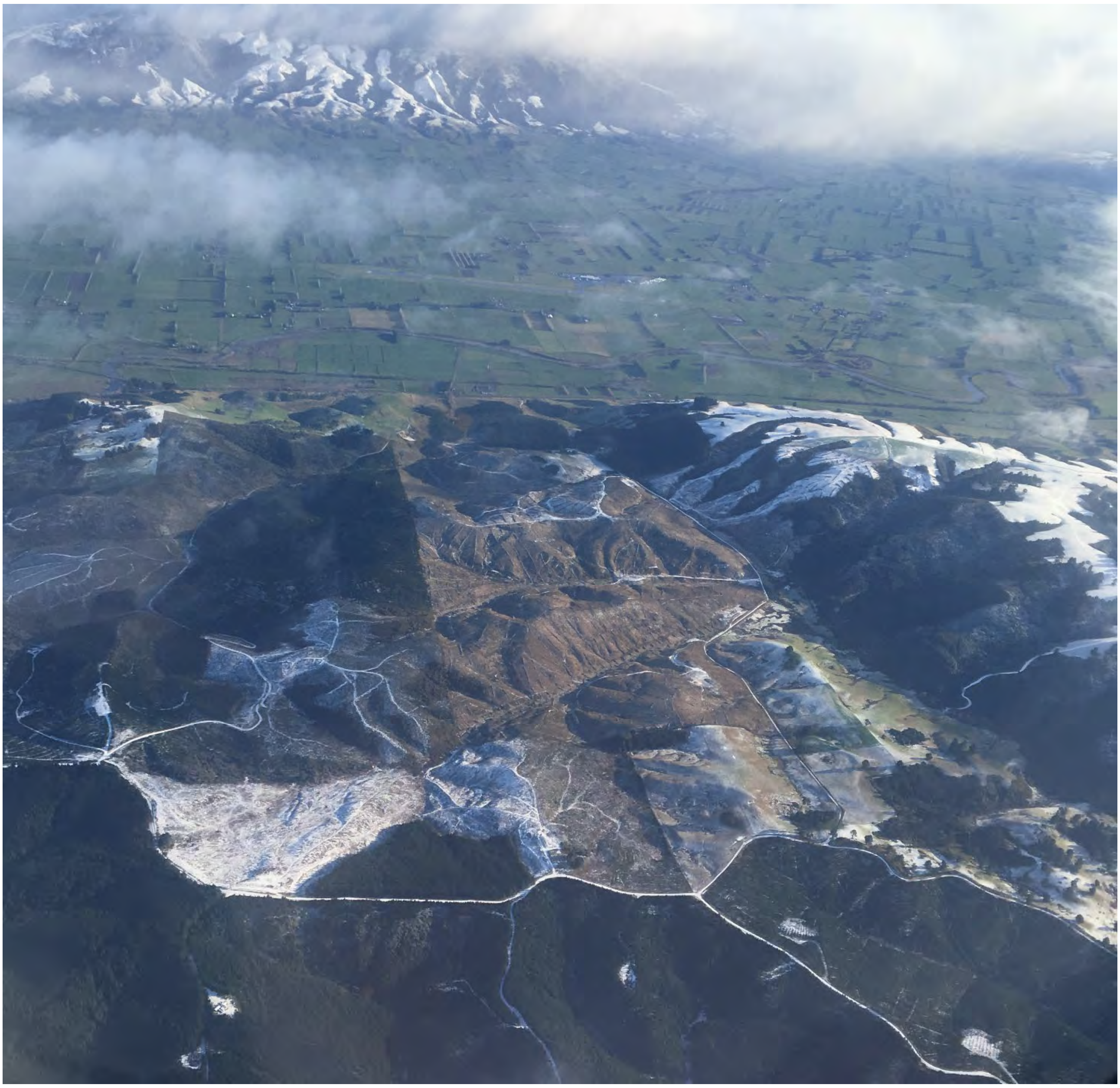
Appendix 5: Geotechnical Interpretive Report



Dunedin City Council

Waste Futures - Smooth Hill Landfill
Geotechnical Interpretive Report

August 2020

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1. Introduction

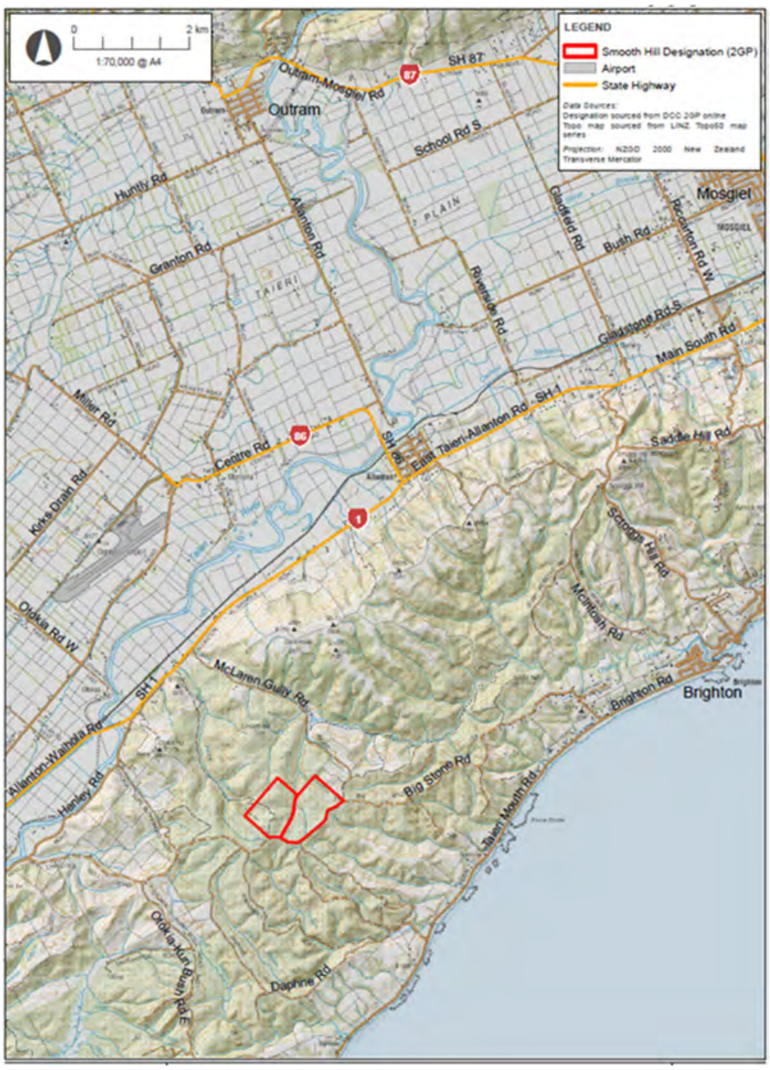
1.1 Background

The Dunedin City Council (Council) collects residential waste and manages the disposal of both residential and the majority of commercial waste for 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 likely to come to the end of its functional life sometime between 2023 and 2028..

The Council commenced a search for a new landfill location in the late 1980s and early 1990 and selected the Smooth Hill site in south-west Dunedin, shown in Figure 1 below, as the preferred option. 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 current landowner, Fulton Hogan Ltd, to purchase the land. Over the following period, 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

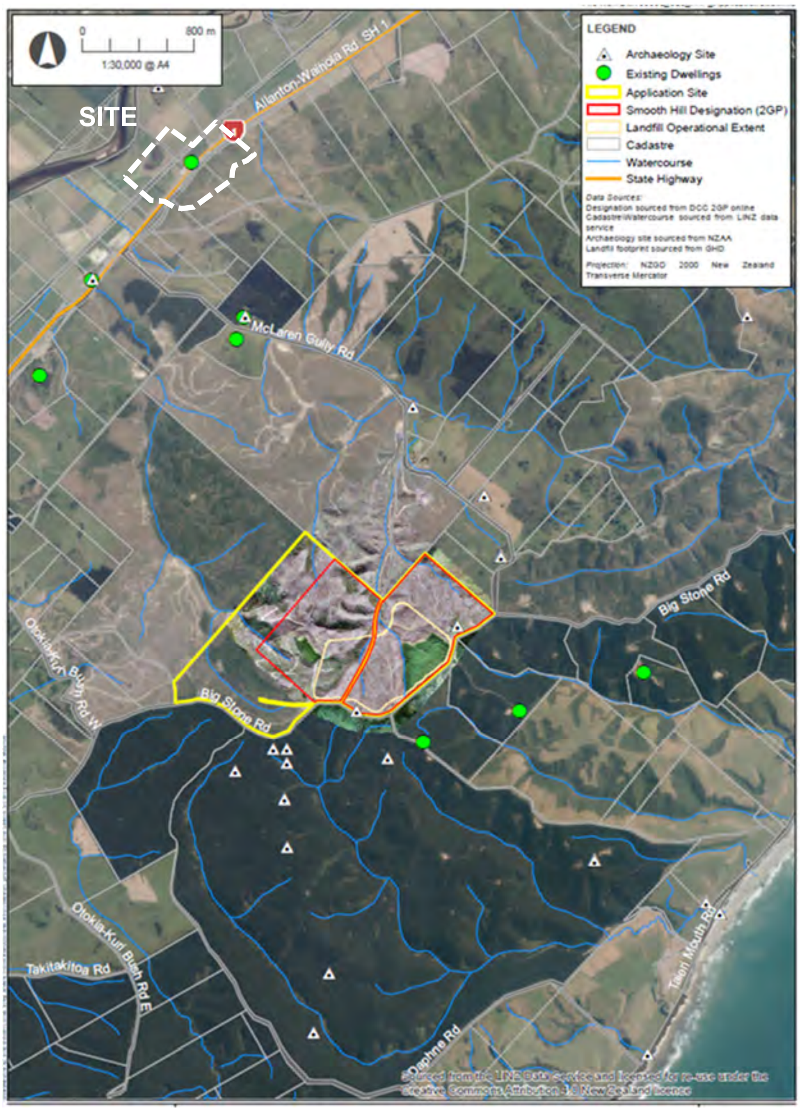


As part of the Waste Future's Project, the Council has reconfirmed the technical suitability of Smooth Hill for the disposal of waste.. The Council has proceeded to develop a concept design for the landfill, site access via existing rural roads, and associated road upgrades. The concept design (the subject 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 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



1.2 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 engineeered fill material; and
- Assess stability of liner and capping slopes (both intermediate and final), as well as proposed cut and fill slopes.

1.3 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 Report (2019): "Waste Futures – Smooth Hill: Geotechnical Factual Report", dated August 2020, reference 12506856.

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 2 to 5 m thick layer of loess. 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. A variation of note from the published geology is 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 occurring to the south of the site. However, their occurrence appears to be more widespread than previously mapped.

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.

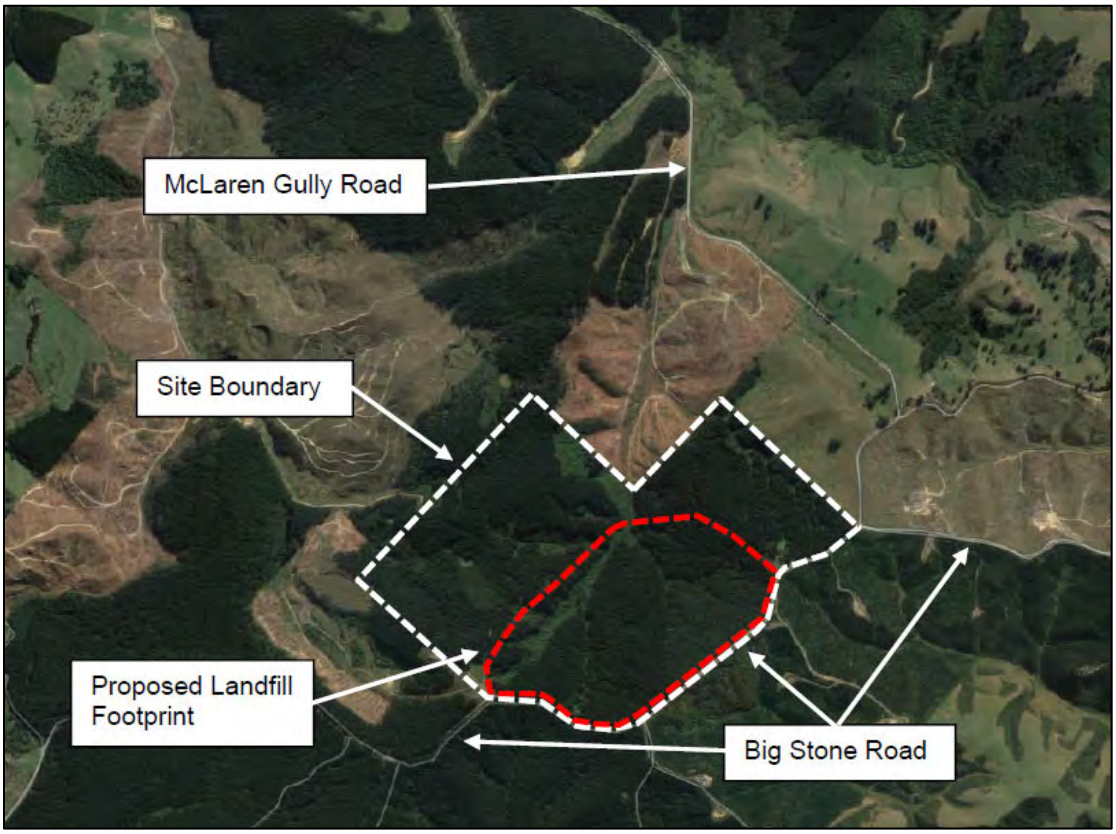


Figure 3 - Proposed landfill site (base image sourced from Google Maps)

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. 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.

