REPORT

Tonkin+Taylor

For a comprehensive understanding of this report, please also refer to the relevant s92 responses

Auckland Regional Landfill

Geotechnical Interpretive Report

Prepared for
Waste Management NZ Ltd

Prepared by
Tonkin & Taylor Ltd

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Table of contents

1 Introduction

Tonkin & Taylor Ltd (T+T) has been engaged by Waste Management NZ Ltd (WMNZ) to undertake geotechnical investigations for a proposed Auckland Regional Landfill project located in the Wayby Valley area approximately 13 km north west of Warkworth. T+T has completed a geotechnical site investigation for the project, with the results presented in the Geotechnical Factual Report (GFR).

This Geotechnical Interpretive Report (GIR) describes the site geological and groundwater conditions and associated geotechnical model. This report is intended to support feasibility level design of the proposed landfill and access and application for resource consent. This report also identifies potential geotechnical risks to the project and recommends further investigation and/or analysis to avoid, remedy or mitigate these risks.

1.1 Scope of work

The scope of the geotechnical assessment was to:

- Review the factual information gathered during the geotechnical investigations and develop a site geological model;
- Assess site slope stability including feasibility stability review of the existing slopes and design slopes;
- Assess other potential geotechnical hazards. A site specific seismic hazard assessment is being prepared under separate cover;
- Assess geotechnical design input parameters;
- Assess the suitability of the site rock and soil materials for earthworks design and construction, including assessment of the suitability of site material for use as a clay liner; and
- Provide geotechnical conclusions and recommendations for future work.

1.2 Site description

The proposed landfill is located in the Wayby Valley area approximately 6 km southeast of Wellsford. The proposed landfill valley is northwest facing and currently vegetated with pine forest.

Access to the landfill is proposed off State Highway 1, 800 m south of the Hōteo River Bridge, where a sealed road will be constructed up a neighbouring farm valley (Southern Block). Existing access to WMNZ's landholding is via Forestry Road off State Highway 1, which then turns into Wilson Rd (Figure 1). A full description of the project is provided in the Assessment of Environmental Effects (Tonkin + Taylor, 2019m)

1.3 Proposed development

The project comprises construction of a landfill with a capacity of approximately 25.8 Mm^3 and estimated to provide waste storage for the greater Auckland area for in excess of 35 years. A full description of the project is provided in the Assessment of Environmental Effects (Tonkin + Taylor, 2019m).

Key features of the proposed works include:

- Earthworks (cut and fill) to modify the existing valley landform to meet the required storage volume (air space);
- Construction of a clay and HDPE lining system along the base of the landfill;

- Construction of an access road from the existing State Highway 1 up the Southern Block and into the proposed landfill (Eastern Block), involving multiple cut slopes and earth fills along the alignment;
- Construction of a bin exchange area on the eastern side of Waiteraire stream; and
- Construction of a bridge over the Waiteraire stream.

2 Previous reports

2.1 Desk study

A geotechnical desk study assessment was undertaken by T+T in April 2017 (ref. 24838.4010) for a prospective landfill in the Wayby Valley area. The current site was subsequently selected as the preferred site and is the subject of this report.

The report found that Pakiri Formation sedimentary rocks of the Waitemata Group are the main geological unit likely to underlie the project footprint. Northland Allochthon has not been identified beneath the project footprint but may be present. It has been identified in the Western Block in the proposed clay borrow area. Tauranga Group sediments generally occur at elevation of less than 50 m above sea level and therefore unlikely to be present within the project sites, with the exception of the low-lying farmland of the Western Block, and alluvium within or adjacent to drainage channels. Numerous old landslide features were identified at the sites.

3 Geotechnical investigations

3.1 Field investigations

T+T carried out field investigations between 26 February and 7 June 2018, which comprised:

- 14 machine cored boreholes drilled to a depth of between 25 and 50 m;
- 21 hand augured boreholes drilled to a maximum depth of 4.0 m;
- 10 test pits excavated by a hydraulic excavator to a maximum depth of 4.5 m;
- Geophysics consisting of downhole shear wave velocity and Multi-channel Analysis of Surface Waves (MASW) testing;
- Installation of groundwater monitoring wells in all boreholes; and
- Rock mass permeability (Packer Testing) testing.

T+T also carried out field investigations in the Western Block, with the objective to locate additional clay material for liner and cap construction on 13 August 2018, which comprised:

- 9 hand auger boreholes drilled to a maximum depth of 4.0 m.

The investigation locations are shown on Figure GM-F01 in Appendix A, the geological logs from the field investigations are presented in the Geotechnical Factual Report (GFR) (Technical Report A, Volume 2).

3.2 Laboratory testing

Laboratory testing was carried out for the purposes of assessing earthworks suitability including fill compaction, low permeability liner and capping material.

Testing was completed to assess the strength of the soil and rock including Unconfined Compressive Strength (UCS) testing for rock and undrained Triaxial testing for soil.

Geotechnics Ltd was engaged to complete the laboratory testing and the results of the testing are located in Appendix C of the GFR (Technical Report A, Volume 2).

4 Ground conditions

The following subsections describe the ground conditions within the proposed landfill footprint and immediate surrounding area. The geological descriptions and commentary regarding the geological conditions have been based on information obtained from subsurface investigations (boreholes, test-pits and hand augers) and observation of surface outcrops. Geological conditions away from the investigations are inferred, and therefore it must be appreciated that actual conditions could vary from the assumed model.

Detailed and location-specific geological descriptions can be found in the borehole logs (Appendix B of the GFR). The main characteristics and distribution of the lithologies identified at the site are presented below. Geological sections for Valley 1 and the access road are presented in Appendix B – Figures GM-F02 to GM-F11.

4.1 Regional geology

The site is situated within an area that evolved as part of the Waitemata Group Basin, 24 to 18 million years ago, which extended from Whangarei to North Waikato. Much of the Waitemata Group consists of gently inclined undulating sedimentary strata, interrupted by some geological faulting, and localized highly deformed intervals.

The uplift and ongoing tectonic extension has produced regional folding on NW-trending axes forming ridgelines with arc–shaped structural forms and regional NNW tending faults. Bedding-parallel clay seams (much weaker very thin horizons) are thought to have formed within the flanks of these folds during uplift.

A relatively quiet tectonic period has since occurred (approximately 18 to 16 million years before present) leading to deep weathering and erosion. The current landform is strongly influenced by weathering and slope movements. Landslides have formed on the fold flanks along bedding-parallel clay seams and at the soil/rock interface.

During the early stage of the Waitemata depositional period thrust sheets from an allochthonous mass were emplaced onto Northland. An allochthon is a geological mass that was formed elsewhere, in this case slivers of oceanic and continental crust that peeled from the subsiding Pacific plate north of current New Zealand. These sheets subsequently moved and detached blocks advanced into the Waitemata Basin to as far south as Albany. The emplacement of the Northland Allochthon resulted in intensive shearing of the typically extremely weak allochthonous rock mass.

Sea-level fluctuations through the Late Pliocene and Pleistocene have resulted in a series of broad scale alluvial terraces, approximately 30 to 40 m above sea level. Deposition of geologically recent (late Pleistocene) stream, colluvial and peat deposits of the Tauranga Group has occurred within paleo-valleys and paleo-coastal margins during periods of higher sea-levels. It is generally accepted that the current sea-level has remained more or less constant over the past 7,000 years.

The site is predominantly underlain by Pakiri Formation sedimentary rocks of the Waitemata Group. Northland Allochthon has been identified on the low rolling farmland of the Western Block as shown in Figure 4.1 below (marked "Kk"[1]). Tauranga Group alluvial sediments were encountered at the base of the road access valley in the Southern Block, and these materials will likely be encountered around low lying streams and may underlie the airfield part of the Western Block as indicated on the geological map.

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[1] Edbrooke, S.W. 2001. Geology of the Auckland area. Institute of Geological and Nuclear Sciences 1:250,000 geological map 3

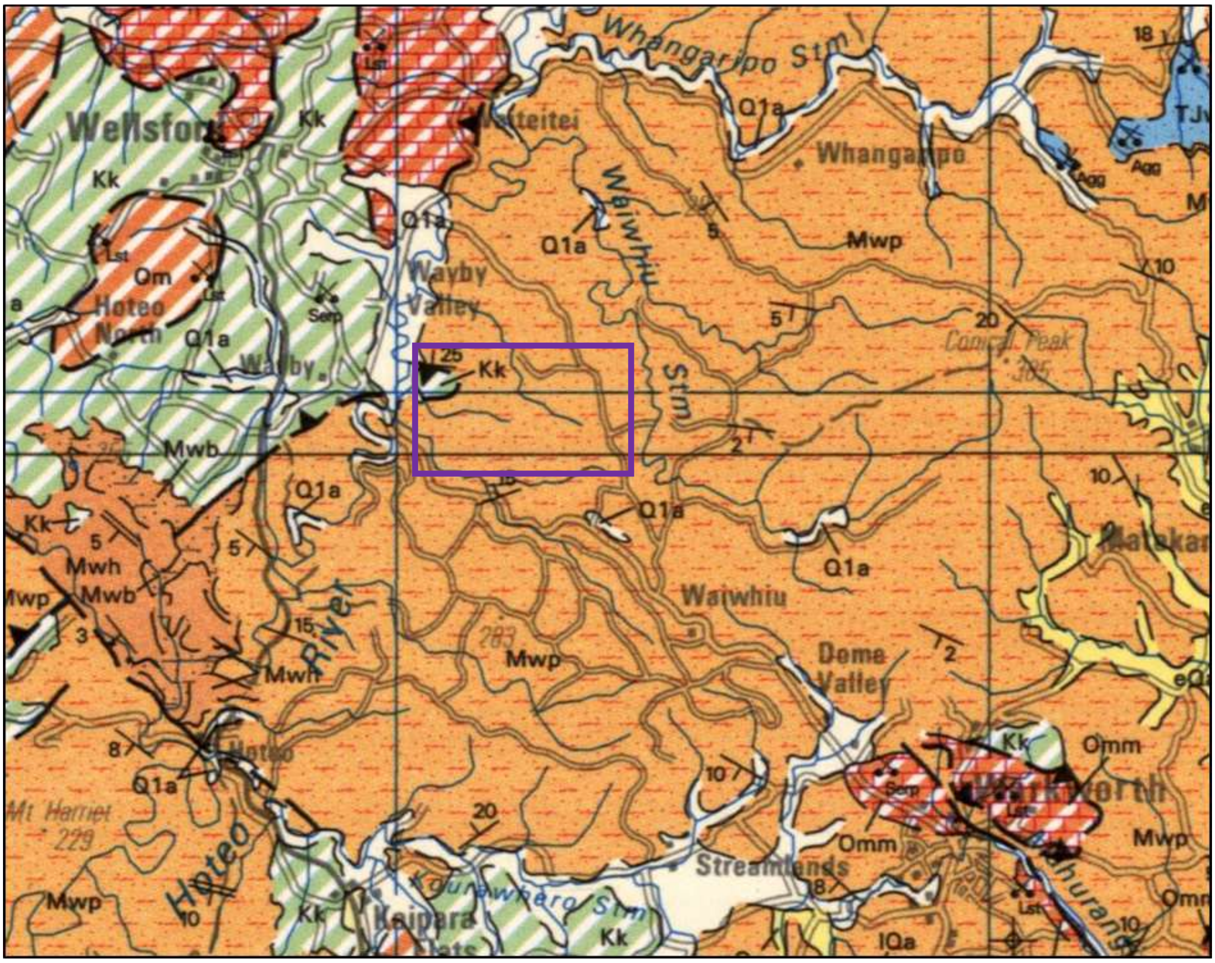


Figure 4.1: Geological setting (source: Edbrooke, 2001)

4.2 Geomorphology

The project site is characterised by a ridge and gully topography, which has been deeply incised by west-north-west draining water courses that form tributaries of the Hōteo River.

The majority of natural slopes encountered in the project area are gently (≤18°) to moderately (19° to 25°) inclined. The south facing slopes are generally steeper than the north facing slopes indicating that the north facing slopes are bedding concordant (dip slopes) and the south facing slopes are bedding discordant (scarp slopes).

Steeply inclined slopes are concentrated within gully and the lower valley side slopes between RL 50 m and RL 150 m, with some localised arcuate shaped, steeply inclined slope areas near the ridge lines.

Site surface conditions were assessed using LiDAR, interpretation of aerial photography and field mapping. Characteristic site geomorphological features include:

- Ridge crest areas that are narrow and flat, which is locally the result of forestry access track and skid site formation;
- Localised arcuate scarps adjacent to the ridge lines that are probably historic landslide scarps although some may be erosional features associated with underlying structural features (folding or faulting); and

- Steeply inclined gully side slopes, particularly at the lower elevations.

The geomorphology of the proposed landfill site and access road is currently subdued by forest cover. We have assessed aerial photographs dated 1940 and 1973 (Figure 4.2), when the project area was devoid of vegetation and geomorphological features are more readily observed. Observed features on the aerial photographs include:

- There is a well-defined northeast (NE) / southwest (SW) structural lineation south of SH1, which appears to project through the valley of the Southern Block. Ridges and gullies are generally aligned NE-SW south of SH1 but are more variable north of SH1 and aligned E-W in the project area;
- Probable historic landslide features (Figure 1, Appendix A) in the vicinity of BH13 on the access road alignment, and below BH1 in the landfill footprint; and
- Numerous active shallow landslips within gully tributaries appearing to coincide with springs. Similar landslips and springs exist throughout Western Block slopes to the west of the landfill footprint.

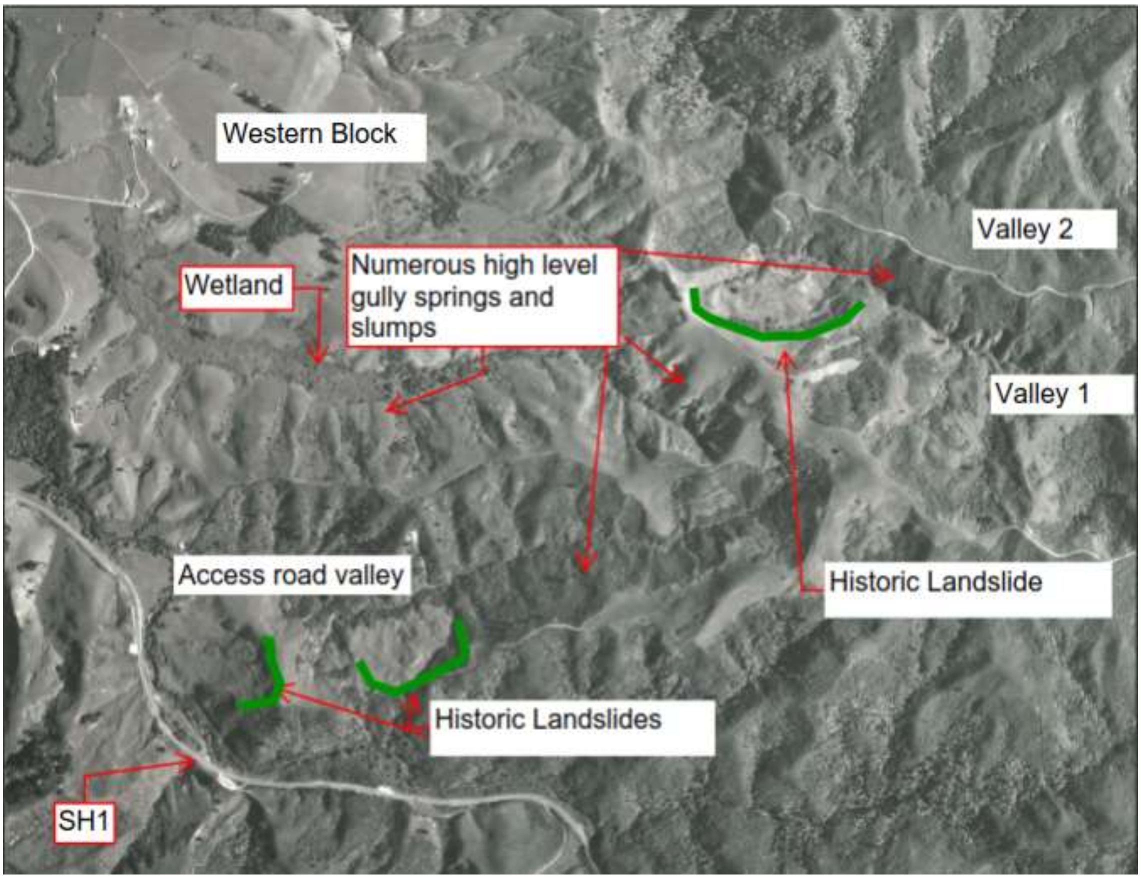


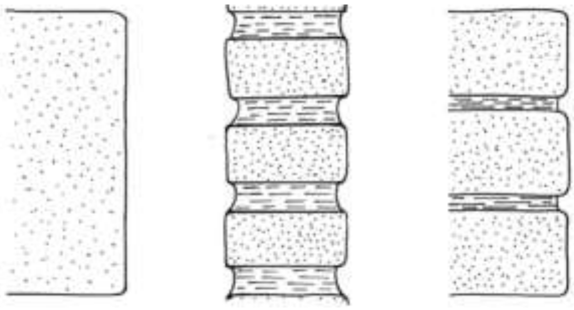
Figure 4.2: 1973 aerial image showing slope geomorphology. Selected identified historic landslides depicted.

4.3 Site geology

4.3.1 Pakiri Formation (Waitemata Group)

Pakiri Formation sediments consist of alternating thick bedded, volcanic rich, graded sandstone and siltstone (Edbrooke, 2001). The following are typical characteristics of the Pakiri Formation:

- The formation typically comprises interbedded weak to moderately strong sandstones and very weak to weak siltstone. Typical bedding sequences are shown in Figure 4.3 below;
- Pakiri Formation also contains moderately strong (20 to 50 MPa) massive volcaniclastic grit beds (gritstone) to conglomeritic coarse sandstones, with mixed source clasts;
- The siltstones and residually weathered soils contain considerable clay content;
- Volcaniclastic sandstones and conglomerates commonly weather to thick residual soils that are known to comprise significant smectite (swelling clay) and some allophane (sensitive clay); and
- Typically the bedding is sub-horizontal to gently dipping with an orthogonal to conjugate jointing pattern. Soft sediment deformation can however result in large increases in bedding orientation over relatively short distances.



1(a)

1(b)

Figure 4.3: Pakiri Formation Rock Structure – a) Schematic showing typical interbedding of sandstone (dotted beds) and Siltstones (dashed beds) b) Typical Pakiri Formation interbedding

Pakiri Formation volcaniclastic muddy sandstones weather forming sensitive silt/sand mixtures. The contact between weathered and unweathered rock may be very sharp and on sloping ground may form a basal shear surface.

Residual Pakiri Formation soils were observed within forestry access road cuts in the project area. These generally consisted of reddish and orange brown silty sands with clay (Photograph 4.1). Limited outcrops of unweathered Pakiri Formation bedrock were observed on the cut slopes, and some bedrock was observed in the stream channels (Photograph 4.2). Bedding structures intersected within boreholes were typically dipping at shallow angles of between 0° and 15°.

The weathering profile of the sandstone and siltstone rock observed in the project area generally occurs within a narrow band, going from completely weathered to slightly weathered within a 5 m interval.

Photograph 4.1: Pakiri Formation residual soil

Photograph 4.2: Pakiri Formation sandstone bedrock observed at the north western end of the site

4.3.2 Northland Allochthon

Northland Allochthon typically comprises an upper residual clay soil overlying highly sheared and broken bedrock. Translational sliding in Northland Allochthon geology can occur on slopes as low as 8 to 10°, typically at the sheared contact at the base of the residual soil. Northland Allochthon was suspected of being present on the lowland west of the site in the Western Block, which was confirmed following the hand auger investigations for clay borrow material. The approximate extent of the Northland Allochthon in this area is shown in Figure 4.4.

Figure 4.4: Inferred extent of the Northland Allochthon within the Western Block

4.3.3 Tauranga Group

The Tauranga Group soils comprise Late Pleistocene to Holocene stream deposits, peatlands and colluvium that have infilled valleys up to 50 m above current sea level. They are generally clay rich, with localised clayey sands at the base and localised sand lenses.

Peat consists of decaying organic matter that typically accumulates in the back swamps of floodplains between levees and the toe of valley side slopes. The type of plant material in the peat can be variable and due to the anaerobic conditions is inhibited from decaying fully.

Colluvium consists of unconsolidated soil and variably weathered rock that have moved downslope by gravity and deposited on the sides and at the base of hillslopes. Colluvium is usually a heterogeneous range of rock and soil types of various particle sizes and is common in the base of the steeply incised gullies.

4.3.4 Residual soil

Pakiri Formation residual soils were encountered throughout the site at depths ranging from 0.8 to 12.5 m below ground level. Thicker residual soil profiles were encountered along the ridgelines of the site, which became thinner downslope towards the valley floor.

The interbedded sandstone and siltstone rock weathers to form interbedded silty sands or sandy silts and clayey silts or silty clays. Silty CLAY and clayey SILT soils recorded undrained shear (vane) strengths typically ranging between 100 to 200 kPa (very stiff) and exhibited moderate plasticity. The silty SAND and sandy SILT materials were the most commonly encountered soils at the site. SPT testing within these soils typically ranged between N = 8 to 18 blows indicating loose to medium dense material.

