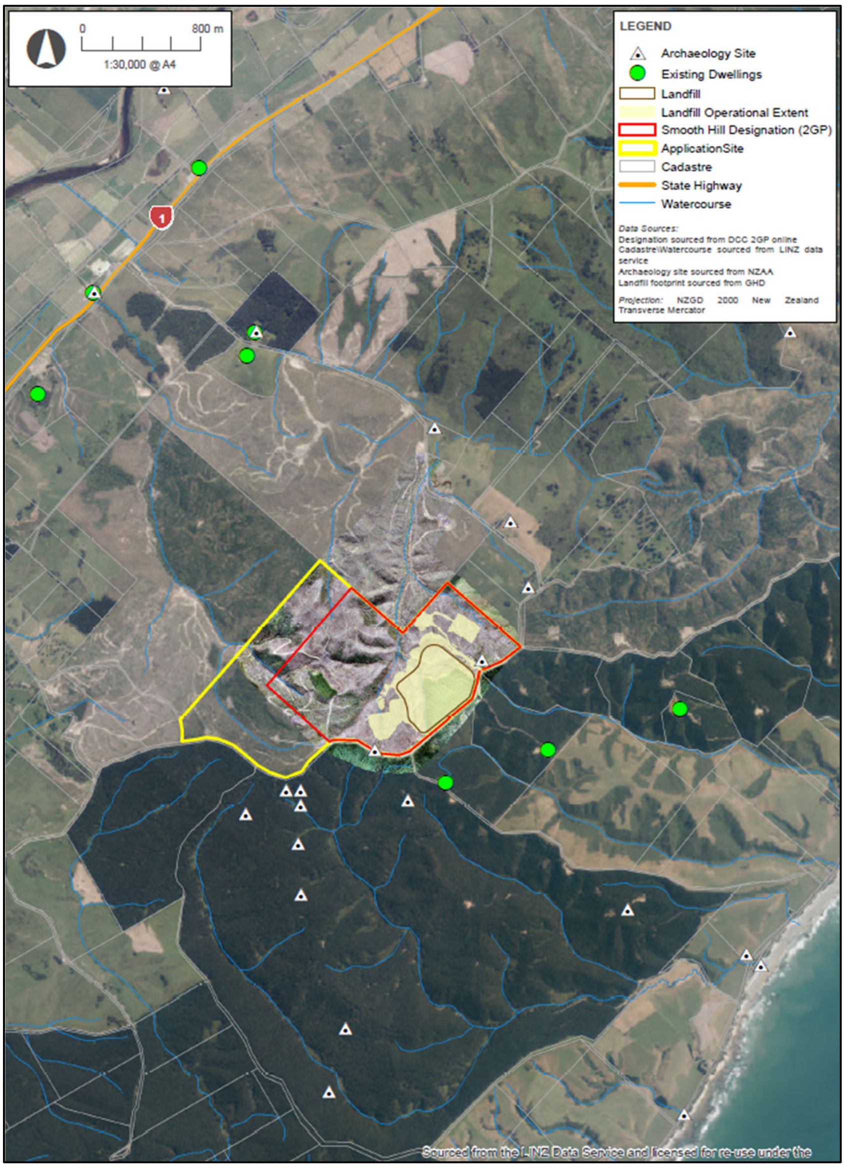
- Practical adjustments to the general construction of the landfill, including:
  – Landfill staging and construction sequencing, to a more typical 'bottom-up' filling methodology, which improves the intermediate and overall landform stability of the new design (Drawings C210 to 214)
  – Leachate containment and collection systems adjusted to reflect the revised construction sequencing
  – Construction phase systems for stormwater diversion, treatment and control
  – Relocation of the attenuation basin to the west of the revised landfill footprint rather than immediately downstream of the landfill toe.

1.2 Project Overview

The proposal includes the following key components:

- ~~The staged construction, operation, and aftercare of a Class 1 landfill within the existing designated site to accept municipal solid waste. The landfill will have a capacity of approximately 6 million cubic metres (equivalent of 5 million tonnes), and expected life (at current Dunedin disposal rates) of approximately 55 years.~~ The landfill will receive waste only from commercial waste companies or bulk loads
- Infrastructure to safely collect, manage, and dispose of landfill leachate, gas, groundwater, and stormwater to avoid consequential adverse effects on the receiving environment
- Facilities supporting the operation of the landfill, including staff and maintenance facilities.
- Environmental monitoring systems
- Landscape and ecological mitigation, including planting
- Upgrades to McLaren Gully Road including its intersection with State Highway 1, and Big Stone Road, to facilitate vehicle access to the site

Figure 2 Site Environs (Updated May 2021)



1.3 Purpose and Scope of Report

The purpose of this Geotechnical Interpretive Report (GIR) is to provide a geotechnical interpretation of the investigation data; provide preliminary recommendations for the geotechnical aspects of the design and construction of the waste facility; and to support the applications for resource consent. In particular~~,~~ this GIR is to:

- Confirm the ground conditions and underlying geology at the site
- Provide geotechnical design parameters for use in the concept landfill design
- Identify any geological or geotechnical hazards presenting a risk to the proposed design
- Assess suitability of excavated materials for re-use as landfill liner, capping and engineered fill material
- Assess stability of liner and capping slopes (both intermediate and final), as well as proposed cut and fill slopes

1.4 Previous Reporting

All recommendations and interpretations made within this GIR are based on findings from the geotechnical investigation reported in the Geotechnical Factual Report (GFR), which is summarised below:

- GHD, 2021a~~19~~, *Smooth Hill Landfill Consenting - Geotechnical Factual Report* ~~, dated August 2020~~ (updated May 2021)

The site investigation detailed in the GFR confirmed that the geology beneath the site is generally in accordance with the published geology. The investigation results show the basement geology to be the Henley Breccia Formation. The Henley Breccia Formation comprises breccia, sandstone, siltstone, and conglomerate. Overlying the majority of the site is a 1~~2~~ to 5 m thick layer of loess and loess derived soils. Thin (<3 m thick) alluvial deposits were encountered in gully bases. Localised deposits of shallow instability debris were encountered at several locations around the site. Pockets of fill are present in a number of locations, as a result of previous forestry activities on the site.

Based on the presence of quartz conglomerate in the cores, a number of boreholes were originally logged as the Taratu Formation. If this were so, this would suggest a ~~A~~ variation ~~of note~~ from the published geology and the occurrence of Taratu Formation conglomerate capping hills and ridges at the eastern and western margins of the landfill footprint. The geology map shows these deposits occur to the south of the site but not within the landfill site and its immediate environs. ~~However, their occurrence appears to be more widespread than previously mapped.~~

Further research of available technical publications on the Henley Breccia has confirmed that quartz conglomerates are present within the sequence. This suggests that the material previously identified as Taratu Formation is likely to be Henley Breccia.

Given the similarity in geotechnical performance of the two units, it is proposed to assume all materials present on site are Henley Breccia and manage them as one unit during the design process. For this reason, the presence of more extensive or less extensive areas of Taratu Formation will have no significant impact on the design.

For consideration, the Taratu Formation sits unconformably over the Henley Breccia and this contact has the potential to be a weaker plane. While it is now considered unlikely to be present on site, during detailed design and construction the presence of this contact will be investigated further.

2. Regional Geology

2.1 Site Description

The site is bordered by Big Stone Road along its southern boundary. Access from State Highway 1 (SH1) is typically via McLaren Gully Road. The site is bounded to the north and west by forestry land, and to the northeast by farmland. Figure 3 provides a closer view of the site.

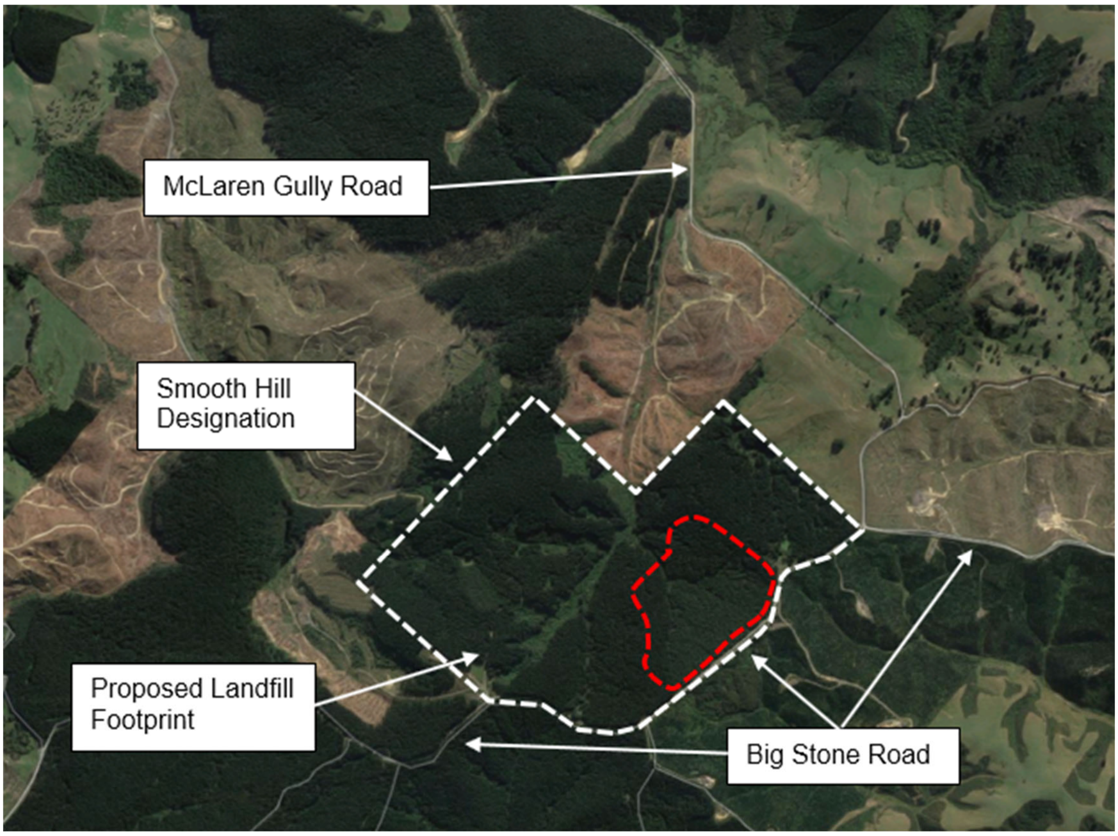


Figure 3 Proposed landfill site (base image sourced from Google Maps)

The site is located in a south to north trending gully, which is fed by smaller gullies from the east, west and south. The flow direction for water exiting the gully is from the south to the north. The slopes around the southern half of the site form a natural "amphitheatre" shape, which is bisected by a larger central ridge and a smaller ridge in the south-western corner – both trending south to north.

Until recently the site was planted with mature Radiata pine. The site cover now is a mixture of scrub, bare earth and forestry slash with replanted pine saplings with small areas of native vegetation in the gully bottoms. A number of forestry tracks provide access around the site. ~~An area of remnant plantation (~ 9.8 Ha) covers the south-eastern side of the site.~~

The ground is typically wet and boggy in the base of the gullies where there is standing or flowing water at times following rainfall.

2.2 Published Geology

A review of the published geological maps (Bishop [1994], and Bishop and Turnbull [1996]) covering the site show that the main lithology expected to be encountered is the Henley Breccia Formation. Although not shown on the geological map, it is expected that the Henley Breccia is overlain by several metres of loess deposits, and locally by alluvium and shallow slope instability debris. Outcrops of Taratu Formation at the tops of ridges are mapped to the south and east of the site.

Figure 4 presents an excerpt from the Bishop (1994) geological map.

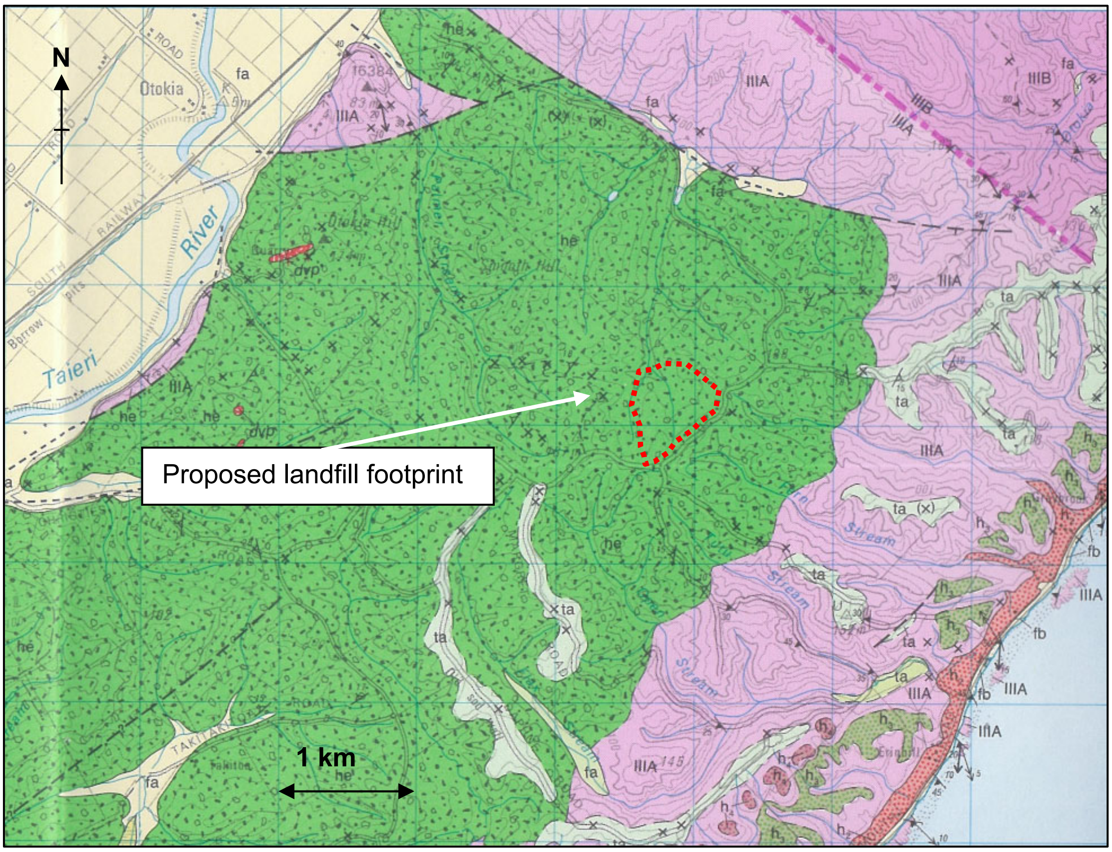
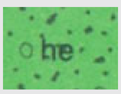


Figure 4 Excerpt from 1:50,000 Geology of the Milton Area (Bishop, 1994)

Table 1 presents a summary of the relevant geological units.

Table 1 Summary of relevant geological units

Map Graphic	Geological Unit	Description (from Geology of the Milton Area (Bishop, 1994))
Not mapped	Colluvium	Composition depends on the underlying material, typically disturbed and mixed; typically weak.
	Alluvium	Alluvial sand and gravel **(fa)**
Not mapped	Loess	Yellow-brown, massive layer or series of layers, mixed at the base with weathered bedrock and overlain by darker organic-rich soil. Columnar “jointing” (due to soil shrinkage) and shrinkage cracks are common.
	Taratu Formation	Yellow quartz sand and pebble conglomerate, with minor clay, carbonaceous siltstone and lignite. Limonite and silica cemented quartz conglomerate **(ta)**
	Henley Breccia	Greywacke and schist breccia and conglomerate. Sand lenses, coal streaks **(he)**

2.3 Geological Structure

2.3.1 Rock outcrop - Taratu Formation?

A single rock outcrop was mapped adjacent to the northwest boundary of the site. A photograph of the outcrop is presented in Figure 5 below. Bedding is most evident in the interface between the sandstone and conglomerate beds and appears to display cross-bedding in the sandstone beds. The bedding observed in the outcrop is consistent with that indicated in the published geological map for the Henley Breccia.

As discussed in Section 1.4 of this report, this outcrop has now been mapped as Henley Breccia rather than Taratu Formation. Similar material was logged in boreholes (BH09, BH209 & BH10). ~~Although not indicated by published geology as outcropping on the site, the Taratu Formation was mapped on site in a single outcrop (adjacent to the northwest boundary of the site) A photograph of the outcrop is presented in Figure 5 below. Bedding is most evident in the interface between the sandstone and conglomerate beds, and appears to display cross-bedding in the sandstone beds. The bedding observed in the outcrop is consistent with that indicated in the published geological map~~

Figure 5 Rock~~Taratu Formation~~ outcrop near the northwest extent of the site

2.3.2 Henley Breccia Formation

Much of the Henley Breccia Formation is massive with rare bedding planes. Bedding ~~, but where bedding is evident, it typically~~has been mapped dipping~~s~~ towards the west or northwest at 15-30° (Bishop, 1994). No outcrops of Henley Breccia were located within the landfill footprint or in the area around the site (only road cuttings were inspected due to access constraints), to confirm this bedding orientation. Where it could be discerned, observed bedding~~unit contacts~~ in the boreholes ~~drilled through the Henley Breccia Formation as part of this project~~ generally confirmed this ~~bedding~~ dip angle ~~where it could be discerned~~, however, the breccia units tended to be very weak and broken at these contacts, which obscured the~~any bedding~~ dip. None of the boreholes were oriented, therefore dip directions were not able to be confirmed. Bedding thicknesses as encountered in the investigation are detailed in Section 3.2.

A shallow angle bedding orientation dipping to the northwest would be favourable for the western slopes of the landfill but potentially unfavourable for the eastern slopes.