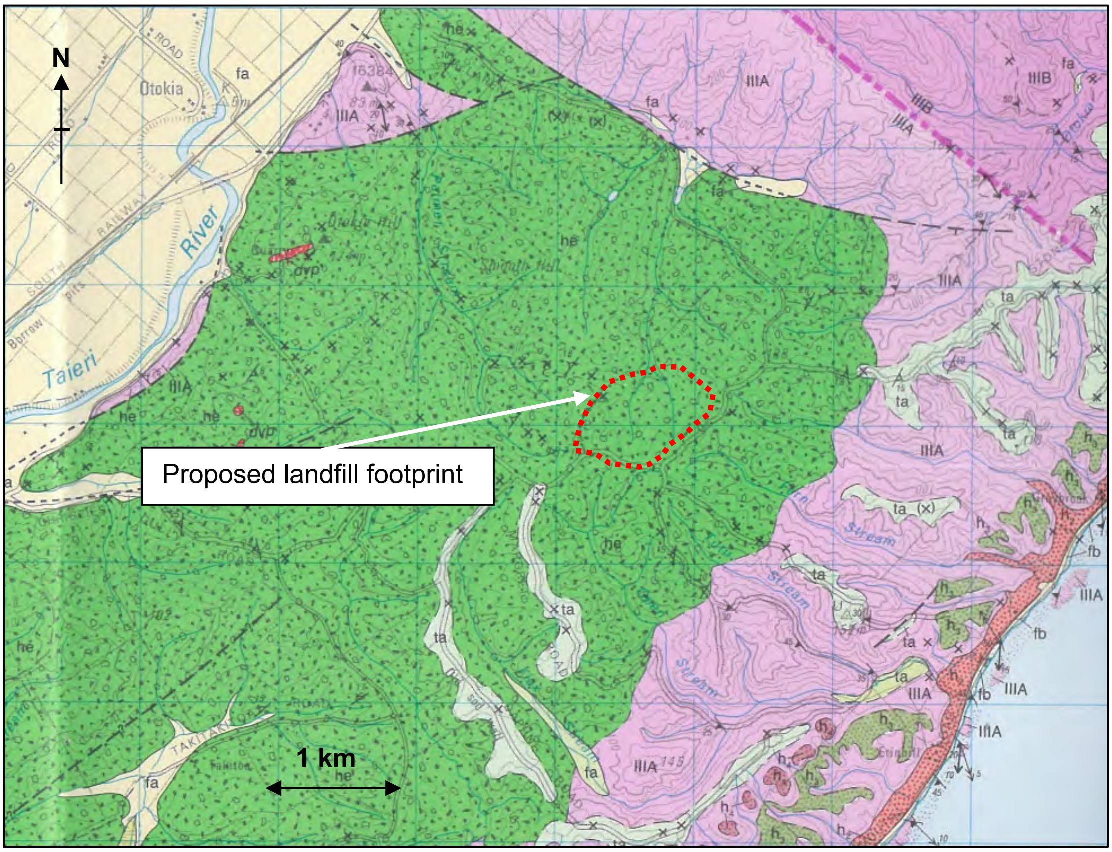
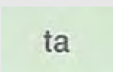


Figure 4 - Excerpt from 1:50,000 Geology of the Milton Area (Bishop, 1994)

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 Taratu Formation

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) and encountered in boreholes (BH09, BH209 & BH10). 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 - Taratu Formation outcrop near the northwest extent of the site

2.3.2 Henley Breccia Formation

Much of the Henley Breccia Formation is massive, but where bedding is evident, it typically dips 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. Observed unit contacts in 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 at the contacts and broken, which obscured 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

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.

2.4 Faults

There are a number of faults near to the site, which typically bound the Henley Breccia. 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.

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 GHD (2020) GFR. 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, as 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 being 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).

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 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 of it underlying instability debris, and some underlying the loess. 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. Both buried topsoils however 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.

The buried topsoil typically comprises silt, with varying amounts of sand and clay, and is 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.

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.

3.2.5 Loess

Loess was 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 were observed at the site during investigations and mapping.

3.2.6 Taratu Formation

Taratu Formation was encountered in BH09/BH209, BH10, BH203, TP10-12, and in an outcrop as described in Section 2.3. Core recovery in the Taratu Formation in BH09 was poor (<40%) in the upper 10 m, so a second borehole (BH209) was drilled nearby. Taratu Formation core recovery in BH209 was consistently above 90%.

With the exception of BH203 (as discussed earlier), the Taratu Formation was encountered below the loess at depths ranging from 1.5 m to 3.15 m bgl. The Taratu Formation overlies the Henley Breccia and the base of the formation was encountered at 129.52 m RL at BH10 in the east of the site, at 123.7 m RL, at BH09 in the northwest of the site, and at 177.2 m RL at BH203 in the southwest of the site. BH209 terminated in Taratu Formation at 122.0 m RL (10.0 m bgl). The maximum thickness of Taratu Formation proven on site was 7.15 m in BH10, BH209 was terminated in Taratu Formation deposits at 10 m bgl. The elevation difference of the basal contact between these investigation locations suggests that the basal contact of the Taratu Formation is dipping to the west or northwest.

The Taratu Formation typically comprised sandstone, siltstone and conglomerate, and was predominantly highly to moderately weathered, with some slightly weathered material in BH209. The conglomerate layers have weathered to gravelly silt or gravelly sand. The conglomerate was typically matrix supported, and the matrix typically comprised sand or sandy silt. The clasts were rounded to sub-angular, fine to coarse gravel-sized quartz and schist.

The Taratu Formation is typically weakly cemented. The conglomerate layers were typically extremely weak to very weak, while the sandstone and siltstone layers were typically very weak to weak. The colour ranged from cream/white, through brown to grey for the conglomerate. The siltstone and sandstone layers were typically grey or brown. No defects were observed in the Taratu Formation.

The conglomerate layers were typically massive, with moderately thick bedding noted in BH203 and in the outcrop. The sandstone and siltstone layers were typically laminated to moderately thickly bedded. The interbedded layers do not appear to be contiguous across the site.

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 was typically underlying 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 was typically 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.

Sandstone (Moderately Weathered to Unweathered)

Sandstone is a common rock type within the Henley Breccia and moderately to unweathered (MW-UW) sandstone was encountered in TP09 and 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 depths. The sandstone was typically interbedded with siltstone, conglomerate and breccia.

The sandstone was typically fine to medium grained, but occasionally coarse grained.

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 all test pits except TP08 and TP09, and all boreholes except BH09, BH201, and BH209. 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.

Conglomerate (Moderately Weathered to Unweathered)

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 that the thicker layer tended to have poor recovery during drilling.

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 underlying sandstone and/or siltstone, 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 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 GHD (2020) Hydrogeological Assessment report.

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 have restricted the placement of investigation points for this concept stage site investigation. These are:

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

- The south-eastern portion of the site is still currently 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 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 areas on **Error! Reference source not found.**.

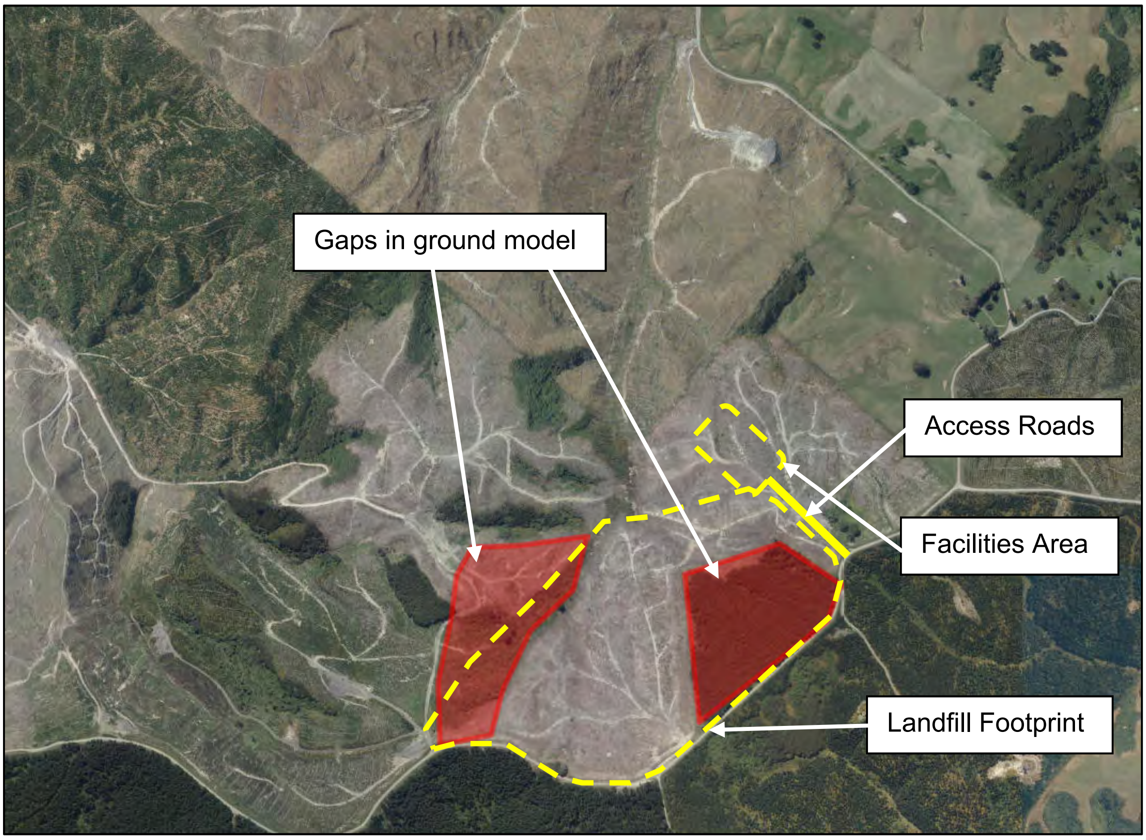


Figure 6 - Identified gaps in the ground model (image sourced from DCCWebmap)

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 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 construction activities. Where they occur within the landfill footprint they will be 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 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. Groundwater is described in more detail in the Hydrogeology Report (GHDb).

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 GHD (2019a) GFR, 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 these 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), the landfill has been assumed to have an Importance Level of 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, IL2 structures are required to be designed to resist earthquake loadings with return periods of 500 and 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 2.2). This table returns a design return period of 1/500 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 0.24 g for damage control limit state (DCLS) (equivalent to ultimate limit state (ULS)) and 0.06 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 requires further assessment.

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 five geotechnical units.

- Unsuitable material – comprising topsoil, fill, alluvium, and unstable slope debris
- Loess
- Completely Weathered to Highly Weathered Henley Breccia Formation – comprising sandstone, siltstone, breccia, conglomerate
- Moderately Weathered to -Unweathered Henley Breccia Formation – comprising 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 2 below.

5.2 Preliminary Geotechnical Design Parameters

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

Geotechnical Unit	Bulk Unit Weight, γ ($kN.m^{-3}$)	Internal Friction Angle, Φ' (°)	Cohesion, c' (kPa)	Uniaxial Compressive Strength (MPa)	Undrained Shear Strength, S_u (kPa)
Re-compacted Loess	16-18	25	0	-	130
CW-HW Rock (including residual soil)	18-20	30-35	0-30	<1	-
MW-UW Rock	21-23	30-40	60-100	5 – 30	-

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 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. Alluvials 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

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. Slope stability was analysed on critical cross sections using Slope/W limit equilibrium software. Slope stability assessment results are presented in Appendix C.

7.2 Landfill Slopes

The construction of the landfill will involve modification of the existing valley landform. Based on the preliminary earthworks plan, a number of cross sections have been generated to identify and analyse the following key features:

- Engineered slopes comprising engineered fill from site won material.
- Cut slopes in the existing topography.
- Toe bund stability and perimeter embankment stability.