Approximately half of the investigation locations encountered reddish residual soil materials, which are often associated with low activity clays such as allophane. Allophanic soils are sensitive and can be difficult to condition and compact to a required earthworks compaction specification. They typically exhibit high in-situ (undisturbed) shear strengths and can perform satisfactorily in cut slopes.

Northland Allochthon soils were encountered in hand auger boreholes in the Western Block on the lower lying, gently sloping land west of landfill Valley 1 (Figure 4.4). The materials were recovered using a hand auger and therefore the structure was disturbed, and only the shallow completely weathered material was recovered. The Northland Allochthon material was encountered at depths between 1.4 to 3.5 m below ground level and was typically comprised of clayey residual soil underlain by very stiff to hard, Silt with occasional limestone gravels.

4.3.5 Alluvium

Alluvium material was only encountered in BH14 at the base of the access road valley between State Highway 1 and the Waiteraire Stream. The material extended to 12.5 m depth and comprised mixtures of clays, silts and sands, exhibiting consistencies/densities of firm and loose respectively.

4.3.6 Topsoil

Test pit and hand auger investigations show that the site area is overlain by a relatively thin (0.0 – 0.3 m) layer of topsoil, the reason for this could possibly due to the disturbance to the land through forestry.

4.4 Geological structure

4.4.1 General

The Pakiri Formation rock mass is known to contain structural defects including rock joints, fault zones and bedding plane shears. The degree of structural disturbance can be highly variable over short distances, and the Pakiri Formation bedrock is known to be highly disturbed adjacent to faulted contacts with the Northland Allochthon. Rock mass defects encountered in the site investigation boreholes included bedding, widely spaced steeply dipping joints and some ‘broken zones’.

Few rock outcrops were encountered in the field with limited available structural orientation data. Future borehole investigations for detailed design should include downhole methods to obtain structural orientation of rock mass defects including bedding, joints and faults. Downhole methods include acoustic (ATV) or optical (OTV) telemetry or orientated core retrieval.

4.4.2 Structural trends (lineation) and geological faults

Lineation patterns evident on aerial photographs indicate structural trends aligned northeast (NE) to southwest (SW) and west-north-west (WNW) to east-south-east (ESE). These trends are generally concordant with mapped structural orientation on the published geological map (Figure 4.1 above, Edbrook 2001). Major structural defect orientations within the site area, including rock joints, faults and bedding strike and dip, are likely to be orientated within these regional structural alignment trends.

A northeast / southwest lineation is evident through the project area but is more strongly defined to the south of the project area (south of SH1). The proposed Valley 1 landfill footprint area and the adjacent Valley 2 are located within gullies and ridges that are aligned WNW to ESE.

There are no active faults mapped within or near the proposed landfill footprint. The nearest mapped (inactive) fault is the thrust fault contact between Northern Allochthon and Pakiri Formation on the western margin of the project area (Figure 4.1, Edbrooke 2001). No major fault zones were encountered in the boreholes, and MASW geophysical profiling did not reveal any obvious signs of faulting within the subsurface profile. However, we consider that geological fault structures that have not been previously mapped, or identified within our current investigation, may exist within the footprint of the proposed landfill.

4.4.3 Bedding

Bedding structure observed in the borehole core is mostly gently inclined (5° to 15°) and locally moderately inclined up to 25°. Limited rock outcrop in the field mostly consisted of massive sandstone, and bedding orientations were generally not discernible. From surface geomorphology, bedding appears to dip to the north and northeast with north to north east facing slopes (dip slopes) being more gently inclined than south facing slopes. Bedding structure was measured dipping 25° to the north east in the streambed in the vicinity of BH7.

4.4.4 Rock joints

Rock joints sets observed within road cuttings formed in Pakiri Formation in the area between Puhoi and Warkworth are dominated by orthogonal to conjugate sets that are generally steeply dipping and highly persistent. These rock joint sets appear to strongly influence the bedrock weathering profile, and be the dominant groundwater conduits within the rock fracture aquifer system.

Rock joints observed within the site investigation borehole cores vary in orientation with respect to the core axis, but the majority are steeply inclined at greater than 45° to the core axis (vertical boreholes). The Lugeon testing data indicate that rock joints with dips steeper than 60° are more likely to be water bearing structures within the rock mass aquifer system.

Based on regional structural trends, the likely orientations of the dominant rock joint sets are likely to be NE to SW and WNW to ESE. The dominant joint sets are likely to be steeply dipping (>60°) and have highly persistent face length, often in excess 20 m.

These rock joint structures are likely to have significant controlling influence on exposed rock slope stability and groundwater seepage, particularly groundwater seepage beneath a proposed landfill liner system.

4.5 Groundwater

4.5.1 Groundwater levels

Standpipe piezometers were installed in all of the machine cored boreholes to allow for on-going groundwater monitoring. Regular groundwater readings were taken during the drilling investigation programme, and groundwater level data recorded during and immediately after drilling are recorded on the borehole logs presented in the GFR (Technical Report A, Volume 2).

The Level loggers were installed in borehole BH1, BH2, BH7 and BH9 on 3 May 2018, BH3 and BH5 on 25 May 2018, and BH10 on 31 May 2018. The continuous groundwater level readings are presented on charts attached as Appendix B of the Hydrogeological Assessment Report (HAR) (Technical Report E, Volume 2), together with daily rainfall recorded at the Warkworth weather station (Network Number: A64464).

The HAR concludes that the water levels in BH1, BH2, BH3 and BH10 represent low permeability sections of the Pakiri Formation, with little or no response to rainfall. The groundwater levels in the remaining bores represent higher permeability zones within the rock formations, showing varying response to rainfall.

As part of the hydrogeological assessment study, a deep water bore (TB01) was drilled on the ridge line between geotechnical boreholes BH1 and BH2 (Figure 1). The static water level in TB01 was recorded at 147 m below ground surface (RL 35 m), being much deeper than the water level in BH1 and BH2. The TB01 water level indicates a deeper, regional groundwater level with geological separation (Pakiri Formation aquitard) from the more shallow aquifer indicated by the geotechnical boreholes.

4.5.2 Packer testing

Hydraulic conductivity testing was undertaken in all machine drilled boreholes (BH1 to BH14) by undertaking single packer Lugeon tests at 3.0 m to 6.0 m intervals within the rock profile. The Lugeon test results are presented in Appendix B of the GFR (Technical Report A, Volume 2).

Lugeon testing is an in-situ testing method used to estimate the hydraulic conductivity of the rock mass in a portion of a borehole isolated by pneumatic packers. The packers create a water-tight seal in the borehole, where water is then injected into the isolated portion of the borehole at various pressures, and the amount of water loss is recorded for each pressure.

The groundwater level and Lugeon permeability testing data indicate a typical rock mass fracture aquifer with groundwater storage within the rock mass fracture network. Larger volume water take and higher Lugeon values generally coincided with discrete steeply dipping joints, closely spaced fracture zones or zones of core loss.

Where water take occurred, most tests showed laminar, void filling or dilatant flow behaviour. Negligible to zero water take was observed in many of the Lugeon tests, which were typically associated with intact rock intervals with few joints. Hydraulic conductivities within the Pakiri Formation rock are generally within the range of 1×10^{-9}m/s to 3×10^{-6} m/s but may be in the order of $1x10^{-5}$ m/s where fracture zones occur.

No water take was recorded in any of the Lugeon tests carried out within borehole BH7, located within the valley floor at the mouth of the proposed Valley 1 landfill. An observation from Lugeon and pump testing within Pakiri Formation for other projects within the wider region, is that water takes were more prevalent within boreholes drilled in ridgelines compared with valley floor boreholes. This is considered to be due to a more open or dilated rock mass associated with relaxed ridge slopes compared with confined (tight) valley floor areas.

4.5.3 Perched groundwater and springs

Groundwater seepage from discrete discharge points within the underlying bedrock fracture aquifer systems result in local springs and seeps daylighting within the overlying impermeable residual soil cover. These springs and seeps cause soil saturation and localised slope instability within the surficial soils. This phenomenon is clearly visible in the Western Block slopes where forest cover has been removed.

Groundwater springs and seeps at the soil and rock interface is an important consideration for slope stability modelling for proposed earthworks in the Pakiri Formation rock and soil materials.

Feedback received during community consultation by WMNZ in October and November 2018 was that the land is prone to forming sub-surface voids, which are likely caused from piping effects in the residual soil. No sub-surface voids were encountered in these geotechnical investigations, however the ecology team identified a couple of these features at shallow depths in the Western Block. These features are isolated and can be easily removed during construction, and filtered drainage installed where required.

4.5.4 Groundwater model

A technical Hydrogeological Assessment has been carried out (Technical Report E, Volume 2). Detailed discussion of the groundwater levels, groundwater aquifer systems and aquifer hydraulic characteristics are presented in Section 3 (Environmental Setting) of that report.

The Hydrogeological Assessment identifies three groundwater systems (aquifers) beneath Valley 1, as detailed on Table 4.1 below.

Table 4.1: Groundwater systems

Groundwater system	General characteristics
Shallow perched	Found at the interface of the residual soil with the highly weathered Pakiri Formation. Contributes baseflow to streams.
Upper Pakiri Formation	Found in the higher elevations of the Pakiri Formation around Valley 1. Horizontal flow along fracture zones and bedding planes, proliferates as seepages in Valley 1. Seeps on the valley walls and springs near the floor.
Regional groundwater	Encountered at depth in the Pakiri Formation beneath Valley 1 (TB01). Is estimated to have a relatively shallow hydraulic gradient that flows predominantly toward the Hōteo River. Flow could also occur to the south toward the Waiteraire Stream.

The Hydrogeological Assessment makes the following conclusions

- Shallow groundwater flow direction is anticipated to largely follow the topographical contours, flowing away from the ridgelines and toward the valley floors. These are a muted reflection of the terrain.
- Flow directions may also be influenced by preferential pathways in fracture zones within the Pakiri Formation. These features will result in variable flow directions and likely form a number of local shallow groundwater divides beneath the ridgelines around Valley 1.
- The regional groundwater level beneath Valley 1 is based on readings from the test bore (TB01), which was recorded at approximately 147 m depth or 35 m RL. Based on the comparison of the regional groundwater level against that of the Hōteo River (20 mRL), the regional groundwater is expected to flow to the west, with a low hydraulic gradient (0.006).
- A downward pressure gradient, or vertical gradient is evident beneath Valley 1 between the groundwater in the Upper Pakiri Formation and the regional groundwater. The downward flow of groundwater however, is retarded by the layers of low permeability unweathered siltstone and sandstone, which are at least locally present and may exist more widely.

- Groundwater flow to the south in the regional aquifer has also been considered based on the location and approximate height of the Waiteraire Stream. A low hydraulic gradient of 0.005 has been estimated.
- Rock mass permeability was tested during the geotechnical investigations by using Packer Testing. The T+T geotechnical interpretive report indicates that the permeability within the Pakiri Formation is generally within the range of 1×10^{-9} to 3×10^{-6} m/s, but may be in the order of 1×10^{-5} m/s where fracture zones occur.
- The permeability testing on the weathered soils indicates that they have very low permeability values, in the order of 6.5 x 10-10 m/s. The regional aquifer beneath Valley 1 has a permeability of 1.7×10^{-6} m/s.

5 Geotechnical hazard assessment

5.1 Slope instability

Slope instability on a range of scales is evident at the project site including localised slumping of side cast fill associated with formation of forestry access on ridge lines, to potential large scale old landslide features (Figure 4.2 above, 5.1 & 5.2 below, and Figure 1 Appendix A). Options to mitigate and remediate existing and potential slope instability will include bulk earthworks to form the landfill land form and other ground improvement measures.

The identified historic landslides appear typical of Pakiri Formation slope forming processes where the following conditions may apply:

- i Low angle Pakiri Formation dip slopes with translational dip slope landslides and deeper seated block slides. Sliding may be occurring at the soil/rock interface or along planes of structural weakness such as bedding-parallel clay seams or fault zones;
- ii Instability associated with groundwater seepage at the soil/rock interface;
- iii Shallow rotational landslides with circular failure surfaces;
- iv Rock fall – wedge failures can occur where bedding planes daylight out the slope and joints intersect creating rock blocks; and
- v Shallow soil creep.

We consider that the primary slope instability hazard is likely to be associated with instability associated with groundwater seepage at the soil and rock interface (item ii above) Failure mechanisms in the proposed Pakiri Formation rock cuttings are likely to involve failure on pre-existing defects in the rock (joints and bedding) that form unstable sliding blocks and wedges.

Probable historic landslide features are evident in the vicinity of BH13 on the access road alignment and below BH1 in the landfill footprint (Figures 5.1 and 5.2 below). These appear to be translational landslides located at, or in close proximity, to the soil / rock interface and in association with groundwater seepage. We have not observed any evidence of more deep seated slope instability within the Pakiri Formation bedrock. Clay seams and some polished defects were observed in boreholes BH12 and BH13 at, or near, the soil bedrock interface.

There are numerous shallow landslips concentrated in gully slope areas associated with groundwater springs.

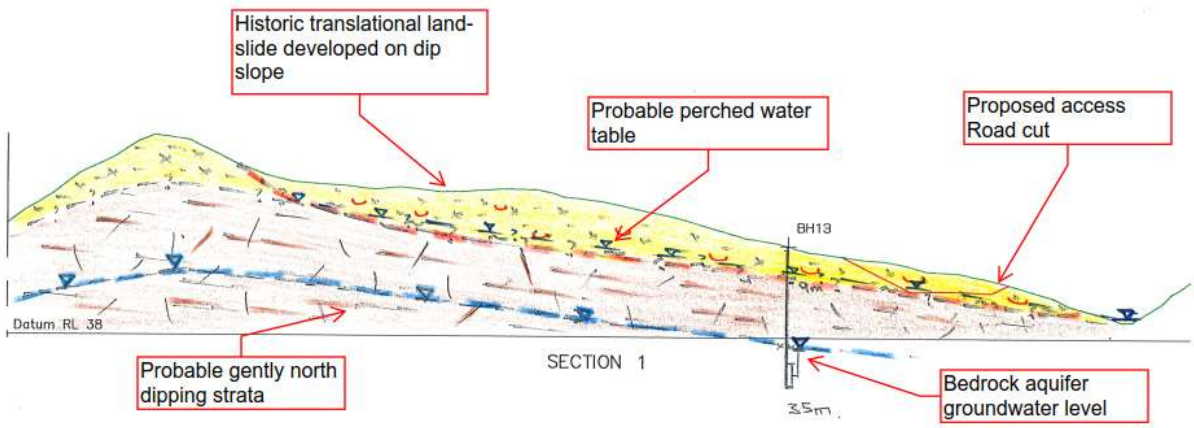


Figure 5.1: Conceptual ground model for historic translational landslide on proposed access road (Chainage 800 m approx.)

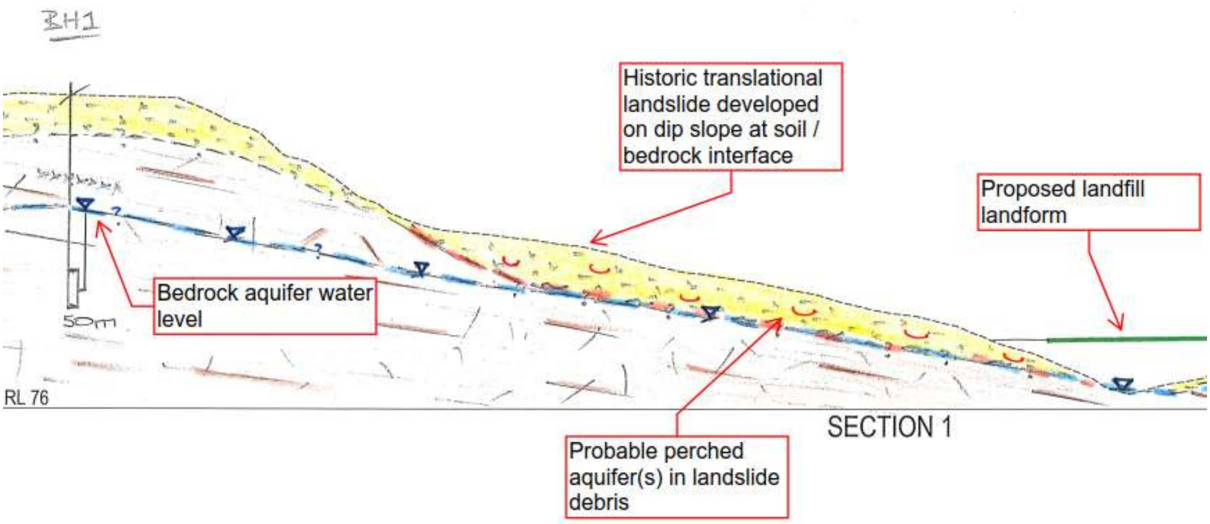


Figure 5.2: Conceptual ground model for historic translational landslide in lower southern flank of Valley 1

Existing slope instability is also present in the Western Block, where it is proposed to source clay and stockpile material for the landfill construction. Northland Allochthon is present on part of the Western Block, which is characterised by low strength residual soil and disturbed bedrock materials with associated slope instability. Slope modifications within both the Pakiri Formation and Northland Allochthon need to be carefully investigated, designed and constructed to mitigate initiation of slope instability. Slope instability can easily be triggered by slope modification such as loading the slope (fill embankments) or cutting into the slope whereby removing toe support to the slope above.

5.2 Groundwater seepage

There is site wide evidence of active springs and seeps occurring on slopes within the proposed landfill project area. Groundwater springs are controlled by water bearing fracture systems day lighting at the surface and seepage at the soil / bedrock interface.

A subsoil drain network will be required beneath the liner to intercept springs and seeps to mitigate the development of excess pore water pressure and hydrostatic head beneath the liner, and associated potential adverse effects on the liner and landfill stability. Additional site investigation data will be required to support detailed design and further model the ground aquifer systems, and potential for hydrostatic pressures to develop beneath the proposed impermeable landfill liner system.

5.3 Seismic hazard

The project area is located in a low seismic hazard area relative to other parts of New Zealand, (Bruxton, 2015). There are no active faults present within 20 km of the project site according to the NZGS New Zealand Active Faults Database.

T+T has reviewed the options for deriving design ground motion values including a Probabilistic Seismic Hazard Assessment (PSHA). The aim of a PSHA is to assess the seismic hazard for a specific site, incorporating recent advances in knowledge and the state of practice[2]. A PSHA provides

[2] Bradley B. A. (2015). Benefits of Site-Specific Hazard Analyses for Seismic Design in New Zealand. *Bulletin of the New Zealand Society for Earthquake Engineering*. Copy attached for reference.

earthquake design parameters values that may be used as an alternative to the loading parameters used for routine engineering design.

A preliminary assessment[3] of design seismic ground motions and site-specific seismic hazard for the project was issued on 13 April 2018 (T+T Ref. 1005069.1120). The full PSHA has been presented under separate cover (see Technical Report C, Volume 2). The PSHA concluded that:

- The Auckland Regional Landfill project is in an area of relatively low seismicity compared to the rest of New Zealand.
- This is supported by the results of the PSHA, with lower levels of shaking than specified in the design standards calculated for the site. However, for regions of relatively low seismicity, NZS1170.5 and the Bridge Manual prescribe a minimum criteria that is to be considered in determining the ultimate limit state (ULS) seismic actions for design, even if a PSHA indicates a low probability of this occurring. The minimum criteria is to provide a margin of safety against collapse if a major earthquake occurs on a currently unknown fault.

5.4 Liquefaction

Liquefaction hazards primarily exist in areas of saturated, unconsolidated, granular soils. Liquefaction can lead to significant loss of strength causing subsidence and/or lateral spreading of embankments and can impact structural foundations.

Due to the plastic behaviour of Pakiri Formation residual soils liquefaction is not expected to be a risk at the site. However, the alluvial soils encountered in BH14 (alongside the Waiteraire Stream) consisted dominantly of firm sandy silt with low plasticity and loose silty sands. These materials are potentially susceptible to liquefaction and lateral spread given the proximity of an open stream channel.

Site specific liquefaction assessments, including assessments of susceptibility of the soils to liquefaction and the liquefaction vulnerability of earthworks and structures will be undertaken at detailed investigation and design stage, notably for the bridge over the Waiteraire Stream. Soils that may be prone to liquefaction within the proposed project areas could occur in the low lying valley floor areas but large volumes are not anticipated. Where these soils occur they will likely be excavated as part of construction of the landfill landform.

5.5 Compressible soils

Compressible soils may be encountered within low lying gully or valley floor areas underlain by Tauranga Group soils or recent alluvium. In particular, peat is highly compressible and prone to large settlements of up to 50% of the peat layer thickness. The possible presence of peat at the site is not yet known as none was encountered during the investigations. Secondary consolidation may also be significant. By contrast, the Pakiri Formation soils will not be prone to significant consolidation settlement due to their formation by weathering of over-consolidated sedimentary rocks.

Compressible soil within the proposed project areas are likely to occur in the low lying valley floor areas but large volumes are not anticipated. Where these soils occur they will be excavated as part of construction of the landfill landform.

[3] Tonkin & Taylor (2018). Dome Valley site: Preliminary assessment of design seismic ground motions and site specific seismic hazard

5.6 Overland flow

Meteorological data indicates that the landfill site area experiences some of the highest rainfall in the Auckland region, receiving on average approximately 1600 mm per year, compared with Auckland which experiences approximately 1200 mm per annum. The rainfall data indicates the landfill site receives both higher annual rainfall and peak intensities compared to other Auckland sub-regions. High rainfall and peak intensities result in increased overland flow, which can create the following hazards:

- Erosion along overland flow channels;
- Increased sediment run-off on non-vegetated slopes;
- Flooding in low lying areas;
- Difficulty placing fill material during earthworks; and
- Slope instability.