Joints, and other rock defects, were rare in both the sandstone / siltstone and breccia units of the Henley Breccia investigated. Most boreholes drilled as part of this project encountered few logged defects, while those that did typically had defect spacings in excess of 10 m. Defect dips ranged between 10° and 80°. Too few defects were logged to obtain any meaningful sense of predominant defect sets or trends.

The potential for discontinuity (joint, defect or bedding) controlled instability is assessed as low due to the low angle, and infrequency of discontinuities. During excavation of the eastern slopes, as the rock is fully exposed, it is recommended that face mapping is undertaken to confirm design assumptions. The current design has an overall slope angle of 1V:5H (~11°) which is favourable for the observed bedding.

2.4 Faults

There are a number of faults near to the site, which bound the Henley Breccia to the western boundary. These faults were instrumental in the formation of the unit, as the breccia was emplaced as a result of uplift of the western side of the Titri Fault, resulting in debris movement to the east; this debris was subsequently buried and lithified to form the Henley Breccia. Subsequent uplift of the eastern side of the Titri Fault brought the Henley Breccia to the surface.

None of the mapped faults are inferred to pass through the site, or within 1 km of the landfill footprint. No evidence of faulting was found during site mapping or in borehole core during these investigations.

2.5 Natural Slope Instability

The published literature (Bishop, 1994) states: *"Shallow slope failures are widespread in Henley Breccia, especially on cleared land north of Waihola. Small failures on steeper slopes underlain by other lithologies are also widespread but are commonly restricted to the cover of loess and colluvium. Localised tunnel gullying 1-2 m deep occurs in loess in a few areas".*

A number of instances of shallow slope instability have been identified on slopes both within the site boundaries and in the surrounding area. Shallow slope instability was identified either by onsite mapping during fieldwork, or by utilising stereo pair aerial photographs, obtained from the Retrolens Historical Image Resource. The areas of shallow slope instability appear to be constrained to the surficial soils (loess) and weathered rock and are up to a few metres deep. A map of the identified areas of shallow slope instability is presented in Appendix A.

The intention is that during the earthworks associated with creating the landfill slopes, these shallow surficial features will be excavated and removed. It is recommended these features are inspected during construction by an Engineering Geologist.

3. Site Geology

3.1 Site Investigations

Site investigations were carried out in two phases, and comprised 15 machine boreholes, 10 test pits and 8 bulk sample pits. The geotechnical investigation results are presented in full in the GFR (GHD, 2021~~0~~). A plan of all test locations is presented in Appendix A of this report.

3.2 Encountered Geology

A number of soil and rock units were encountered during the investigation. These are summarised below in Sections 3.2.1 to 3.2.7. In general, the encountered geology is consistent with the mapped geology in Bishop (1994) and Bishop & Turnbull (1996)~~ with the exception of the occurrence of Taratu Formation on the site~~.

Topsoil was observed to be covering the majority of the site~~,~~ and was encountered from surface at a number of test locations. However, since it will be excavated from the entirety of the landfill footprint, it has not been described below.

3.2.1 Fill

Fill was encountered at a number of locations around the site, in BH04, BH09, BH202, BH209, BH211, TP08, and TP12. None of the fill is engineered, all of it is associated with the recent forestry work on the site, with skidder pads at the locations of BH04, BH09, BH211, and TP12. The fill at both BH209 and TP08 is likely a result of track formation – at TP08 the fill overlays a saturated layer of vegetation.

At BH04, BH09 and BH209, the fill overlies loess. At BH202 and TP12, the fill overlies buried topsoil.

The fill was encountered at the surface at all of the above locations and was typically 0.25 to 1.5 m thick. The fill is typically moist, firm to stiff, brown, and dark grey. The fill occasionally contained intermixed organic matter (forestry slash).

All fill will be removed during the construction of the landfill.

3.2.2 Areas of Shallow Instability

Shallow instability debris was encountered in BH01, TP02, TP05 and TP09. Several other small, areas of shallow instability were noted around the site but were not drilled or excavated. These areas will~~are expected to~~ be entirely removed as part of the landfill earthworks, and thus would have no effect on the landfill construction or operation. The debris material was encountered at the surface (occasionally with a thin veneer of topsoil), and was typically 0.4 to 2.7 m thick and associated with topsoil/loess and possibly weathered rock.

Given the nature of debris deposition (i.e. in-situ material moving downslope), this unit's composition varied across the site depending on the underlying in-situ material. Typically, the instability debris comprised disturbed gravelly silt, silty sand, sand, silt and organic material such as tree roots and branches. The cohesive soils were typically inferred to be stiff and moist, while the granular soils were typically inferred to be loose and wet to saturated.

To assess the strength of the material, one shear vane reading was taken in the instability debris, in TP02, with a peak vane shear strength of 65 kPa. This indicates the material tested is stiff, however, given the variability of the material, this strength descriptor cannot be applied to the unit as a whole.

3.2.3 Buried Topsoil

Two distinct layers of buried topsoil were encountered~~ in BH04, BH08, BH202, TP02, TP08, TP09 and TP12~~; some underlying fill and instability debris, and some underlying the loess.

Buried topsoil was identified underlying instability debris (TP02, TP09), fill (TP08, TP12), and overlying alluvium (TP02, TP09). In BH202 the buried topsoil is overlying completely weathered Henley Breccia siltstone.

The buried topsoil in BH04 and BH08 was encountered below the loess, which was deposited during the last glaciation, making it a much older deposit.

All buried topsoils share the majority of their mechanical properties.

~~Generally, the buried topsoil is underlying instability debris (TP02, TP09) or fill (TP08, TP12), and overlying alluvium (TP02, TP09) or loess (TP08, TP12). In BH202 the buried topsoil is overlying completely weathered Henley Breccia siltstone.~~

They typically comprise silt, with varying amounts of sand and clay, and are either brown or grey. The buried topsoil in BH202 also contained small branches. The buried topsoil was typically inferred to be dry to moist, and firm to very stiff.

One vane shear strength reading was taken in the buried topsoil, in TP02, with a peak vane shear strength of 90 kPa.

~~The top of the buried topsoil layers was typically encountered between 100.54 to 142.5 m RL. The base of the buried topsoil layers was typically encountered between 100.34 to 142.4 m RL.~~ The layers of buried topsoil were typically 0.2 to 0.3 m thick, but ranged up to 0.7 m thick. The buried topsoil is not contiguous across the site, being found only in localised areas.

Topsoil will be removed during construction of the landfill. For historic buried topsoil under the loess, this will be removed and separated out during excavation of the loess.

3.2.4 Alluvium

Alluvium was encountered in the base of the gullies in BH01, TP01, TP02, TP03, ~~TP06, ~~and TP09. The top of the alluvium was typically encountered between 0.2 to 2.7 m bgl. The thickness of the alluvium ranged between 0.3 m to 2.0 m.

The alluvium was encountered underlying topsoil, buried topsoil and occasional instability debris, and overlying weathered rock.

The alluvium typically comprised sand, silt and gravel in varying amounts with organic material. The alluvium was typically moist to wet and grey or light grey. No strength testing was undertaken in the alluvium, but it is noted as generally exhibiting low strength.

TP01, TP02 and TP03 now fall outside of the re-design of the landfill.

At the location of TP09, this young material will be excavated and removed during construction of the landfill.

3.2.5 Loess

Loess derived soils were encountered across most of the site.

~~, except in BH01, BH202, BH203, TP01, TP02, TP03, TP05, TP06 and TP09. The locations where the loess was not encountered are predominantly in the base of gullies. BH202 had fill and buried topsoil directly overlying Henley Breccia, suggesting the loess had been removed at this location. BH203 was located on a cut platform and was drilled directly into Taratu Formation, suggesting the loess had also been removed at this location.~~

The loess soil typically comprised silt, with varying amounts of clay, sand and traces of fine gravel. The top of the loess was typically encountered at depths from the ground surface to 0.6 m bgl, but ranged as deep as 1.2 m bgl. The loess was typically between 1.0 and 3.0 m thick.

The loess was encountered underlying topsoil, fill and buried topsoil, and overlying buried topsoil, completely to highly weathered rock, and less weathered rock.

Peak vane shear strength readings taken in the loess ranged from 58 kPa to greater than 140 kPa, with most readings being greater than 130 kPa. These readings are consistent with what would be typically expected for in-situ loess in the South Island. The loess was typically brown, grey, orange-brown or yellow-brown in colour, dry to moist, and typically exhibited non-plastic to low plasticity behaviour - though occasional layers had a higher clay content and displayed high plasticity.

Published literature reports tunnel gullies and columnar jointing being reported as occurring within the loess in the region. However, neither of these phenomena were observed at the site during investigations and mapping.

Loess will be progressively stripped from the site during construction of the landfill and used as part of the liner system.

The locations where the loess was not encountered (BH01, BH202, BH203, TP01, TP02, TP03, TP05, TP06 and TP09). are predominantly in the base of gullies. BH202 had fill and buried topsoil directly overlying Henley Breccia, suggesting the loess had been removed at this location,-;likely by erosion. BH203 was located on a cut platform and was drilled directly into rock, suggesting the loess had also been removed at this location through excavation.

3.2.6 Taratu Formation

In updating this report in May 2021, it has been decided to remove the Taratu Formation section from the Encountered Geology Section and provide a description in the Regional Geology (Section 2.3.1).

3.2.7 Henley Breccia Formation

The Henley Breccia Formation comprises interbedded sandstone, siltstone, conglomerate, and breccia. The encountered Henley Breccia units are described below.

Completely Weathered to Highly Weathered rock

Completely to highly weathered (CW-HW) rock was encountered at most locations around the site.~~, except for BH01, BH02, BH09, BH203, BH209, TP02-03, TP06-07, and TP09~~. The CW-HW rock is typically encountered below the loess, though locally underlying buried topsoil, alluvium or surface instability debris.

The top of the CW-HW rock was encountered at depths ranging from 0.7 to 4.4 m bgl, and ranged from between 1.0 and 5.0 m thick. This unit has been weathered entirely to a soil. The composition of the residual soil depends on the parent rock type, but typically comprised some combination of sand, silt, and gravel, with occasional clayey layers – the source rocks being sandstone, siltstone and breccia respectively.

The strength of CW-HW Henley Breccia ranges from very stiff to hard (soil strengths) to predominantly extremely weak, locally very weak (rock strengths).

The moisture content of this unit was predominantly dry to moist, and the soils were typically non-plastic to low plasticity. This unit was logged as variously grey, brown, orange-brown, yellow-brown, cream/white, red, and purple.

CW-HW Henley Breccia was not encountered in BH01, BH02, BH09, BH209, TP02, TP03, TP06, TP07, and TP09.

Sandstone (Moderately Weathered to Unweathered)

Moderately weathered to unweathered (MW-UW) sandstone is the predominant rock type encountered in the Henley Breccia and was encountered in all boreholes and TP09 and the sandstone was typically interbedded with siltstone, conglomerate and breccia. The sandstone was fine to medium grained, but occasionally coarse grained. ~~all boreholes, except BH209 The top of the sandstone was typically encountered between 90 to 100 m RL or 120 to 140 m RL, and extended beyond the borehole/test pit termination depth~~

Strength ranged from extremely weak to strong, but typically ranged from very weak to moderately strong. Whilst the weathering grade decreased with depth, the strength of the unit was more dependent on the cementation between the grains, resulting in a highly variable strength profile down-hole.

Bedding was occasionally evident and ranged from thinly laminated to moderately thickly bedded. There were typically few defects logged, suggesting a very wide spacing (ie. >2 m spacing). The colour ranged between light grey, grey, orange-brown and yellow-brown.

Siltstone (Moderately Weathered to Unweathered)

Moderately weathered to unweathered (MW-UW) siltstone was encountered in the base of all test pits except TP08 and TP09. Thin beds (~0.2 m to 1.0 m thick) of siltstone was encountered in all boreholes except BH09, BH201, and BH209. The siltstone was typically interbedded with sandstone, conglomerate, and breccia. The siltstone appears to be a minor unit in the Henley Breccia sequence.

Occasional thin (typically 2-10 mm thick, occasionally up to 150 mm thick) lignite interlaminations / interbeds were encountered within the siltstone. This layer was often associated with a distinctive orange iron staining at ~97 m Reduced Level (RL).

~~The top of the siltstone was typically encountered between 90 and 110 m RL or 130 to 140 m RL, and extended to the borehole/test pit termination depths~~. ~~The siltstone was typically interbedded with sandstone, conglomerate and breccia. Occasional thin (typically 2-10 mm thick, occasionally up to 150 mm thick) lignite interlaminations / interbeds were encountered within the siltstone.~~ Strength ranged from extremely weak to moderately strong, but typically ranged from very weak to weak. The degree of weathering decreased with depth, but strengths generally remained the same.

Few defects were logged and were typically moderately to very widely spaced bedding partings. The colour varied between grey, yellow-brown, orange-brown, red-brown and brown. Bedding was not always evident but was thinly to moderately thickly bedded dipping at 10-15°, where observed. Note, the boreholes are not oriented and dip angles are indicative only.

Conglomerate (Moderately Weathered to Unweathered)

Rare MW-UW conglomerate was encountered in BH04, BH06, and BH07. ~~It was encountered at depths ranging between 4.4 to 13.0 m bgl (103.75 m RL to 137.3 m RL).~~ The conglomerate layers were typically 0.3 to 1.0 m thick. Of note is the poor core recovery in the thick layer.

The strength ranged from extremely weak to moderately strong but was predominantly extremely weak to weak.

The conglomerate was matrix supported, and the matrix typically comprised silt or sand. The clasts were sub-angular to rounded quartz and schist. No defects were logged within the conglomerate layers.

Breccia (Moderately Weathered to Unweathered)

MW-UW breccia was encountered in TP07 and all boreholes, except BH09, BH10, BH202 (cored/logged section from 0.0 to 10.6 m bgl) and BH209. The breccia was predominantly unweathered to slightly weathered. The breccia was typically encountered below the sandstone, at depths ranging from 2.4 to 17.6 m bgl, ~~(81.0 m RL to 138.7 m RL)~~ and extended to the borehole/test pit termination depths.

Strength ranged from extremely weak to strong, though was typically weak to moderately strong. The weathering grade increased with depth, but the strength of the breccia was more dependent on the matrix cementation, resulting in a highly variable strength profile, as the degree of cementation varies significantly downhole.

There were no~~typically few~~ logged defects~~, with very wide spacing~~. The breccia contained both matrix supported and clast supported layers. The matrix was typically coarse sand. The clasts were typically sub-rounded to angular quartz and schist. Bedding was not always present, but was moderately thickly bedded where observed. The colour ranged between grey, white and pink.

3.3 Groundwater

Groundwater levels have been measured in piezometers a number of times across both phases of site investigation.~~ This report only presents a summary of groundwater conditions~~ A full assessment of the site's hydrogeological conditions is presented in the Groundwater Report (GHD, 2021c~~0~~)

3.4 Laboratory testing

Laboratory testing was carried out for the purposes of assessing the suitability of site materials (loess and CW-HW rock) for re-use in the bulk earthworks expected to be required for the project.

Tests carried out on samples comprised the following:

- Atterberg limit testing, NZS 4402:1986, Test 2.2, 2.3 & 2.4
- Particle size distribution, NZS 4402:1986, Test 2.8.1, 2.8.4
- NZ Standard Compaction, NZS 4402:1986, Test 4.1.1
- Pinhole dispersion, crumb tests, ASTM D4647 & ASTM D6572
- Constant head triaxial permeability testing on recompacted soil, ASTM D5084
- UCS of recompacted soil sample, NZS 4402:1986,Test 6.3.1
- Lime demand test, NSW Transport; Roads & Maritime Services Test Method T144

Testing was undertaken on natural soils and lime / bentonite stabilised soils.

Central Testing Services was engaged to complete the laboratory testing, and the results of the testing are presented in Appendix C of the GHD GFR (2020a).

3.5 Gaps in the Ground Model

Several factors restricted the ~~original~~ placement of investigation points for the concept stage site investigation. These were:

- Difficult terrain making some areas hard to access with machinery in generally poor weather conditions

- The south-eastern portion of the site until recently was forested with a stand of macrocarpa trees resulting in machinery being unable to access this area
- Ecological restrictions were in place to protect native fauna (potential for any lizards, skinks, falcons) during investigations– predominantly in the west of the site

Due to these factors, some gaps in the ground model remain where limited information on the underlying geology is available. These are highlighted on Figure 6 below.

However, the investigations to date have generally encountered consistent geological conditions across the site and reasonable and confident assumptions can be made regarding the likely ground conditions within the highlighted area remain within the revised landfill footprint Figure 6.