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 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 the following geotechnical properties:

Table 3 Base Liner Parameters

Material	Bulk Unit Weight, γ (kN.m^{-3})	Internal Friction Angle, Φ' (°)	Cohesion, c' (kPa)
Re-compacted Loess	18	*25*	0

Assessment of the landfill geomembrane liner stability is not considered within this analysis and is discussed in the “Waste Futures Phase 2 - Workstream 3 Smooth Hill Landfill - Landfill Concept Design Report”, GHD (2020).

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 4 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

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.

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 Hydrogeology Report (GHD, 2019b).

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 the landfill has been assumed to have an importance level of 2 (IL2) (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 respectively.

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 (0.25 for SLS) for Importance Level 2 and return Period 1/500 (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 0.24g for ULS and 0.06 for SLS.

7.5 Stability Modelling Cases

7.5.1 Concept Level Analysis

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.

7.5.2 Critical Cross Sections and Structures

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.

The structures modelled for stability are outlined in the concept sketch below:

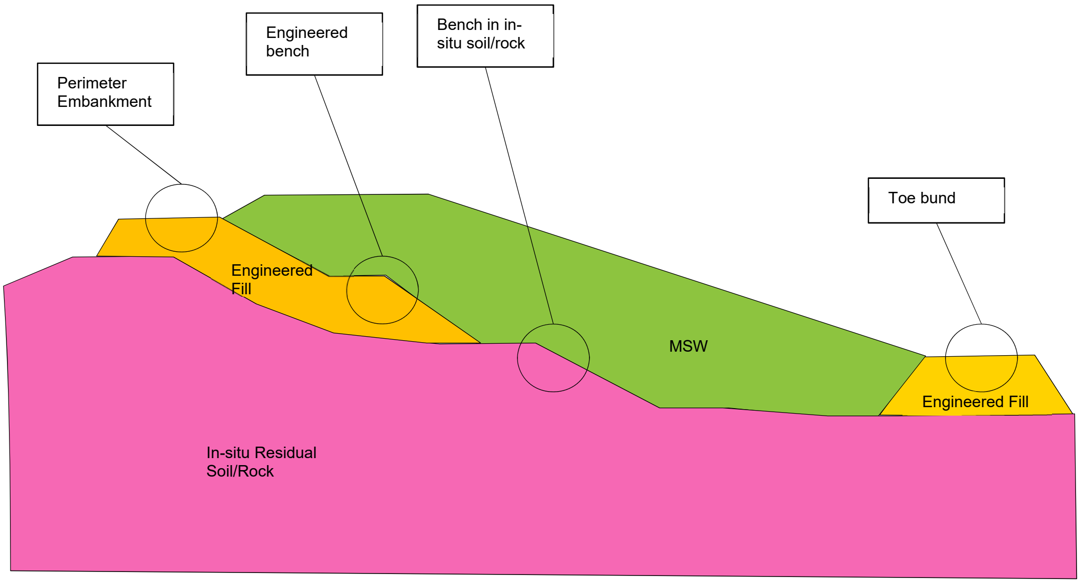


Figure 7 - Indicative Landfill Sketch

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

7.5.3 Target Factor of Safety

It is generally accepted good engineering practice that the required factors of safety (FOS) 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 FOS required for seismic slope stability is FOS >1.0.

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

Table 6 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

7.6.1 Section A Western Slope

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:

Table 7 01 Section A Western Slope

Analysis Case	Slope	Target FoS	Calculated FoS of Critical Slip	Figure in Appendix
Static	Engineered bench	≥1.5	2.7	C.2
SLS Seismic	Engineered bench	>1.0	2.1	C.2
ULS Seismic	Engineered bench	>1.0	1.2	C.2

7.6.2 Section B South Western Slope

Section B comprises an engineered perimeter embankment along the southern boundary of the landfill. The embankment is approx. 17 m in height. The stability of the perimeter embankment has been analysed with the full waste placement.

Table 8 02 South Western Slope

Analysis Case	Slope	Target FoS	Calculated FoS of Critical Slip	Figure in Appendix
Static	Perimeter Embankment with waste surcharge	≥1.5	2.1	C.3
Short term bench construction	Perimeter Embankment without waste surcharge	≥1.3	1.7	C.3
SLS Seismic	Perimeter Embankment with waste surcharge	>1.0	1.7	C.3
ULS Seismic	Perimeter Embankment with waste surcharge	>1.0	1.1	C.3

7.6.3 Section C Eastern Slope

On the eastern slope the base of the proposed landfill will be excavated approximately 10 m into the in-situ soils. As such, the proposed benches are assumed to be formed in very weak to weak, moderately weathered to unweathered rock.

Table 9 03 Section C Eastern Slope

Analysis Case	Slope	Target FoS	Calculated FoS of Critical Slip	Figure in Appendix
Static	Rock Bench	≥1.5	5.2	C.4
SLS Seismic	Rock Bench	>1.0	4.0	C.4
ULS Seismic	Rock Bench	>1.0	2.3	C.4

7.6.4 Section D Toe Bund (Northern)

The toe bund in the bottom of the valley which forms the northern boundary of the landfill will comprise site won engineered fill. The approximate height of the bund is 15 m. Stability of the bund has been considered with the full waste placement of the completed landfill.

Table 10 04 Section D Toe Bund (Northern)

Analysis Case	Slope	Target FoS	Calculated FoS of Critical Slip	Figure in Appendix
Static	Bund with waste surcharge	≥1.5	2.0	C.5
Internal Bund Stability	Bund with waste surcharge	≥1.5	2.0	C.5
Short Term Elevated Leachate	Bund with waste surcharge	≥1.3	2.4	C.5
SLS Seismic	Bund with waste surcharge	>1.0	1.6	C.5
ULS Seismic	Bund with waste surcharge	>1.0	1.0	C.5

7.7 Summary

The results indicate adequate slope stability for the proposed benches in engineered fill and into in-situ rock cuts. The results also indicate adequate stability for the landfill with full waste placement.

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.
- Areas of shallow instability have been identified across the site primarily associated with the loess and highly weathered Henley Breccia. These will either 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.
- 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.
- 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.

9. References

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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.

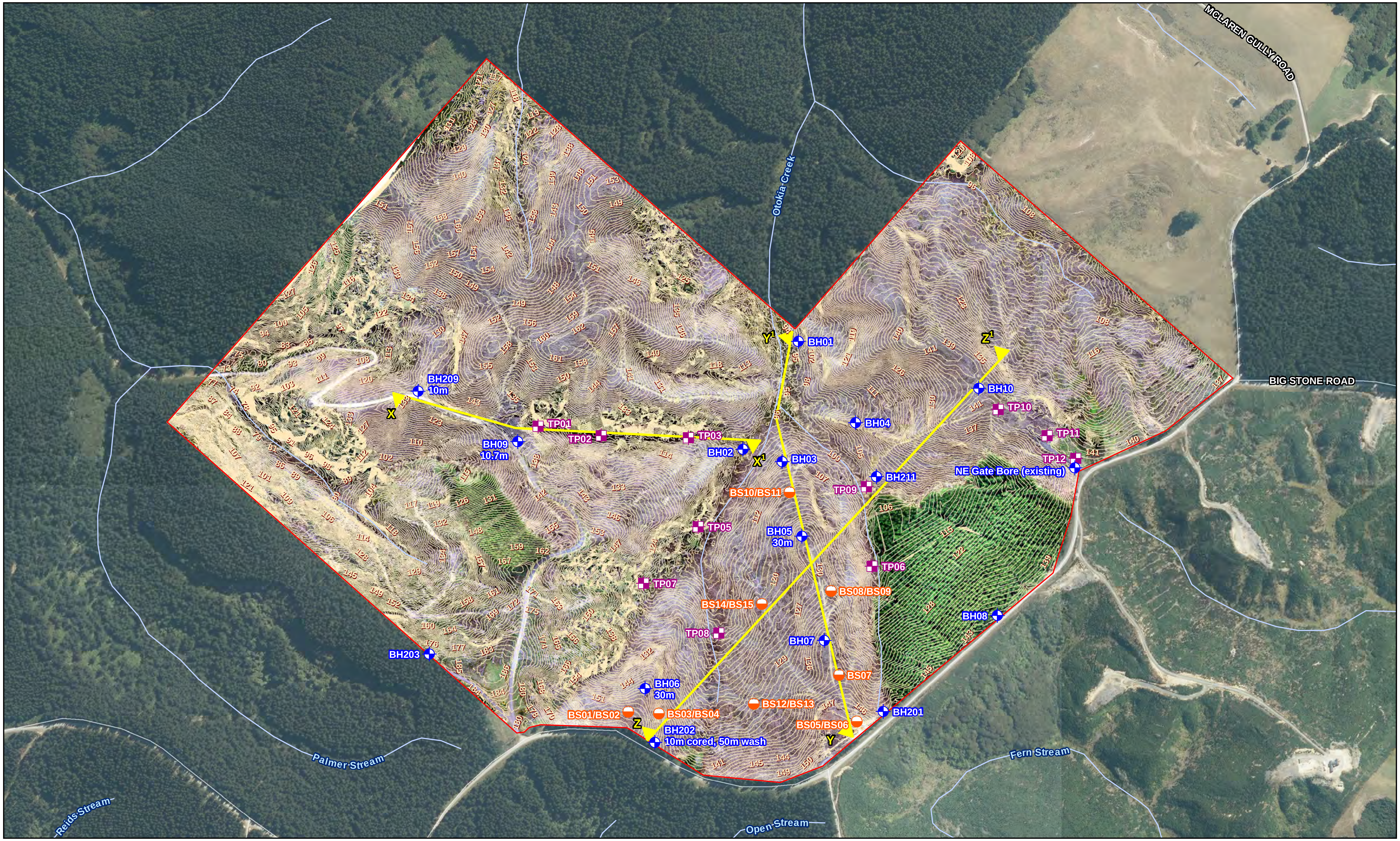
Site conditions (including the presence of hazardous substances and/or site contamination) may change after the date of this Report. GHD does not accept responsibility arising from, or in connection with, any change to the site conditions. GHD is also not responsible for updating this report if the site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

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LEGEND

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

Dunedin City Council
Smooth Hill Landfill
Geotechnical and Hydrogeological

Job Number	12506381
Revision	A
Date	17 Aug 2020

Investigation Location Plan

Figure A1

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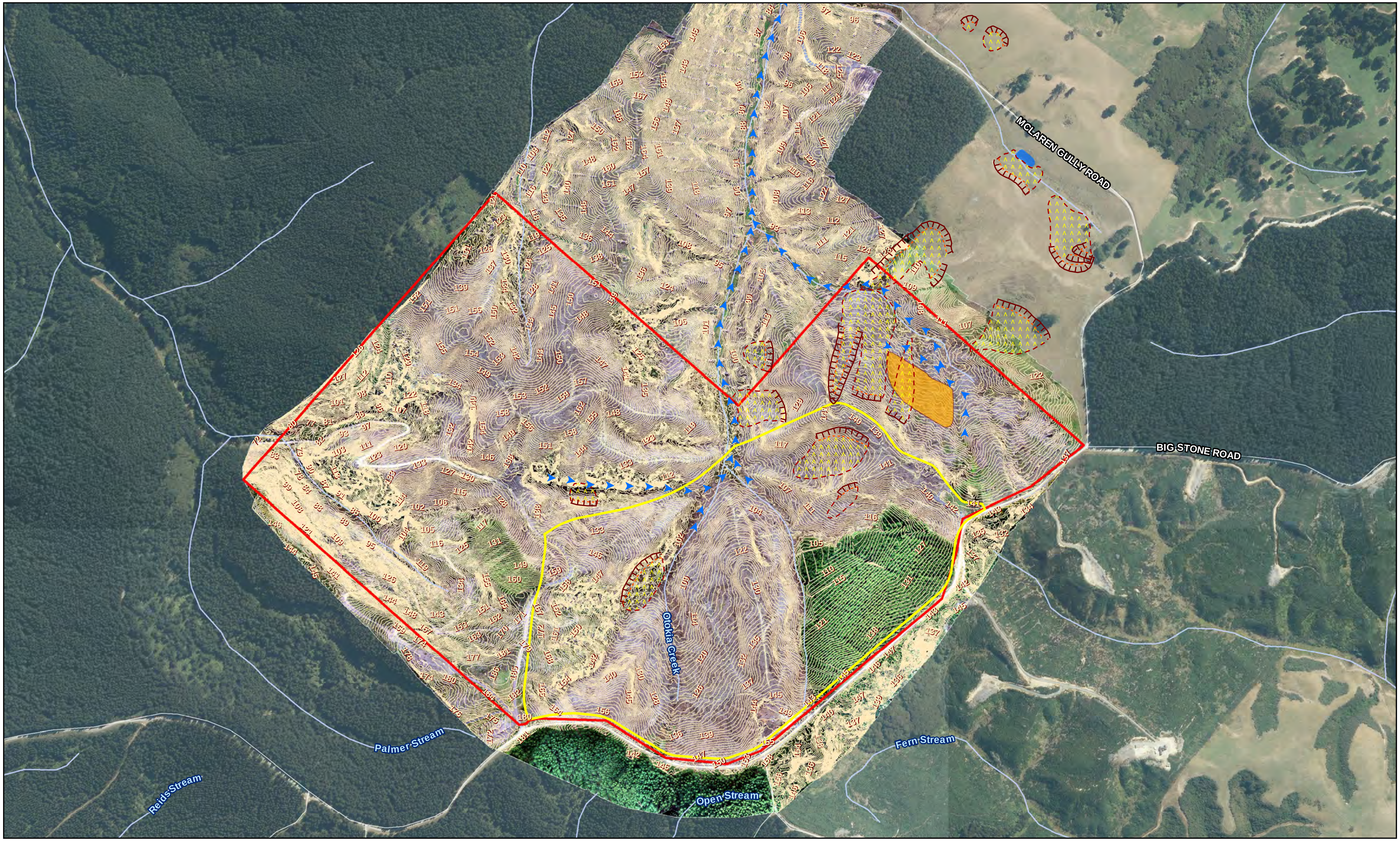
Data source: Aerial imagery - Stantec, LINZ & esri (); General topo - LINZ 2019; Contours - Stantec Created by:jrprice