An assessment of stormwater management and overland flow hazard for the proposed landfill is being prepared under separate cover. Earthworks construction will need to be sequenced and managed to minimise exposure of erodible and sensitive soils to rainfall.

5.7 Acid sulphate soils

Acid sulphate soils in the Auckland and Northland regions are generally associated with unconsolidated Holocene sediments deposited in low lying coastal areas. Disturbance (earthworks) and oxidation of acid sulphate soils can release sulphuric acid, which can lead to leaching and mobilisation of minerals and increased metal concentrations into the environment. Whangarei District Council provides information on acid sulphate soils at the following link:

http://www.wdc.govt.nz/BuildingandProperty/Property-Information/Pages/Acid-Sulphate-Soil.aspx

Acid sulphate soils were not encountered in the site investigations, and we consider that they are unlikely to exist on site due to the site elevation, distance from the coastal environment and absence of Holocene geology within the project footprint. However, if there are any field indicators for acid sulphate soils during earthworks then we recommend that appropriate action is taken (including laboratory testing) in accordance with relevant standard such as the Australian national guidance document for acid sulphate soils

(http://www.waterquality.gov.au/SiteCollectionDocuments/sampling-identification-methods.pdf)

6 Geotechnical design parameters

6.1 General

The geotechnical design parameters discussed in this report were developed based on interpretation of all existing field investigation and laboratory test results, considerations of published correlations and engineering judgement based on site observations and experience in other projects with similar materials.

Summary geotechnical parameters, including Pakiri Formation rock and soil materials and associated engineered fill, are presented in the following Tables 6.1 to 6.4 below. These are based on previous work from the nearby Puhoi to Warkworth (P2W) corridor project[4] and modified where appropriate in accordance with laboratory testing data from the current project.

6.2 Soil parameters

Effective shear strength parameters from laboratory testing of the residual soil materials ranged from 25° to 37° for the friction angle (Ø’) and 8 to 24 for cohesion (c’). The remoulded, compacted coefficient of permeability ranged from 3.5 x 10^{-10} to 7.6 x 10^{-10} m/s. The Allophane clay content ranged from less than 5% to 5% to 7% and the soils were tested as being non dispersive to moderately to slightly dispersive.

We consider that the soil parameters presented in Tables 6.1 and 6.2 below are appropriate for geotechnical design for the Pakiri Formation derived materials within the proposed landfill and access road areas.

6.3 Rock mass parameters

Unconfined compressive strength testing of the Pakiri Formation bedrock included 0.22 MPa for highly weathered sandstone and 22 MPa for unweathered sandstone. Further testing of the highly weathered rock was not possible due to its typically highly fractured nature and thin profile across the site. We consider that the rock mass parameters presented in Table 6.3 are appropriate for preliminary geotechnical design for the Pakiri Formation bedrock materials within the proposed landfill and access road areas.

[4] Puhoi to Warkworth Motorway PPP, Geotechnical Interpretive Report, Doc No: 650-RPT-008-NX2, Design Work Pack#:6.0-03, Contract No: NZTA-PA4030

Table 6.1: Summary of design strength parameters for Pakiri Formation soils (SPT N<50)

Unit Code	Geotechnical Unit Description	Unit Weight (kN/m^3)	Effective Stress				Total stress		Young's Modulus		v Poissons Ratio
			Lower bound		Characteristic		Lower bound	Upper bound	Lower bound	Characteristic	
			c' (kPa)	ϕ' (°)	c' (kPa)	ϕ' (°)	S_u (kPa)	S_u (kPa)	E (MPa)	E (MPa)	
Pakiri Formation Soils											
P1	Firm, residually weathered to completely weathered	17	2	27	5	28	25	50	5	10	0.35
P2	Stiff, residually weathered to completely weathered	17	2	29	7	30	50	100	10	15	0.35
P3	Very stiff to hard, highly weathered	18	7	32	10	34	100	200	20	35	0.30

Table 6.2: Summary of design characteristic consolidation parameters for Pakiri Formation soil

Unit Code	Geotechnical Unit Description	Characteristic mv (m2/MN)	Characteristic cv (m2/yr)
P1	Firm, residually weathered to completely weathered	0.15	20
P2	Stiff, residually weathered to completely weathered	0.10	20

Table 6.3: Summary of design rock mass parameters

Unit Code	Geotechnical Unit Description	Unit Weight (kN/m^3)	m_i	MR	Characteristic UCS	Typical GSI Range	Mohr-Coulomb		v Poissons Ratio
							c' (kPa)	ϕ' (°)	
P4	Highly weathered to moderately weathered Pakiri Formation rock	21	8	200	2.0	55-65	60	35	0.30
P5	Slightly weathered to unweathered, interbedded Pakiri Formation rock	23	8	200	13	55-65	150	40	0.26
P6	Tectonised and/or frequent bedding plane partings in highly weathered to moderately weathered interbedded Pakiri Formation	21	8	200	1.5	25-35	10	26	0.30
P7	Tectonised and/or frequent bedding plane partings in slightly weathered to unweathered interbedded Pakiri Formation	23	8	200	10	30-40	100	28	0.26

Note: Where unfavourably oriented continuous weak planar discontinuities are present, these will dominate the stability of the rock mass.

c' = effective cohesion

Ø' = effective friction angle

mi = Intact rock constant

MR = Modulus Ratio

GSI = Geological strength index

Erm = Youngs modulus (rock mass)

6.4 Earth fill

Compaction testing of elected Pakiri Formation soils recorded maximum dry density in the range 1.49 to 1.69 tonnes / m^3, and optimum moisture content of 20 to 27%. We consider that the fill geotechnical parameters presented in Table 6.6.4 below are appropriate for earthworks design.

The tested natural water content of a selection of the site soils ranged from 48.7% to 61.9%, which indicates that the soils are wet of optimum and will require conditioning (drying back) to achieve the optimum soil moisture content for compaction. It will be essential to carry out bulk earthworks during the summer earthworks season. In addition, the soils are generally high in silt and sand content and the soil compaction will be sensitive to moisture conditions.

Table 6.6.4: Summary of geotechnical fill design parameters

Fill Unit Code	Fill Type	Unit Weight (kN/m^3)	Effective Stress		Total stress	Young's Modulus E (MPa)	v Poissons Ratio
			Cohesion c' (kPa)	Friction Angle ϕ' (deg)	Undrained Shear strength S_u (kPa)		
Rock Fill							
RF0	Imported Rockfill (GAP 65, GAP 150 or suitable run-of-pit)	21 -23	0	45	N/A	80	0.30
RF1 & RF2	Local source Pakiri Formation rockfill (Rockfill Class 1 & 2)	19	2	40	N/A	80	0.30
Soil Fill							
F2	Structural Fill	18.5	7	30	140	30	0.35
F3	Structural 120 Fill	18	6	29	120	20	0.35
F4	Buttress Fill	18	5	28	80	N/A	N/A
F5	Landscape Fill	17	3	26	60	N/A	N/A
F6	Unsuitable Fill	15-17	0	24	40	N/A	N/A

7 Slope stability assessment

7.1 Methodology

Slope stability analyses have been undertaken on the geological cross sections provided in Appendix B using proprietary Slope/W limit equilibrium software. The analyses were run using an entry-exit failure method and all results reported are optimised for the critical slide surface unless they were considered to be unrealistic. The analysis considered the following slope stability design cases:

- Static (long term stability, using effective stress parameters);
- Static (short term stability, using total stress parameters);
- Static with elevated groundwater conditions;
- Seismic Ultimate Limit State (ULS earthquake loading); and
- Seismic Serviceability Limit State (SLS earthquake loading).

Analyses have been limited to larger scale instability which is primarily controlled by rock mass properties. Smaller (bench-scale) 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.

7.2 Landfill slopes

7.2.1 Landfill design

The construction of the landfill will involve earthworks modification of the existing valley landform and installation of an extensive surface and subsurface drainage network. The proposed earthworks and drainage measures will enhance the existing slope stability, including excavation and / or buttressing of existing landslides.

The landfill design involves the placement of geosynthetic liner materials, which requires stability checks for base sliding failure mechanisms within the landfill for both static and seismic loading cases using the material properties listed in Table 7.1 below.

Table 7.1: Geotechnical properties of Landfill components

Material	Unit Weight (kN/m^3)	Cohesion c' (kPa)	Friction Angle ϕ' (deg)
Refuse	9.3	5	25
Liner interface (peak)	17	0	25
Liner interface (post-peak)	17	0	16

To assist feasibility level landfill design, slope stability analyses were carried out for potential landfill cut slopes formed at 2.5H:1V and steeper slopes at 1H:1V. Further to our initial slope stability analyses work, the proposed slopes up to 75 m high within the landfill footprint have been designed at 3H:1V on the southern slopes and 2.5H:1V on the northern slopes, with a 12 m wide bench cut every 20 m of vertical slope.

Two cross valley sections (Section 2 and Section 4, Figure GM-F02, Appendix B), and a long section (Section 8, Figure GM-F02, Appendix B) through the landfill valley, were selected as critical for slope stability analysis. Slope stability analyses of Sections 2 and 4 were analysed with no refuse material in the valley i.e. end of landfill construction and prior to placement of refuse. This is considered the most critical stage case in terms of slope stability because there is no refuse to provide additional buttress support to the valley slopes.

It is proposed to construct a clay toe bund at the toe of the landfill to act as a buttress for the landfill refuse during operations. The clay bund is proposed to be a minimum of 12 m high, and 12 m wide at the top and sloping at 3H:1V on either side of the bund. Slope stability of the landfill long section has been modelled with the full design height, which is the critical design case.

The summary results of the slope stability assessment are summarised in Table 7.2, Table 7.3 and Table 7.4 below, and the Slope/W output presented are in Appendix C.

7.2.2 Analyses results

Table 7.2: Landfill Cross Section 2 Slope/W results

Analysis Case	Slope	Target FoS	Calculated FoS	Appendix C
Static long term	2.5H:1V	1.5	1.7	Figure 12
	1H:1V	1.5	1.2	Figure 13
Static short term	2.5H:1V	1.5	3.9	Figure 12
	1H:1V	1.5	3.5	Figure 13
Elevated groundwater	2.5H:1V	1.2	1.5	Figure 12
	1H:1V	1.2	1.2	Figure 13
Seismic ULS	2.5H:1V	1.0	1.2	Figure 12
	1H:1V	1.0	1.1	Figure 13
Seismic SLS	2.5H:1V	1.0	1.5	NA
	1H:1V	1.0	1.2	NA

Table 7.3: Landfill Cross Section 4 Slope/W results

Analysis Case	Slope	Target FoS	Calculated FoS	Appendix C
Static long term	2.5H:1V	1.5	1.9	Figure 14
	1H:1V	1.5	1.3	Figure 15
Static short term	2.5H:1V	1.5	2.8	Figure 14
	1H:1V	1.5	3.5	Figure 15
Elevated groundwater	2.5H:1V	1.2	1.7	Figure 14
	1H:1V	1.2	1.2	Figure 15
Seismic ULS	2.5H:1V	1.0	1.2	Figure 14
	1H:1V	1.0	0.9	Figure 15
Seismic SLS	2.5H:1V	1.0	1.6	NA
	1H:1V	1.0	1.2	NA

Table 7.4: Landfill long section Slope/W results (Appendix C Figure 16)

Analysis Case	Target FoS	Calculated FoS
Static long term	1.5	2.6

Analysis Case	Target FoS	Calculated FoS
Static short term (clay bund)	1.5	5.4
Elevated groundwater	1.2	2.6
Seismic ULS	1.0	1.1
Seismic SLS	1.0	1.7

The results indicate adequate stability for the landfill valley design cut slopes constructed at batter angles of 2.5H:1V or less. The slope stability safety factors would be expected to improve with the buttressing effect of increasing refuse fill height over time.

The stability assessments indicate that the toe bund provides satisfactory support to the compacted refuse material upslope, and mitigates the potential for slope failure along the liner interface under static and seismic design conditions. It is proposed to place the refuse material at a maximum slope of 5H:1V and minimum slope of 10H:1V.

7.3 Landfill access road slopes

The proposed landfill access road will extend from SH1 and up through the Southern Block to connect to the southern ridgeline of Landfill Valley 1. The access road has a proposed corridor width of 20 m, requiring cut slopes and fill embankments to achieve the required design grade. Concept fill slopes of 2.5H:1V (22°), and cut slopes of 1H:1V (45°), with a 4 m wide bench proposed every 8 m height resulting in an overall cut slope of 1.5H:1V (34°) were considered.

Slope stability analyses have been undertaken for two critical sections along the proposed access road, at chainage 1300, where one of the highest cuts is proposed, and chainage 1450, where one of the highest fill embankments is proposed. The results of the slope stability assessment are summarised in Table 7.5 and Table 7.6 below, and presented in Appendix C.

Table 7.5: Access road chainage 1300 Slope/W results for cut slopes – trial design 1H:1V (Appendix C Figure 17)

Analysis Case (1H:1V)	Target FoS	Calculated FoS
Static long term	1.5	1.2
Static short term	1.5	2.8
Elevated groundwater	1.2	1.1
Seismic ULS	1.0	0.9
Seismic SLS	1.0	1.1

Table 7.6: Access road chainage 1450 Slope/W results for fill slopes – trial design 2.5H:1V (Appendix C Figure 18)

Analysis Case (2.5H:1V)	Target FoS	Calculated FoS
Static long term	1.5	1.7
Static short term	1.5	1.8
Elevated groundwater	1.2	1.6
Seismic ULS	1.0	1.2
Seismic SLS	1.0	1.5

Based on these stability analyses it is concluded that additional slope stability modelling was required to refine the cut slope design for the overlying weaker soil materials, and the results of these analyses are presented in Table 7.7 below, and presented in Appendix C.

Table 7.7: Cut slope design refinement

Analysis Case	Target FoS	1.5H:1V calculated FoS (Appendix C Figure 19)	2H:1V calculated FoS (Appendix C Figure 20)
Static long term	1.5	1.3	1.5
Static short term	1.5	2.6	2.7
Elevated groundwater	1.2	1.2	1.2
Seismic ULS	1.0	1.0	1.0
Seismic SLS	1.0	1.2	1.3

The results indicate adequate slope stability for 2H:1V cut slopes formed in the upper soil strength materials, overlying steep cut slopes in the underlying rock. These preliminary slope designs in soil, natural soil and rock will require confirmation during detailed design, including the potential impact of defect-controlled instability e.g. wedge and planar block failures within the bedrock.

Following the formation of the proposed cut and fill slopes outside the landfill valley footprint, bare soil slopes will be susceptible to surface rilling and erosion. Slope protection measures will be required such as placement of hydroseed, topsoil and / or erosion matting.

7.4 Clay borrow and stockpile access road stability

It is proposed to construct an access road from the ridgeline east of Valley 1 down to the clay borrow area in the Western Block. As discussed in section 4.3.2, Northland Allochthon material was encountered in the lower lying, rolling hills of the Western Block where the proposed road and clay borrow area is located.

Investigation and design of the access road has not yet been undertaken, however, it is likely to require some form of ground improvement work to meet design stability requirements. Ground improvement options could include slope drainage, slope retention or flattened slope cut/fill geometries.

7.5 Historic landslide features

The existing landfill and access road slopes are currently vegetated in forest and the subtle geomorphic features are masked. Numerous landslide features and groundwater springs are evident on observed aerial photographs dating back to 1940. Numerous steep sided and deeply incised gullies are also evident on aerial photographs and were observed during site walkovers.

Large historic landslide features located within the access road alignment and southern slope of Valley 1 will require further specific investigation and design, including hazard and risk assessment (Figure 1, Appendix A, & Figure 5.1 and 5.2 above).

These landslide features should be anticipated and allowed for in the final ground works design. Ground improvement work including installation of subsoil drains and slide surface shear keys will probably be required.

The proposed cut-to-fill earthworks to form the landfill ground profile should mitigate or remediate much of the existing slope instability features where they will be cut out or buttressed.

BH13, located within a probable large historic landslide within the access road alignment (Figure 1), encountered a deep soil layer and closely fractured bedrock with high groundwater flows with Lugeon testing (Figure 5.1 above). The deep soil layer and high Lugeon takes may reflect ground disturbance associated with the slope instability, which will require further investigation at the detailed design stage.

8 Earthworks

The construction of the landfill and access road will require a significant volume of cut to fill earthworks. Various types of fill material will be required including bulk fill, rock fill, clay liner, leachate drainage layer, landfill final cap, and landscape fill.

8.1 Excavation / Ripability

The bulk of the proposed excavations will occur at the south-eastern end of the landfill Valley 1, with proposed excavations up to 40 m deep.

The excavatability of the Pakiri Formation is expected to vary depending on the rock type (sandstone v siltstone), weathering and fracture spacing. The highly to moderately weathered rock is expected to be rippable with a large excavator. The closely fractured, unweathered to slightly weathered rock may also be easily rippable where bedding planes and fractures provide planes of weakness within the rock mass, which allow it to be excavated more readily. This is illustrated in the ripability chart by Pettifer and Fookes, 1994. Massive sandstone beds with widely spaced joints will probably require blasting.

A summary graph showing the average fracture spacing of the rock with regard to depth for all machine cored boreholes has been produced in Figure 8.1 below. The figure shows that as depth increases the fracture spacing also increases. This is particularly noticeable from 20 m, where between 10 to 20 m depth the average fracture spacing is typically less than 1.0 m, but from 20 to 50 m depth the spacing generally widens. This has been illustrated on Figure 8.2 below, the green shaded area (rock less than 20 m depth) is expected to be hard digging to hard ripping, whereas the orange shaded area (rock greater than 20 m) is expected to be easy ripping to potentially requiring blasting.

Experience with formation of road cuts on the Puhoi to Warkworth motorway extension project has shown that strong unweathered Pakiri Formation rock requires blasting for excavation, and particularly the blocky, strong and massive sandstone lithotype. In addition, pre splitting may be required in the blocky rock mass with widely spaced joints, or post blasting trimming following blasting to form the design profile.

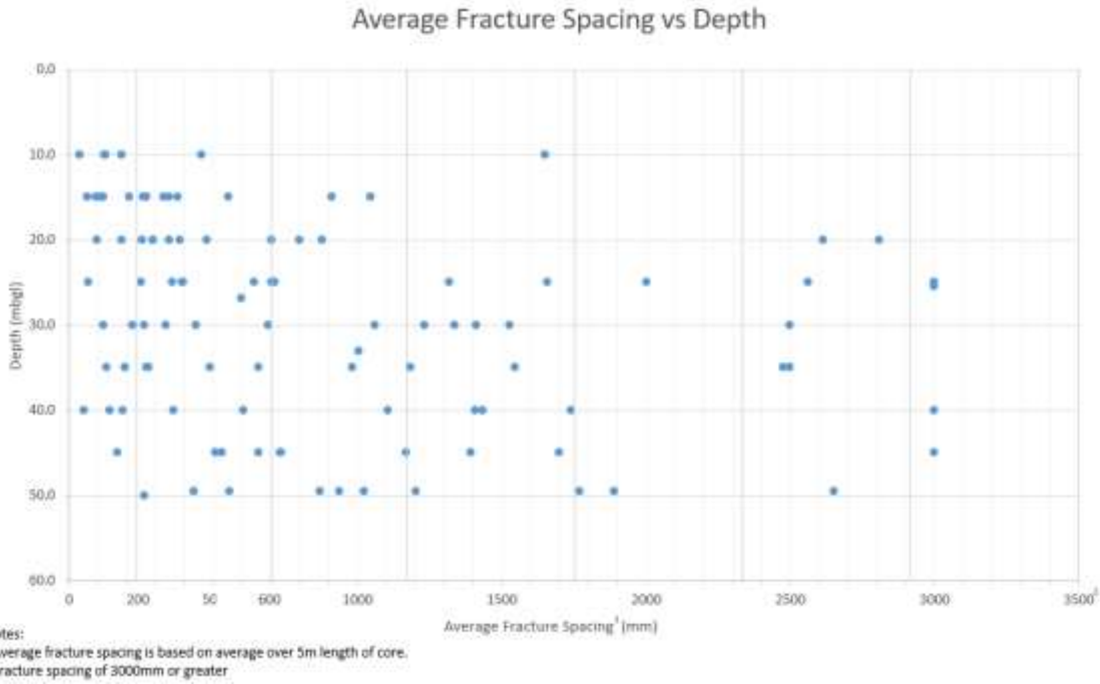


Figure 8.1: Average fracture spacing vs depth of in-situ rock

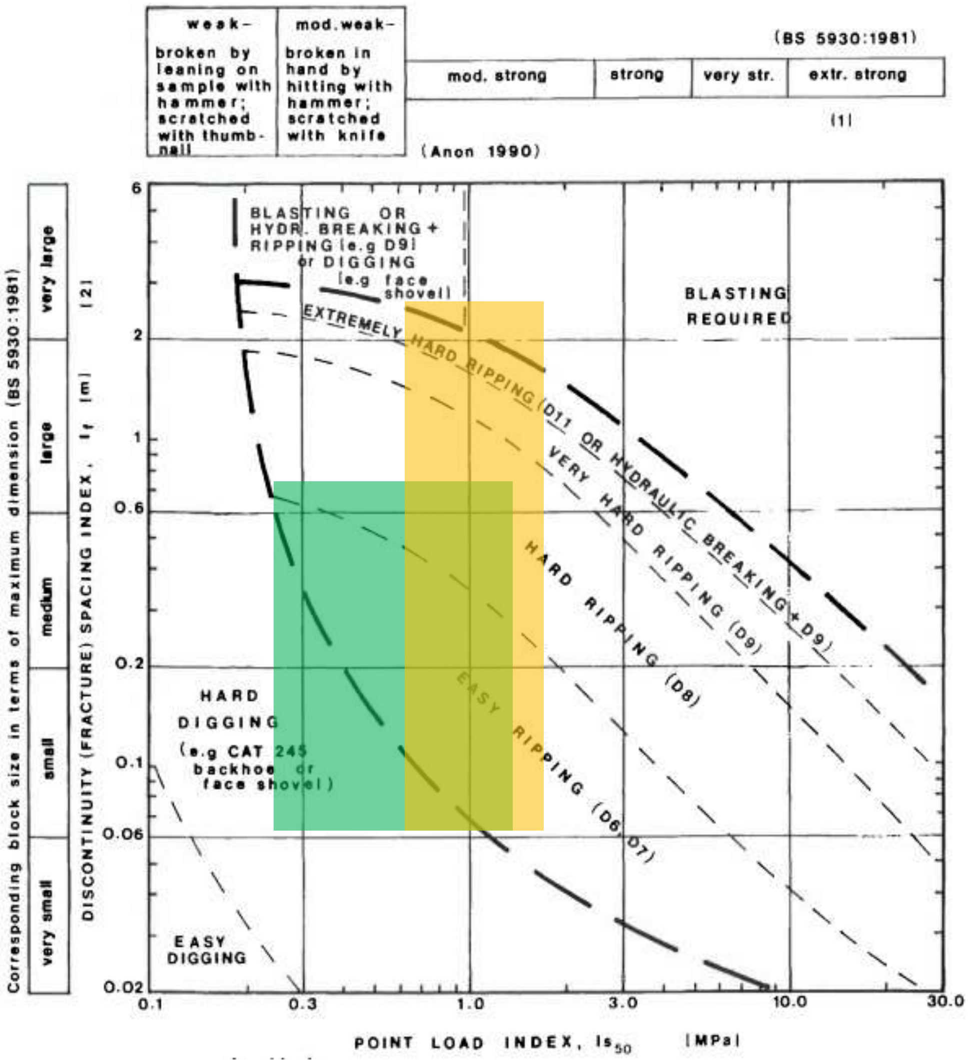


Figure 8.2: Ripability chart (Pettifer and Fookes, 1994)

8.2 Bulk / Rock fill

Slake durability tests were undertaken in the Puhoi to Warkworth motorway project (P2W) to assess the disintegration characteristics of the Pakiri Formation rock. The results indicate that the sandstone rock has a slake durability of >88% compared to >60% for the siltstone and interbedded units. In accordance with Grainger (1984), rocks with a UCS greater than 3.6 MPa and a slake durability index higher than 90% are considered durable, and therefore acceptable for use as Rockfill.