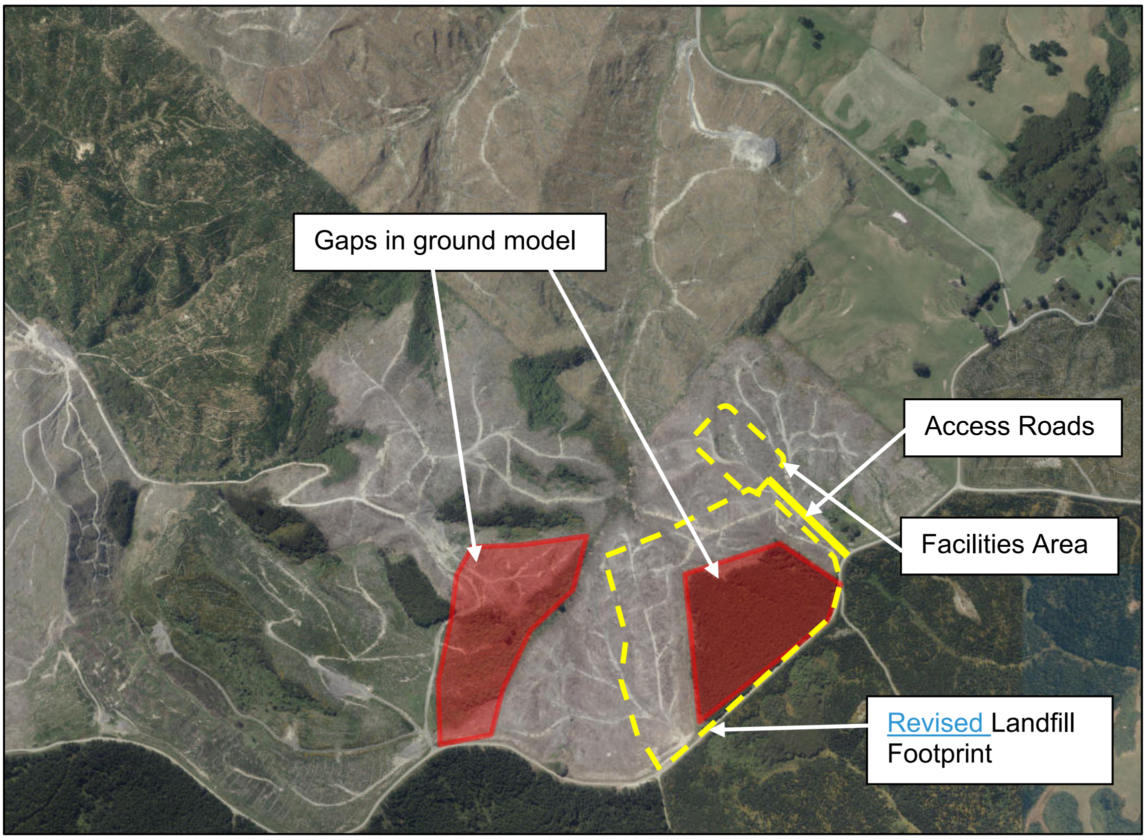


Figure 6 Identified gaps in the ground model (image sourced from DCC Webmap)

The footprint of the revised design no longer coincides with the western "gap", thus reducing our gap in site specific knowledge.

4. Geotechnical Hazards

4.1 Shallow Slope Instability Features

The identified shallow instability features in the site area and surrounds typically take the form of shallow ground movement in the loess cover or completely weathered rock mass. Instability debris was investigated in several places. These observations tend to agree with the descriptions in published literature with instability being generally confined to loess and possibly the completely weathered underlying rock. It is likely that features have become mobilised following saturation during periods of high rainfall. Depth of instability is typically less than 1 or 2 metres.

Based on site mapping and review of historical aerial photographs, a number of features have been identified across the site and are shown on Figure 2 in Appendix A. However, recent forestry activities on the site have obscured some of the historic features identified on older aerial photographs. Cut benches for skidder pads and shifted fill material, combined with the lack of the usual raking of slash into windrows, makes it difficult to determine the natural landform in places.

Given the shallow and discrete nature of most of these features, the majority of the identified areas of shallow instability in and around the site are likely to have little to no effect on development. Where they occur outside the development footprint they are unlikely to impact the construction or operation of the site unless immediately adjacent to or above construction activities. Where they occur within the landfill footprint, or can affect the operation of the landfill, they will be fully excavated and removed as part of the landfill development earthworks and will not impact development. Areas where further consideration will be required are where they potentially intersect areas of cut or fill associated with the landfill footprint and appurtenant structures. A ~~a~~ number of small features are mapped around the slopes immediately below the proposed site facilities area.

Further investigation will be required during detailed design to spatially define and understand these features but as described previously these areas of instability are likely to be associated with the top few metres of loess and completely weathered rock. Geotechnical risk can be mitigated through either stabilisation or removal of unstable materials during detailed design and construction. This is a common issue that is dealt with on a regular basis during design and construction in areas of loess in Otago and Canterbury.

4.2 Compressible Soils

It is expected that topsoil, some of the loess, alluvium in the base of gullies, unstable materials, and fill all have the potential to be compressible under load, due to their typically weak / loose and variable nature. These materials, and any other potentially compressible soils, will be removed from the landfill footprint and from beneath any areas on which engineered fill is to be placed, including the bund around the southern boundary of the landfill~~.~~. Provided this material is removed, there should be little risk of settlement due to soil consolidation.

4.3 Groundwater seepage

Groundwater seepage was noted in a number of locations around the site; however, the seepages were predominantly coming from areas of fill or alluvium near the base of gullies~~.~~ Groundwater is described in more detail in the Groundwater Report (GHD 2021b). ~~Groundwater beneath the landfill will be managed during site development by the placement of drainage material beneath the landfill liner to collect and direct groundwater to the base of the landfill from where it will be pumped out for discharge to the surface water system~~.

4.4 Liquefaction

Alluvium encountered in the base of gullies comprised limited thicknesses of saturated soft/loose sand, silt and gravel. In theory the sand layers within these deposits have the potential to liquefy during an earthquake, however, given that all of the alluvium will be removed from the landfill footprint liquefaction will not be a hazard to the landfill.

4.5 Site Seismicity

As discussed in Section 2.2.3 of the GFR (GHD, 2021a), there are a number of mapped faults within 100 km of the site.

It should be noted that the faults described in Table 1 of the GFR, whilst they are faults that are listed in the GNS Active Faults Database, all of those listed, except the Alpine Fault, do not meet the definition of "Active" as defined by GNS Science (i.e. recurrence interval <2000 years). Furthermore, the closest active fault to the proposed landfill site, as defined in NZS 1170.5:2004, is the Alpine Fault, which is located 240 km to the northwest.

In lieu of specific guidance on determining the ground acceleration to use for designing landfills to resist earthquakes, Structural Design Actions, Part 5 Earthquake Actions,New Zealand (NZS 1170.5 2004) and the New Zealand Transport Agency Bridge Manual (NZBM 3rd Edt Oct2108) have been considered.

Whilst landfills are not specifically referenced in NZS 1170.5 2004 (and 1170.5 Section 1.1 specifically excludes slopes), on the basis of leachate being classed as a hazardous substance, the landfill has been assumed to have an Importance Level of 3 (…*containing hazardous materials capable of causing hazardous conditions that do not extend beyond the property boundaries*)~~2 (IL2 - normal structures and structures not in other importance levels)~~ to give some guidance as to possible design lifetimes and resultant return periods. For a design working life of 50 or 100 years, IL3~~2~~ structures are required to be designed to resist earthquake loadings with return periods of 1000~~500~~ and 2500~~1000~~ years respectively.

The site investigation results show the ground conditions at the site should be classified as subsoil site class 'C' (shallow soil), as per NZS 1170.5.

For slope stability assessment under seismic load, the New Zealand Transport Agency Bridge Manual (NZBM) provides a method for determining a design ground acceleration, however, NZBM does not use design life and defines annual probability of exceedance (Table 2.3~~2~~). This table returns a design return period of 1/1000~~500~~ years. Seismic coefficients for preliminary geotechnical design for slope stability have been calculated using NZBM. Using this methodology, the peak ground accelerations (PGA) derived for the site are 0.31~~24~~ g for damage control limit state (DCLS) (equivalent to ultimate limit state (ULS)) and 0.08~~6~~ g for service limit state (SLS) (¼ DCLS).

At detailed design stage, a site specific probabilistic seismic hazard assessment could be completed if seismic shaking is deemed a risk that cannot be mitigated through liner design and leachate management practices. ~~requires further assessment.~~ Recent papers [1]by GNS on the Titri Fault and by Taylor-Silva on the Akatore Fault are consistent with the recurrence interval data already considered. On this basis we do not believe a SSSHA is required for the site.

[1] Investigation of past earthquakes on the Titri Fault, coastal Otago, New Zealand, DJ Barrel et al ,GNS Science Report 2017/35 October 2020
Paleoseismology of the Akatore Fault, East Otago, B Taylor-Silva April 2017, Masters Thesis University of Otago

5. Geotechnical Design

5.1 Preliminary Geotechnical Units

For the purposes of this report, the encountered geology described in Section 3.2 has been grouped into four~~ive~~ geotechnical units. The Henley Breccia has been further sub-divided into its major constituent fractions

- Unsuitable material – comprising topsoil, fill, alluvium, and unstable slope debris
- Loess
- Completely Weathered to Highly Weathered Henley Breccia Formation – comprising predominantly silt and sand~~sandstone, siltstone, breccia, conglomerate~~
- Moderately Weathered to -Unweathered Henley Breccia Formation – comprising and sub-divided into interbedded sandstone, siltstone, breccia, conglomerate
- ~~Taratu Formation – comprising sandstone, siltstone, conglomerate~~

The extent of the completed investigations and the geotechnical design set out in this section of the report are considered to provide a reasonable level of confidence for the concept design process and associated resource consent applications. The site has been adequately understood and conceptualised and is deemed suitable for the proposed development based on the information available. However, during detailed design further investigations and analysis may result in the refinement of the above geotechnical units and the preliminary design parameters set out in Table 4 below. Additional investigation would focus on orienting the core to confirm the dip and direction of bedding and shear strength testing of unit boundaries.

Data on preliminary geotechnical design parameters is incorporated into Section 7 of this report.

6. Earthworks

6.1 General

Earthworks will be ~~a major~~ part of the construction and operation of the Smooth Hill landfill and are expected to include:

- Cut and fill to create the required landfill base slopes and storage volume. Note all unsuitable soils and loess will be removed from beneath the landfill footprint. Engineered fill will be placed in any areas requiring back-fill to form the landfill base. It is likely that weathered rock will be used for engineered fill
- Construction of a toe bund to form a buttress at the low point of the landfill and containment for leachate
- Construction of an attenuation basin to the west of the landfill footprint ~~downstream of the toe bund.~~
- Cut and fill for the internal roads and site facilities
- Liner construction
- Landfill capping
- Landscaping on completed landfill cells

6.2 Material Reuse

6.2.1 Liner and Capping Materials

The proposed earthworks requires the removal of significant volumes of material from beneath the landfill footprint including all loess materials. Of this material, the loess could potentially be used for landfill liner and capping material. Completed laboratory testing of the loess indicates it can be compacted to achieve a permeability of 3×10^{-8} to 5×10^{-10} m/s, which is a relatively low permeability and desirable for a liner or capping material. However, loess soils typically become dispersive when disturbed and are prone to erosion from water flow and/or seepage. Completed dispersion testing on samples of loess collected at the site confirms that these materials are potentially dispersive - see results in the GFR. This is an undesirable property for a landfill capping or liner material where long term integrity is important.

However, loess materials can be made non-dispersive through stabilisation by the addition of lime. Completed lab testing (see GFR) has shown the addition of 2.5% lime by weight results in a non-dispersive material and indicates that this type of stabilisation may result in a material suitable for a landfill liner or capping layer. Because plasticity (see below) is a desirable property for a liner material, stabilisation of loess using bentonite was also carried out. Preliminary testing indicates the addition of bentonite does not significantly impact dispersivity.

Atterberg Limit testing of the untreated loess indicates it plots on the A-Line of the Casagrande plasticity chart (see GFR), suggesting that is has some plastic properties. Completed Atterberg testing on lime stabilised loess samples indicate the material remains on the A-Line. Further testing is required to confirm the effect of stabilisation on the plasticity of compacted loess and its ability to self-anneal. If used as a part of a liner system or a capping layer non-plastic behaviour and development of cracks would not be acceptable.

6.2.2 Bulk fill

It is expected that CW-HW Henley Breccia, ~~(and possibly the Taratu Formation),~~ will be suitable for use as bulk-fill. Laboratory testing undertaken to determine the suitability of the Henley Breccia for reuse as engineered fill beneath the base of the landfill suggests the material is generally suitable. Testing results are presented in the GFR.

Loess will generally be reserved as a low permeability liner, capping material or intermediate cover. While its use as a bulk fill is possible, careful consideration would be required regarding its dispersive nature.

6.2.3 Daily and Intermediate cover

It is expected that all materials including unsuitables could be suitable for use as daily cover during landfill operations. Intermediate cover will be generally restricted to loess and CW-HW Henley Breccia ~~and Taratu Formation~~ along with any hard fill delivered to the site. Alluvial may also be appropriate but represent a relatively small proportion of the site materials.

6.3 Material Excavatability

The landfill concept design shows that excavation is expected to occur over the entirety of the landfill footprint, whether this is just removing unsuitable soils and loess, or excavating the rock mass to achieve the desired landform. This section discusses the ease of excavating the underlying rock

A number of stronger, more cemented, sandstone and breccia layers were encountered across the site; although the borehole spacing is too great to allow us to definitively constrain the extents of these layers at this time. However, it is thought these layers are not contiguous.

The strength of the rock mass does not uniformly increase with depth; rather, the strength changes with the degree of cementation, which is highly variable with depth. The weathering grade does have some influence on the rock strength, but only in the sense that more weathered rocks are *generally* weaker, and less weathered rocks are *generally* stronger. Typically, defects in the rockmass also influence the overall rockmass strength and excavatability. However, in this case the rockmass is typically massive and very few defects have been observed in drill holes at this site.

Due to the high variability in the rock strength and without being able to constrain the lateral extent of the weaker and harder layers better, a full rippability assessment cannot be carried out at this stage.

During the course of the site investigations, it was observed that in most cases, a 20 tonne excavator could excavate the soil and CW-HW rockmass without undue difficulty.

It is expected, based on the core retrieved from the boreholes, that the majority of the MW-UW rockmass should be somewhere between easy and hard ripping, but a hydraulic rock breaker may be required on occasion, when the stronger cemented layers are encountered. This will need to be confirmed during detailed design and investigation.

7. Slope Stability Assessment

7.1 ~~Introduction~~Update

An initial slope stability assessment was undertaken in the 2020 Geotechnical Interpretive Report. This was undertaken on the anticipated natural landform and engineered benches that would form the landfill basin. This was a high-level assessment based on the information available.

Due to the redesign of the proposed landfill footprint and questions raised by the T&T reviewer, GHD has undertaken a new slope stability assessment.

Unlike the 2020 slope stability assessment, which focussed solely on SlopeW limit equilibrium software analysis, the updated s~~As part of the landfill concept design, slope stability analysis has been completed for the natural landform, and engineered benches that will form the landfill basin. S~~lope stability assessment has combined a review of the structural geomorphology of the Henley Breccia and only limited Slope/W ~~was analysed~~ on critical cross sections ~~using Slope/W limit equilibrium software~~. Slope stability assessment results are presented in Appendix C.

As part of the revised landform design, the existing landform will involve excavation of the in-situ soils and rock. At concept level design, it is assumed that the loess covering the entire site will be excavated to be reused elsewhere. It is also assumed that all topsoil, or soft or otherwise unsuitable soils will be excavated and removed. As such, it is assumed that the formation materials of the landfill will comprise a mixture of weathered rock and engineered fill.

7.2 Landfill Slopes

For concept design we have considered both the permanent slope and the temporary cut slopes

7.2.1 Permanent slopes

Landfill base

Permanent landfill base slopes are designed to achieve an overall slope grade of 1V:5H, with two 10 m wide benches in each slope, inter-bench slopes grades achieve 1V:4H.

Toe bund

The proposed bund will be built up to heights in excess of 10 m. The berm slopes angles are designed at 1V:4H.

Access Road

The access road along the northern end of the landfill cuts into the existing hillside resulting creating a cut in excess of 10 m. At concept stage design, a cut slope of 1V:3H has been assumed.

7.2.2 Temporary Slopes

Development of the landfill will be undertaken in stages. Staging will require temporary cut faces.

To facilitate landfill construction temporary cuts of 1V:3H have been proposed. A 1V:3H results in a slope angle of 18.4 degrees. The angle of the temporary slopes matches the regional dip of the Henley Breccia to minimise daylighting of bedding in cut faces.

7.3 ~~Landfill~~ Design Inputs into the Slope Models

7.3.1 Landfill Liner

The landfill design involves the placement of a composite mineral and synthetic liner. At concept stage the mineral liner will comprise recovered stabilised loess (600 mm) over engineered fill (200 mm). ~~which requires stability checks for base sliding failure mechanisms within the landfill for both static and seismic loading cases. It is understood the liner material will comprise recovered stabilised loess (600 mm) over engineered fill (200 mm).~~ For the slope stability analysis, the liner has been modelled as a single material with geotechnical properties. Stability checks for base sliding failure mechanisms have been considered for both static and seismic loading cases.

Assessment of the landfill geomembrane liner stability is not considered within this analysis and is discussed in the Concept Design Report (GHD, 2021~~19~~c).

7.3.2 Landfill Capping

For slope stability analysis at concept level stage, these materials have not been modelled as an individual unit in the models because they do not uniquely contribute to the slope stability analysis. They are included as part of the overall mass of the landfill waste in the toe bund model.

7.3.3 Municipal Solid Waste (MSW)

At concept level design, the Municipal Solid Waste (MSW) has been modelled using typical published values. The determination of appropriate material properties is based on the following assumptions:

- Heavy compaction in layers of no greater than 1.0 m lifts
- Daily cover of clean, locally won soil comprising 10%-20% of total fill volume
- Underdrainage of leachate with no recirculation

Bulk density municipal solid waste (MSW) has considered inclusion of 20% daily cover of soil.

The following MSW parameters have been used from published values.

Table 2 MSW Parameters

Landfill Development	Bulk Unit Weight, γ ($kN.m^{-3}$)	Internal Friction Angle, Φ' (°)	Cohesion, c' (kPa)
Initial Landfill Development*	12	33	0
Decomposing Landfill post closure**	12	22	9

* Kavzanjian et al. (1995)

** Hossain and Hague (2009), Fassett et al. 1994, Dixon et al. 2005

7.3.4 Leachate Management

It is assumed during the working life of the landfill; leachate levels will be mechanically maintained at a 300 mm head above the liner. It is possible that the pumps could fail at some time or, in the long term, it is possible that the pumps could be switched off if the leachate quality becomes acceptable. Therefore, a higher leachate level has been modelled to check these circumstances. This has been modelled considering a 1.5 m thick saturated MSW layer at the base of the landfill.