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)

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Geotechnical and Hydrogeological

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Revision	A
Date	17 Aug 2020

Instability Plan

Figure A2

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

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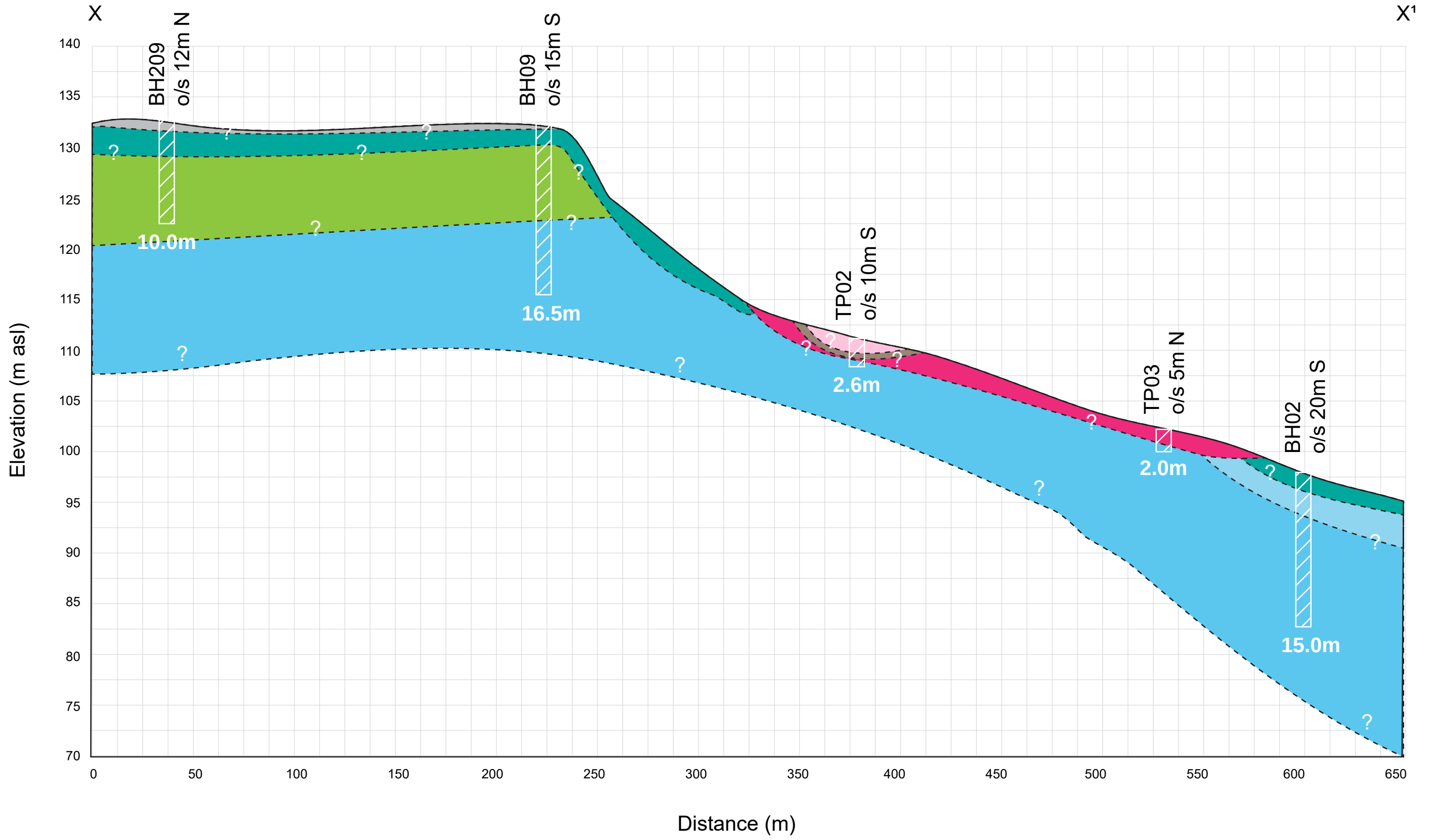
Data source: Aerial imagery - Stantec, LINZ & esri (); General topo - LINZ 2019; Contours - Stantec Created by:jrprice

Appendix B – Ground Model Sketches

Section X-X’

Section Y-Y’

Section Z-Z’



Note: Refer to Figure 1 for section alignment

DRAWING NOT TO SCALE

LEGEND

- Fill
- Slip debris
- Buried topsoil
- Alluvium
- Loess
- Taratu Formation
- CW-HW Henley Breccia Formation
- MW-UW Henley Breccia Formation

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Date | 24 Jan 2020

Cross Section X-X¹
Looking North

Figure 3

N:\NZ\Christchurch\Projects\51\12506381\GIS\Maps\Deliverables\Geo\12506381_Z003_Geo_CrossSectionX.ai

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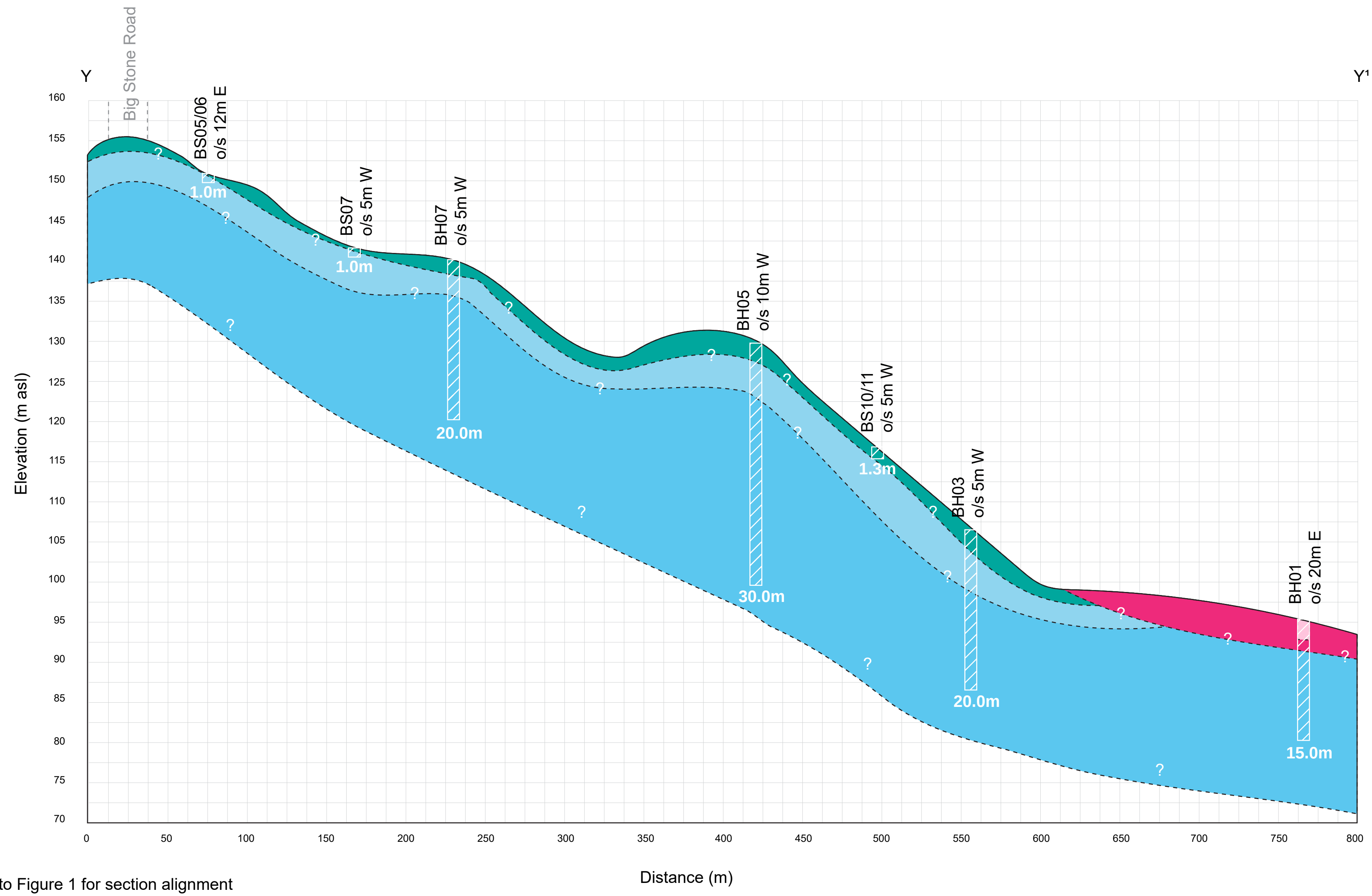
Data source: GHD. Created by:jrprice



Note: Refer to Figure 1 for section alignment

DRAWING NOT TO SCALE

LEGEND

- Fill
- Slip debris
- Buried topsoil
- Alluvium
- Loess
- Taratu Formation
- CW-HW Henley Breccia Formation
- MW-UW Henley Breccia Formation

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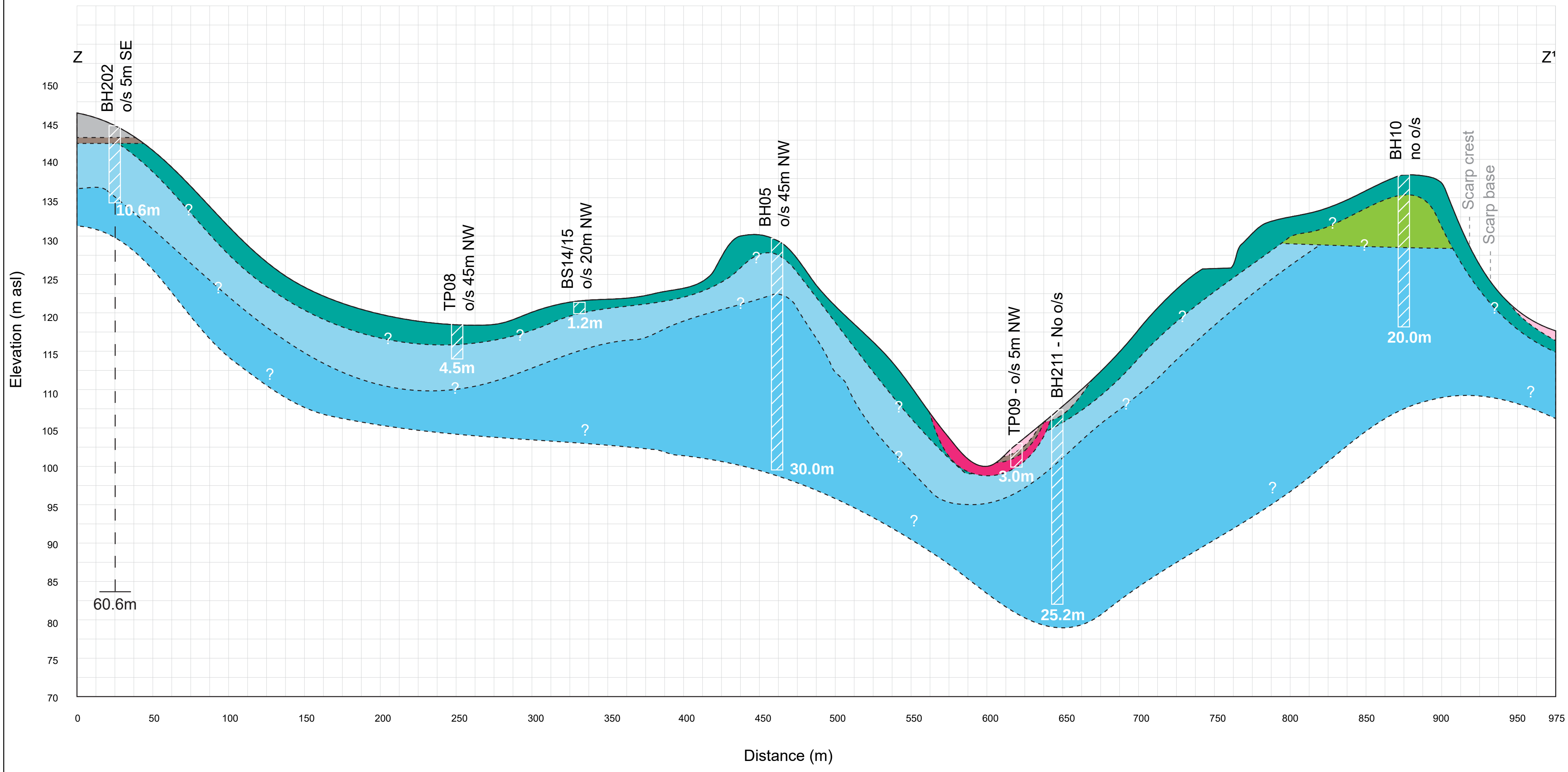
Cross Section Y-Y¹
Looking North-West