Slightly weathered to unweathered siltstone rock typically has unconfined compressive strengths (UCS) of 5 to 10 MPa, however, are more susceptible to slaking and could be used as Rockfill with caution. This material could be placed in combination with soil as bulk fill with compaction control of air voids. Sandstone and conglomerate units are not considered to be particularly susceptible to slaking especially if they are not subject to wetting and drying or repetitive loading and are of relatively high strength (>10 MPa).

Sandstone beds ranging in UCS strength from 10 to 20+ MPa with thickness of greater than 1 m could be selectively quarried. The typical sandstone UCS ranges from 15 to 20 MPa and the siltstone typically 5 to 10 MPa.

8.3 Soil materials

The residually weathered Pakiri Formation soils may be suitable for placement as compacted fill for clay liner or other purposes. However, where they are wet of optimum moisture content, or contain significant allophanic clay, they may not be suitable as they will be difficult to condition and meet a fill placement specification. In addition, if in future they are exposed to fluctuating moisture conditions they may prone to shrink or swell behaviour causing disruption to built structures. They could be selectively taken for use as fill in places other than critical structural fills.

Allophanic soils within the Pakiri Formation are typically recognised by their deep red or pink colour. They are also characterised by being highly sensitive i.e. high ratio of peak strength to remoulded strength.

8.4 Clay liner and cap

Selected samples from the test pit investigations underwent laboratory testing to assess their suitability for construction of the clay liner and capping material for the landfill. The laboratory testing undertaken was:

- Heavy compaction - to assess the optimum moisture content required to achieve the optimum bulk density for the potential clay liner fill materials;
- Triaxial permeability – to assess the hydraulic conductivity of the potential clay liner fill material. The material was tested at 3% wet of optimum moisture content and 5% below optimum bulk density; and
- Pinhole permeability – to assess the sensitivity to erosion from water flow.

The results from triaxial and pinhole permeability laboratory testing are summarised in Table 8.1 below.

Table 8.1: Summary results of permeability testing

Sample ID	Depth (m bgl)	Material	Permeability (m/s)	Dispersion class
TP03	1.0	Clayey SILT	5.8×10^{-10}	Non dispersive
TP06	0.8	Sandy SILT	9.01×10^{-10}	Slightly dispersive
TP08	3.8	Sandy SILT w some clay	7.57×10^{-10}	Non dispersive
TP30	1.0	Silty CLAY	3.53×10^{-10}	Non dispersive

Compaction testing of Pakiri Formation soils indicated recorded maximum dry density in the range 1.49 to 1.69 tonnes / m^3, and optimum moisture content of 20 to 27% (% of dry mass). The natural moisture content of the soils was higher than the optimum moisture content and therefore some conditioning (drying) of the soil will be required. It will be better to construct the clay liner during the summer months to allow the material to dry out prior to compaction.

The triaxial permeability tests produced coefficient of permeability values less than 1×10^{-9} m/s indicating that the remoulded soil has a very low permeability, and therefore should be suitable for clay liner and cap construction, allowing a margin of difference between lab and field performance of up to one order of magnitude, to meet a lining system element spec requiring 1×10^{-8} m/s.

Three of the pinhole permeability tests produced non-dispersive soil behaviours. The sample from TP6 at 0.7 to 1.5 m depth displayed moderate to slightly dispersive characteristics, which is probably due to the higher sand content within this sample. Potentially dispersive soils may be mitigated by soil mixing.

The results from these tests indicate that the existing site soils derived from weathering of Pakiri Formation are probably suitable for use as clay liner material but that there are potential risks in terms of compaction and erodibility. It is likely that the allophanic soils will be suitable for clay liner construction, perhaps with mixing with other soils.

Additional investigations have been undertaken within the Western Block in order to locate additional clay liner and cap material. The investigations encountered silt/clay material 1.4 to 3.5 m thick overlying Northland Allochthon material, which we anticipate will be suitable for liner and cap construction. To specifically assess how these materials will behave as liner and cap materials it is recommended that further sampling and testing be undertaken at the time of detailed design.

8.5 Leachate drainage layer

It is expected that in the early stages of construction there may not be sufficient on site cut material to use for the leachate drainage layer, therefore this material will need to be imported. This material is expected to be a 7/20 drainage aggregate with no fines content.

During later stages of the landfill construction, large volumes of rock will be excavated from the eastern end of Valley 1, this rock material may be suitable for use as leachate drainage aggregate. The Sandstone rock has a moderate strength and is relatively unweathered. The crushing resistance of the rock will need to be tested in order to assess its suitability at the time of its proposed use for use as leachate drainage aggregate.

8.6 Access road bridge

As part of the access road to the landfill a bridge is required to cross the Waiteraire stream. BH14 was drilled on the southern side of the stream at the proposed bridge abutment, ground conditions at this location consisted of alternating firm silt and loose sand materials before encountering moderately strong rock at 12.55 m. Artesian groundwater conditions were encountered within the BH14, with groundwater head measured greater than 1.3 m above ground level.

Two foundation options for the proposed bridge include bored piles or a culvert. Due to the firm/loose, saturated and potentially liquefiable ground conditions encountered at the bridge location traditional shallow foundations will not be appropriate for the bridge abutments.

Bored piles would need to be embedded into the rock in order to satisfy the vertical and horizontal design loads. Due to the artesian groundwater conditions the top of the pile casing would need to be at least 1.5 m above ground level to prevent water flowing out the casing during construction. Fixity of the bridge deck and piles may be required to resist potentially high horizontal loads generated during lateral spreading.

Installation of a precast box culvert may be the most cost effective option to allow vehicles to cross the stream. This option would reduce the lateral spreading risk to the bridge structure and would ease construction due to there being no disturbance to the artesian surface. However this option presents ecological impacts worth consideration, minor ground improvement work may be required to improve the bearing capacity and installation of erosion protection such as riprap, otherwise this would be a very viable option. The choice of method must also consider ecological effects that are not addressed in this report.

9 Site suitability

We assess that the land area contained within the proposed project area is suitable for landfill development in general accordance with the Technical Guidelines for Disposal to Land[5]. The geology within the proposed landfill footprint includes variably weathered Pakiri Formation bedrock and associated residual soils.

The variably fractured bedrock and residual soils generally have low permeability, which should provide good natural containment. The site is not close to the coast, is not close to any active faults and does not overlie Karst geology or high permeability sand and gravel. The rock and soil materials available on site are generally suitable for liner construction and landfill operation. .

[5] WasteMINZ, August 2018, Technical Guidelines for Disposal to Land

10 Conclusions and recommendations

10.1 Conclusions

- Geotechnical investigations have been carried out to support feasibility level design and consenting of the proposed Auckland Regional Landfill. Investigations were carried out between 26 February 2018 and 7 June 2018, which comprised 14 machine cored boreholes, 21 hand augured boreholes, 10 test pits and geophysics consisting of downhole shear wave velocity and Multi-channel Analysis of Surface Waves (MASW) testing;
- Additional geotechnical investigations have been carried out to source clay material for liner and cap construction. The investigations were completed on 13 August 2018 and comprised of 9 hand augers drilled to a maximum depth of 4.0 m;
- Laboratory testing was carried out for the purposes of assessing slope stability and earthworks suitability including fill types and compaction, low permeability liner and capping material;
- The proposed landfill site and access alignment is underlain by Pakiri Formation bedrock consisting of interbedded sedimentary sandstone and siltstone with some conglomeritic layers. The bedrock is overlain by a variable thickness of residual, colluvial and landslide soil;
- Existing slope instability hazards include some large historic landslide features and numerous active shallow landslips (within upper soil) within gully tributaries appearing to coincide with spring lines. It is expected that the risk of instability associated with these features will be mitigated by earthworks to form the landfill landform and additional ground stabilisation and improvement work;
- Northland Allochthon material was encountered in the proposed clay borrow area, which is a relatively unstable geological rock formation.
- Other geotechnical hazards and constraints include potentially compressible or liquefiable soils, groundwater seepage and hydrostatic forces beneath a future liner system, and strong massive sandstone bedrock that will require blasting to form the landfill basegrade. The potentially liquefiable soils, generally confined to the gully axis will be removed from the landfill footprint, and subsoil drainage will be installed to intercept groundwater seepage.
- There are no active geological faults mapped in the site area, and no significant fault zones were encountered in the boreholes or MASW testing. Rock mass defects encountered in boreholes included bedding, widely spaced steeply dipping joints and some broken zones. Bedding is gently inclined to the north. It is anticipated that some fault disturbed bedrock is likely to encountered within the proposed landfill footprint and access road alignments;
- The hydrogeological assessment has identified three potential groundwater systems including upper and deep level fracture aquifer systems in the Pakiri Formation, and perched groundwater in the overlying residual soil. Additional data and groundwater testing will be required to verify and confirm the hydrogeological model;
- Downhole Lugeon testing indicated variable permeability in the upper bedrock aquifer associated with various bedrock joint and fracture sets.
- A network of subsoil drains will be required beneath the landfill liner to intercept groundwater from springs and seeps.
- A site specific seismic hazard assessment has been prepared for the site and reported separately. The study concluded that the proposed Auckland Regional Landfill site is in

an area of relatively low seismicity compared to the rest of New Zealand, and the PSHA assessed lower levels of shaking than specified in the design standards calculated for the site. However, for regions of relatively low seismicity, NZS1170.5 and the Bridge Manual prescribe a minimum criteria that is to be considered in determining the ultimate limit state (ULS) seismic actions for design;

- Geotechnical input design parameters are presented, which follow on from previous large scale corridor projects in Pakiri formation rock and soil materials and reflect laboratory testing results undertaken for this project;
- Slope stability analyses generally indicate adequate slope stability for he proposed cut and fill design slopes. Design modifications will probably be required during the detailed design and construction phases. Existing landslide features identified within the Southern Valley road access alignment and in the Valley 1 landfill footprint will require additional investigation and design of appropriate remedial and mitigation for detailed design;
- The existing rock and soil materials should generally be compatible with proposed landfill earthworks and construction including formation of the base grades and clay liner. Potential geotechnical constraints include rippability of rock within deep cuts, and the available volume and geochemistry (allophane content) of low permeability clayey soils.
- Laboratory testing indicates that the site soils are wet of optimum and will require conditioning (drying back) to achieve the optimum soil moisture content for compaction. It will be appropriate to carry out earthworks during the summer earthworks season and / or utilise lime. The soils are generally high in silt content with some sand and the soil compaction will sensitive to moisture fluctuation.
- Additional investigations within the Western Block encountered silt/clay material 1.4 to 3.5 m thick overlying Northland Allochthon material, which should be suitable for liner and cap construction. To specifically determine how these materials will behave as liner and cap materials it is recommended that further sampling and testing be undertaken at the time of final design.

10.2 Recommendations

We make the following recommendations to support detailed design and construction of the proposed Auckland Regional Landfill:

- Carry out additional drilling investigations for detailed ground and groundwater modelling and slope stability analyses to support detailed design. These investigations should be carried out at the time of final design, preferably after vegetation clearance;
- Additional borehole and rock mass data in areas of large proposed cuts to assess rock mass structure, rippability, re-use as hard fill and groundwater seepage modelling. Utilise downhole core orientation methods to support rock defect structure orientation analyses for slope stability and groundwater flow and seepage modelling;
- Additional rock and soil sampling and laboratory testing to support detailed design;
- CPT testing at the proposed location of bridge over the Waiteraire stream to assess the liquefaction susceptibility risk at the time of detail design;
- The investigation, final design and specification of landfill and appurtenant structure earthworks should be carried out or reviewed by a Chartered Professional Engineer practicing in geotechnical engineering or an experienced Engineering Geologist;
- Earthworks should include subsoil drainage, where required;

- A detailed construction methodology will be required to ensure that the proposed earthworks are staged and carried out in a manner that will not contribute to slope instability;
- During construction cut slopes will need to be assessed by a geotechnical professional for the presence of adverse geological conditions including landslide deposits, geological faults and the groundwater seepage.
- On satisfactory completion of earthworks the Geotechnical Engineer or Engineering Geologist should submit a completion report and appropriate land use and earthfill suitability statements;
- All earthworks should be carried out in accordance with NZS4431:1989 and all fill foundations should be stripped, benched and drained; and
- An erosion and stormwater control plan must be prepared prior to the commencement of earthworks and should specify measures to avoid adverse offsite effects arising from the construction works.

11 Applicability

This report has been prepared for the exclusive use of our client Waste Management NZ Ltd, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

Recommendations and opinions in this report are based on data from boreholes, test pits and hand augers. The nature and continuity of subsoil away from the boreholes, test pits and hand augers are inferred and it must be appreciated that actual conditions could vary from the assumed model.

Tonkin & Taylor Ltd

Report prepared by:	Report prepared by:
....	....
Alex Naylor Senior Engineering Geologist	Tim Coote Senior Engineering Geologist

Authorised for Tonkin & Taylor Ltd by:

....

Simonne Eldridge

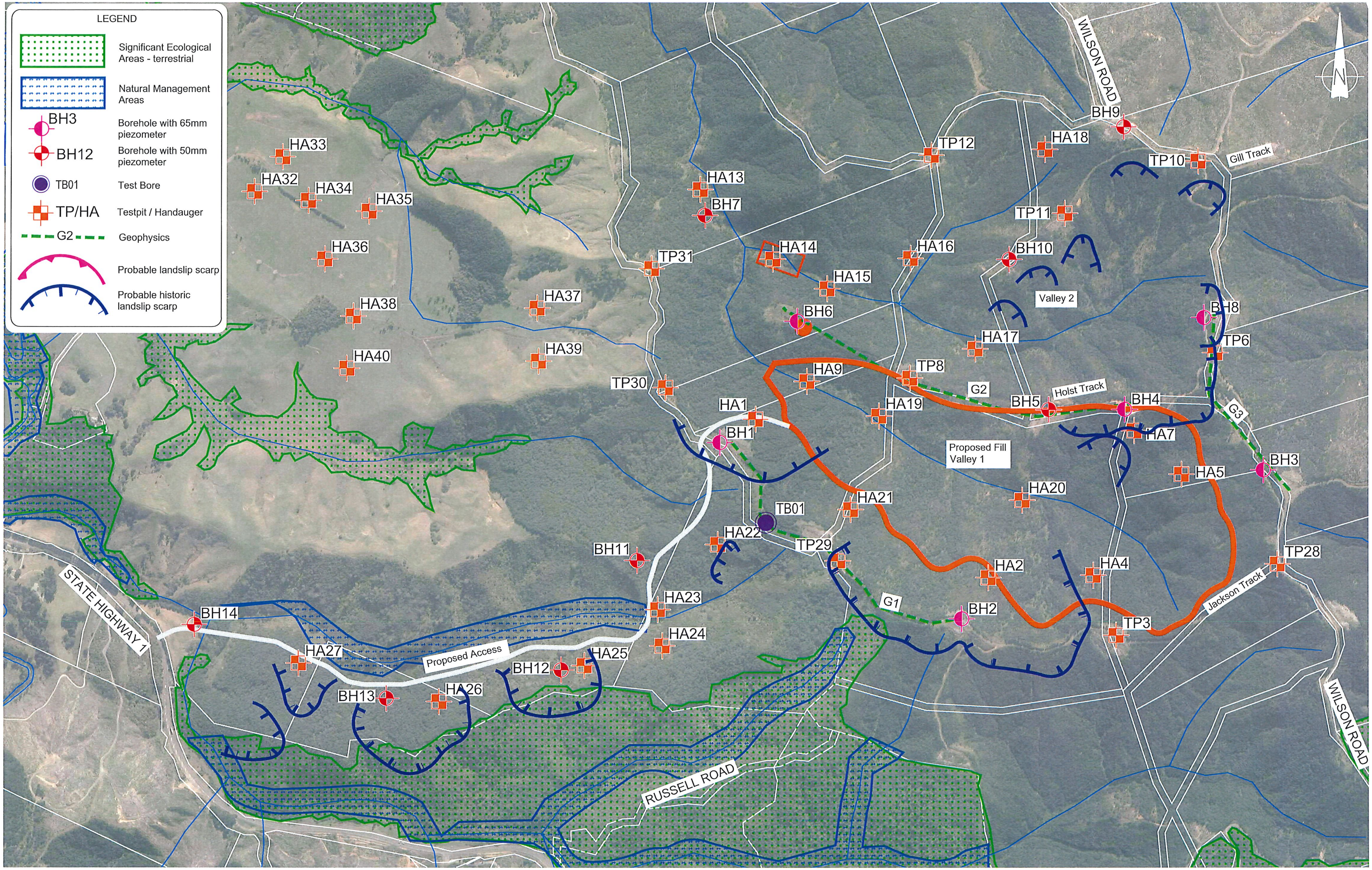
Project Director

ALNA
p:\1005069\issued documents\final application for lodgement may 2019\volume 2 - technical reports\technical report b - geotechnical interpretive report\technical report b - geotech interpretive .docx

Appendix A: Figures

- **Figure GM-F01 – Site Investigation Plan**

C:\12dSynergy\data\ALBTCAD\Polaris_WOL_Concept_55\CAD\FIG\1005069.1120-Interp-F1.dwg 2019-May-24 3:05:59 PM Plotted By: LOVE JADVANI



SCALE 1:10,000

0 100 200 300 400 500 (m)