7.4 Geotechnical inputs into the Slope Models

7.4.1 Henley Breccia

Rock mass strength or discontinuity/ bedding-controlled slope failure

~~As part of the landform design, preparation of the existing landform will involve excavation into the in-situ soils and rock. At concept level design, it is assumed that the loess covering the entire site will be excavated to be reused elsewhere. It is also assumed that all topsoil, or soft or otherwise unsuitable soils will be excavated and removed. As such, it is assumed that the formation materials of the landfill will comprise either variable weathered rock or engineered fill~~Few defects were logged in the Henley Breccia units and as such the units are generally considered to be massive in nature. Weaker sections in the rock profile were noted on the shallow dipping bedding boundaries between units, however some of this apparent weakness could have been exacerbated during drilling. It is considered that slope instability will be driven either by the rock mass strength of the highly weathered units or by bedding.

We have no data on bedding discontinuity strengths. GHD has assessed the potential range of rock mass strengths based on the rock strengths described in the borehole logs. This may change following detailed design geotechnical investigations.

Rock mass strength

The four distinct geological units (Sandstone, Siltstone, Breccia and Conglomerate) were described as ranging from very weak to strong (with localities of extremely weak – i.e. residual soil). No laboratory tests were undertaken to confirm the strength descriptions given, instead, the NZ Geotechnical Society Field Description of soil and rock has been used to provide an estimate of Unconfined uniaxial compressive strength of these materials.

GHD have used RocData V5.013 by Rocscience Inc. to provide Mohr Column envelopes and estimated cohesion and friction ranges.

Intact uniaxial compressive strength

This was based on the log descriptions of the rock ranging from very weak to strong. Where a strength range has been given, the lower UCS provided by NZ Geotechnical Society has been used.

Geological Strength Index (GSI)

Based on the geological logs, the majority of the rock units were massive-intact with large defect spacing, however, bedding thicknesses ranged from 0.5 m thick to > 2.0 m thick. The GSI will vary depending on location specific rock quality, however, from the logs and estimated GSI range of 60-80 has been used.

7.4.2 Preliminary geotechnical parameters

The geotechnical design parameters recommended are based on the results of the investigations, in-situ test results, laboratory test results, empirical relationships, local experience, and other available public domain data. Our level of confidence is based on the limitations of the field logging and laboratory testing. However, this is appropriate for concept level design for consenting purposes and we remain confident the structural controlled slope failure mechanism considered in this report is not likely for this site and is not a mechanism observed in the region within these rock types (see Section 7.5). Note that as any unsuitable material is to be removed from the landfill footprint, this has not been included.

Table 3 Preliminary geotechnical design parameters (Updated May 2021)

Geotechnical Unit	Bulk Unit Weight, γ (kN/m3)	Internal Friction Angle, ϕ (°)	Cohesion, c' (kPa)	Unconfined uniaxial Compressive Strength (MPa)	Undrained Shear Strength Su (kPa)
In-situ Loess	16	25	0	-	130
Recompacted Loess	16.5	25	0	-	130
Henley Breccia Residual Soils (Silt mixtures)	18-20	30	2	--	-
Recompacted Bulk/Engineered Fill	18	30	0	-	-
Very Weak to Weak Henley Breccia (rock mass strength)					
VW-W Sandstone	18-20	38-44	70-100	1-5	-
VW-W Siltstone	18-20	30-35	50-90	1-5	-
VW-W Breccia	18-20	39-45	70-100	1-5	-
VW-W Conglomerate	18-20	40-46	70-100	1-5	-
Weak to Moderately Strong Henley Breccia (rock mass strength)					
W-MS Sandstone	18-20	38-44	340-480	5-20	-
W-MS Siltstone	18-20	30-35	270-450	5-20	-
W-MS Breccia	18-20	39-45	350-490	5-20	-
W-MS Conglomerate	18-20	40-46	350-500	5-20	-
Moderately Strong to Strong Henley Breccia (rock mass strength)					
MS-S Sandstone	18-20	38-44	1300-1900	20-50	-
MS-S Siltstone	18-20	30-35	1000-1800	20-50	-
MS-S Breccia	18-20	39-45	1400-2000	20-50	-
V MS-S Conglomerate	18-20	40-46	1400-2000	20-50	-

~~The geotechnical design parameters recommended in Table 2 are based on the results of the investigations, in-situ test results, laboratory test results, empirical relationships, local experience, and other available public domain data. Note that as any unsuitable material is to be removed from the landfill footprint, this has not been included in Table 2 below.~~

~~**Table 2 Preliminary geotechnical design parameters (Table deleted May 2021 and replaced with Table 4)**~~

~~7.4.1 In-situ Materials~~

~~The site investigation indicates that following site excavation, the in-situ materials beneath the landfill will comprise variably weathered rock ranging from:~~

- ~~Breccia (and gravelly silt residual soil)~~
- ~~Siltstone (and silt residual soil)~~
- ~~Sandstone~~

~~All rock was noted to grade from extremely weak (residual soil) to very weak and weak to moderately strong.~~

~~For concept level analysis, two in-situ materials have been considered for slope stability modelling from Section 5.2. These have been chosen based on the borehole logs near the cross sections. The materials used are:~~

- ~~Breccia (Residual Soil) – CW to HW Rock~~
- ~~Sandstone – MW to UW Rock~~

~~7.4.2 Engineered Slopes~~

~~Some areas of the landfill formation will need to be built up from the existing ground level using site-won engineered fill comprising a mixture of residual soils and very weak rock.~~

~~It should be noted, that in some areas the fill is up to 16 m in height above existing ground level.~~

~~The proposed landfill liner benches comprise 10 m wide benches with 1V:20H slopes (spaced at 10 m vertical interval). It is assumed that topsoil and loess will be cleared and that benches will be cut into residual soil or rock.~~

~~The following geotechnical parameters have been assumed for the engineered fill:~~

~~Table 5 Engineered Fill Parameters~~

~~Layer~~	~~Bulk Unit Weight, γ (kN.m^{-3})~~	~~Internal Friction Angle, Φ' (°)~~	~~Cohesion, c' (kPa)~~
~~Engineered Fill~~	~~18~~	~~30~~	~~0~~

7.4.3 Groundwater

Groundwater monitoring indicates the following:

1. Shallow groundwater (within 5 m of the proposed base of the landfill) at the northern extents of the proposed landfill where the natural valley floor forms.
2. Deep groundwater levels at the ridges (southern, western and eastern) sides of the proposed landfill.

A more detailed discussion of the site groundwater regime is presented in the Groundwater Report (GHD, 2021~~19~~b).

Beneath the base of the landfill and the toe bund, groundwater will be managed through subsoil drains to maintain a drained condition.

7.4.4 Seismic Loading

The calculation of the Peak Horizontal Ground Acceleration (PGA) used for the slope stability modelling has been outlined in Section 4.5.

The design peak horizontal ground acceleration (PGA), expressed as a fraction of earth gravitational acceleration, has been calculated for the site using NZS1170.5:2004 and NZTA Bridge Manual (SP/M/022), Third Edition, 2018, as a guideline.

~~Landfills are not specifically referenced in these documents, however~~ T~~t~~he landfill has been assumed to have an importance level of 3~~2~~ (IL3~~2~~). ~~Normal structures and structures not in other importance levels). IL2 provides some guidance as to the design life and earthquake return periods. IL2 structures are required to be designed to resist earthquake loadings with return periods of 500 and 1000 years for a design working life of 50 or 100 years respective~~

For this analysis, the PGA has been derived using Section 6.2 of the Bridge Manual for 'Slope Stability' with the following equation:

$$\mathbf{PGA = C_{0,1000} \times (R_u/1.3) \times f \times g}$$

Where:

PGA = Peak Ground Acceleration

C0,1000 = 1000 year return period PGA coefficient

= 0.23 for Subsoil Class C at Mosgiel

Ru = return period factor derived from NZS 1170.5

= 1.3 (0.25 for SLS) for Importance Level 3~~2~~ and return Period 1/~~5~~1000 (1/25 for SLS)

F = 1.33 for subsoil class C

The PGA has been derived for Damage Control Limit State (DCLS, i.e. ULS) and SLS.

With the above values, the PGA values derived are 0.31~~24~~g for ULS and 0.08~~6~~ for SLS.

7.5 Stability Modelling Cases

7.5.1 Modelled slopes – critical cross sections

For the concept design, both temporary cut slopes and permanent slopes cross sections have been developed for qualitative and quantitative assessment.

For the temporary slopes of Stage 1, two cross-sections per face have been considered. These have been identified by using the chainage along the first bench.

For the permanent slopes, a cross section~~s~~ has~~ve~~ been cut through the toe bund and the completed landfill.

The following cross sections have been selected:

- Section CH120 – through NE landfill slope
- Section CH240 – through NE landfill slope
- Section CH400 - through SE landfill slope
- Section CH500 - through SE landfill slope
- Section CH560 - through SW landfill slope
- Section CH700 - through SW landfill slope
- Section A - through landfill bund

The selected cross sections in relation to the proposed landfill footprint are shown in Appendix B. The existing geotechnical investigation data has been overlain onto these cross sections with the broad regional dip superimposed. This helps identify where additional data may be required to complete detailed design.

The cross sections are available in Appendix B.

~~A number of cross sections were developed through the proposed landform and critical/representative sections selected for analysis. The sections analysed include:~~

- ~~01 Section A Western Slope~~
- ~~02 Section B South Western Slope (and perimeter embankment)~~
- ~~03 Section C Eastern Slope~~

- ~~04 Section D Toe Bund (Northern)~~

~~The cross sections incorporated the following structures, and were modelled using Slope/W:~~

- ~~Formation benches composed of engineered fill (site won material) and/or in-situ rock cuts~~
- ~~Stability of upslope perimeter embankment comprising engineered fill~~
- ~~Stability of downslope toe bund compromised of engineered fill (at end of landfill life)~~

~~Sections A, B and D consider the stability of engineered slopes formed on in-situ residual soil and rock. Because the critical slip surfaces form through the engineered fill, geotechnical strength parameters for rock have been excluded from these analyses. Section C models cut benches in moderately to unweathered in-situ rock.~~

~~Where slope stability has been considered in rock, the analysis has been limited to large scale instability which is primarily controlled by rock mass properties. Smaller (bench-scale) local instability is more likely to be controlled by the presence of individual discontinuities. The assessment of smaller scale instability is an issue for detailed design and construction.~~

~~For concept level landfill design, slope stability analyses were carried out for potential cut slopes and engineered fill slopes at 1V:4H with 10 m wide benches and maximum slope heights of 10 m.~~

~~7.4.5~~7.5.2 Concept Level Analysis

Based on the published geological information, bedding is expected to be dipping shallowly to the north-west (16° - 18° towards 329° NW). As such, slopes at risk of bedding driven failure are those that dip towards the north-west such that the bedding units can daylight in the cut face.

Proposed slopes that may be affected by these criteria are the south-east landfill slopes and some proposed cut slopes in the north-west of the landfill where the proposed access road truncates the hillside.

Mapping the apparent dip of the geological units on the critical cross-sections demonstrates that a bedding-controlled slope failure is very unlikely. On this basis not all identified critical slopes have been modelled using SlopeW.

~~All slope stability analysis has been carried out using the conditions and parameters listed in the previous sections. The following stability scenarios have been considered:~~

- ~~Static (long term stability)~~
- ~~Seismic Ultimate Limit State (ULS earthquake loading)~~
- ~~Seismic Serviceability Limit State (SLS earthquake loading)~~

~~As underdrainage below the landfill liner will be present, and ground water has been encountered at significant depth below the proposed landfill base, short-term elevated ground water stability modelling has not been considered.~~

~~7.4.6 Critical Cross Sections and Structures~~

~~**Figure 7 - Indicative Landfill Sketch (Removed May 2021)**~~

~~The plan and cross sections can be found in Appendix C.~~

~~7.4.7~~7.5.3 Target Factor of Safety

It is generally accepted good engineering practice that the required factors of safety (Fo~~O~~S) for long term stability should be ≥1.5 for static conditions. ~~For the temporary condition the static factor of safety should be ≥1.3.~~

At concept level of analysis, the Fo~~O~~S required for seismic slope stability is Fo~~O~~S >1.0.

The target Factors of Safety for modelling are summarised in the table below:

Table 4 Target factors of safety

Analysis Case	Target FoS
Static	≥1.5
~~Short Term Static including Elevated Leachate Condition~~	~~≥1.3~~
SLS Seismic	>1.0
ULS Seismic	>1.0

7.6 Stability Modelling Results

~~7.4.8~~7.6.1 SlopeW Analysis

~~Section A comprises benches of engineered fill placed on in-situ rock. The fill thickness reaches up to 16 m thick in places. Each bench comprises a 1V:4H slope with a 10 m wide bench at the top. The critical slips Factor of Safety from SlopeW are recorded below:~~SlopeW has been used to model the completed landfill design in Cross Section A. To mimic the driving force of the landfill waste against the toe bund, selected user defined failure planes were chosen to force the failure through the base of the landfill. The results of these are included in Appendix C.

Three different potential failure slips were analysed for the static, SLS and ULS cases which were:

- Local Toe bund stability
- Global toe bund stability
- Waste stability

The results are provided in the following three tables below.

Table 5 Results of SlopeW modelling – local toe bund (Updated May 2021)

Analysis Case	Target FoS	Modelled FoS
Static	≥1.5	3.720
SLS Seismic (0.08 g)	>1.0	2.479
ULS Seismic (0.31 g)	>1.0	1.225

Table 6 Results of SlopeW modelling – global toe bund (Updated May 2021)

Analysis Case	Target FoS	Modelled FoS
Static	≥1.5	4.058
SLS Seismic (0.08 g)	>1.0	2.666
ULS Seismic (0.31 g)	>1.0	1.312

Table 7 Results of SlopeW modelling – waste (Updated May 2021)

Analysis Case	Target FoS	Modelled FoS
Static	≥1.5	2.748
SLS Seismic (0.08 g)	>1.0	1.890
ULS Seismic (0.31 g)	>1.0	0.978

~~7.4.9~~7.6.2 Qualitative slope stability analysis

The temporary (landfill base) slopes shown in cross sections CH120, CH240, CH400, CH500, CH560 and CH700 have not been modelled in SlopeW. Alternatively a qualitative approach has been taken.

-To assist in the qualitative stability assessment of these slopes, the available boreholes have been superimposed on the cross-section and the "apparent" bedding dip overlain.

CH 120 and CH240

The cross sections at CH120 and CH240 are looking to the south east along the cut slope of Stage 1. The "apparent" dip of the bedding is very shallow and demonstrates favourable bedding (Figure 7). Bedding controlled failure is assessed as low.

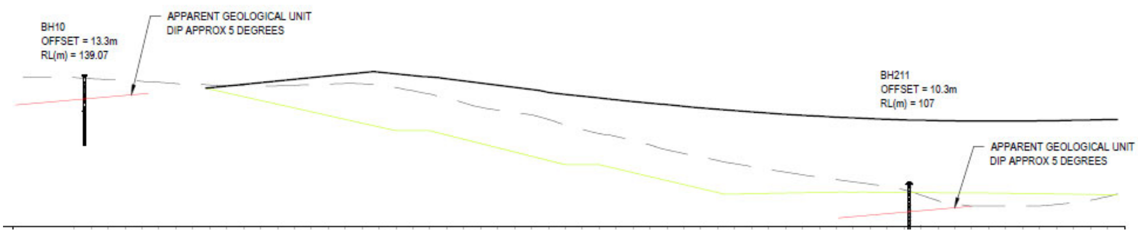


Figure 7 CH120 (excerpt from Drawing 12506381-01-Q003)

As a temporary slope, CH240 will not be altered. Engineered fill will be ultimately placed across it during Stage 2.

CH 400 an CH 500

These cross sections are looking to the southwest. This face of the landfill is in the least structurally favourable orientation for bedding controlled failure. The cut of the temporary slopes has been chosen to minimise exposure of bedding planes in the face of the cut face.

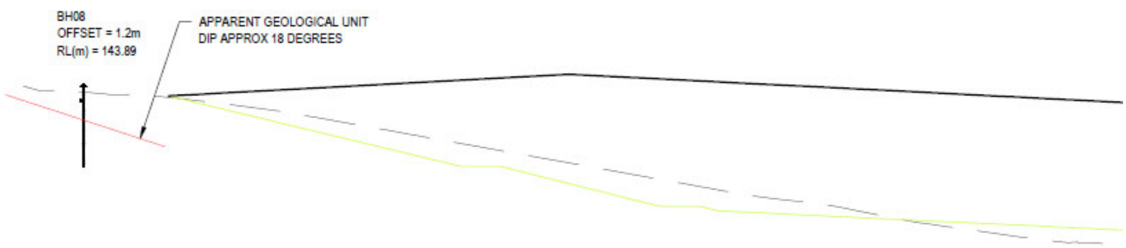


Figure 8 CH400 (excerpt from Drawing 12506381-01-Q003)

The risk of bedding controlled failure is considered low.

Ch 560 and Ch 700

The cross sections at CH560 and CH700 are looking to the north west along the cut slope of the later stages of Stage 1. The apparent dip of bedding is very shallow and favourable

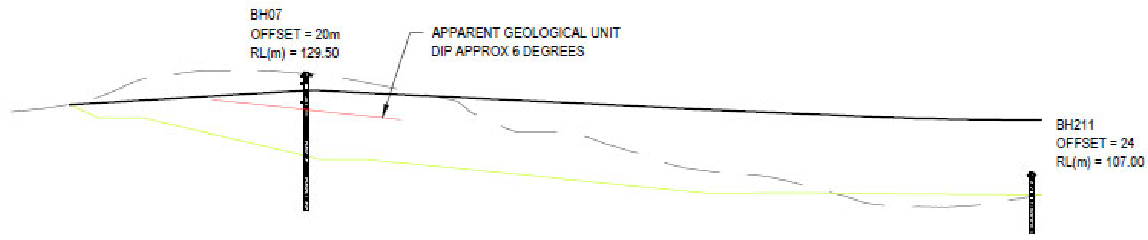


Figure 9 CH700 (excerpt from Drawing 12506381-01-Q003)

These ground models identify two important things

The horizontal distance between boreholes and depth of boreholes limits the ability to match up geological units. The model therefore relies on the mapped dip and dip direction of the regional geology.