Figure 4

N:\NZ\Christchurch\Projects\51\12506381\GIS\Maps\Deliverables\Geo\12506381_Z004_Geo_CrossSectionY.ai

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: GHD. Created by:jrprice



Note: Refer to Figure 1 for section alignment

DRAWING NOT TO SCALE

LEGEND

- Fill
- Slip debris
- Buried topsoil
- Alluvium
- Loess
- Taratu Formation
- CW-HW Henley Breccia Formation
- MW-UW Henley Breccia Formation

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Cross Section Z-Z¹
Looking North-West

Figure 5

N:\NZ\Christchurch\Projects\51\12506381\GIS\Maps\Deliverables\Geo\12506381_Z005_Geo_CrossSectionZ.ai

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Appendix C – Slope Stability Modelling Results

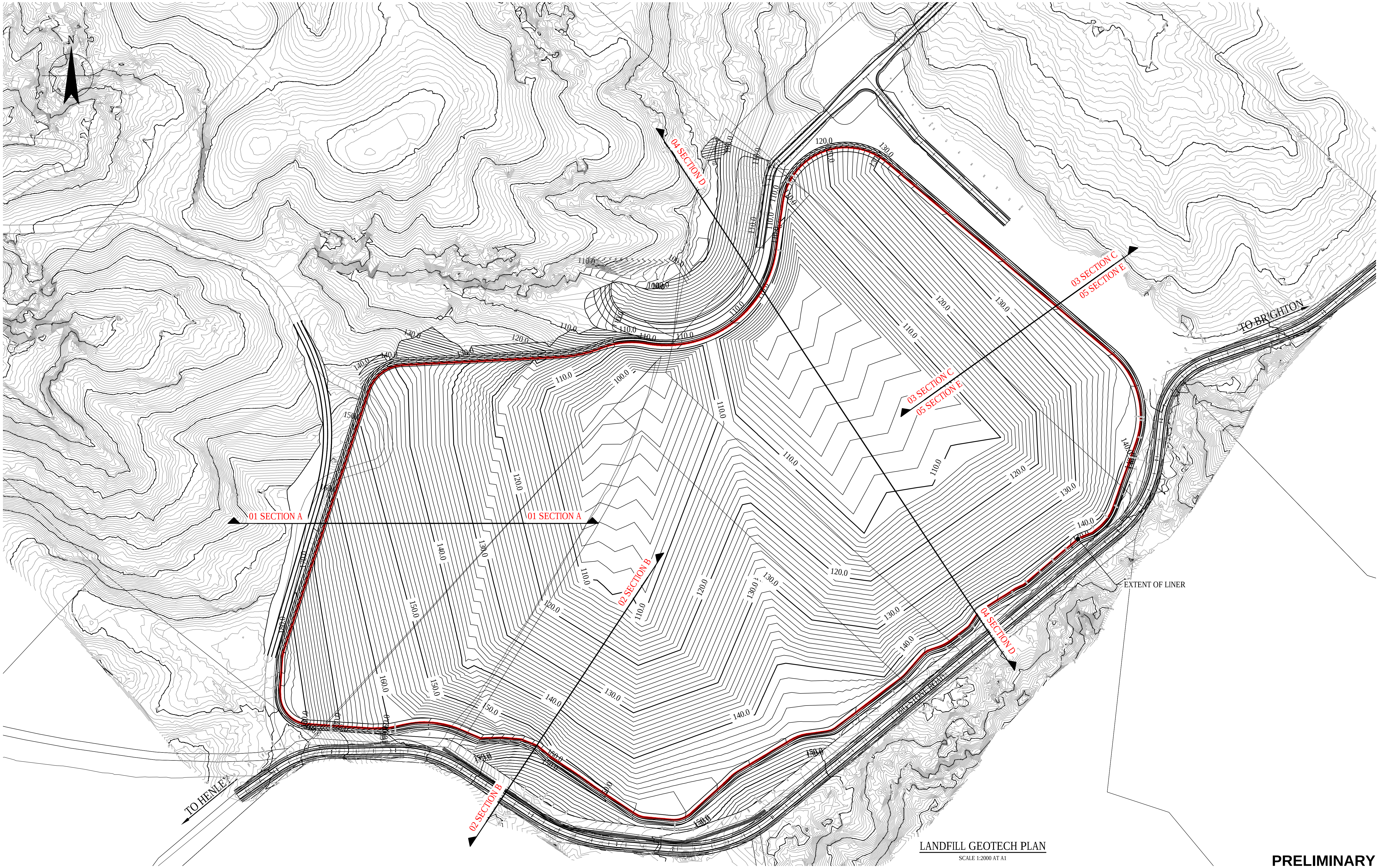
Appendix C.1 - Cross Section Plan

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)



LANDFILL GEOTECH PLAN

SCALE 1:2000 AT A1

PRELIMINARY

No	Revision	Drawn	Job Manager	Project Director	Date
A	PRELIMINARY				

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

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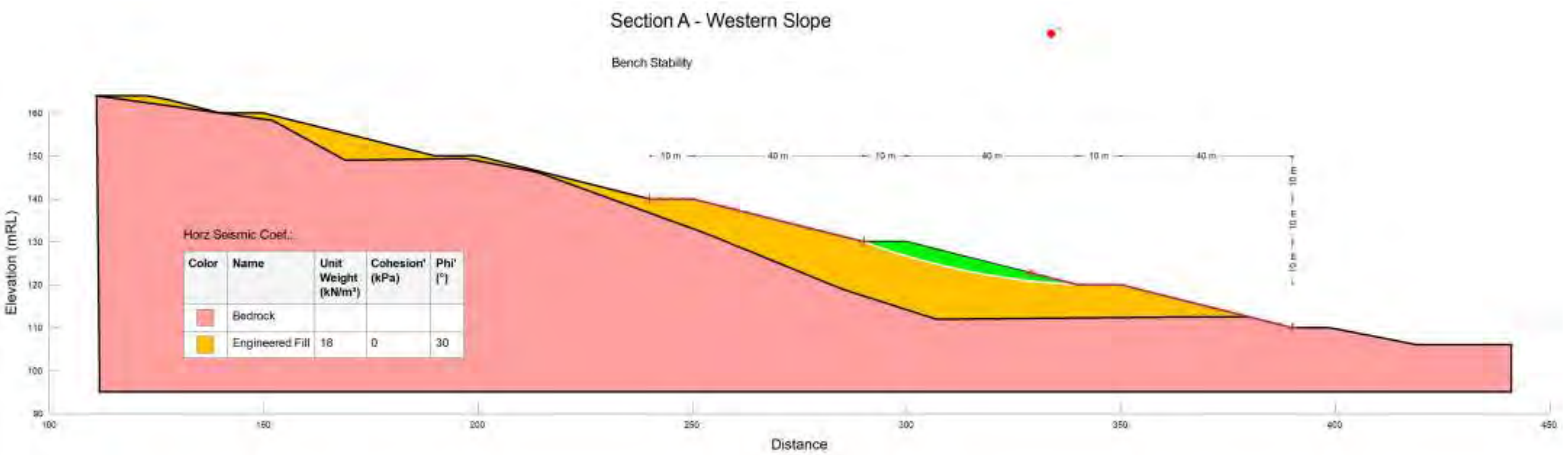
Drawn		Designer	
Drafting Check		Design Check	
Approved (Project Director)			
Date			
Scale	1:2000 AT A1	This Drawing must not be used for Construction unless signed as Approved	

Client **DUNEDIN CITY COUNCIL**
Project **SMOOTH HILL LANDFILL**
Title **LANDFILL GEOTECH PLAN**

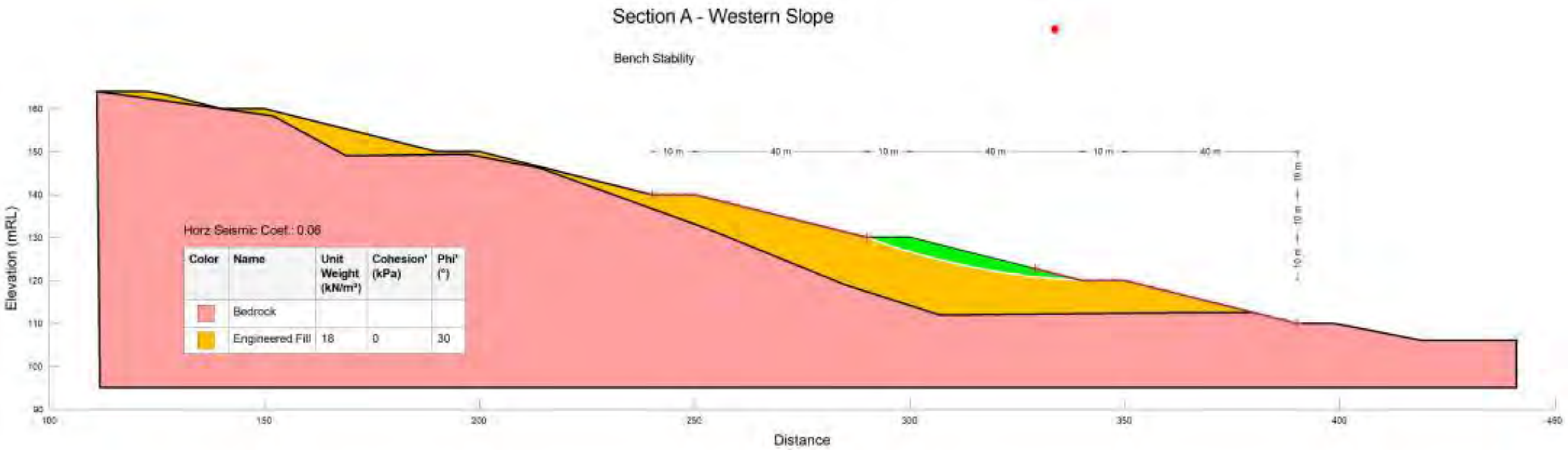
Original Size **A1** **Drawing No: 51-12506381-01-Q101** **Rev: A**

Plot Date: 4 December 2019 - 1:54 PM Plotted by: Kelly Millar Cad File No: N:\NZ\Christchurch\Projects\51\12506381\CADD\Drawings\51-12506381-01-Q101_GEOTECH.dwg