Aerial photo sourced from Auckland Council GIS Website.
Property boundaries sourced from Land Information New Zealand data as at 16-Feb-2017
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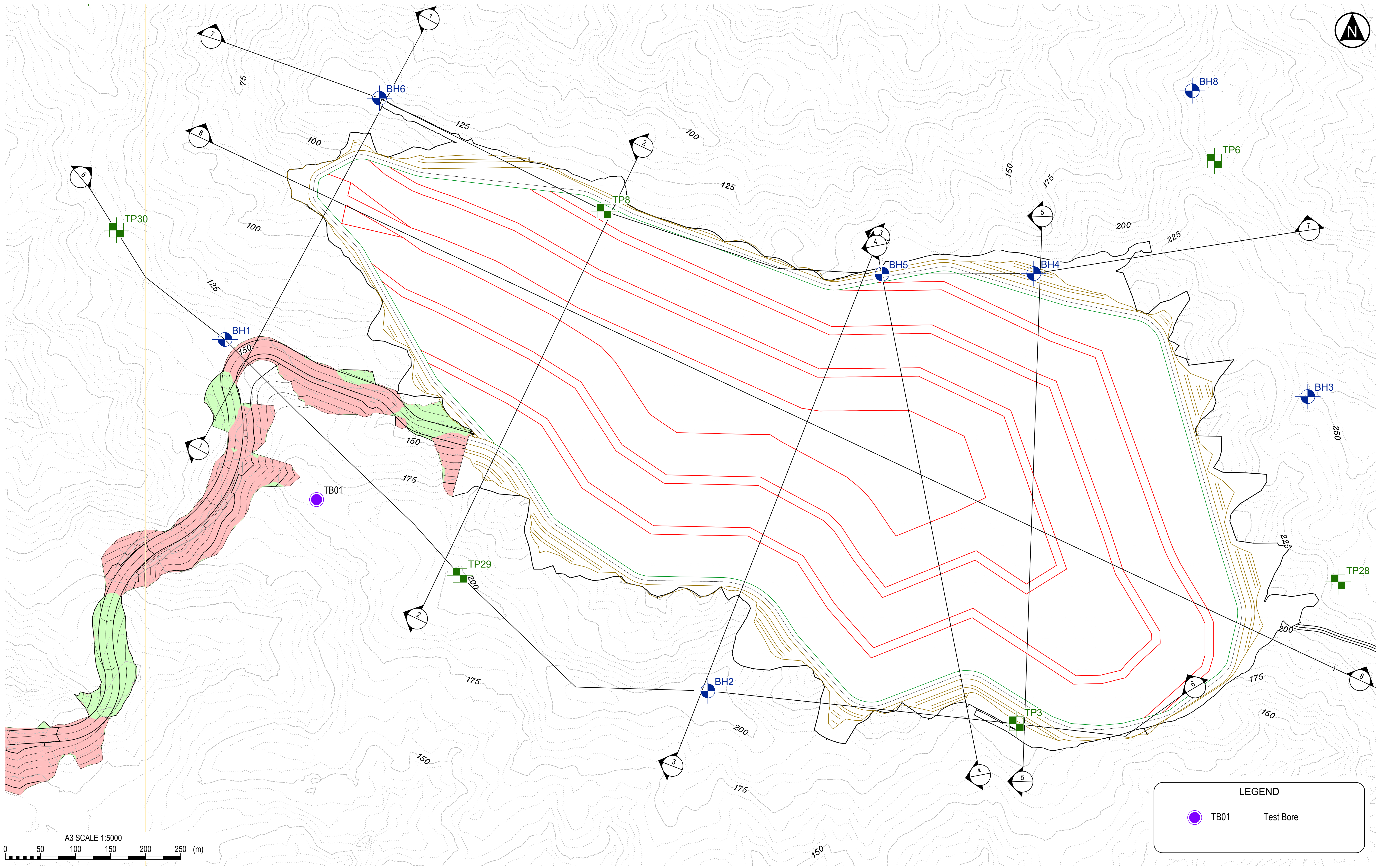
DESIGNED	TC	Aug.18	
DRAWN	RBS	Sept.18	
CHECKED			
APPROVED	DATE		

CLIENT	WASTE MANAGEMENT NZ Ltd
PROJECT	AUCKLAND REGIONAL LANDFILL PROJECT
TITLE	GEOLOGICAL SECTION Site and Investigation Plan
SCALE (A3)	1:10,000
FIG No.	GM-F01
REV	1

Appendix B : Geological Sections

- **Figures GM-F02 to GM-F11**



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DRAWN	BKD	Jun.18	
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APPROVED	DATE		

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PROJECT **AUCKLAND REGIONAL LANDFILL PROJECT**

TITLE GEOLOGICAL SECTION

Geological Section Plan

SCALE (A3) 1:5000 FIG No. GM-F02 REV 1

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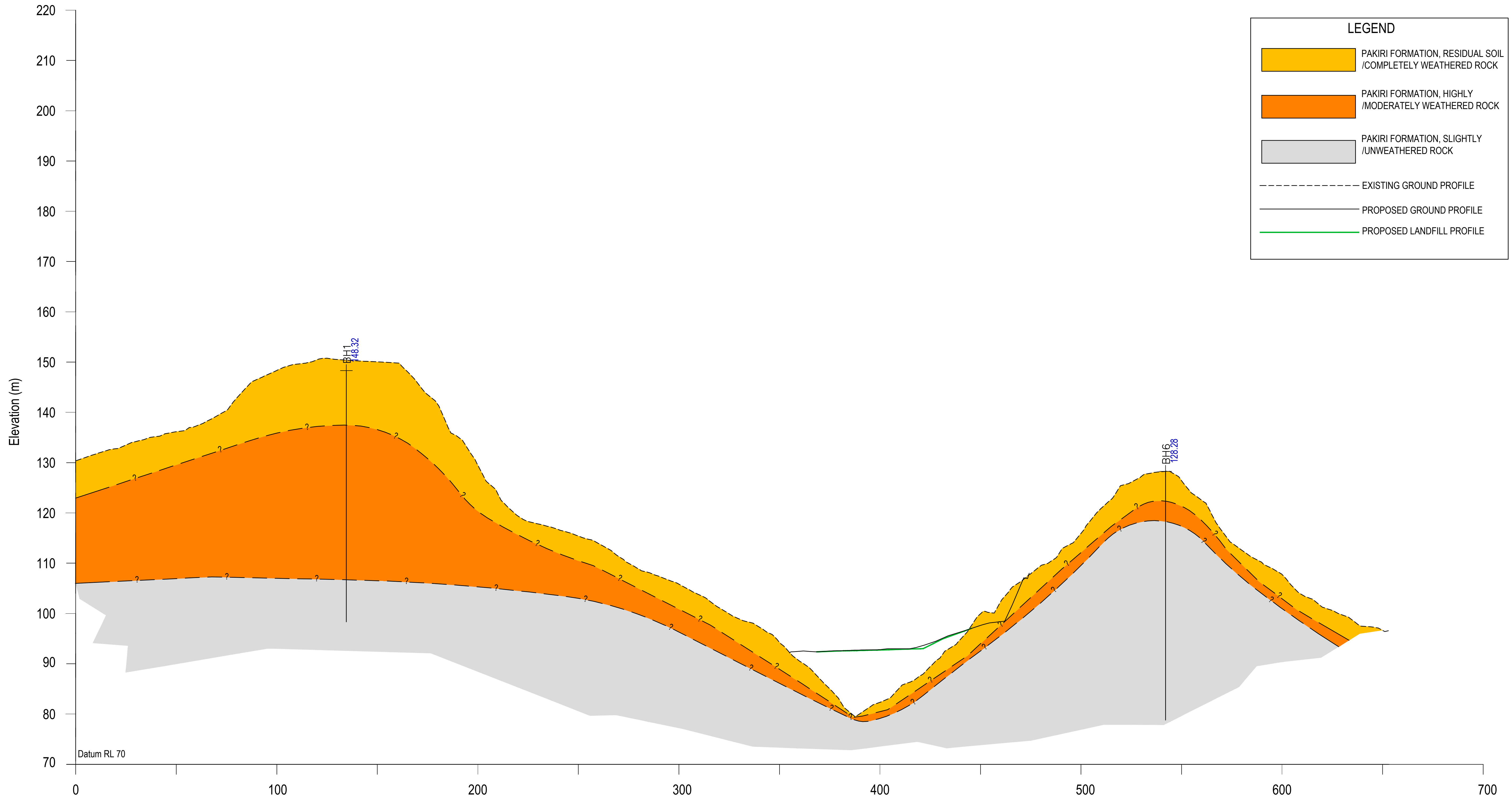

SECTION 1

A3 SCALE 1:2000
0 20 40 60 80 100 (m)

ORIGINAL IN COLOUR

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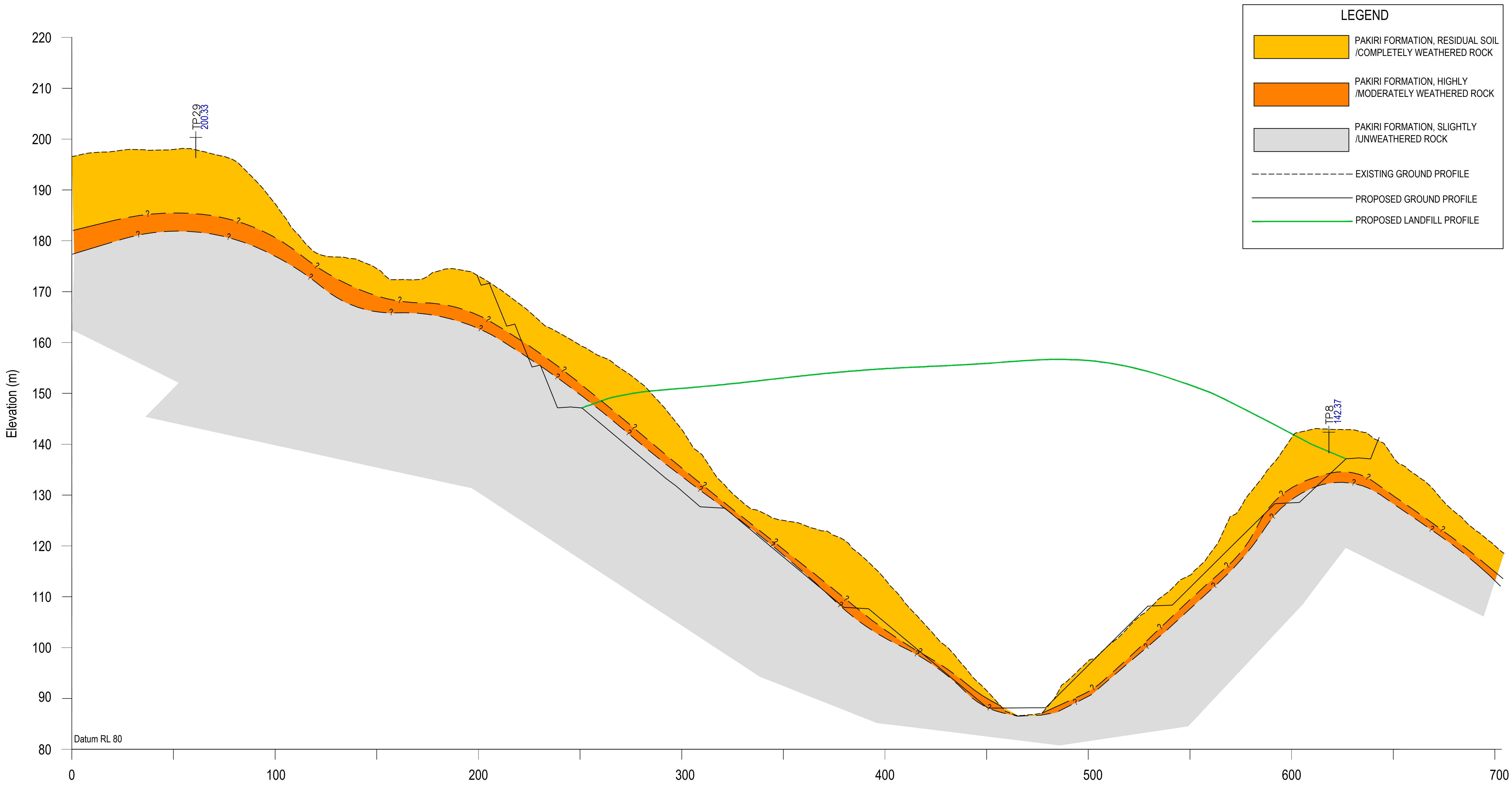
CLIENT WASTE MANAGEMENT NZ Ltd
PROJECT AUCKLAND REGIONAL LANDFILL PROJECT
TITLE GEOLOGICAL SECTION
Section 1
SCALE (A3) 1:2000 H (1:800V) FIG No. GM-F03 REV 1



SECTION 2

A3 SCALE 1:2000
0 20 40 60 80 100 (m)

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TITLE GEOLOGICAL SECTION
Section 2
SCALE (A3) 1:2000 H (1:800V) FIG No. GM-F04 REV 1

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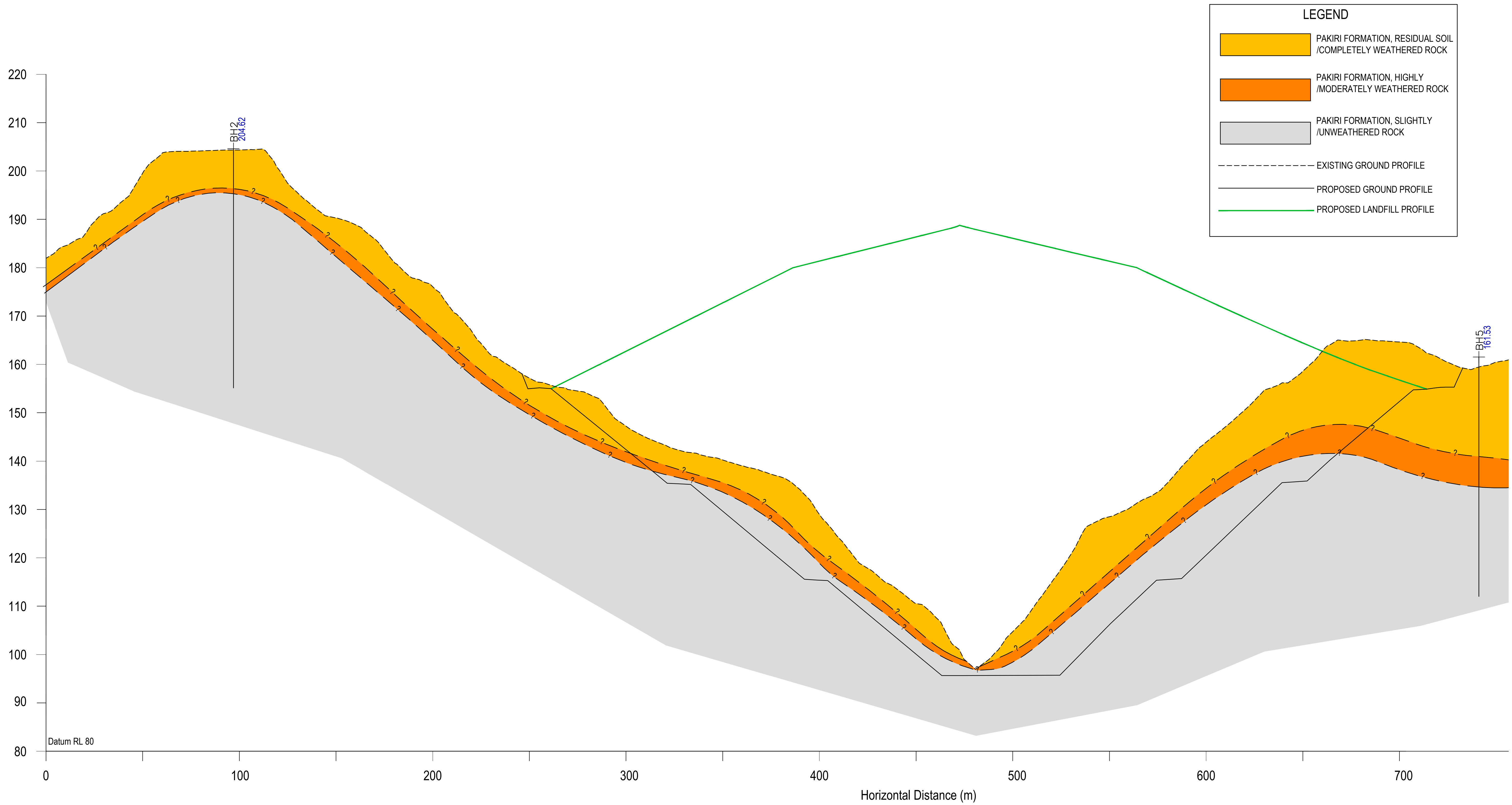

SECTION 3

A3 SCALE 1:2000
0 20 40 60 80 100 (m)

ORIGINAL IN COLOUR

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TITLE GEOLOGICAL SECTION
Section 3

SCALE (A3) 1:2000 H (1:800V) FIG No. GM-F05 REV 1

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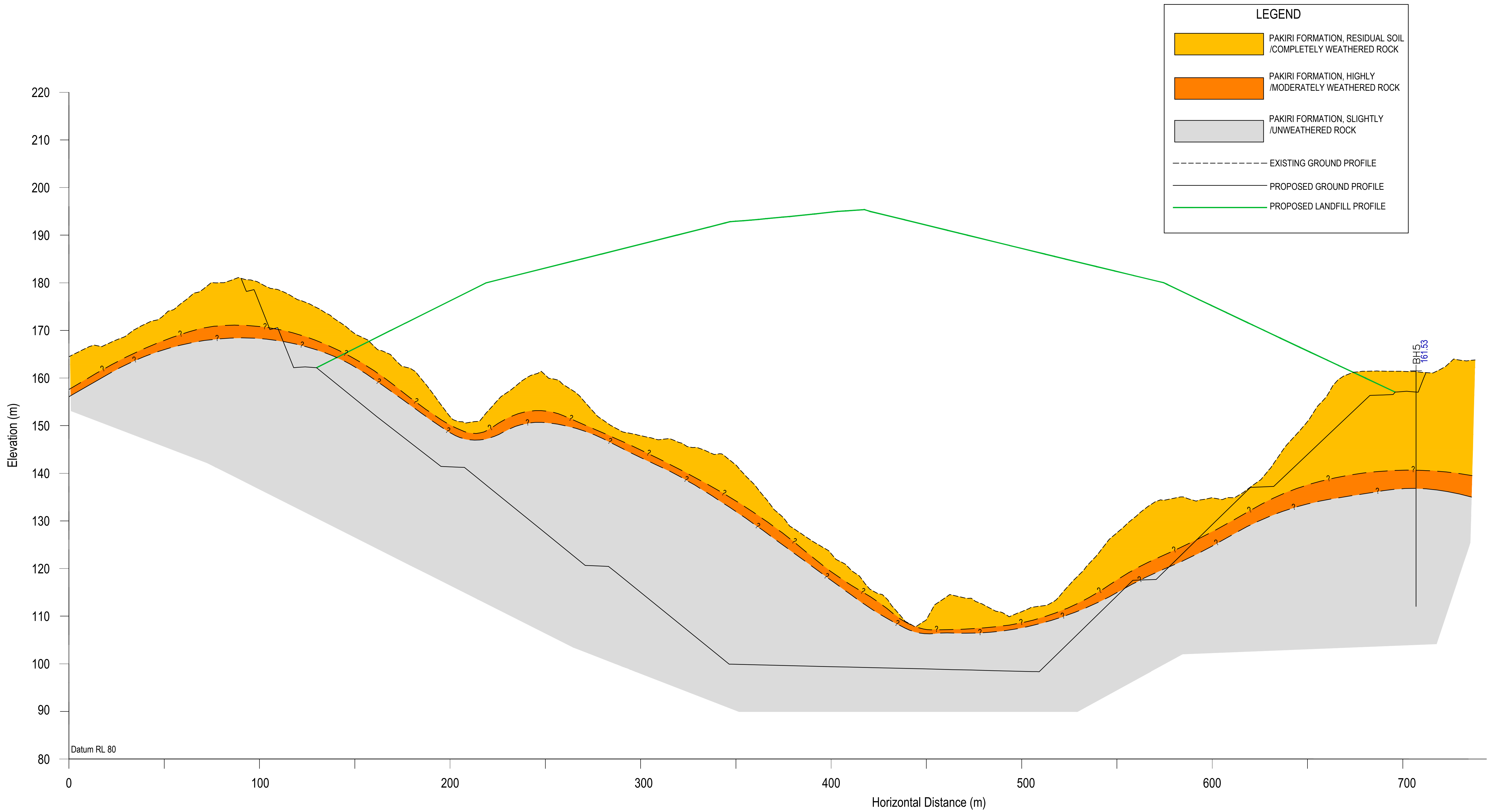

SECTION 4

A3 SCALE 1:2000

0 20 40 60 80 100 (m)

ORIGINAL IN COLOUR

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TITLE GEOLOGICAL SECTION
Section 4