As identified earlier in Section 3.5, there is a gap in data on the northwestern facing slope

~~7.5~~7.7 Summary

7.7.1 Temporary Slopes

All temporary slopes are proposed to be cut at 1V:3H which generally match the existing slopes on site and will be cut into favourably dipping rock. As such, it is considered unlikely any slope instability would occur. We consider this sufficient assessment at concept level design however further, targeted, investigations should be undertaken and these slopes assessed under the new data during detail design.

7.7.2 Permanent Slopes

The results of the modelling of the landfill with full waste placement demonstrate~~indicate~~ adequate slope stability of the toe bund under static and seismic load cases.

8. Site Suitability

Smooth Hill was selected from a number of potential sites during the late 1980's as the preferred location for a future landfill for Dunedin. At that time a range of parameters were taken into account including the likely favourable geotechnical characteristics of the site. This study has generally confirmed a number of the assumptions made at that time in the context of current landfill design criteria and modern engineering practice and the site is assessed as being suitable for landfill development in general accordance with the Technical Guidelines for Disposal to Land. The key attributes with respect to geotechnical site suitability are:

- The low permeability of the underlying Henley Breccia provides a high degree of hydrogeological containment for the site.
- The loess soils are likely to be suitable as low permeability materials for the landfill liner and landfill cap with suitable lime or similar stabilisation. However, further work is required during detailed design to confirm.
- The variably weathered Henley Breccia will be excavatable and suitable as a bulk engineered fill.
- Slope stability analysis of the excavated Henley Breccia slopes and the final landfill form under a range of static and seismic scenarios indicates satisfactory factors of safety can be achieved.
- It should be noted that interim stability of the waste during operation will depend on the methodology adopted for placing waste. These are detailed design issues that will need to be addressed during site development.
- The closest active fault is the Alpine Fault 240 km north-west of the site. Rupture of the Alpine Fault has been considered in the above slope stability analysis. Recent studies of faults closer to the site confirm recurrence intervals used in the assessment. The site has been assessed as an Importance Level 3 structure for PGA derivation.
- Areas of shallow instability have been identified across the site primarily associated with the loess and highly weathered Henley Breccia. These will be removed ~~or stabilised~~ during development. ~~Further investigation and definition will be required during detailed design. These features are common across the loess areas of Otago and Canterbury.~~ No deep-seated instability features have been identified.
- ~~Whilst the Taratu Formation is not mapped on the published geological maps as being present on site, its identified location and geological attitude is wholly consistent with that mapped locally to the site in that it forms a remanent layer over the Henley Breccia on the tops of the surrounding hills.~~
- The Big Stone Road access route utilizes existing roads. Earthworks will be required as part of the widening and upgrade of the road. Stability of earthworks will be addressed during detailed design for the access road upgrade. Conditions are generally anticipated to be similar to those seen on site

9. References

- AS/NZS 1170.0:2002. *Structural Design Actions, Part 0: General Principals.*
- Bishop, D.G. 1994: *Geology of the Milton area*. Scale 1:50 000. Institute of Geological & Nuclear Sciences geological map 9. 1 sheet + 32p. Institute of Geological & Nuclear Sciences Ltd, Lower Hutt, New Zealand.
- Bishop, D.G., Turnbull, I.M. (compilers) 1996. *Geology of the Dunedin area*. Institute of Geological and Nuclear Sciences 1:250 000 geological map 21. 1 sheet + 52 p. Lower Hutt, New Zealand: Institute of Geological and Nuclear Sciences Limited.
- Dixon. N, Russell. D, Jones. V, (2005). *Engineering properties of municipal solid waste*, Geotextiles and Geomembranes, Volume 23, Issue 3, Pages 205-233.
- Dunedin City Council Webmap, retrieved 17 December 2019, https://www.dunedin.govt.nz/dunedin-city/maps-and-photos/ratesmap
- Fassett J.B., Leonardo G.A. and Repetto P.C., (1994) *Geotechnical properties of municipal solid wastes and their use in landfill design*. Waste Technical Conference, Charleston SC, USA.
- Gharabaghi, B. Singh, M.K. Inkratas, C. Fleming, I.R. McBean, E. *Comparison of slope stability in two Brazilian municipal landfills,* Waste Management, Volume 28, Issue 9, 2008, Pages 1509-1517.
- GHD, 2021a, *Smooth Hill Landfill Consenting – Geotechnical Factual Report*, Reference 12506381
- GHD, 2021b, *Smooth Hill Landfill Consenting - Groundwater Report*, Reference 12506381
- GHD, 2021c, Dunedin City Council, *Waste Futures Phase 2 – Work Stream 3 Smooth Hill Landfill - Landfill Concept Design Report*, Reference 12506381
- Hossain. S, Haque. M, *The effects of daily cover soils on shear strength of municipal solid waste in bioreactor landfills,* Waste Management, Volume 29, Issue 5, May 2009, Pages 1568-1576.
- Kavazanjian, E., Jr., Matasovic, N. Bonaparte, R., Schmertmann, G.R., 1995. *Evaluation of MSW Properties for Seismic Analysis*. Geoenvironment 2000, ASCE Geotechnical Special Publication No. 46 (2).
- New Zealand Transport Agency, *Bridge Manual (SP/M/022)*, 3rd Edition, Amendment 3, October 2018.
- NZS 1170.5:2004, *Structural Design Actions – Part 5: Earthquake Actions – New Zealand*.
- Retrolens Historical Image Resource, sourced on 9 August 2019: http://retrolens.nz/
- Taylor-Silva, B., *Paleoseismology of the Akatore Fault, East Otago*. Unpublished MSc Thesis, University of Otago, Dunedin, New Zealand.
- WasteMINZ, August 2018, *Technical Guidelines for Disposal to Land*

10. Limitations

This report has been prepared by GHD for Dunedin City Council and may only be used and relied on by Client for the purpose agreed between GHD and the Client as set out in Section 1 of this report.

GHD otherwise disclaims responsibility to any person other than the Client and Council officers, consultants, the hearings panel and submitters associated with the resource consent and notice of requirement process for the Smooth Hill Landfill Project arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report. GHD disclaims liability arising from any of the assumptions being incorrect.

GHD has prepared this report on the basis of information provided by Client and others who provided information to GHD (including Government authorities)], which GHD has not independently verified or checked beyond the agreed scope of work. GHD does not accept liability in connection with such unverified information, including errors and omissions in the report which were caused by errors or omissions in that information.

The opinions, conclusions and any recommendations in this report are based on information obtained from, and testing undertaken at or in connection with, specific sample points. Site conditions at other parts of the site may be different from the site conditions found at the specific sample points.

Investigations undertaken in respect of this report are constrained by the particular site conditions, such as the location of vegetation and topography. As a result, not all relevant site features and conditions may have been identified in this report.

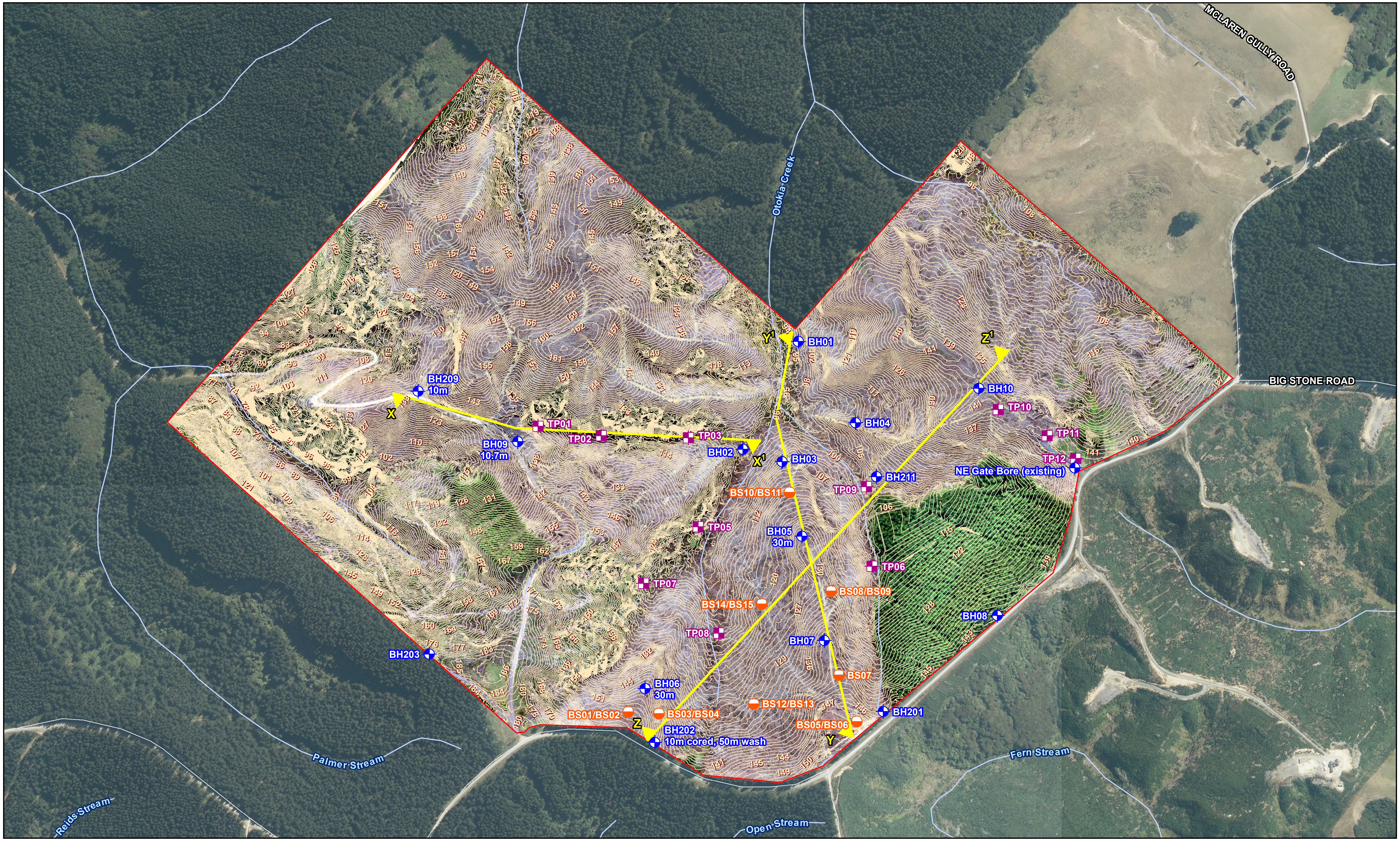
Site conditions (including the presence of hazardous substances and/or site contamination) may change after the date of this Report. GHD does not accept responsibility arising from, or in connection with, any change to the site conditions. GHD is also not responsible for updating this report if the site conditions change.

Appendices

Appendix A – Plans

Test location plan

Slope instability plan



Paper Size A3

0 50 100 200

Metres

Map Projection: Transverse Mercator
Horizontal Datum: NZGD 2000
Grid: NZGD 2000 New Zealand Transverse Mercator

N

LEGEND

- Site boundary
- Cross section location
- Borehole
- Test pit
- Bulk sample location
- Waterways
- Contours (1m)

GHD

Dunedin City Council
Smooth Hill Landfill
Geotechnical and Hydrogeological

Job Number	12506381
Revision	A
Date	17 Aug 2020

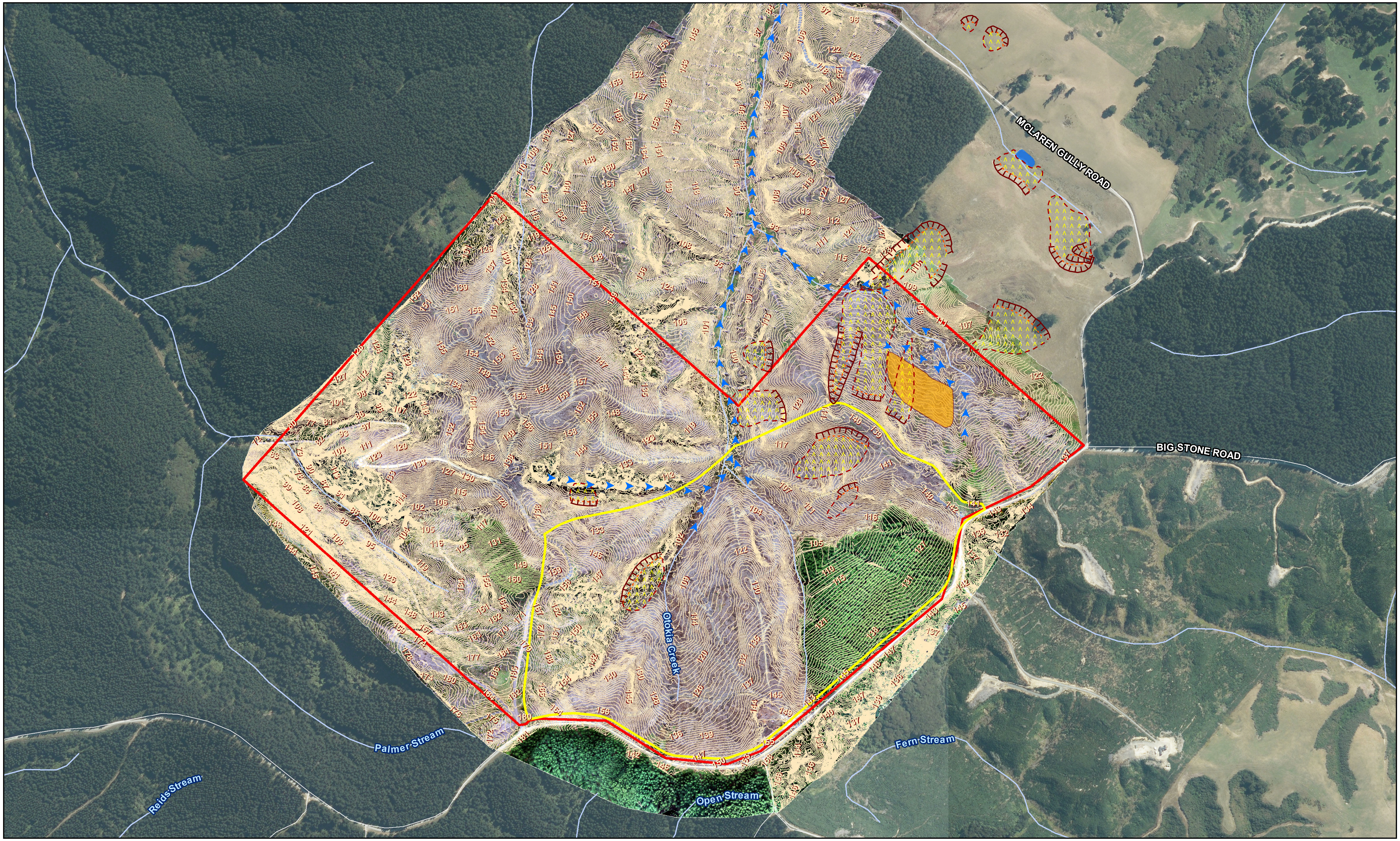
Investigation Location Plan

Figure A1

N:\NZ\Christchurch\Projects\51\12506381\GIS\Maps\Deliverables\Geo\12506381_Z001_Geo_InvestigationLocations.mxd

Level 3, 138 Victoria Street, PO Box 13 468, Christchurch 8141, New Zealand T 64 3 378 0900 F 64 3 378 8001 E chcmail@ghd.com W www.ghd.com

Data source: Aerial imagery - Stantec, LINZ & esri (); General topo - LINZ 2019; Contours - Stantec Created by:jrprice



Paper Size A3

0 50 100 200

Metres

Map Projection: Transverse Mercator
Horizontal Datum: NZGD 2000
Grid: NZGD 2000 New Zealand Transverse Mercator

N

LEGEND

- Site boundary
- Landfill operational extent
- Slope failure headscarp crest
- Slope failure headscarp base
- Slope failure debris runout extent
- Hummocky ground
- Drainage
- Pond
- Possible stockpile area
- Waterways
- Contours (1m)

Dunedin City Council
Smooth Hill Landfill
Geotechnical and Hydrogeological

Job Number	12506381
Revision	A
Date	17 Aug 2020

GHD

Instability Plan

Figure A2

N:\NZ\Christchurch\Projects\51\12506381\GIS\Maps\Deliverables\Geo\12506381_Z002_Geo_InstabilityPlan.mxd

Level 3, 138 Victoria Street, PO Box 13 468, Christchurch 8141, New Zealand T 64 3 378 0900 F 64 3 378 8001 E chcmail@ghd.com W www.ghd.com

Data source: Aerial imagery - Stantec, LINZ & esri (); General topo - LINZ 2019; Contours - Stantec Created by:jrprice

Appendix B – Ground Models ~~Sketches~~

~~Section X-X'~~

~~Section Y-Y'~~

~~Section Z-Z'~~

PLAN

SCALE 1 : 2000

FOR CONSENT
NOT FOR CONSTRUCTION

No	Revision	Drawn	Job Manager	Project Director	Date
0	FOR CONSENT	RG	NE*	SD*	25.05.21

Note: * indicates signatures on original issue of drawing or last revision of drawing