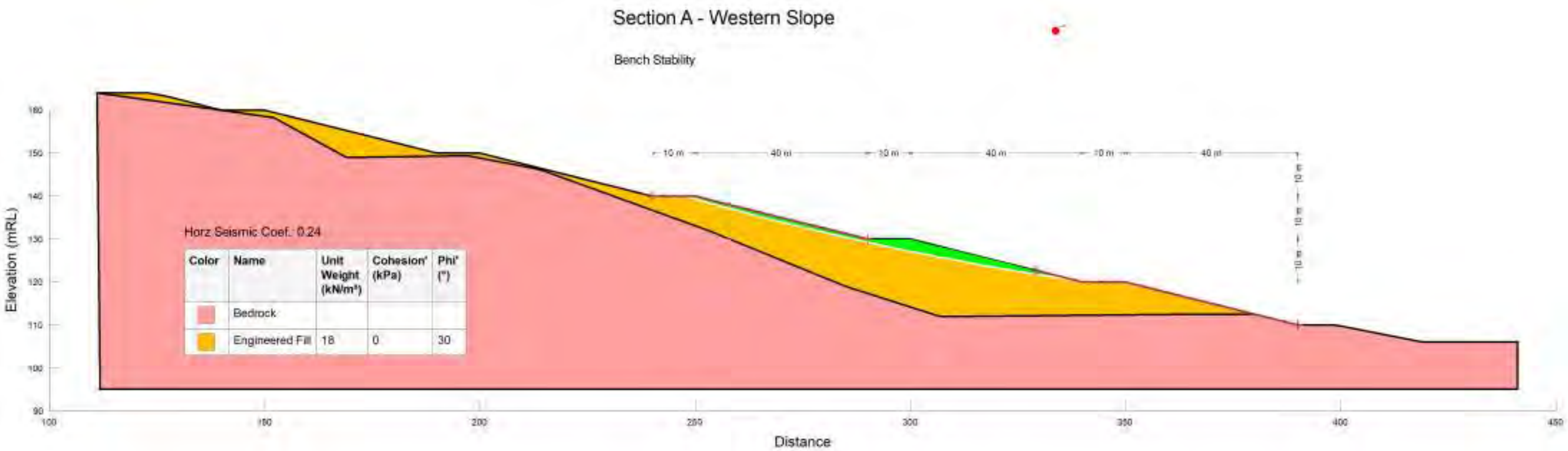
Static Analysis

Critical factor of Safety = 2.7



Seismic SLS Analysis

Critical factor of Safety = 2.1



Seismic ULS Analysis

Critical factor of Safety = 1.2

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				Drawn		
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				Approved		
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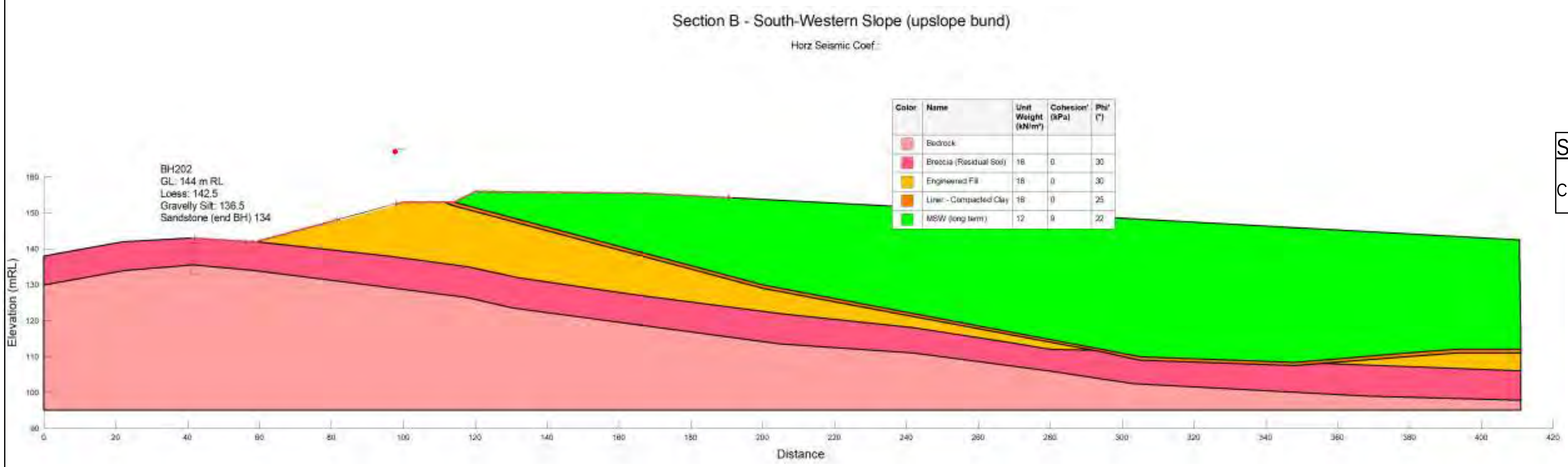
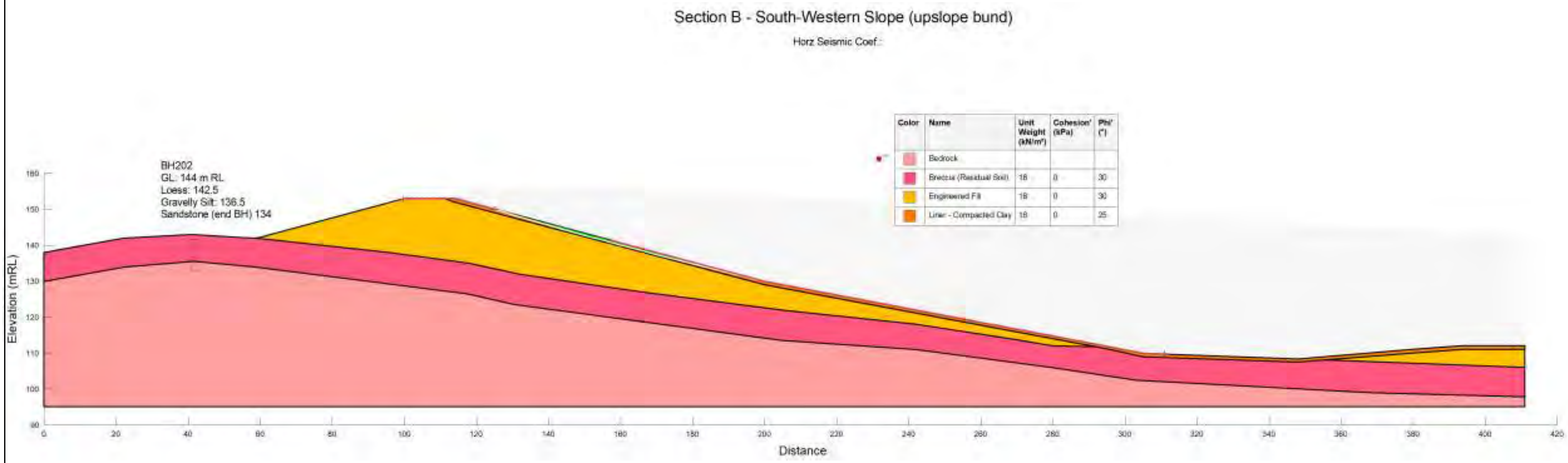


Static Analysis

Critical factor of Safety = 2.1



Short Term Bench Construction Analysis

Critical factor of Safety = 1.7

Issue	Description	Date	Approved		Name	Date
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				Approved		
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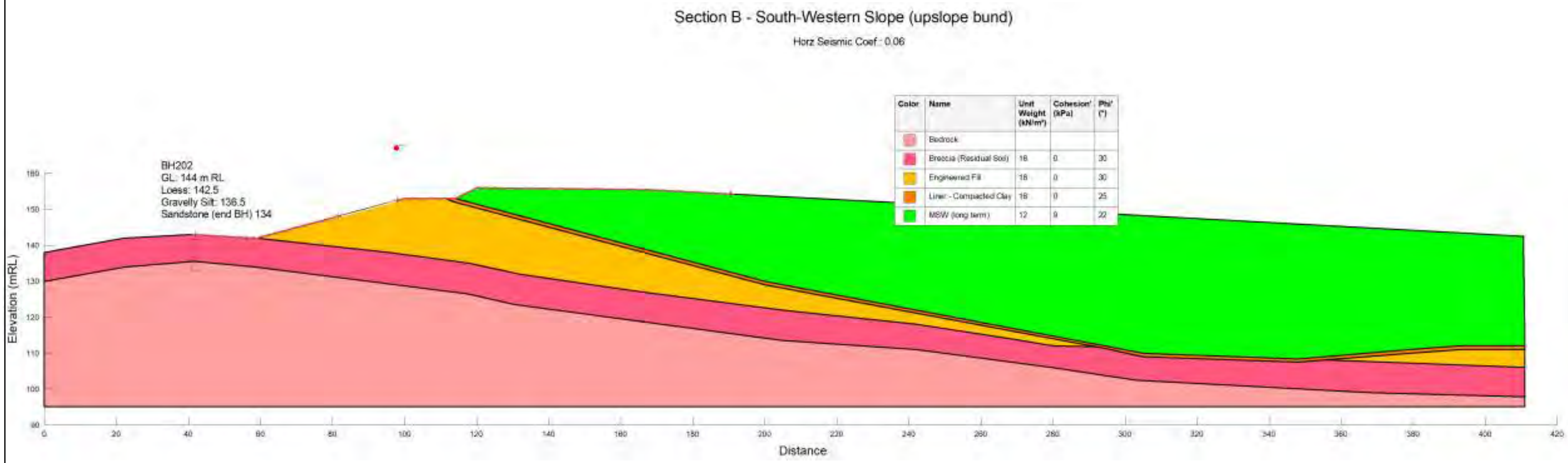
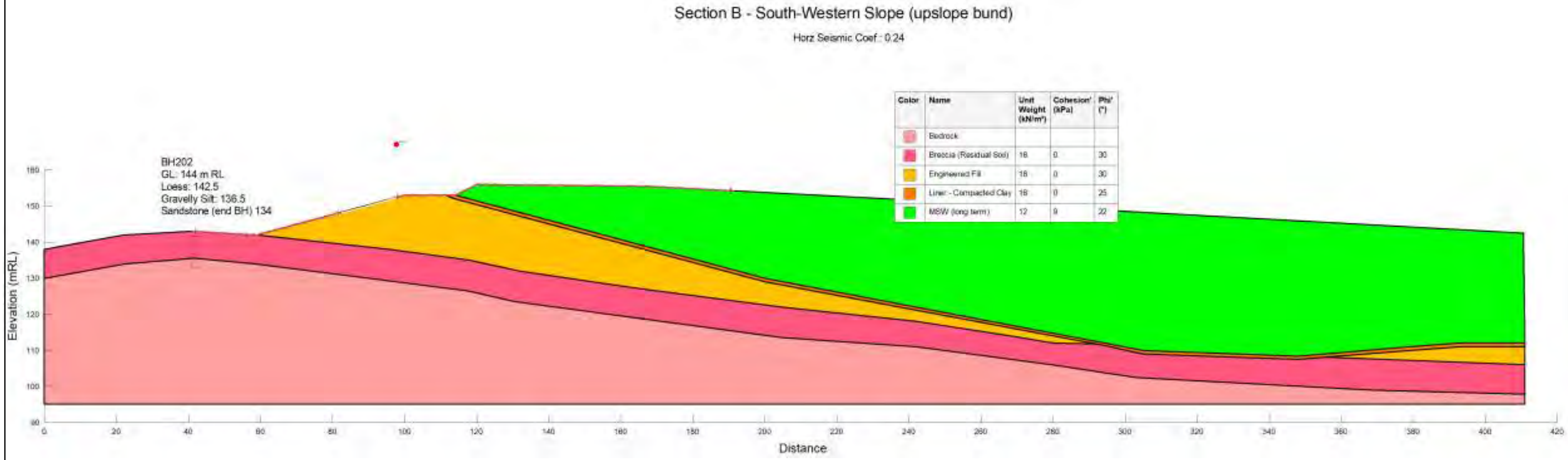