SCALE (A3) 1:2000 H (1:800V) FIG No. GM-F06 REV 1

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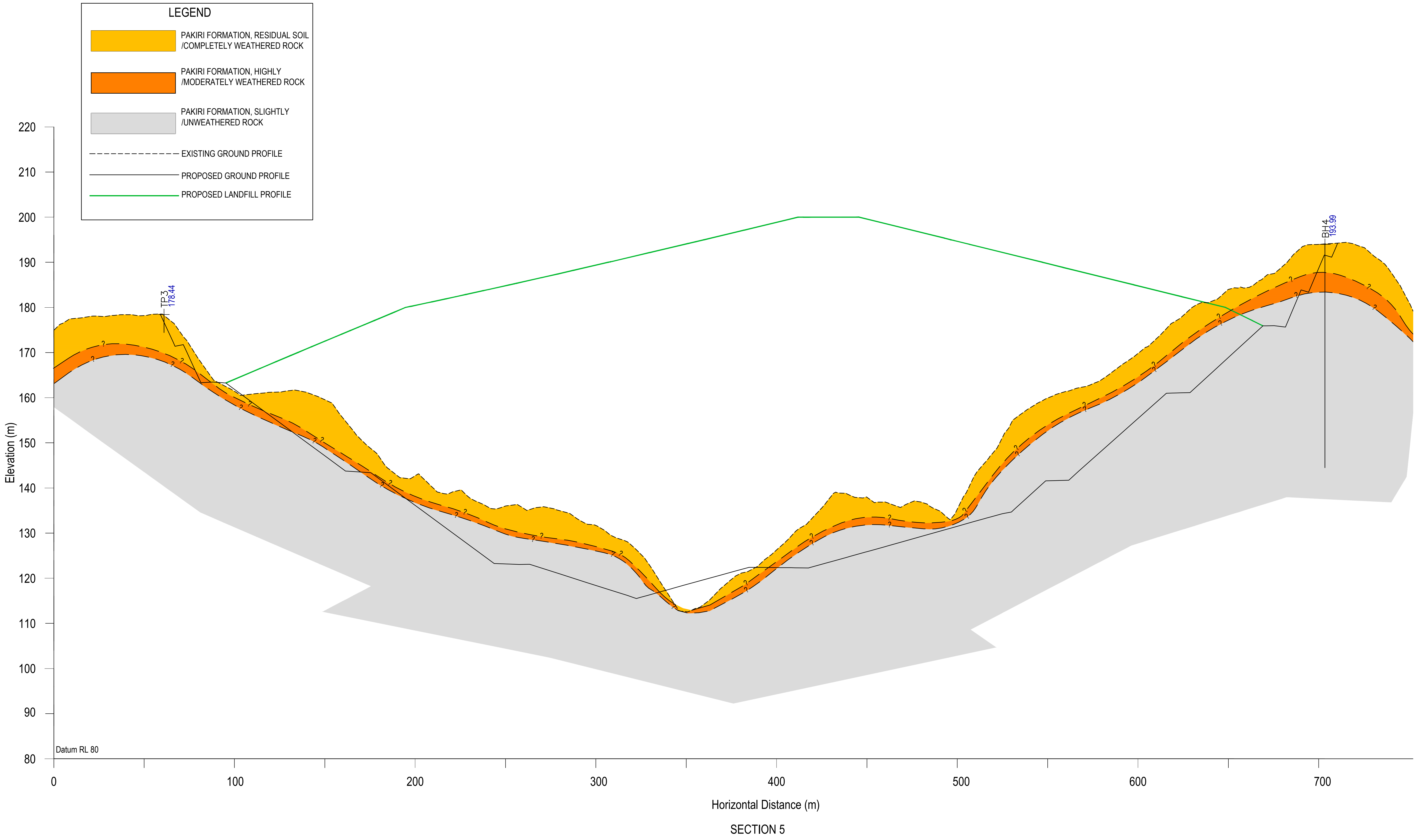

A3 SCALE 1:2000

0 20 40 60 80 100 (m)

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PROJECT	**AUCKLAND REGIONAL LANDFILL PROJECT**
TITLE	GEOLOGICAL SECTION Section 5
SCALE (A3)	1:2000 H (1:800V)
FIG No.	GM-F07
REV	1

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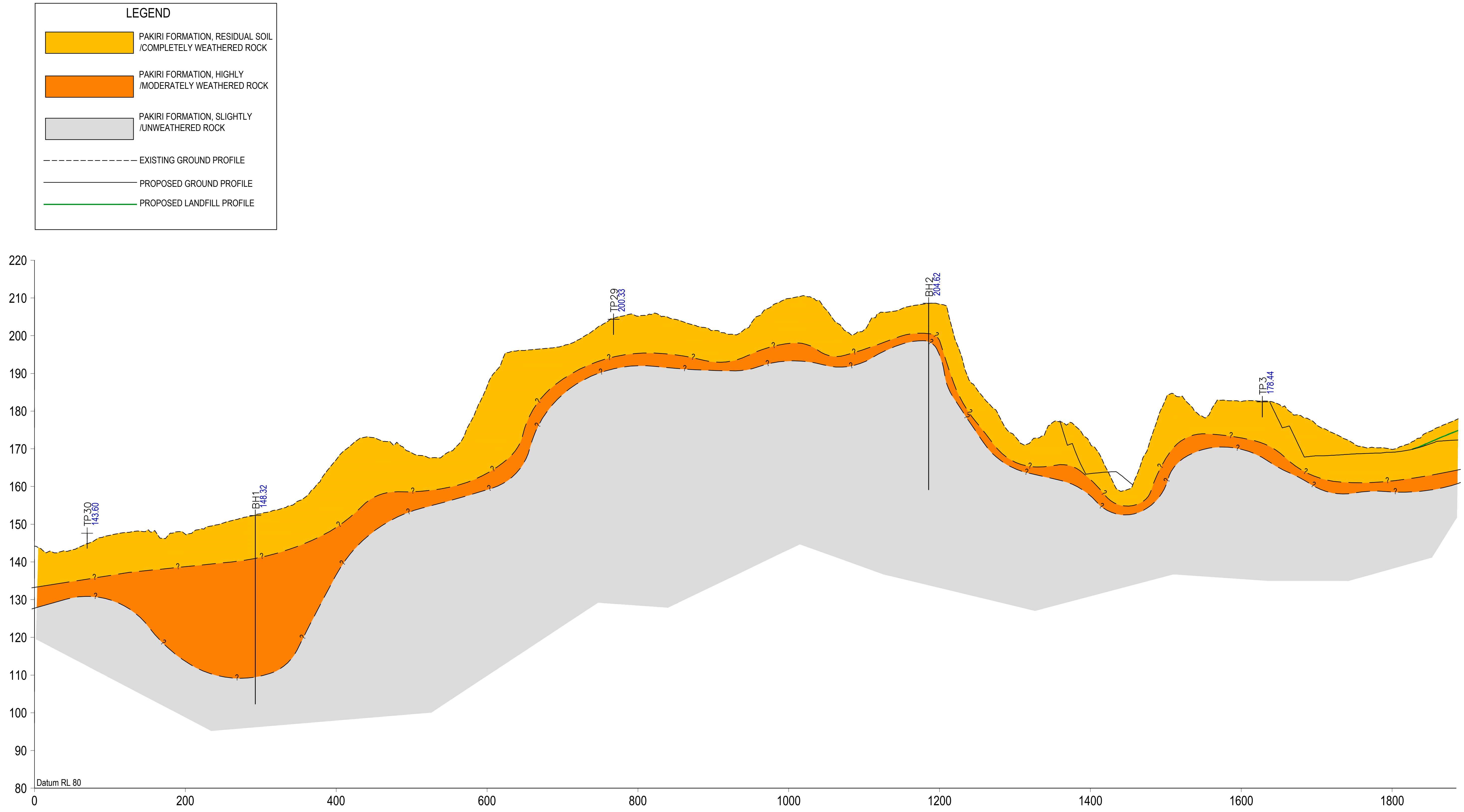

SECTION 6

A3 SCALE 1:5000

0 50 100 150 200 250 (m)

ORIGINAL IN COLOUR

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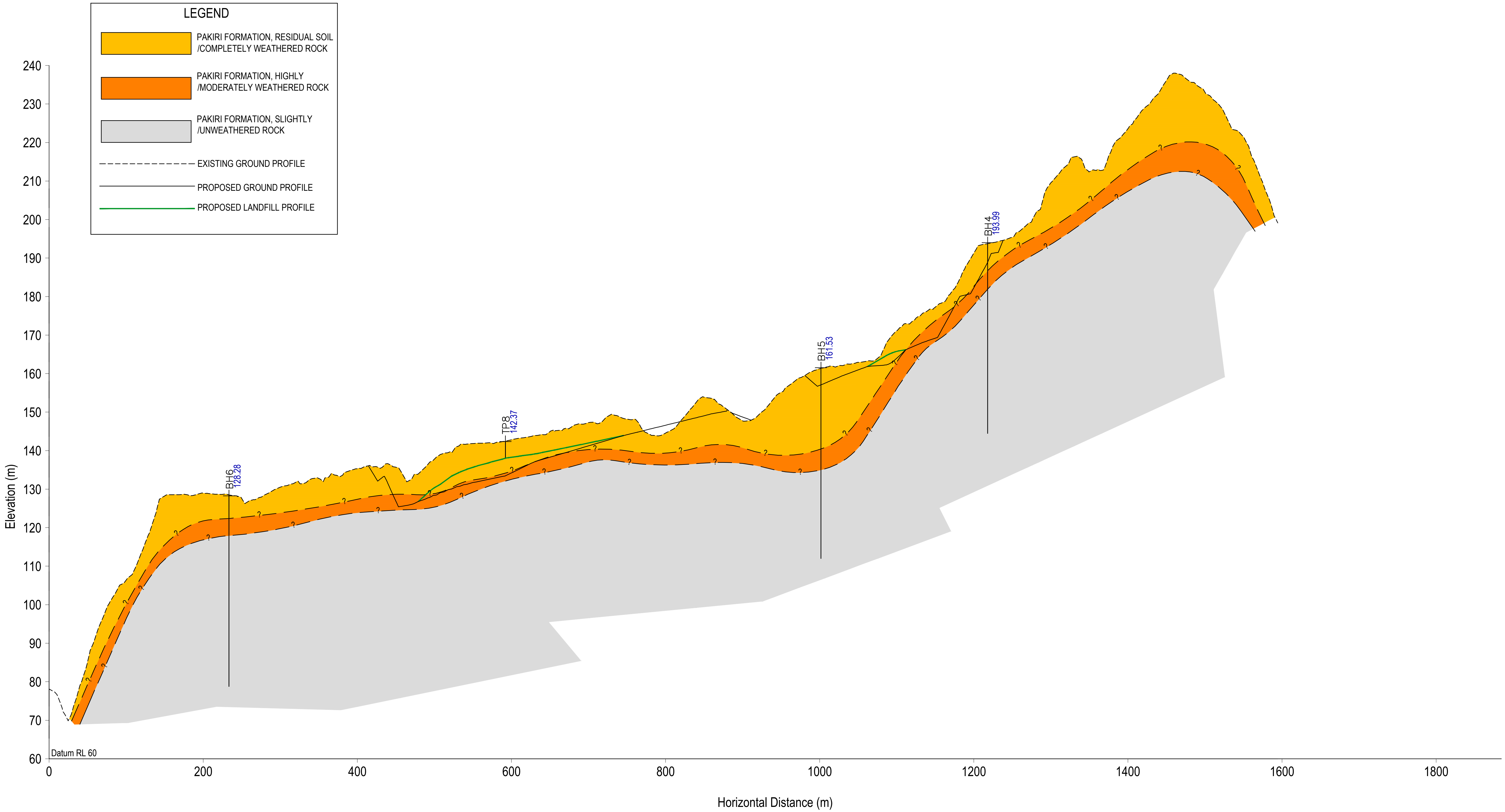
TITLE GEOLOGICAL SECTION

Section 6

SCALE (A3) 1:5000 H (1:1000V) FIG No. GM-F08 REV 1



SECTION 7

A3 SCALE 1:5000

0 50 100 150 200 250 (m)

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TITLE	GEOLOGICAL SECTION Section 7
SCALE (A3)	1:5000 H (1:1000V)
FIG No.	GM-F09
REV	1

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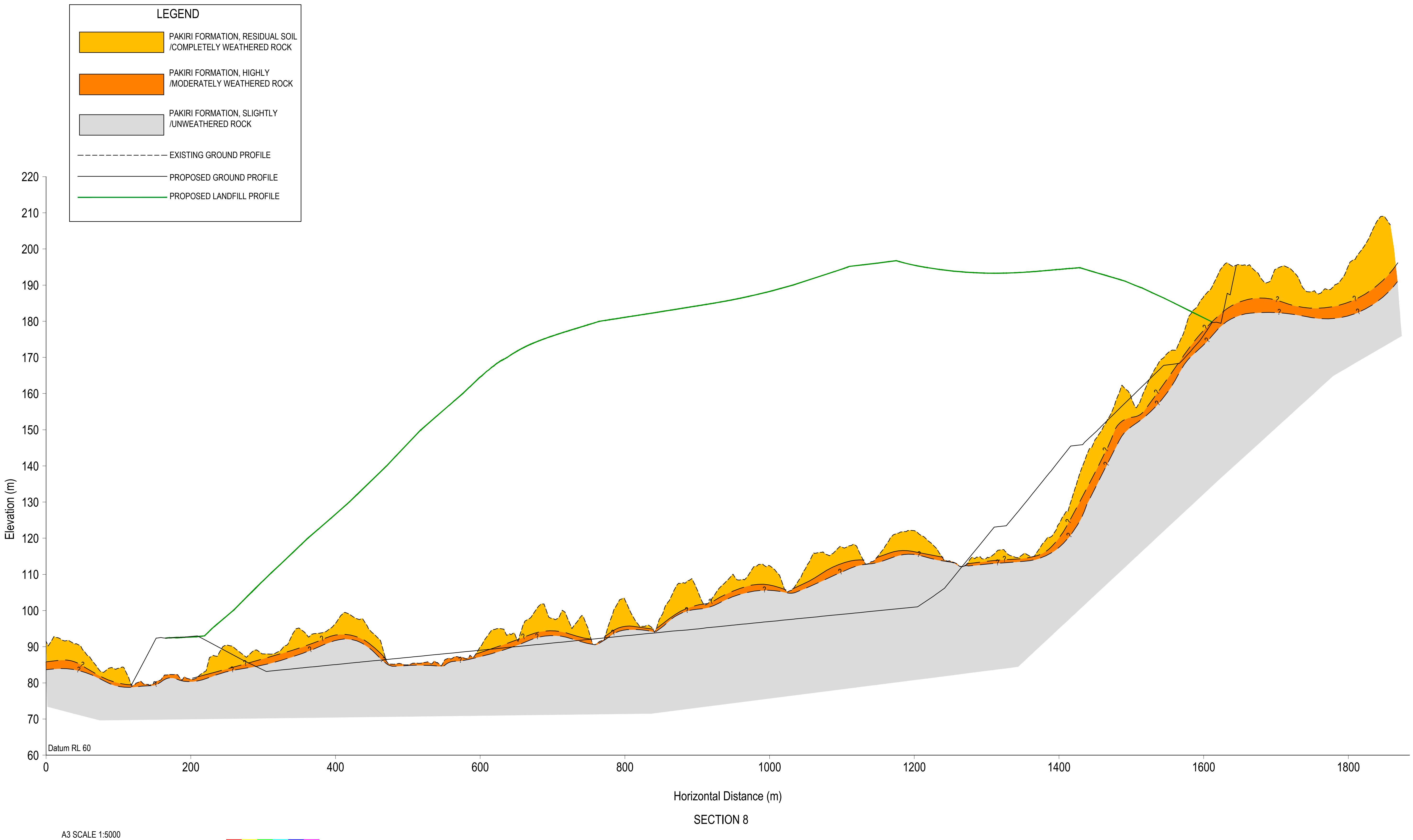

A3 SCALE 1:5000
0 50 100 150 200 250 (m)

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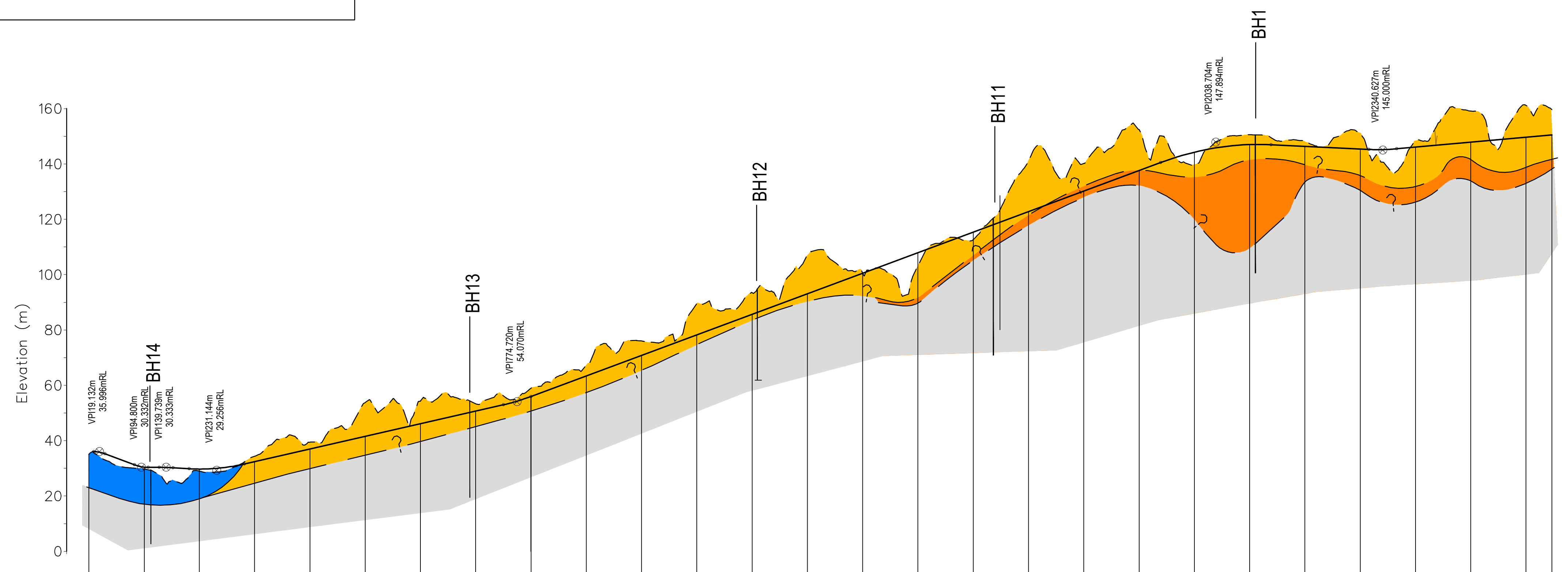
TITLE GEOLOGICAL SECTION
Section 8
SCALE (A3) 1:5000 H (1:1000V) FIG No. GM-F10 REV 1

LEGEND

- PAKIRI FORMATION, RESIDUAL SOIL /COMPLETELY WEATHERED ROCK
- PAKIRI FORMATION, HIGHLY /MODERATELY WEATHERED ROCK
- PAKIRI FORMATION, SLIGHTLY /UNWEATHERED ROCK
- TAURANGA GROUP ALLUVIAL SOIL
- EXISTING GROUND PROFILE



Datum RL(m) -10

CHAINAGE (m)	FINISHED ROAD LEVEL (m)	EXISTING GROUND RL (m)
0.00		35.03
100.00	30.41	29.77
200.00	29.72	29
300.00	32.40	34.34
400.00	36.96	39.45
500.00	41.53	53.72
600.00	46.09	54.31
700.00	50.66	53.34
800.00	55.95	58.94
900.00	63.37	67.26
1000.00	70.79	76.06
1100.00	78.21	89.54
1200.00	85.64	93.49
1300.00	93.06	106.88
1400.00	100.48	100.62
1500.00	107.91	103.21
1600.00	115.33	112.62
1700.00	122.75	141.07
1800.00	130.17	140.12
1900.00	137.60	152.42
2000.00	144.23	139.7
2100.00	146.99	150.6
2200.00	146.35	148.05
2300.00	145.39	151.17
2400.00	146.07	148.34
2500.00	147.88	159.1
2600.00	149.68	161.19
2646.98	150.53	159.64

LONGITUDINAL SECTION: ALIGNMENT RS03 D->RS03 D

Horizontal Scale 1:10000

Vertical Scale 1:2000

A3 SCALE 1: 10000

0 0.1 0.2 0.3 0.4 0.5 (km)

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PROJECT AUCKLAND REGIONAL LANDFILL PROJECT

TITLE GEOLOGICAL SECTION

Access Road - Longitudinal Section - Overall Road

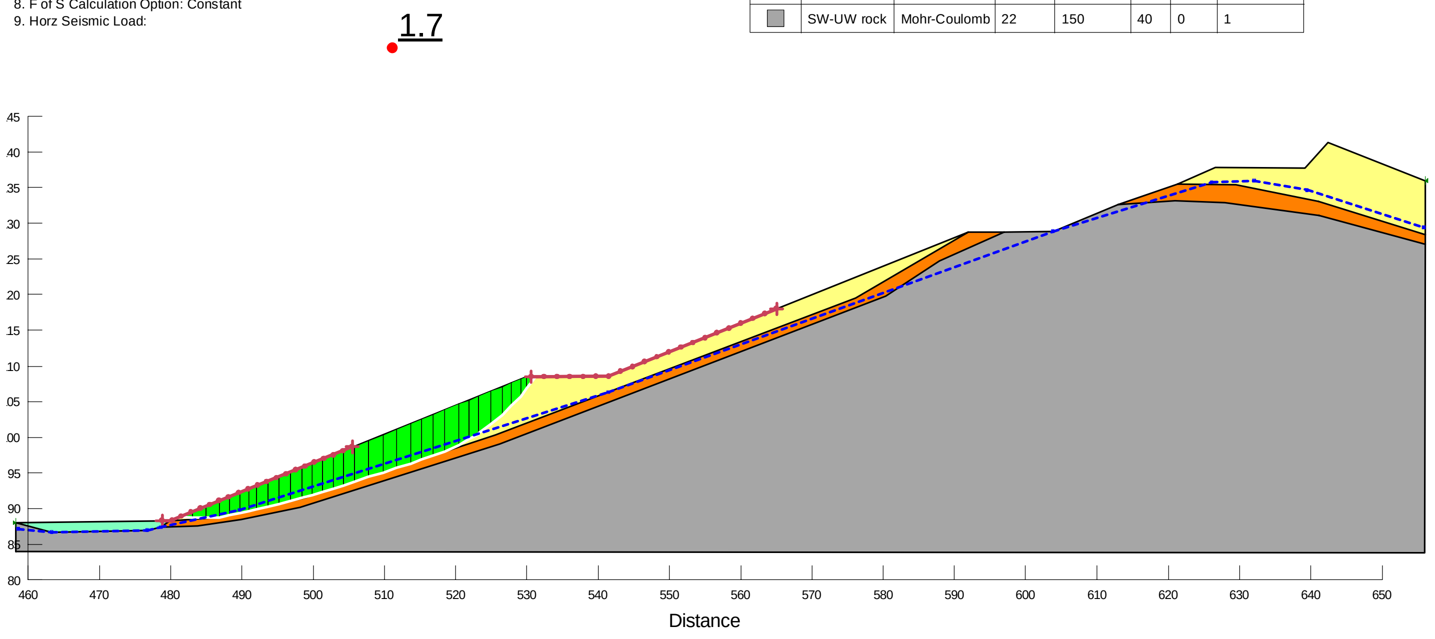
SCALE (A1) 1:10000 H (1:2000V) FIG No. GM-F11 REV 1