0 20 40 60 80 100m
SCALE 1:2000 AT ORIGINAL SIZE

DUNEDIN CITY COUNCIL kaunihera a-rohe o Ōtepoti

GHD
Level 3, 138 Victoria Street
Christchurch 8013, New Zealand
T 64 3 378 0900 F 64 3 370 8001
E chcmail@ghd.com W www.ghd.com

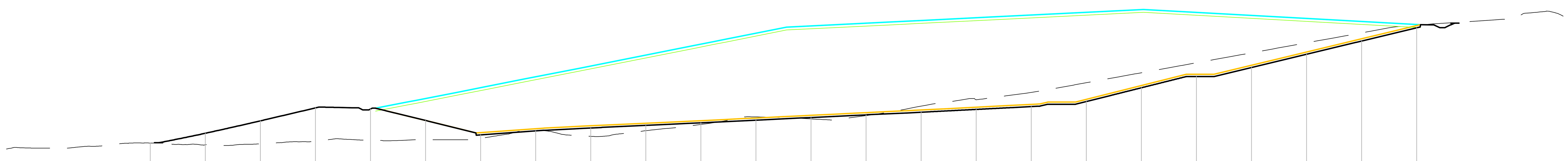
DO NOT SCALE

Drawn	R. GERONIMO	Designer	D. McQUISTAN
Drafting Check	G. DOUGHERTY*	Design Check	S. WEBB*
Approved (Project Director)	S. DOUGLASS*		
Date	25.05.21		
Scale	1:2000 @ A1		This Drawing must not be used for Construction unless signed as Approved

Client **DUNEDIN CITY COUNCIL**
Project **SMOOTH HILL LANDFILL**
Title **SITE INVESTIGATION PLAN**

Original Size **A1** Drawing No: **12506381-01-Q100** Rev: **0**

Plot Date: 25 May 2021 - 1:16 PM Plotted by: Rey Emmanuel Geronimo Cad File No: C:\12d\SW\data\P-00-12D-001\51-12506381 - Smooth Hill Landfill_293\CADD\Drawings\51-12506381-01-Q100.dwg



DATUM RL. 81.00

CHAINAGE	-52.26	-40.00	-20.00	0.00	20.00	40.00	60.00	80.00	100.00	120.00	140.00	160.00	180.00
DESIGN LINER SURFACE LEVEL									107.03	102.62	104.04	105.10	106.03
DESIGN CAPPING SURFACE LEVEL									115.03	118.95	122.91	126.84	130.82
DESIGN BASE LEVEL CUT - / FILL +				0.09	4.22	8.36	12.09	11.21	7.03	1.29	-0.24	3.05	1.90
DESIGN BASE LEVEL BUND / LINER SUBGRADE)				98.93	102.40	106.91	111.62	111.20	107.03	101.82	103.24	104.30	105.23
EXISTING SURFACE LEVEL	96.68	97.00	97.63	98.83	98.18	98.54	99.53	99.99	100.00	100.53	103.48	101.26	103.33

CHAINAGE	200.00	220.00	240.00	260.00	280.00	300.00	320.00	340.00	360.00	380.00	400.00	420.00	440.00	460.00	480.00	500.00	513.27
DESIGN LINER SURFACE LEVEL	106.95	107.92	108.90	109.88	110.84	111.86	112.88	114.71	119.72	123.81	127.10	131.91	136.72	141.53			
DESIGN CAPPING SURFACE LEVEL	134.80	138.78	141.42	142.41	143.41	144.40	145.39	146.38	147.38	146.35	145.24	144.13	143.03	141.92			
DESIGN BASE LEVEL CUT - / FILL +	0.57	-1.22	0.49	0.26	-2.21	-3.51	-5.17	-6.84	-5.47	-5.01	-5.35	-4.16	-2.97	-0.95			
DESIGN BASE LEVEL BUND / LINER SUBGRADE)	106.15	107.12	108.10	109.08	110.04	111.06	112.08	113.91	118.92	123.01	126.30	131.11	135.92	140.73			
EXISTING SURFACE LEVEL	105.59	108.34	107.61	108.82	112.25	114.56	117.25	120.76	124.39	128.01	131.64	135.27	138.89	141.67	143.05	146.08	145.00

LONGITUDINAL SECTION - SECTA

HORZ 1:1000 VERT 1:1000

FOR CONSENT
NOT FOR CONSTRUCTION

No	Revision	Drawn	Job Manager	Project Director	Date
0	FOR CONSENT	RG	NE*	SD*	25.05.21

Note: * indicates signatures on original issue of drawing or last revision of drawing

0 10 20 30 40 50m
SCALE 1:1000 AT ORIGINAL SIZE

DUNEDIN CITY COUNCIL | kaunihera a-rohe o Ōtepoti

GHD

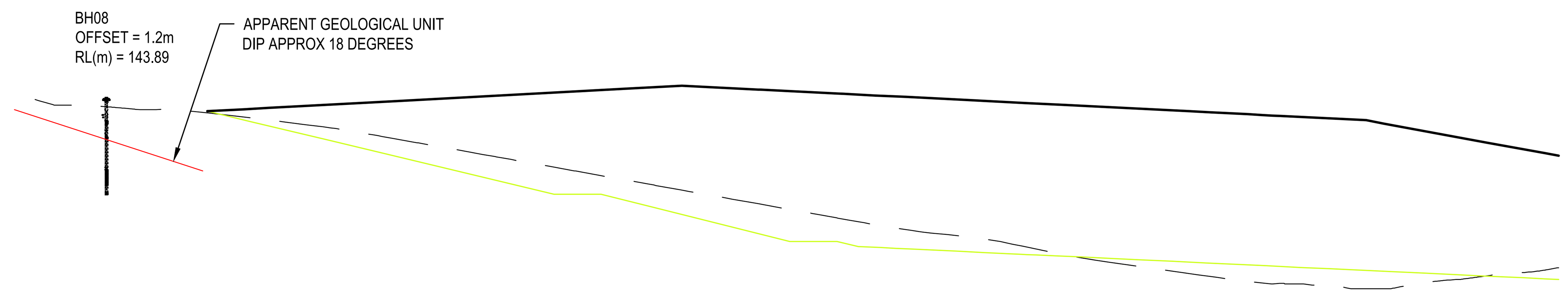
Level 3, 138 Victoria Street
Christchurch 8013, New Zealand
T 64 3 378 0900 **F** 64 3 370 8001
E chcmail@ghd.com **W** www.ghd.com

DO NOT SCALE

Drawn R. GERONIMO | Designer D. McQUISTAN
Drafting Check G. DOUGHERTY* | Design Check S. WEBB*
Approved (Project Director) S. DOUGLASS*
Date 25.05.21
Scale 1:1000 @ A1
This Drawing must not be used for Construction unless signed as Approved

Client **DUNEDIN CITY COUNCIL**
Project **SMOOTH HILL LANDFILL**
Title **SITE INVESTIGATION**
LONGITUDINAL SECTION - SECTION A
Original Size **A1** **Drawing No: 12506381-01-Q101** **Rev: 0**

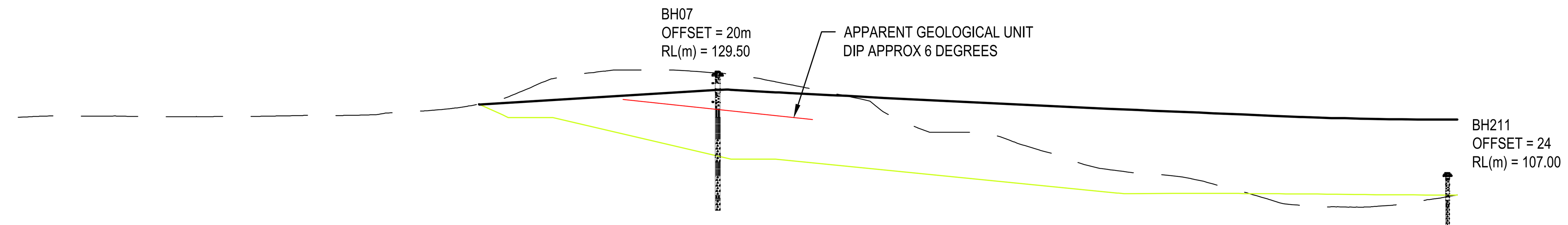
Plot Date: 25 May 2021 - 1:16 PM Plotted by: Rey Emmanuel Geronimo Cad File No: C:\12d\SW\data\P-00-12D-001\51-12506381 - Smooth Hill Landfill_293\CADD\Drawings\51-12506381-01-Q100.dwg



DATUM R.L. 98.00

OFFSET FROM CENTRELINE	DESIGN FINAL CAPPING LEVEL	DESIGN BASE CELL LEVEL	EXISTING SURFACE LEVEL
-160			142.98
-155			142.64
-150			142.22
-145			142.02
-140			142
-135			141.48
-130	141.89	140.85	140.95
-125	142.16	139.64	140.39
-120	142.43	138.44	139.79
-115	142.7	137.24	139.14
-110	142.97	136.04	138.49
-105	143.24	134.84	137.81
-100	143.51	133.65	136.96
-95	143.78	132.45	136.07
-90	144.04	131.25	135.17
-85	144.31	130.05	134.28
-80	144.58	128.85	133.39
-75	144.85	127.65	132.49
-70	145.12	126.45	131.6
-65	145.39	125.25	130.7
-60	145.66	124.05	129.8
-55	145.93	124.05	128.91
-50	146.2	124.05	128.01
-45	146.47	122.8	127.11
-40	146.74	121.54	126.21
-35	147.01	120.29	125.32
-30	146.98	119.04	124.42
-25	146.72	117.79	123.52
-20	146.47	116.53	122.62
-15	146.22	115.28	121.73
-10	145.96	114.03	120.83
-5	145.71	114.03	119.93
0	145.46	114.03	119.03
5	145.2	112.94	118.13
10	144.95	112.7	117.23
15	144.7	112.46	116.48
20	144.44	112.22	115.83
25	144.19	111.98	115.32
30	143.94	111.74	114.71
35	143.69	111.5	113.91
40	143.43	111.26	112.89
45	143.18	111.03	111.87
50	142.93	110.8	110.87
55	142.67	110.57	110
60	142.42	110.33	109.29
65	142.17	110.1	108.58
70	141.91	109.86	107.88
75	141.66	109.62	107.17
80	141.41	109.38	106.47
85	141.15	109.16	105.82
90	140.9	108.92	105.24
95	140.65	108.68	105
100	140.39	108.44	104.86
105	140.14	108.22	104.57
110	139.89	107.98	104
115	139.26	107.74	104
120	138.33	107.5	104.5
125	137.4	107.28	105.31
130	136.47	107.04	105.85
135	135.55	106.8	106.35
140	134.63	106.57	107.04
145	133.72	106.35	107.38
150	132.82	106.11	107.91

CH 400



DATUM R.L. 94.00

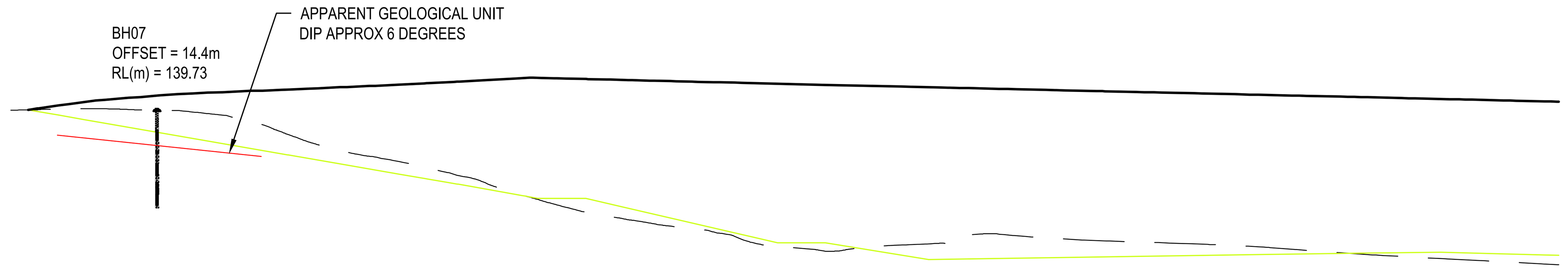
OFFSET FROM CENTRELINE	DESIGN FINAL CAPPING LEVEL	DESIGN BASE CELL LEVEL	EXISTING SURFACE LEVEL
-155			120.65
-150			120.62
-145			120.59
-140			120.49
-135			120.49
-130			120.53
-125			120.55
-120			120.55
-115			120.61
-110			120.7
-105			120.81
-100			120.77
-95			120.83
-90			120.95
-85			121.46
-80			121.72
-75			122.12
-70			122.72
-65	123.35	122.57	123.59
-60	123.65	120.32	124.94
-55	123.96	120.32	127.03
-50	124.26	120.32	129.06
-45	124.57	119.16	129.92
-40	124.87	117.99	130.54
-35	125.18	116.83	131
-30	125.48	115.66	131
-25	125.79	114.5	131
-20	126.09	113.34	130.8
-15	126.4	112.17	130.45
-10	126.53	111.01	130.09
-5	126.26	111.01	129.3
0	125.98	111.01	128.33
5	125.73	110.51	127.36
10	125.46	110.01	126.34
15	125.21	109.51	125.34
20	124.95	109.01	124.27
25	124.69	108.51	121.15
30	124.45	108.01	118.76
35	124.21	107.51	117
40	123.97	107.01	117
45	123.73	106.52	117
50	123.49	106.02	116.11
55	123.24	105.52	114.31
60	123	105.02	112.79
65	122.76	104.52	111.21
70	122.52	104.02	109.59
75	122.28	103.54	108.61
80	122.06	103.24	107.97
85	121.85	103.26	107.63
90	121.63	103.28	107.13
95	121.41	103.3	106.22
100	121.2	103.3	104.94
105	120.98	103.26	103.64
110	120.76	103.23	102.17
115	120.55	103.19	100.88
120	120.37	103.15	100.35
125	120.2	103.09	100.29
130	120.09	103.05	100.2
135	119.97	103	100.4
140	119.92	102.97	100.79
145	119.87	102.94	101.61
150	119.86	102.92	102.42

CH 700

DATUM R.L. 103.00

OFFSET FROM CENTRELINE	DESIGN FINAL CAPPING LEVEL	DESIGN BASE CELL LEVEL	EXISTING SURFACE LEVEL
-155			140.52
-150			140.05
-145			139.17
-140			137.93
-135			137.1
-130			135.61
-125	137.34	136.83	134.16
-120	137.77	135.87	133.28
-115	138.2	134.92	132.27
-110	138.63	133.96	131.27
-105	139.06	133.01	130.13
-100	139.49	132.06	129.08
-95	139.92	131.1	127.76
-90	140.35	130.15	125.85
-85	140.77	129.2	123.81
-80	141.18	128.24	122.78
-75	141.59	127.29	121.49
-70	141.89	126.33	120.21
-65	141.85	125.38	119.28
-60	141.85	124.42	118.53
-55	141.85	124.42	117.86
-50	141.85	124.42	117.26
-45	141.86	123.17	116.68
-40	141.86	121.92	116.17
-35	141.86	120.67	115.24
-30	141.86	119.42	114.1
-25	141.86	118.17	112.77
-20	141.86	116.92	111
-15	141.86	115.67	111.38
-10	141.87	114.42	111.93
-5	141.87	114.42	112.05
0	141.87	114.42	111.94
5	141.87	113.25	111.62
10	141.87	112.08	111.3
15	141.87	110.91	110.99
20	141.87	109.74	110.75
25	141.87	109.28	110.66
30	141.88	109.39	110.57
35	141.88	109.49	110.55
40	141.88	109.6	110.58
45	141.95	109.7	110.57
50	142.04	109.81	110.53
55	142.12	109.91	110.31
60	142.21	110.02	110.01
65	142.3	110.13	109.3
70	142.39	110.23	109.38
75	142.48	110.34	109.56
80	142.56	110.45	109.75
85	142.65	110.55	109.93
90	142.74	110.66	110.15
95	142.83	110.77	110.38
100	142.92	110.88	110.62
105	143	110.96	110.86
110	143.09	110.86	111.12
115	143.18	110.75	111.4
120	143.27	110.64	111.69
125	143.36	110.53	111.97
130	143.44	110.42	112.18
135	143.53	110.32	112.37
140	143.62	110.22	112.5
145	143.71	110.12	112.61
150	143.8	110.02	112.71