Seismic SLS Analysis

Critical factor of Safety = 1.7



Seismic ULS Analysis

Critical factor of Safety = 1.1

Issue	Description	Date	Approved		Name	Date
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				Drawn		
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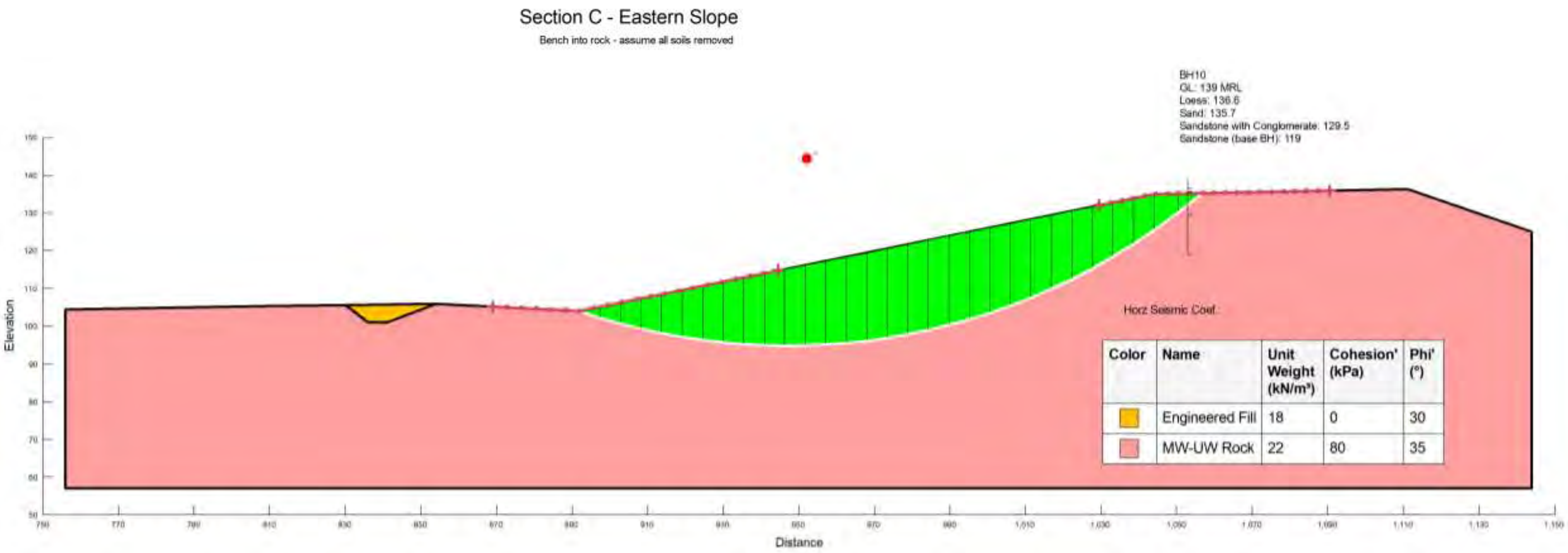
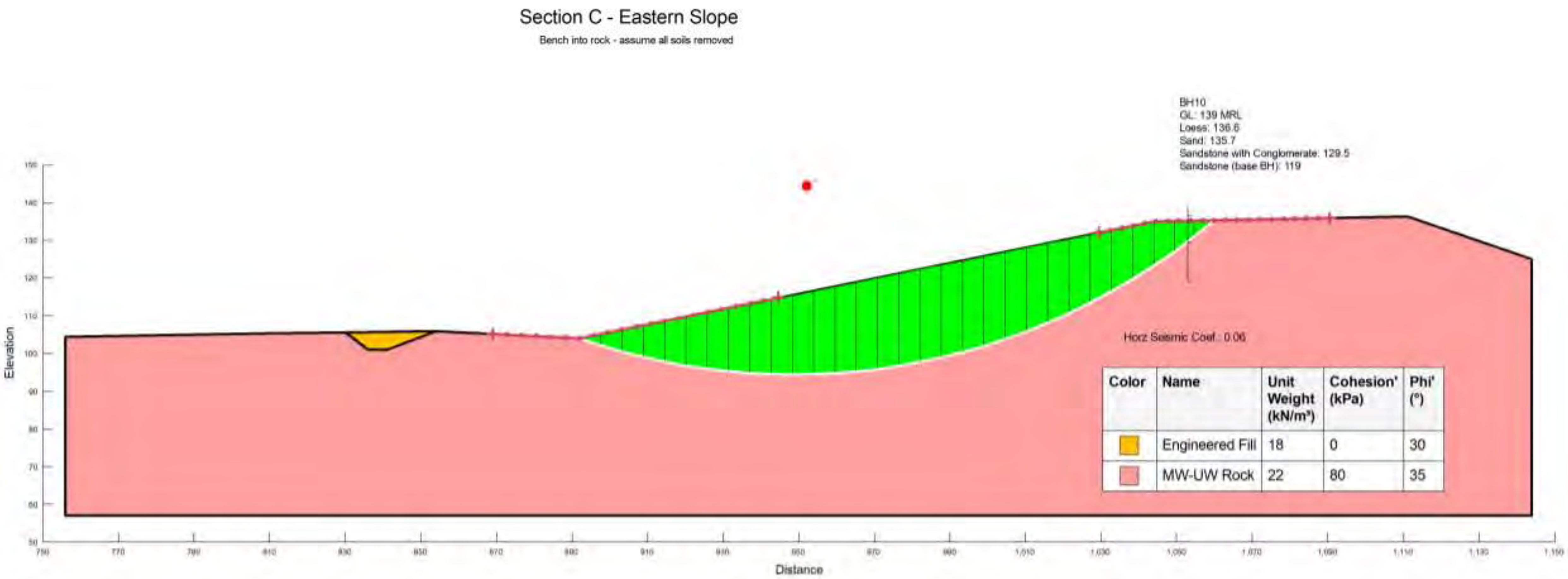


Static Analysis

Critical factor of Safety = 5.2



Seismic SLS Analysis

Critical factor of Safety = 4.0

Issue	Description	Date	Approved		Name	Date
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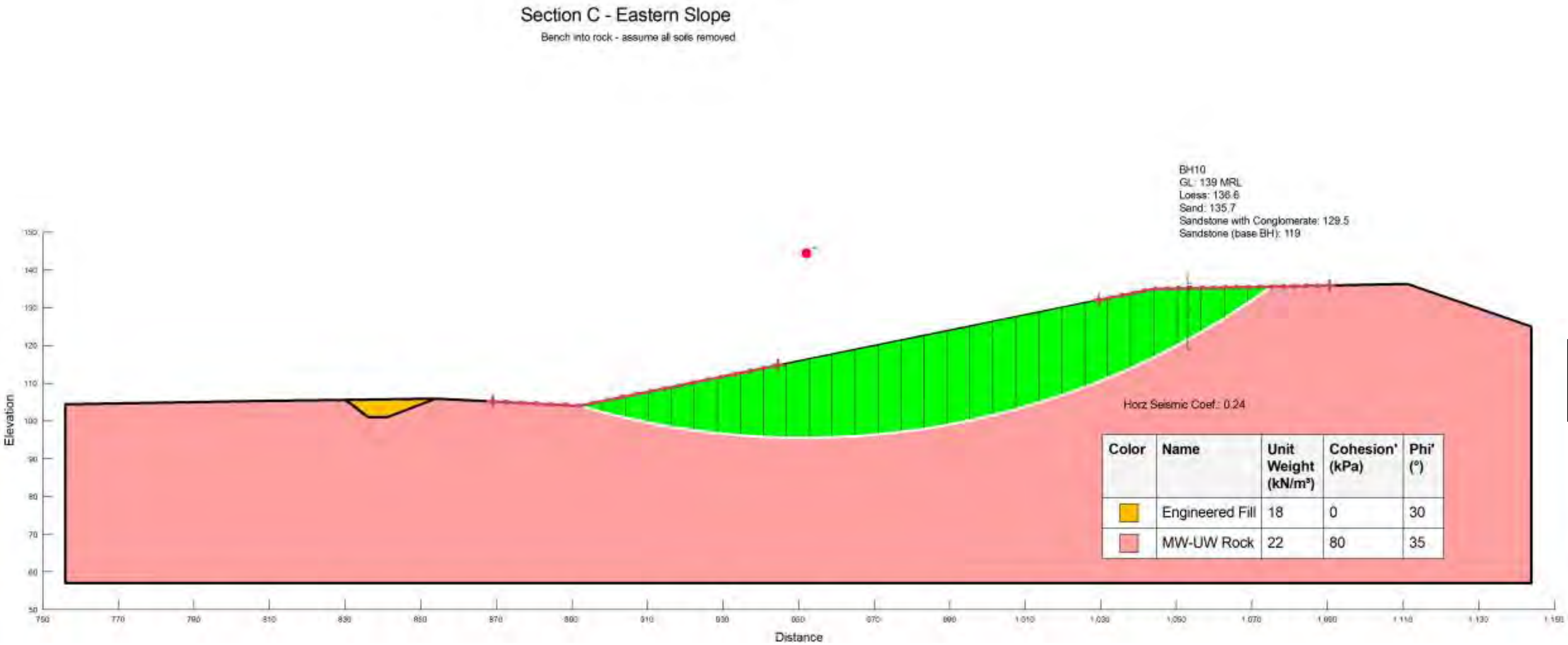

Seismic ULS Analysis

Critical factor of Safety = 2.3

Issue	Description	Date	Approved		Name	Date
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				Approved		
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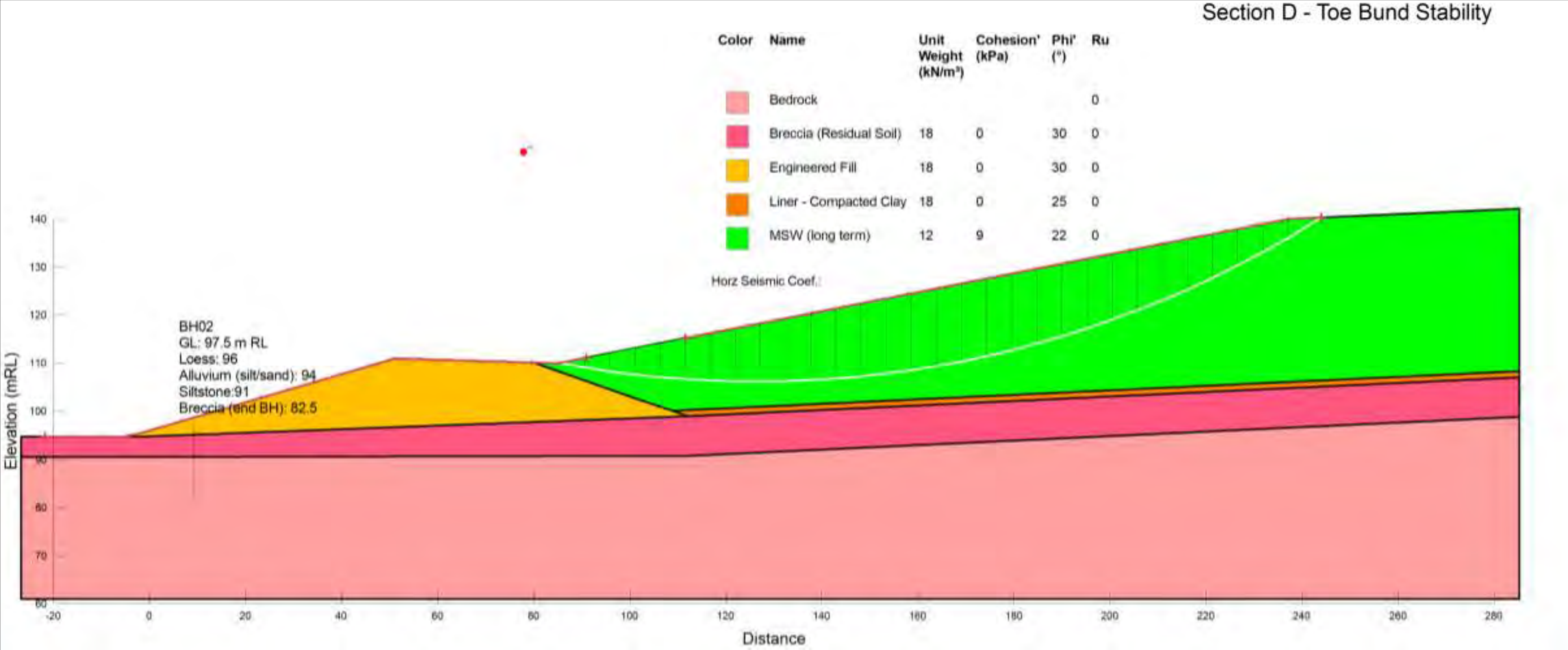