Appendix C : Slope stability results

- Section 2 Northern Slope
- Section 2 Southern Slope
- Section 4 Northern Slope
- Section 4 Southern Slope
- Landfill stability long section
- Access road CH1300 m 1V:1H
- Access road CH4500 m 1V:1H
- Access road CH1300 m 1V:1.5H
- Access road CH1300 m 1V:2H

Analysis Notes:
1. Name: Static long term Northern slope
2. Method: Morgenstern-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	HW MW rock	Mohr-Coulomb	21	60	35	0	1
	Residual Soil	Mohr-Coulomb	17	7	30	0	1
	Rock Fill	Mohr-Coulomb	19	0	40	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	1



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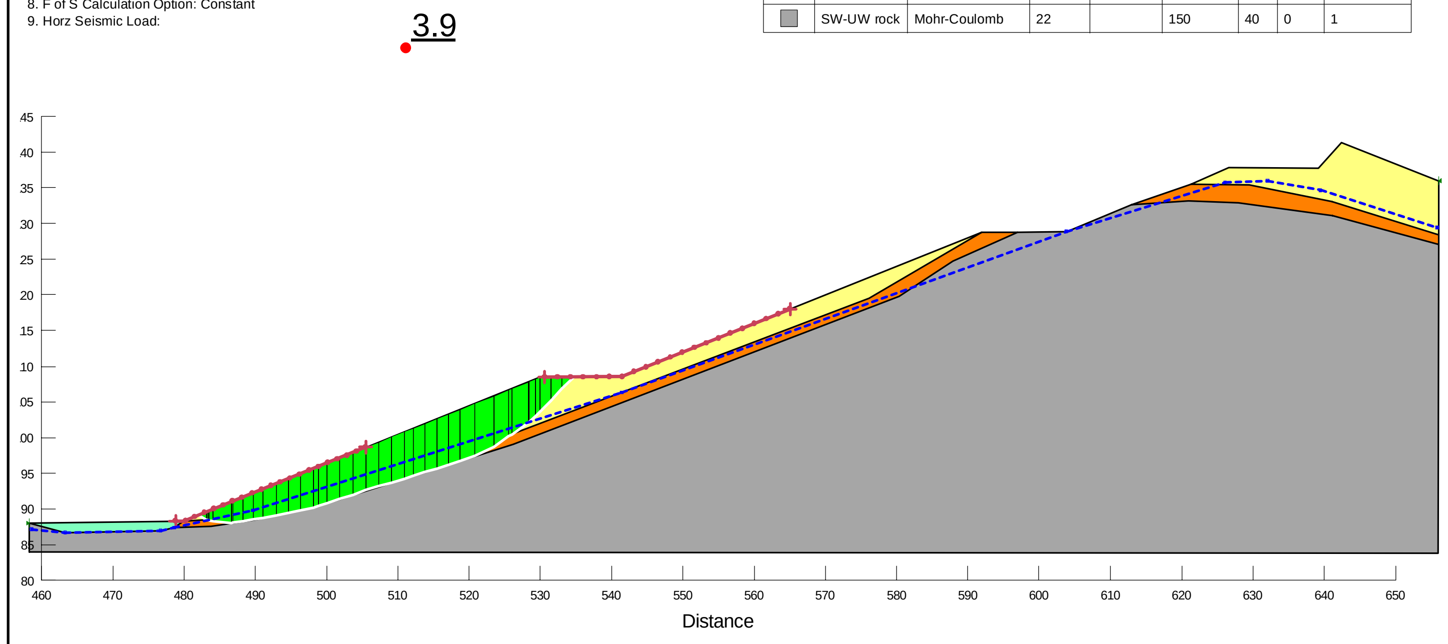
Title: Section 2 northern slope appendix.gsz		DWP Number:	
Analysis: Static long term Northern slope		Analysed by:	ALNA
Comments:	Scale: 1:750 @ A4	Checked by:	TIMC

Directory: C:\Users\alna\Documents\Polaris\

Analysis Notes:
1. Name: Static short term Northern slope
2. Method: Morgenstern-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	HW MW rock	Mohr-Coulomb	21		60	35	0	1
	Residual Soil	Undrained (Phi=0)	17	100				1
	Rock Fill	Mohr-Coulomb	19		0	40	0	1
	SW-UW rock	Mohr-Coulomb	22		150	40	0	1



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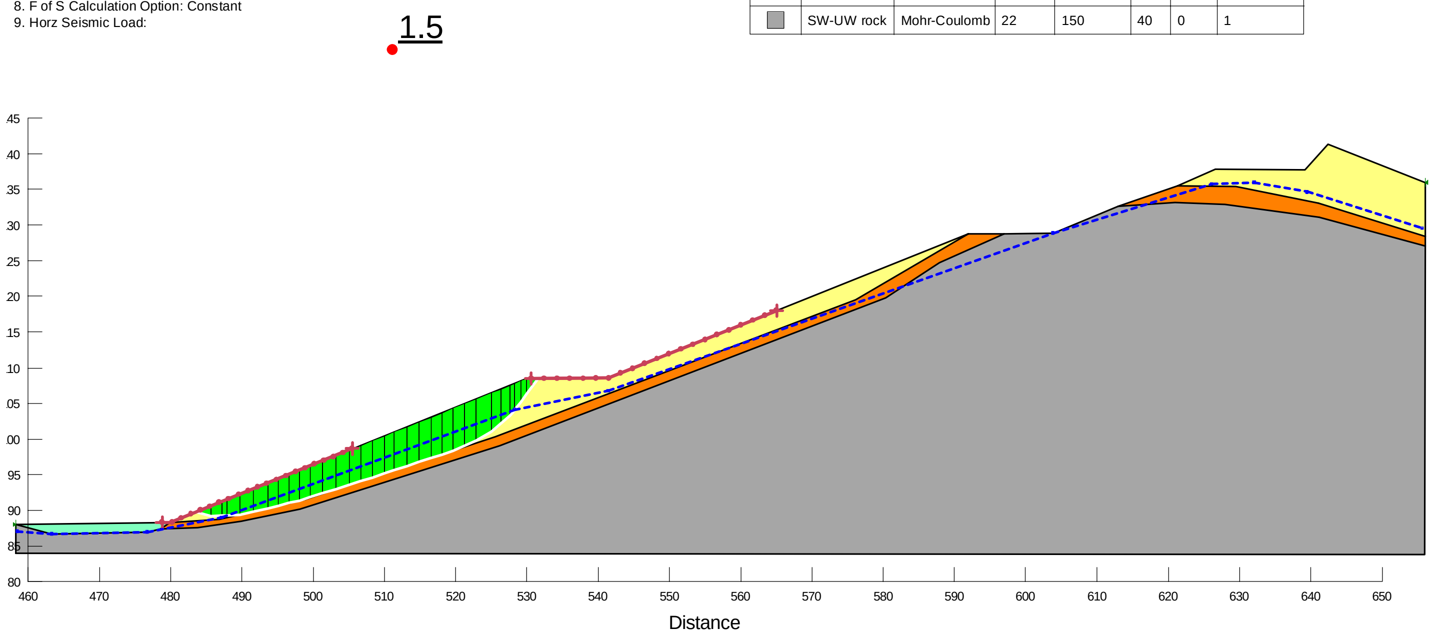
Title: Section 2 northern slope appendix.gsz		DWP Number:	
Analysis: Static short term Northern slope		Analysed by:	ALNA
Comments:	Scale: 1:750 @ A4	Checked by:	TIMC

Directory: C:\Users\alna\Documents\Polaris\

Analysis Notes:
1. Name: Static elevated groundwater Northern slope
2. Method: Morgenstern-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	HW MW rock	Mohr-Coulomb	21	60	35	0	1
	Residual Soil	Mohr-Coulomb	17	7	30	0	1
	Rock Fill	Mohr-Coulomb	19	0	40	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	1



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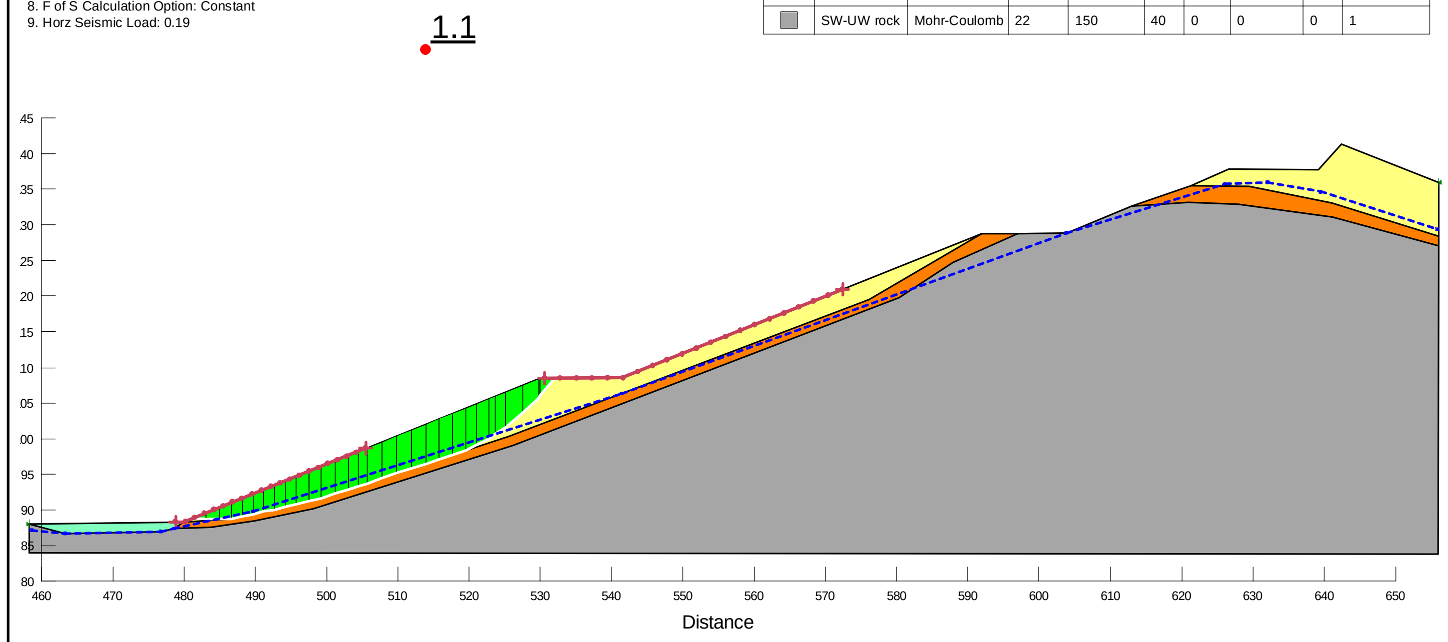
Title: Section 2 northern slope appendix.gsz		DWP Number:	
Analysis: Static elevated groundwater Northern slope		Analysed by:	ALNA
Comments:	Scale: 1:750 @ A4	Checked by:	TIMC

Directory: C:\Users\alna\Documents\Polaris\

Analysis Notes:
1. Name: Seismic ULS Northern slope
2. Method: Morgenstern-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load: 0.19

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	HW MW rock	Mohr-Coulomb	21	60	35	0	0	0	1
	Residual Soil	Mohr-Coulomb	17	7	30	0	0	0	1
	Rock Fill	Mohr-Coulomb	19	2	40	0	0	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	0	0	1



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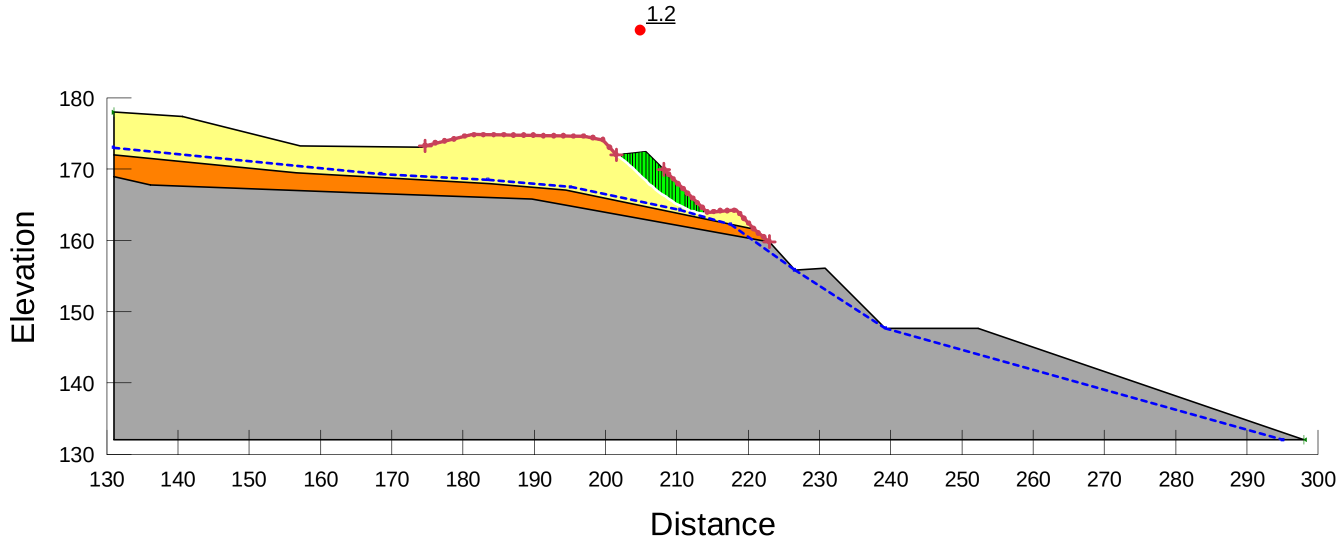
Title: Section 2 northern slope appendix.gsz		DWP Number:	
Analysis: Seismic ULS Northern slope		Analysed by:	ALNA
Comments:	Scale: 1:750 @ A4	Checked by:	TIMC

Directory: C:\Users\alna\Documents\Polaris\

Analysis Notes:
1. Name: Static long term Southern slope
2. Method: Morgenstern-Price
3. Direction of movement: Left to Right
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	HW MW rock	Mohr-Coulomb	21	60	35	0	1
	Residual Soil	Mohr-Coulomb	17	7	30	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	1



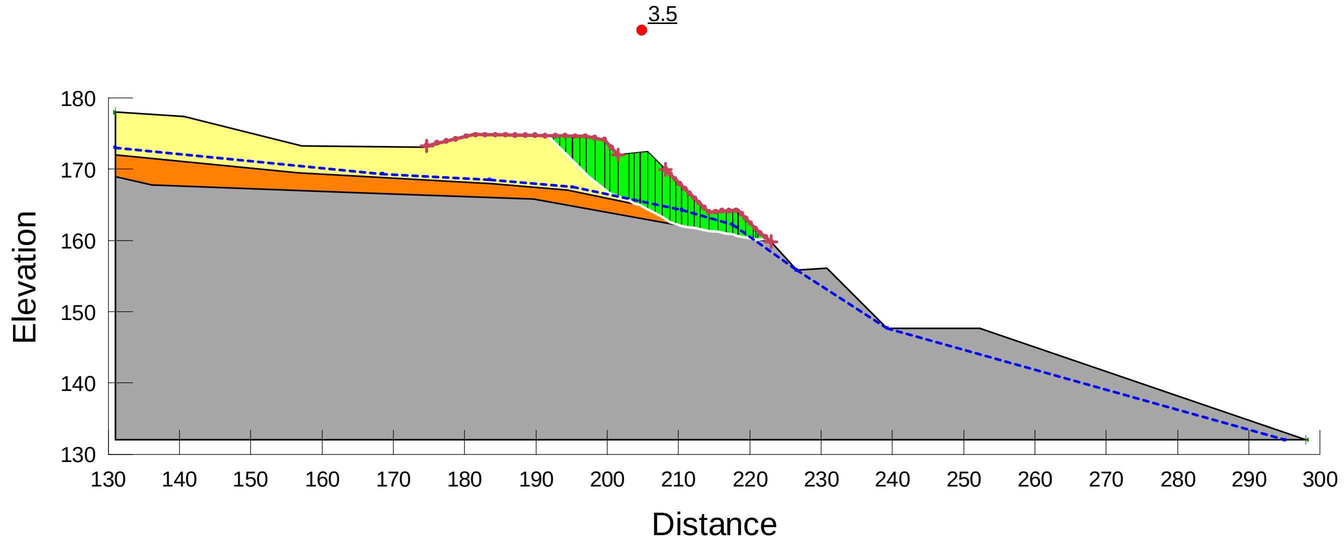
Tonkin+Taylor
Waste Management

Title: Section 2 southern slope appendix.gsz		DWP Number:
Analysis: Static long term Southern slope		Analysed by: ALNA
Comments:	Scale: 1:750 @ A4	Checked by: TIMC

Directory: C:\Users\alna\Documents\Polaris\

Analysis Notes:
1. Name: Static short term Southern slope
2. Method: Morgenstern-Price
3. Direction of movement: Left to Right
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	HW MW rock	Mohr-Coulomb	21		60	35	0	1
	Residual Soil	Undrained (Phi=0)	17	100				1
	SW-UW rock	Mohr-Coulomb	22		150	40	0	1



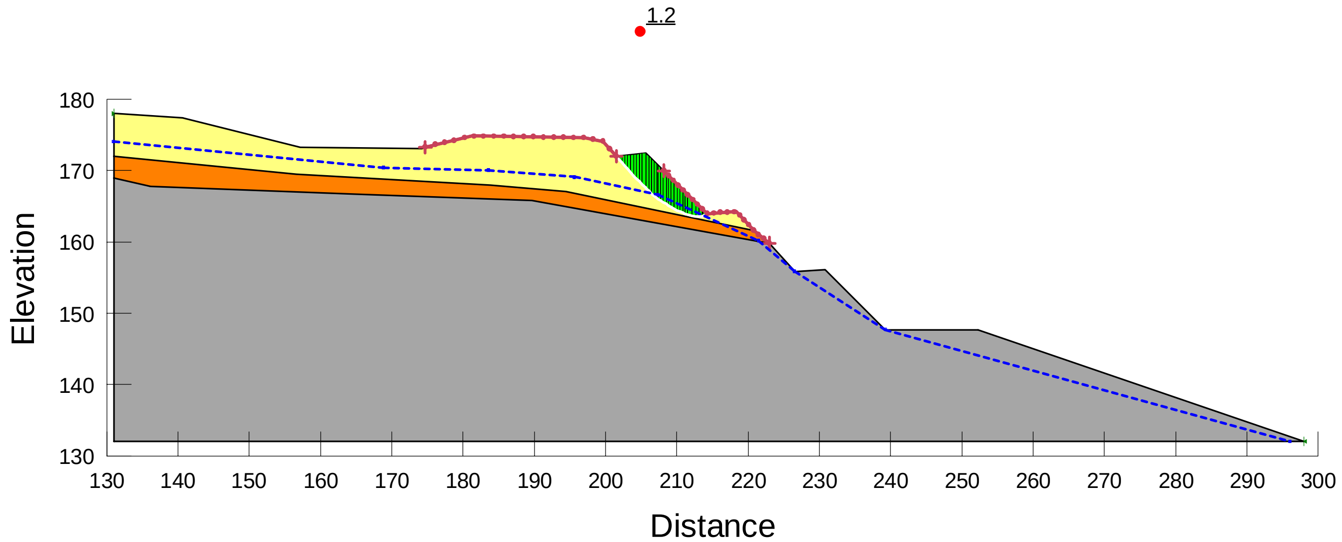
Tonkin+Taylor
Waste Management

Title: Section 2 southern slope appendix.gsz		DWP Number:
Analysis: Static short term Southern slope		Analysed by: ALNA
Comments:	Scale: 1:750 @ A4	Checked by: TIMC

Directory: C:\Users\alna\Documents\Polaris\

Analysis Notes:
1. Name: Static elevated groundwater Southern slope
2. Method: Morgenstern-Price
3. Direction of movement: Left to Right
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	HW MW rock	Mohr-Coulomb	21	60	35	0	1
	Residual Soil	Mohr-Coulomb	17	7	30	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	1