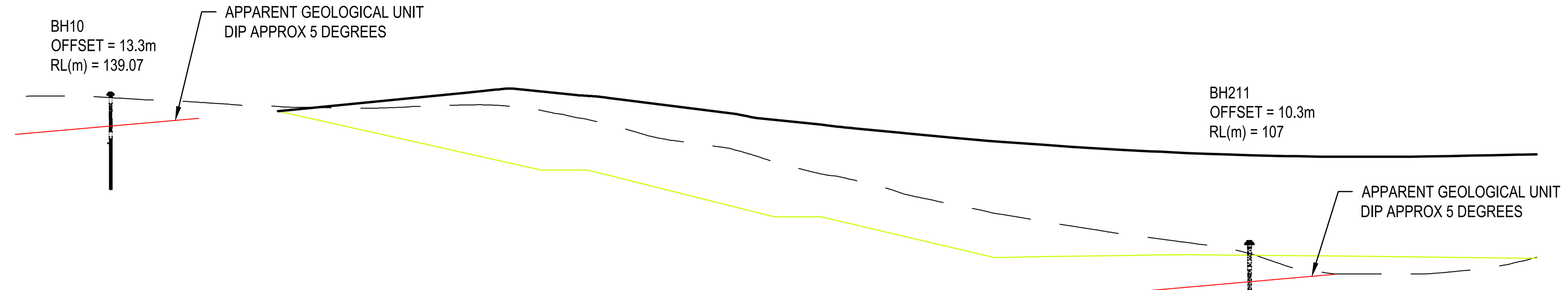
CH 240



DATUM R.L. 102.00

OFFSET FROM CENTRELINE	DESIGN FINAL CAPPING LEVEL	DESIGN BASE CELL LEVEL	EXISTING SURFACE LEVEL
-155	141.79	138.21	140.42
-150	142.29	137.34	140.4
-145	142.75	136.47	140.24
-140	143.12	135.6	140.17
-135	143.49	134.73	140.02
-130	143.73	133.87	139.53
-125	143.92	133	138.99
-120	144.15	132.13	137.76
-115	144.31	131.27	136.11
-110	144.51	130.41	134.26
-105	144.72	129.55	132.63
-100	144.92	128.68	131.4
-95	145.16	127.81	130.47
-90	145.4	126.94	129.57
-85	145.66	126.08	128.54
-80	145.91	125.22	127.34
-75	146.15	124.37	126.16
-70	146.42	123.49	124.5
-65	146.68	122.6	122.89
-60	146.87	121.72	121.47
-55	146.75	121.72	120.06
-50	146.62	121.72	118.8
-45	146.5	120.56	117.88
-40	146.37	119.39	117.09
-35	146.25	118.23	116.25
-30	146.13	117.07	115.62
-25	146	115.91	115.08
-20	145.88	114.74	114.09
-15	145.76	113.58	112.41
-10	145.64	112.42	111.75
-5	145.53	112.42	111.28
0	145.41	112.42	110.67
5	145.29	111.61	111.11
10	145.17	110.8	111.63
15	145.05	109.98	111.91
20	144.94	109.17	112.14
25	144.83	108.98	112.32
30	144.71	109.05	114.07
35	144.6	109.13	114.35
40	144.49	109.2	113.87
45	144.37	109.27	113.5
50	144.26	109.35	113.21
55	144.15	109.42	112.96
60	144.03	109.49	112.79
65	143.92	109.57	112.61
70	143.8	109.64	112.44
75	143.69	109.72	112.26
80	143.57	109.79	112.08
85	143.46	109.86	111.87
90	143.34	109.94	111.59
95	143.23	110.01	111.25
100	143.11	110.08	110.88
105	143	110.16	110.52
110	142.88	110.23	110.17
115	142.77	110.3	109.86
120	142.65	110.36	109.59
125	142.54	110.42	109.32
130	142.42	110.42	109.05
135	142.31	110.29	108.79
140	142.19	110.16	108.52
145	142.07	110.03	108.26
150	141.96	109.91	108.01

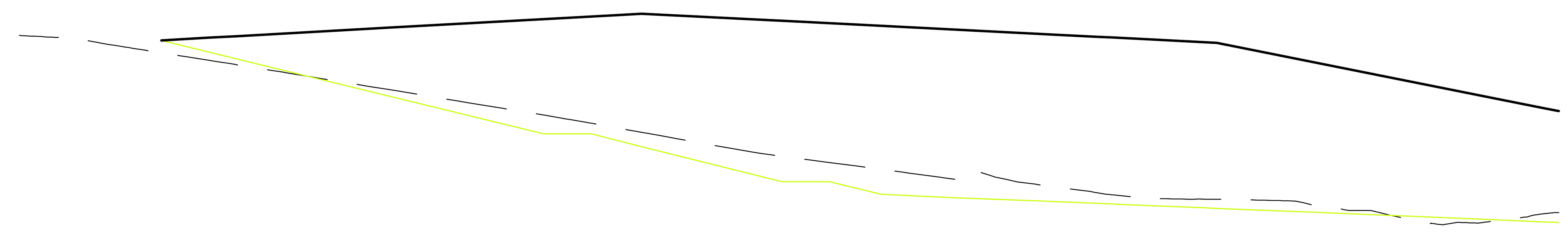
CH 560



DATUM R.L. 95.00

OFFSET FROM CENTRELINE	DESIGN FINAL CAPPING LEVEL	DESIGN BASE CELL LEVEL	EXISTING SURFACE LEVEL
-155			138.85
-150			138.54
-145			138.21
-140			138.08
-135			137.77
-130			137.46
-125			137.09
-120			136.92
-115	135.86	135.49	136.67
-110	136.35	134.37	136.57
-105	136.85	133.26	136.48
-100	137.34	132.14	136.4
-95	137.83	131.03	136.41
-90	138.33	129.91	136.59
-85	138.82	128.8	136.77
-80	139.32	127.68	136.97
-75	139.81	126.57	137.1
-70	140.3	125.46	137.04
-65	140.48	124.34	136.62
-60	139.97	123.22	135.94
-55	139.45	123.22	135.01
-50	139.04	123.22	134.01
-45	138.52	121.97	132.87
-40	137.89	120.72	131.42
-35	137.26	119.47	130.09
-30	136.63	118.22	129.29
-25	136	116.97	128.57
-20	135.37	115.72	127.9
-15	134.5	114.47	126.54
-10	133.93	113.22	124.89
-5	133.41	113.22	123.59
0	132.89	113.22	122.38
5	132.3	112.05	121.52
10	131.82	110.88	120.34
15	131.33	109.71	119.04
20	130.88	108.54	117.57
25	130.39	107.36	116.51
30	129.94	106.19	115.46
35	129.51	105.02	114.44
40	129.13	104.59	113.49
45	128.75	104.68	112.66
50	128.4	104.76	111.82
55	128.05	104.84	111.03
60	127.76	104.91	110.22
65	127.5	104.99	109.5
70	127.21	105.09	108.83
75	126.99	105.12	108.16
80	126.77	105.13	107.5
85	126.59	105.06	106.77
90	126.42	105.03	105.81
95	126.29	104.99	104.25
100	126.19	104.96	102.61
105	126.11	104.92	101.65
110	126.05	104.89	101
115	126.04	104.84	101
120	126.04	104.78	101
125	126.06	104.72	101
130	126.13	104.65	101.08
135	126.19	104.59	101.5
140	126.29	104.53	101.98
145	126.39	104.47	102.75
150	126.49	104.4	103.79

CH 120



DATUM R.L. 98.00

OFFSET FROM CENTRELINE	DESIGN FINAL CAPPING LEVEL	DESIGN BASE CELL LEVEL	EXISTING SURFACE LEVEL
-160			143.15
-155			142.48
-150			141.6
-145			140.8
-140	142.66	142.59	140.01
-135	142.94	141.37	139.22
-130	143.22	140.15	138.43
-125	143.5	138.92	137.64
-120	143.77	137.7	136.84
-115	144.05	136.48	136.02
-110	144.33	135.26	135.2
-105	144.61	134.04	134.38
-100	144.88	132.82	133.56
-95	145.16	131.6	132.74
-90	145.44	130.38	131.92
-85	145.72	129.15	131.1
-80	145.99	127.93	130.28
-75	146.27	126.71	129.46
-70	146.55	125.49	128.64
-65	146.83	124.27	127.82
-60	147.11	123.05	127
-55	147.38	123.05	126.16
-50	147.66	123.05	125.27
-45	147.94	121.8	124.38
-40	148.21	120.54	123.48
-35	148.02	119.29	122.58
-30	147.77	118.03	121.68
-25	147.51	116.78	120.78
-20	147.26	115.53	119.89
-15	147.01	114.27	119.04
-10	146.75	113.02	118.38
-5	146.5	113.02	117.72
0	146.25	113.02	117.06
5	146	111.81	116.4
10	145.74	110.59	115.74
15	145.49	110.22	115.07
20	145.24	109.98	114.41
25	144.98	109.76	113.75
30	144.73	109.56	113.4
35	144.48	109.36	113.95
40	144.22	109.16	112.93
45	143.97	108.96	112.27
50	143.72	108.76	111.61
55	143.46	108.56	110.93
60	143.21	108.35	110.24
65	142.96	108.14	109.68
70	142.71	107.94	109.51
75	142.45	107.75	109.38
80	142.2	107.52	109.36
85	141.38	107.32	109.32
90	140.39	107.12	109.2
95	139.39	106.92	109.07
100	138.39	106.69	108.47
105	137.4	106.49	107.66
110	136.4	106.29	107
115	135.41	106.1	106.67
120	134.41	105.87	105.51
125	133.41	105.67	104.41
130	132.42	105.47	104.22
135	131.42	105.28	104.4
140	130.42	105.06	104.91
145	129.43	104.84	105.51
150	128.43	104.63	106.35

CH 500

FOR CONSENT
NOT FOR CONSTRUCTION

0	FOR CONSENT	RG	NE*	SD*	25.05.21
No	Revision — Note: * indicates signatures on original issue of drawing or last revision of drawing	Drawn	Job Manager	Project Director	Date

0 10 20 30 40 50m
SCALE 1:1000 AT ORIGINAL SIZE

DUNEDIN CITY COUNCIL kaunihera a-rohe o Ōtepoti

GHD
Level 3, 138 Victoria Street
Christchurch 8013, New Zealand
T 64 3 378 0900 F 64 3 370 8001
E chcmail@ghd.com W www.ghd.com

DO NOT SCALE

Drawn	R. GERONIMO	Designer	D. McQUISTAN
Drafting Check	G. DOUGHERTY*	Design Check	S. WEBB*
Approved (Project Director)	S. DOUGLASS*		
Date	25.05.21		
Scale	1:1000 @ A1	This Drawing must not be used for Construction unless signed as Approved	

Client **DUNEDIN CITY COUNCIL**
Project **SMOOTH HILL LANDFILL**
Title **SITE INVESTIGATION CROSS SECTIONS**

Original Size **A1** Drawing No: **12506381-01-Q102** Rev: **0**

Plot Date: 25 May 2021 - 1:16 PM Plotted by: Rey Emmanuel Geronimo Cad File No: C:\12d\SW\data\P-00-12D-001\51-12506381 - Smooth Hill Landfill_293\CADD\Drawings\51-12506381-01-Q100.dwg

Appendix C – Slope Stability Modelling Results

~~Appendix C.1 - Cross Section Plan~~

~~Appendix C.2 – Section A Western Slope~~

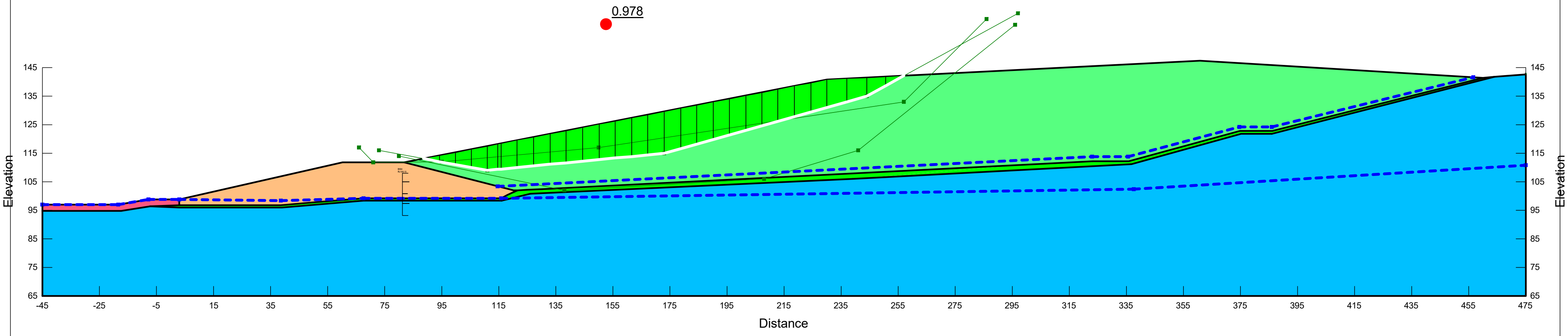
~~Appendix C.3 – Section B South Western Slope~~

~~Appendix C.4 – Section C Eastern Slope~~

~~Appendix C.5 – Section D Toe Bund (Northern)~~

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	03 Residual Soil (SILT)	Mohr-Coulomb	18	2	30	0	1
	10 Bulk/Eng Fill	Mohr-Coulomb	18	0	30	0	1
	11 MSW Long term	Mohr-Coulomb	12	9	22	0	2
	13 SW Siltstone (VW-W)	Mohr-Coulomb	18	50	30	0	1
	14 Residual Soil	Mohr-Coulomb	18	2	30	0	1

Horz Seismic Coef.: 0.31



Level 3
138 Victoria Street
Christchurch Central,
Canterbury
8013
Ph: +64 3 378 0900

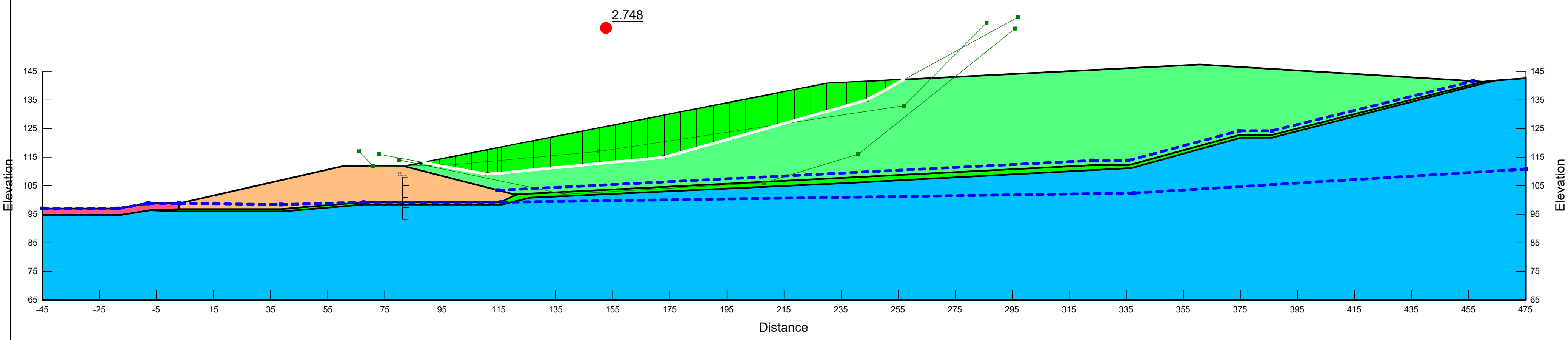
DUNEDIN CITY COUNCIL | kaunihera a-rohe o Ōtepoti

Title: Slope Stability for Section A.gsz		
Analysis: 3.2 Waste Stabilty - ULS (UD)		Date: 29/04/2021
Comments:	Scale: 1:2,000 @ A4	Created By: Dhugal McQuistan

Directory: N:\NZ\Auckland\Projects\51\12529451\Tech\Geotech\02 New design\A. Slope Stability Analysis Working\

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	03 Residual Soil (SILT)	Mohr-Coulomb	18	2	30	0	1
	10 Bulk/Eng Fill	Mohr-Coulomb	18	0	30	0	1
	11 MSW Long term	Mohr-Coulomb	12	9	22	0	2
	13 SW Siltstone (VW-W)	Mohr-Coulomb	18	50	30	0	1
	14 Residual Soil	Mohr-Coulomb	18	2	30	0	1

Horz Seismic Coef.:



GHD
Level 3
138 Victoria Street
Christchurch Central,
Canterbury
8013
Ph: +64 3 378 0900

DUNEDIN CITY COUNCIL | kaunihera a-rohe o Ōtepoti

Title: Slope Stability for Section A.gsz

Analysis: 3.0 Waste Stabilty - Static (UD)

Date: 29/04/2021

Comments:

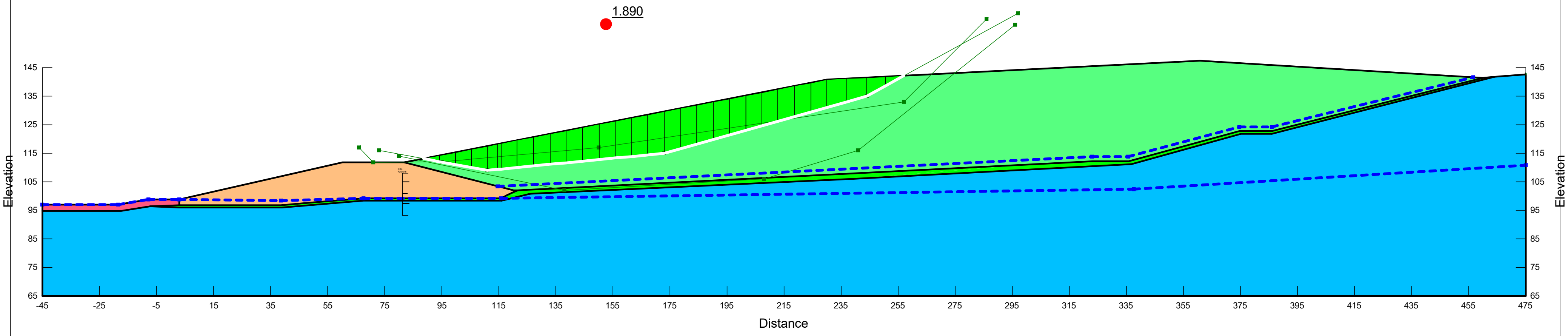
Scale: 1:2,000 @ A4

Created By: Dhugal McQuistan

Directory: N:\NZ\Auckland\Projects\51\12529451\Tech\Geotech\02 New design\A. Slope Stability Analysis Working\

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	03 Residual Soil (SILT)	Mohr-Coulomb	18	2	30	0	1
	10 Bulk/Eng Fill	Mohr-Coulomb	18	0	30	0	1
	11 MSW Long term	Mohr-Coulomb	12	9	22	0	2
	13 SW Siltstone (VW-W)	Mohr-Coulomb	18	50	30	0	1
	14 Residual Soil	Mohr-Coulomb	18	2	30	0	1