Static Analysis

Critical factor of Safety = 2.5

Issue	Description	Date	Approved		Name	Date
				Designed		
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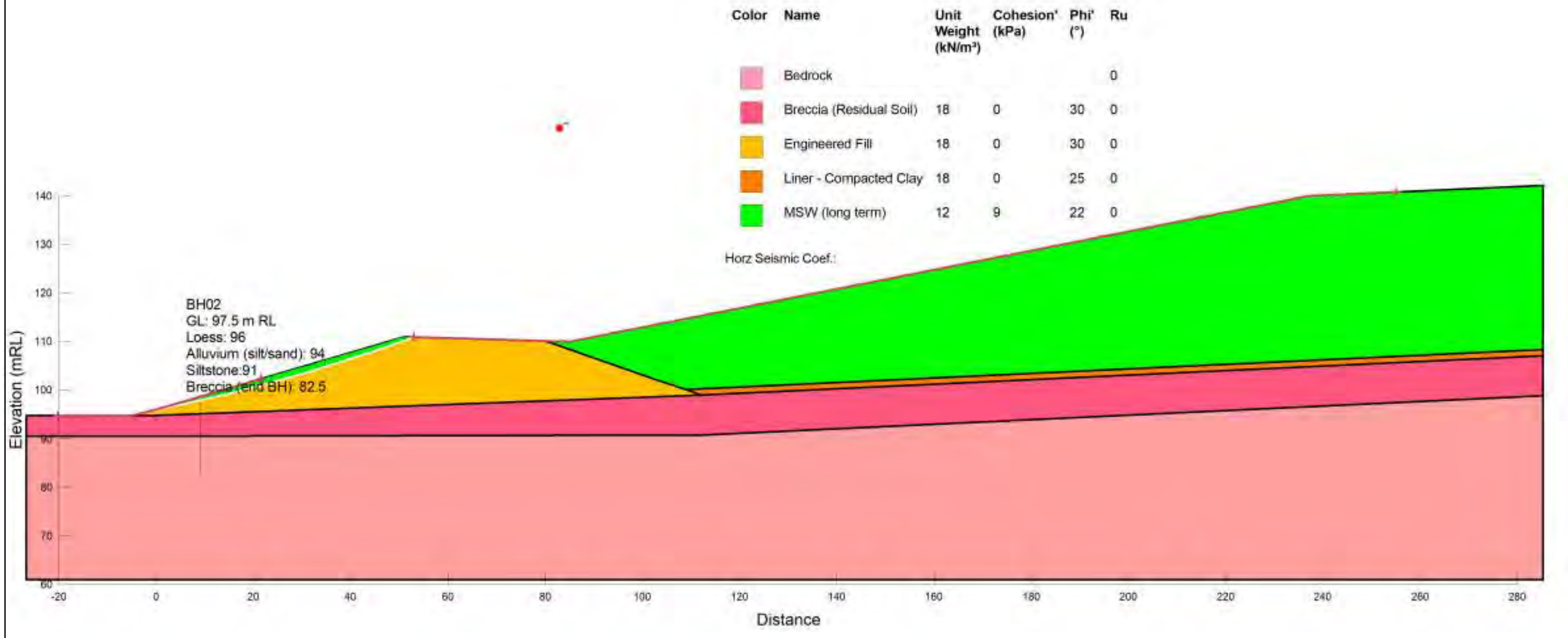
Section D - Toe Bund Stability



Internal Bund Stability Analysis

Critical factor of Safety = 2.0

Issue	Description	Date	Approved		Name	Date
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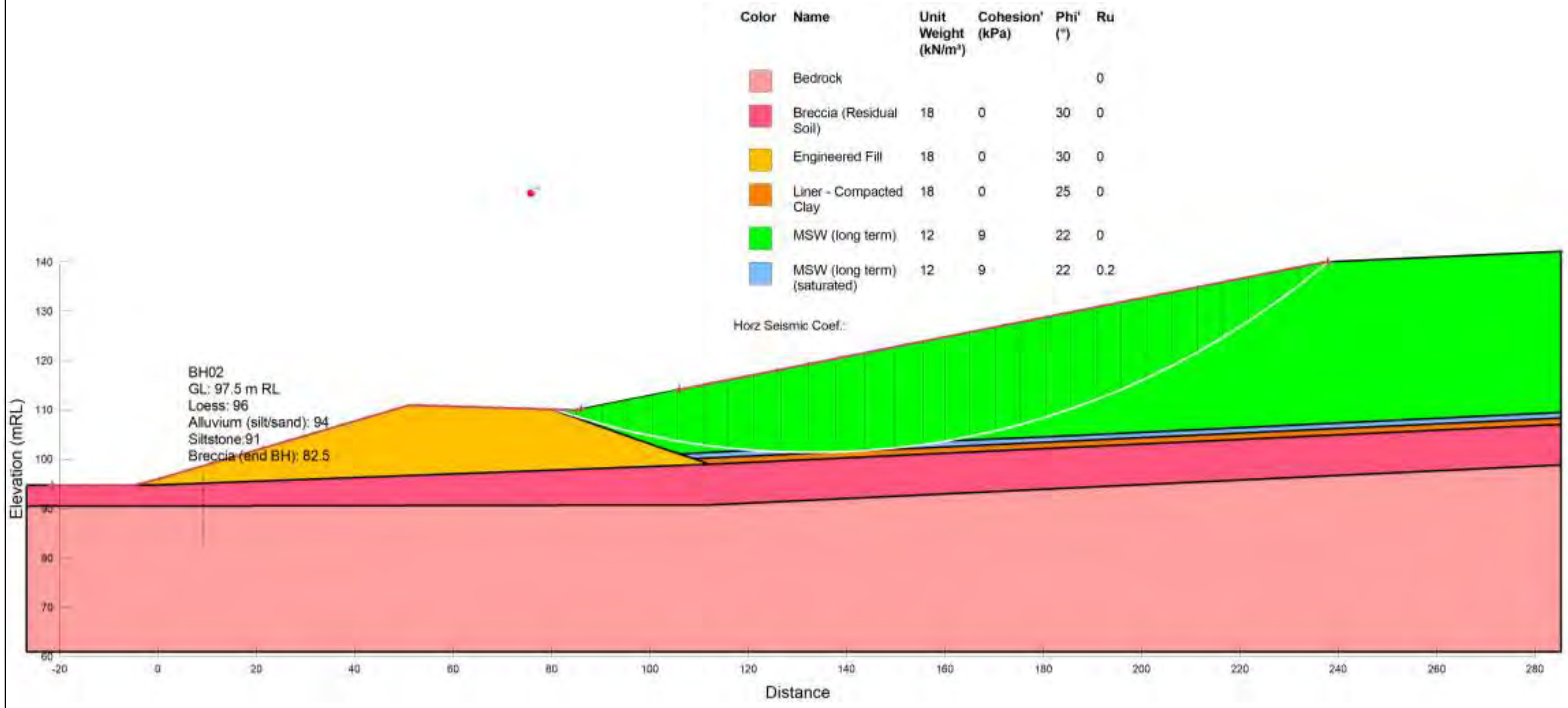

Short Term Elevated Leachate Analysis

Critical factor of Safety = 2.4

Issue	Description	Date	Approved		Name	Date
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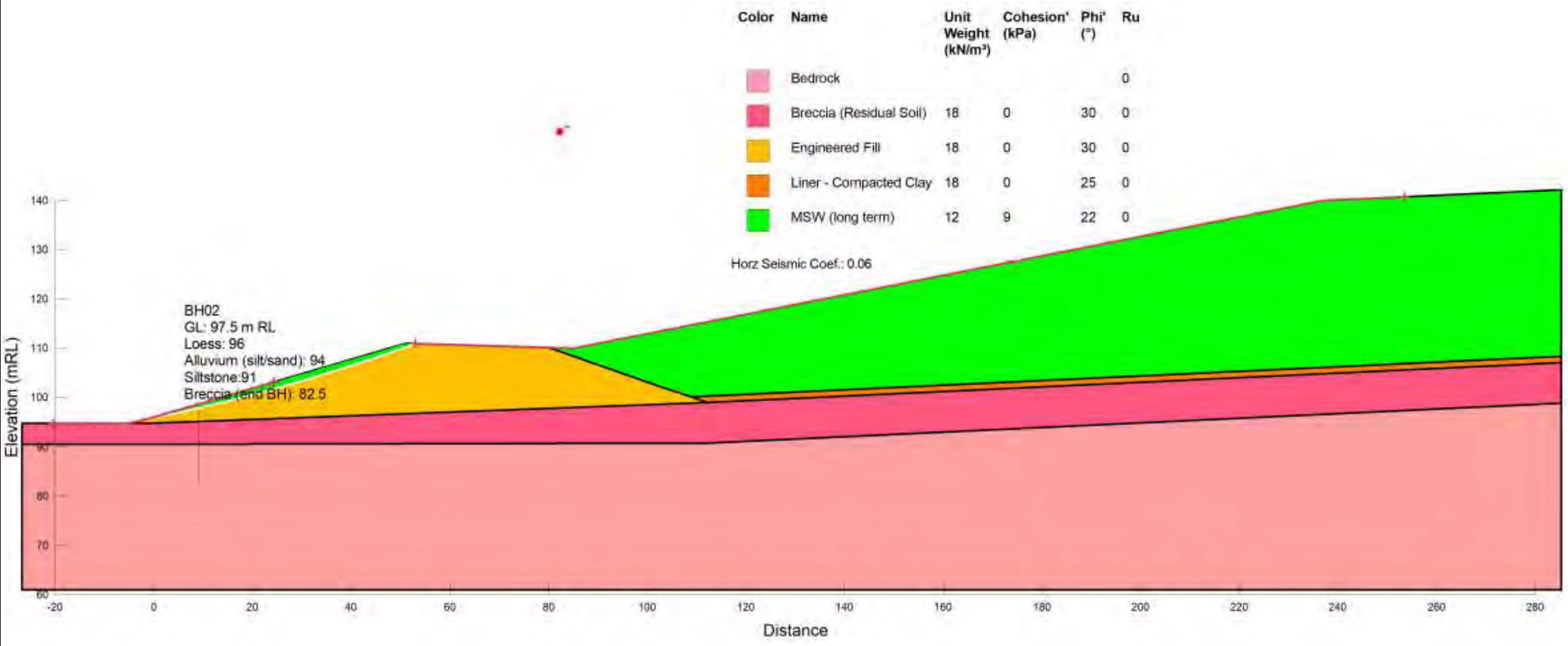
Section D - Toe Bund Stability



Seismic SLS Analysis

Critical factor of Safety = 1.6

Issue	Description	Date	Approved		Name	Date
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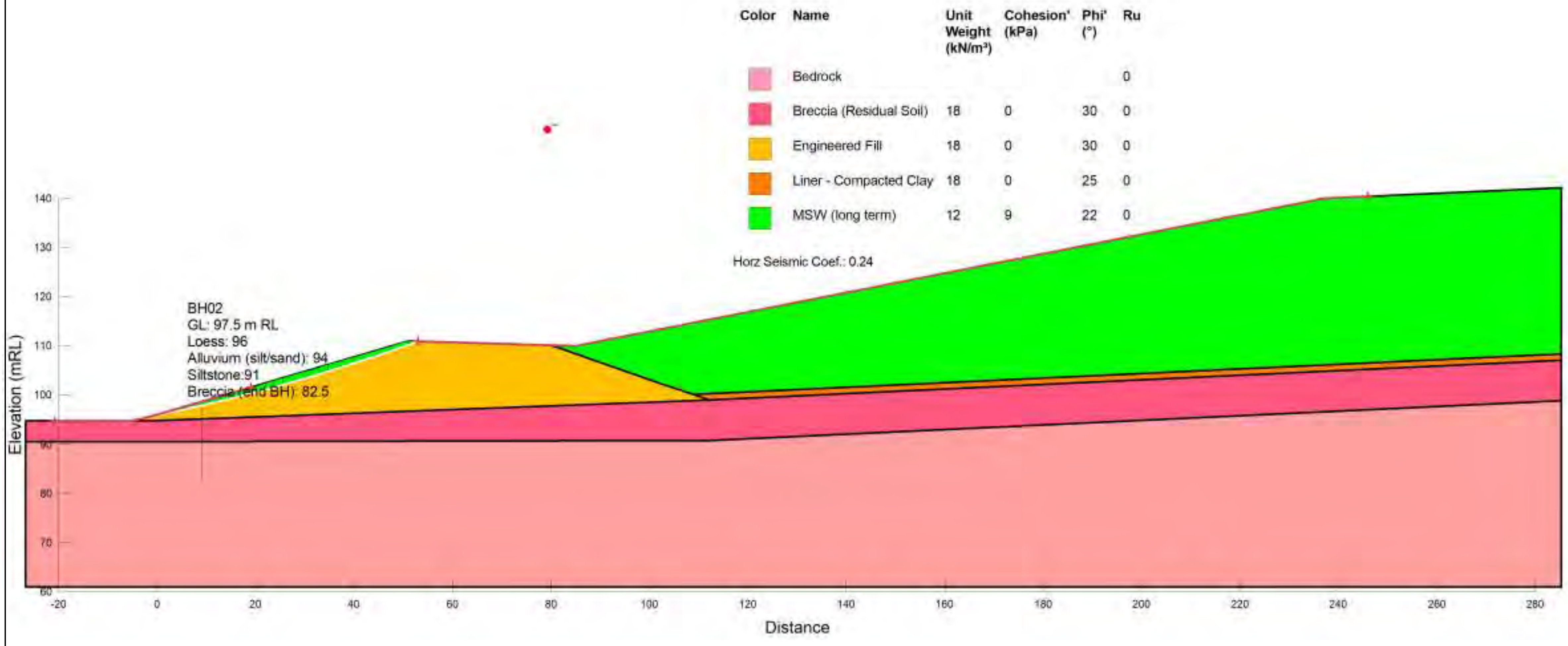

Seismic ULS Analysis

Critical factor of Safety = 1.0

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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.

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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

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