Horz Seismic Coef.: 0.08



GHD
Level 3
138 Victoria Street
Christchurch Central,
Canterbury
8013
Ph: +64 3 378 0900

DUNEDIN CITY COUNCIL | kaunihera a-rohe o Ōtepoti

Title: Slope Stability for Section A.gsz

Analysis: 3.1 Waste Stabilty - SLS (UD)

Date: 29/04/2021

Comments:

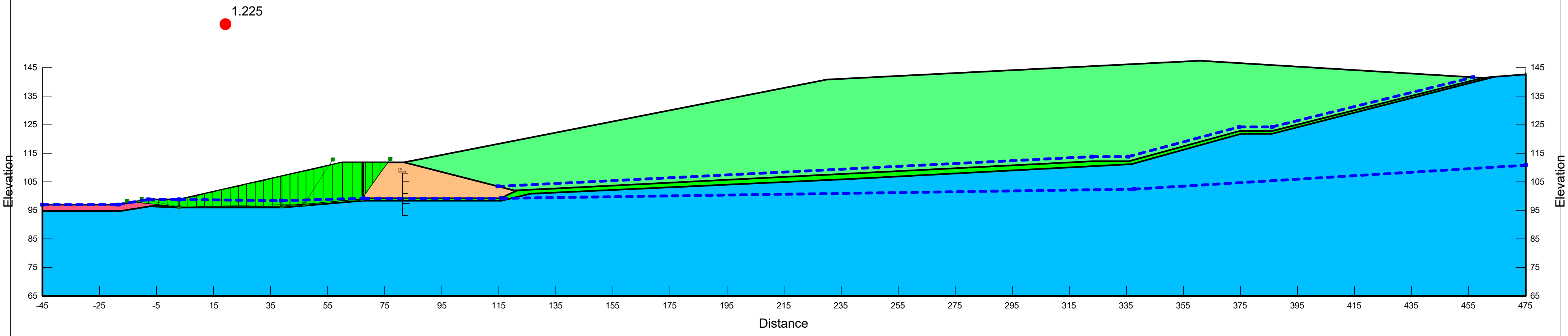
Scale: 1:2,000 @ A4

Created By: Dhugal McQuistan

Directory: N:\NZ\Auckland\Projects\51\12529451\Tech\Geotech\02 New design\A. Slope Stability Analysis Working\

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	03 Residual Soil (SILT)	Mohr-Coulomb	18	2	30	0	1
	10 Bulk/Eng Fill	Mohr-Coulomb	18	0	30	0	1
	11 MSW Long term	Mohr-Coulomb	12	9	22	0	2
	13 SW Siltstone (VW-W)	Mohr-Coulomb	18	50	30	0	1
	14 Residual Soil	Mohr-Coulomb	18	2	30	0	1

Horz Seismic Coef.: 0.31

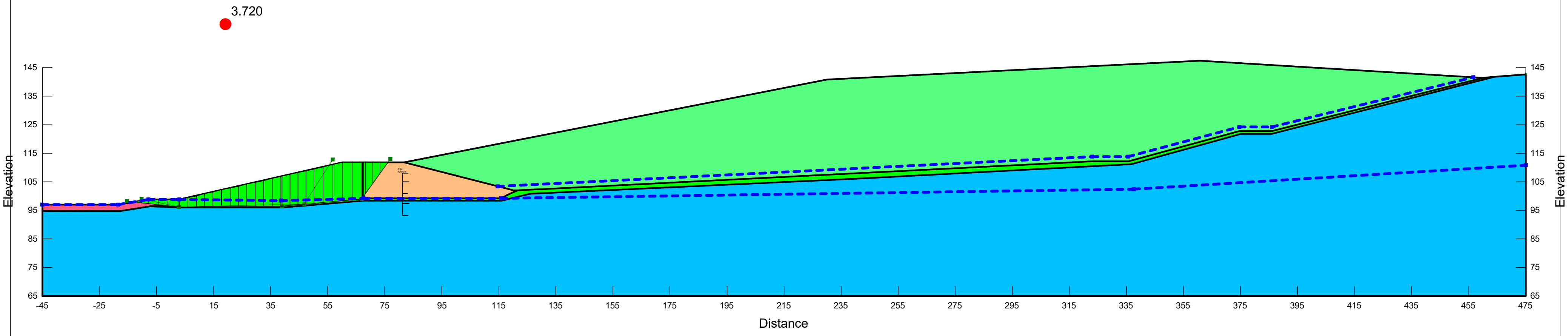


Level 3 138 Victoria Street Christchurch Central, Canterbury 8013 Ph: +64 3 378 0900	DUNEDIN CITY COUNCIL kaunihera a-rohe o Ōtepoti	Title: Slope Stability for Section A.gsz
		Analysis: 1.2 Local Bund Stabilty - ULS (UD) — Date: 29/04/2021
		Comments: — Scale: 1:2,000 @ A4 — Created By: Dhugal McQuistan

Directory: N:\NZ\Auckland\Projects\51\12529451\Tech\Geotech\02 New design\A. Slope Stability Analysis Working\

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	03 Residual Soil (SILT)	Mohr-Coulomb	18	2	30	0	1
	10 Bulk/Eng Fill	Mohr-Coulomb	18	0	30	0	1
	11 MSW Long term	Mohr-Coulomb	12	9	22	0	2
	13 SW Siltstone (VW-W)	Mohr-Coulomb	18	50	30	0	1
	14 Residual Soil	Mohr-Coulomb	18	2	30	0	1

Horz Seismic Coef.:



GHD
Level 3
138 Victoria Street
Christchurch Central,
Canterbury
8013
Ph: +64 3 378 0900

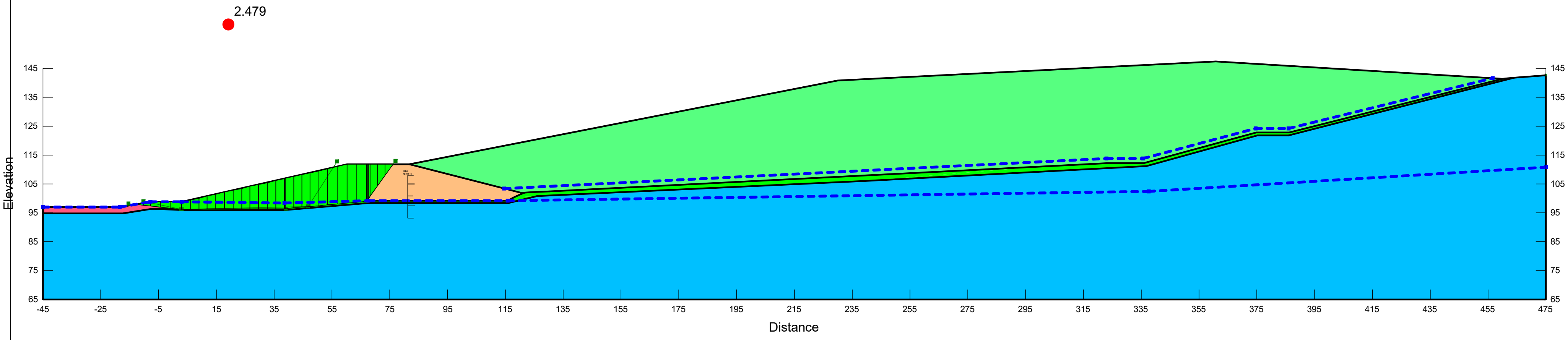
DUNEDIN CITY COUNCIL | kaunihera a-rohe o Ōtepoti

Title: Slope Stability for Section A.gsz		
Analysis: 1.0 Local Bund Stabilty - Static (UD)		Date: 29/04/2021
Comments:	Scale: 1:2,000 @ A4	Created By: Dhugal McQuistan

Directory: N:\NZ\Auckland\Projects\51\12529451\Tech\Geotech\02 New design\A. Slope Stability Analysis Working\

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	03 Residual Soil (SILT)	Mohr-Coulomb	18	2	30	0	1
	10 Bulk/Eng Fill	Mohr-Coulomb	18	0	30	0	1
	11 MSW Long term	Mohr-Coulomb	12	9	22	0	2
	13 SW Siltstone (VW-W)	Mohr-Coulomb	18	50	30	0	1
	14 Residual Soil	Mohr-Coulomb	18	2	30	0	1

Horz Seismic Coef.: 0.08



GHD
Level 3
138 Victoria Street
Christchurch Central,
Canterbury
8013
Ph: +64 3 378 0900

DUNEDIN CITY COUNCIL | kaunihera a-rohe o Ōtepoti

Title: Slope Stability for Section A.gsz

Analysis: 1.1 Local Bund Stabilty - SLS (UD)

Date: 29/04/2021

Comments:

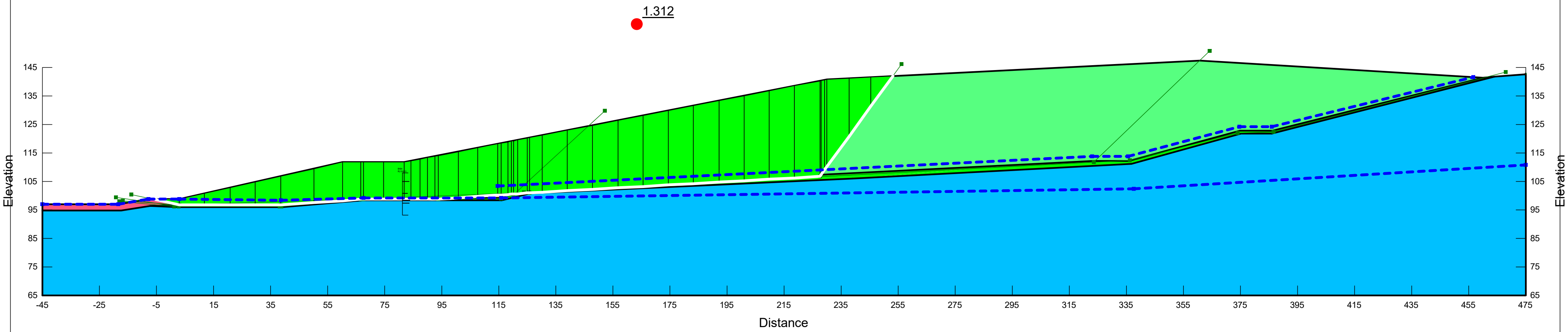
Scale: 1:2,000 @ A4

Created By: Dhugal McQuistan

Directory: N:\NZ\Auckland\Projects\51\12529451\Tech\Geotech\02 New design\A. Slope Stability Analysis Working\

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	03 Residual Soil (SILT)	Mohr-Coulomb	18	2	30	0	1
	10 Bulk/Eng Fill	Mohr-Coulomb	18	0	30	0	1
	11 MSW Long term	Mohr-Coulomb	12	9	22	0	2
	13 SW Siltstone (VW-W)	Mohr-Coulomb	18	50	30	0	1
	14 Residual Soil	Mohr-Coulomb	18	2	30	0	1

Horz Seismic Coef.: 0.31



Level 3
138 Victoria Street
Christchurch Central,
Canterbury
8013
Ph: +64 3 378 0900

Title: Slope Stability for Section A.gsz

Analysis: 2.2 Global Bund Stabilty - ULS (UD)

Date: 29/04/2021

Comments:

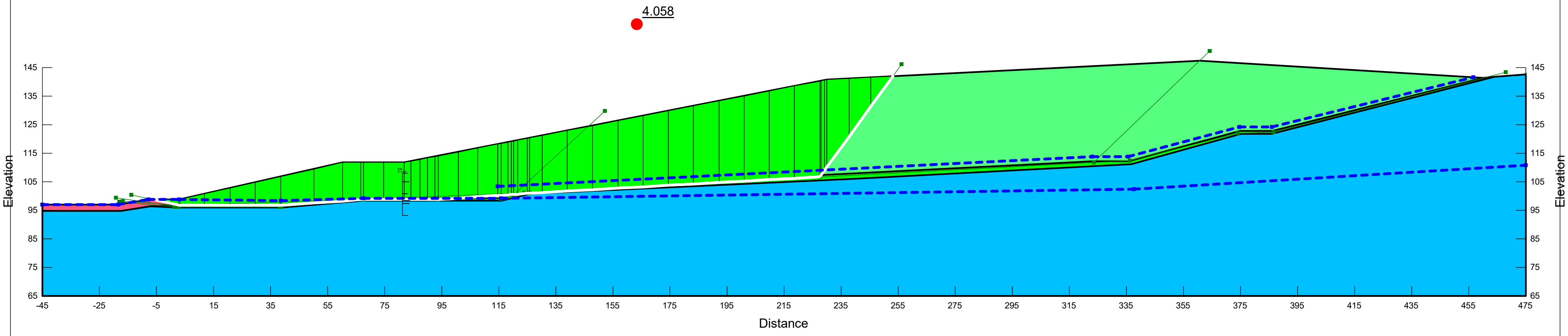
Scale: 1:2,000 @ A4

Created By: Dhugal McQuistan

Directory: N:\NZ\Auckland\Projects\51\12529451\Tech\Geotech\02 New design\A. Slope Stability Analysis Working\

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	03 Residual Soil (SILT)	Mohr-Coulomb	18	2	30	0	1
	10 Bulk/Eng Fill	Mohr-Coulomb	18	0	30	0	1
	11 MSW Long term	Mohr-Coulomb	12	9	22	0	2
	13 SW Siltstone (VW-W)	Mohr-Coulomb	18	50	30	0	1
	14 Residual Soil	Mohr-Coulomb	18	2	30	0	1

Horz Seismic Coef.:



GHD
Level 3
138 Victoria Street
Christchurch Central,
Canterbury
8013
Ph: +64 3 378 0900

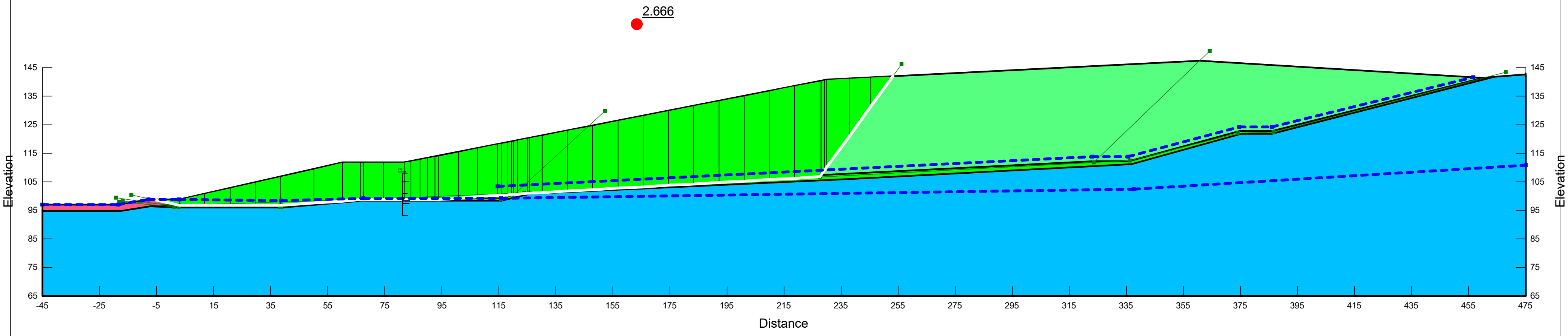
DUNEDIN CITY COUNCIL | kaunihera a-rohe o Ōtepoti

Title: Slope Stability for Section A.gsz		
Analysis: 2.0 Global Bund Stabilty - Static (UD)		Date: 29/04/2021
Comments:	Scale: 1:2,000 @ A4	Created By: Dhugal McQuistan

Directory: N:\NZ\Auckland\Projects\51\12529451\Tech\Geotech\02 New design\A. Slope Stability Analysis Working\

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	03 Residual Soil (SILT)	Mohr-Coulomb	18	2	30	0	1
	10 Bulk/Eng Fill	Mohr-Coulomb	18	0	30	0	1
	11 MSW Long term	Mohr-Coulomb	12	9	22	0	2
	13 SW Siltstone (VW-W)	Mohr-Coulomb	18	50	30	0	1
	14 Residual Soil	Mohr-Coulomb	18	2	30	0	1

Horz Seismic Coef.: 0.08



GHD
Level 3
138 Victoria Street
Christchurch Central,
Canterbury
8013
Ph: +64 3 378 0900

DUNEDIN CITY COUNCIL | kaunihera a-rohe o Ōtepoti

Title: Slope Stability for Section A.gsz

Analysis: 2.1 Global Bund Stabilty - SLS (UD)

Date: 29/04/2021

Comments:

Scale: 1:2,000 @ A4

Created By: Dhugal McQuistan

Directory: N:\NZ\Auckland\Projects\51\12529451\Tech\Geotech\02 New design\A. Slope Stability Analysis Working\

This report has been prepared by Matt Fitzmaurice, John Southworth and Dhugal McQuistan under the direction of Samantha Webb, a Technical Director and Engineering Geologist with GHD Ltd. Matt has 9 years as an engineering geologist, John has 23 years experience as an engineering geologist and Dhugal has 4 years experience as a geotechnical engineer. Samantha has 30 years in all aspects of engineering geology including a number of landfill projects and has the following qualifications BSc (Hons) Earth Sciences and MSc Engineering Geology.

GHD

Level 4 Security Building
115 Stuart Street
T: 64 3 378 0991 F: E:

https://ghdnet-my.sharepoint.com/personal/mette_vanlith_ghd_com/Documents/Desktop/Smooth Hill/20210520 Technical Report G - Geotechnical Interpretive Report_20052021.docx

Document Status

Revision	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
Rev1	M.Fitzmaurice/J.Southworth/ D.McQuistan.	S.Webb		S.Douglass		18-8-2020
Rev2	D, McQuistan	S.Webb		S. Douglass		26-5-2021

www.ghd.com