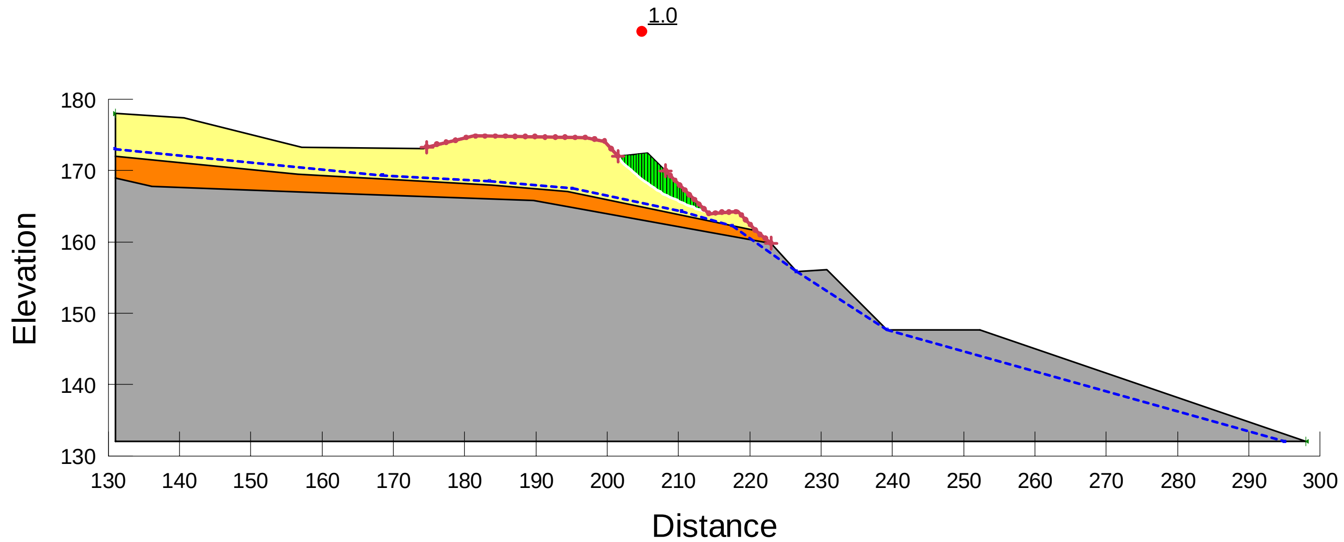
Tonkin+Taylor
Waste Management

Title: Section 2 southern slope appendix.gsz		DWP Number:
Analysis: Static elevated groundwater Southern slope		Analysed by: ALNA
Comments:	Scale: 1:750 @ A4	Checked by: TIMC

Directory: C:\Users\alna\Documents\Polaris\

Analysis Notes:
1. Name: Seismic ULS Southern slope
2. Method: Morgenstern-Price
3. Direction of movement: Left to Right
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load: 0.19

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	HW MW rock	Mohr-Coulomb	21	60	35	0	0	0	1
	Residual Soil	Mohr-Coulomb	17	7	30	0	0	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	0	0	1



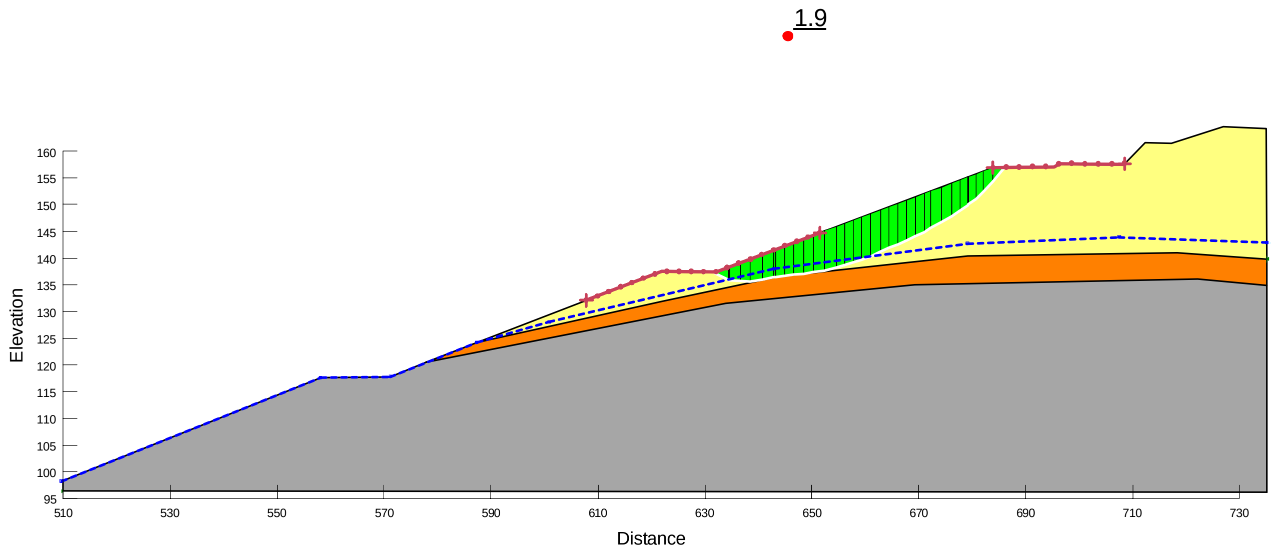
Tonkin+Taylor
Waste Management

Title: Section 2 southern slope appendix.gsz		DWP Number:
Analysis: Seismic ULS Southern slope		Analysed by: ALNA
Comments:	Scale: 1:750 @ A4	Checked by: TIMC

Directory: C:\Users\alna\Documents\Polaris\

Analysis Notes:
1. Name: Static (long term) Northern Slope
2. Method: Morgenstern-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	HW-MW rock	Mohr-Coulomb	21	60	35	0	1
	Residual Soil	Mohr-Coulomb	17	7	30	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	1

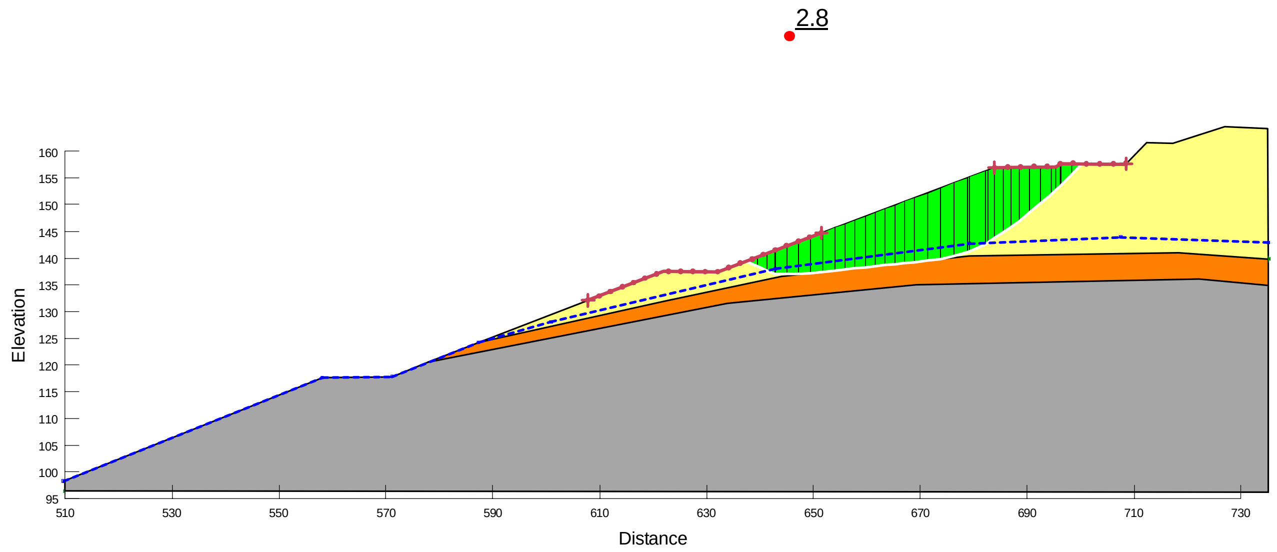


Tonkin+Taylor	Title: Section 4 Northern slope.gsz	DWP Number:
Waste Management	Analysis: Static (long term) Northern Slope	Analysed by: ALNA
	Comments: Scale: 1:1,000 @ A4	Checked by: TIMC

Directory: C:\Users\alna\Documents\Polaris\Appendix C\

Analysis Notes:
1. Name: Static (short term) Northern Slope
2. Method: Morgenstern-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	HW-MW rock	Mohr-Coulomb	21		60	35	0	1
	Residual Soil	Undrained (Phi=0)	17	100				1
	SW-UW rock	Mohr-Coulomb	22		150	40	0	1

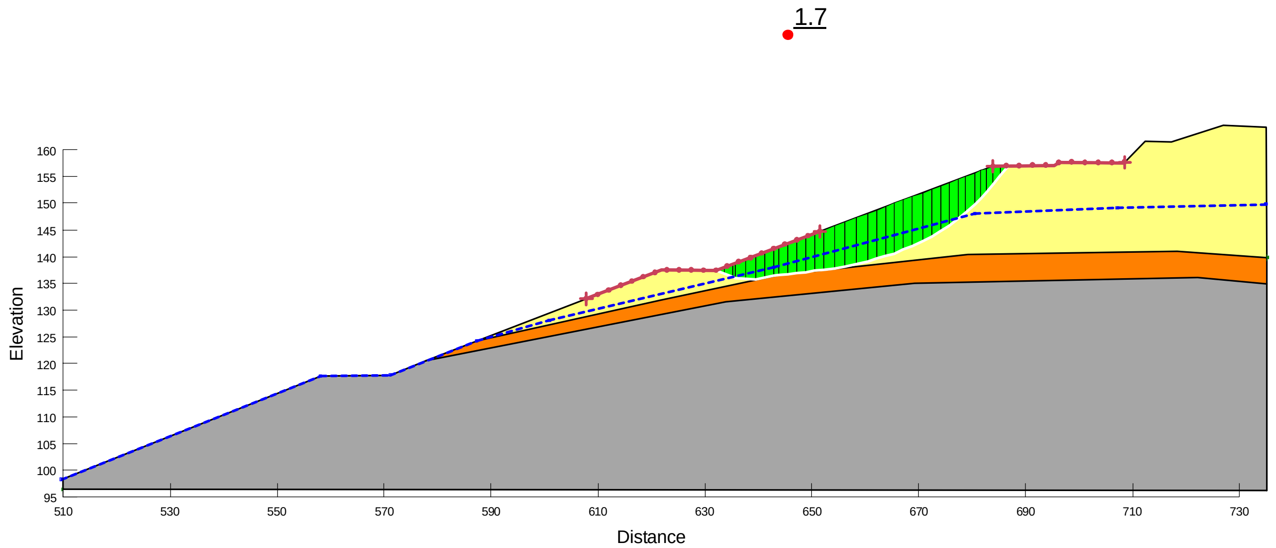


Tonkin+Taylor	Title: Section 4 Northern slope.gsz	DWP Number:
Waste Management	Analysis: Static (short term) Northern Slope	Analysed by: ALNA
	Comments: Scale: 1:1,000 @ A4	Checked by: TIMC

Directory: C:\Users\alna\Documents\Polaris\Appendix C\

Analysis Notes:
1. Name: Static (elevated groundwater) Northern Slope
2. Method: Morgenstern-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	HW-MW rock	Mohr-Coulomb	21	60	35	0	1
	Residual Soil	Mohr-Coulomb	17	7	30	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	1

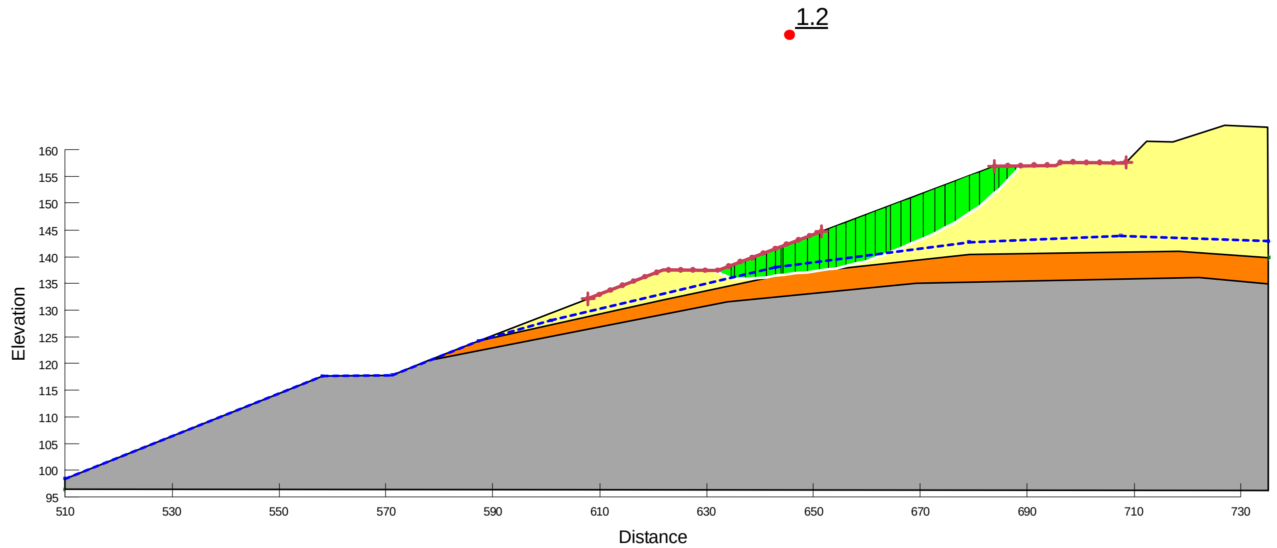


Tonkin+Taylor	Title: Section 4 Northern slope.gsz	DWP Number:
Waste Management	Analysis: Static (elevated groundwater) Northern Slope	Analysed by: ALNA
	Comments: Scale: 1:1,000 @ A4	Checked by: TIMC

Directory: C:\Users\alna\Documents\Polaris\Appendix C\

Analysis Notes:
1. Name: Seismic ULS Northern Slope
2. Method: Morgenstern-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load: 0.19

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	HW-MW rock	Mohr-Coulomb	21	60	35	0	0	0	1
	Residual Soil	Mohr-Coulomb	17	7	30	0	0	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	0	0	1

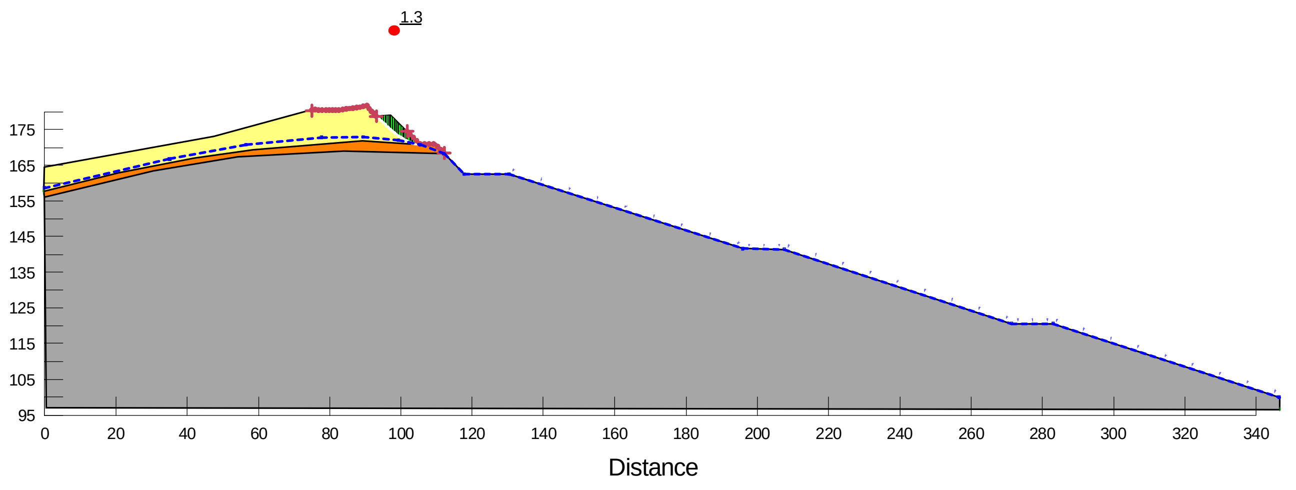


Tonkin+Taylor	Title: Section 4 Northern slope.gsz	DWP Number:
Waste Management	Analysis: Seismic ULS Northern Slope	Analysed by: ALNA
	Comments: Scale: 1:1,000 @ A4	Checked by: TIMC

Directory: C:\Users\alna\Documents\Polaris\Appendix C\

Analysis Notes:
1. Name: Static (long term) Southern Slope
2. Method: Morgenstern-Price
3. Direction of movement: Left to Right
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	HW-MW rock	Mohr-Coulomb	21	60	35	0	1
	Residual Soil	Mohr-Coulomb	17	7	30	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	1

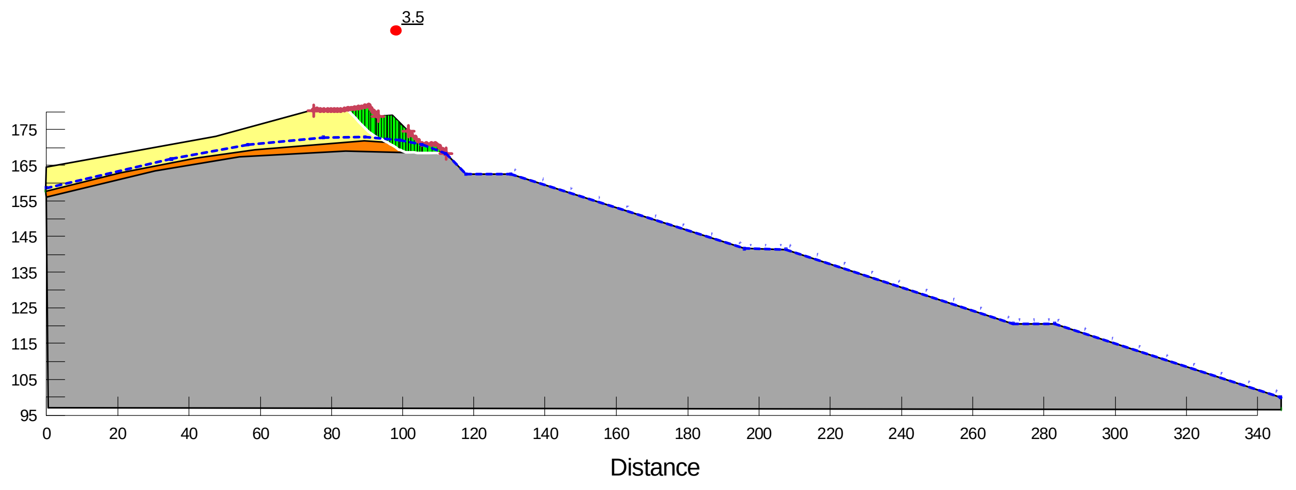


Title: Section 4 Southern slope.gsz		DWP Number:
Analysis: Static (long term) Southern Slope		Analysed by: ALNA
Comments:	Scale: 1:1,500 @ A4	Checked by: TIMC

Directory: C:\Users\alna\Documents\Polaris\Appendix C\

Analysis Notes:
1. Name: Static (short term) Southern Slope
2. Method: Morgenstern-Price
3. Direction of movement: Left to Right
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	HW-MW rock	Mohr-Coulomb	21		60	35	0	1
	Residual Soil	Undrained (Phi=0)	17	100				1
	SW-UW rock	Mohr-Coulomb	22		150	40	0	1

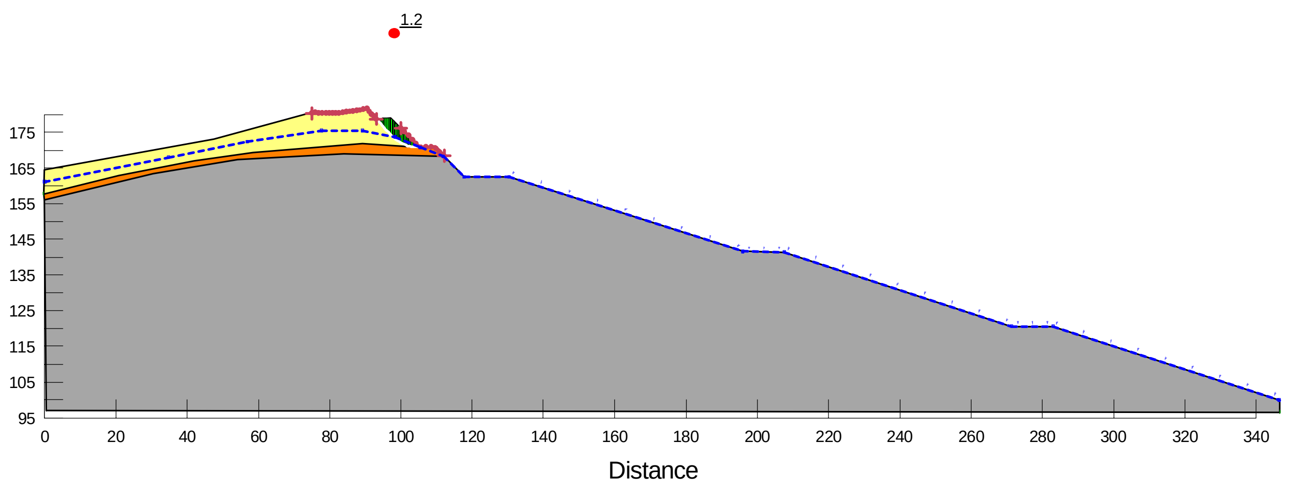


Title: Section 4 Southern slope.gsz		DWP Number:
Analysis: Static (short term) Southern Slope		Analysed by: ALNA
Comments:	Scale: 1:1,500 @ A4	Checked by: TIMC

Directory: C:\Users\alna\Documents\Polaris\Appendix C\

Analysis Notes:
1. Name: Static (elevated groundwater) Southern Slope
2. Method: Morgenstern-Price
3. Direction of movement: Left to Right
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	HW-MW rock	Mohr-Coulomb	21	60	35	0	1
	Residual Soil	Mohr-Coulomb	17	7	30	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	1

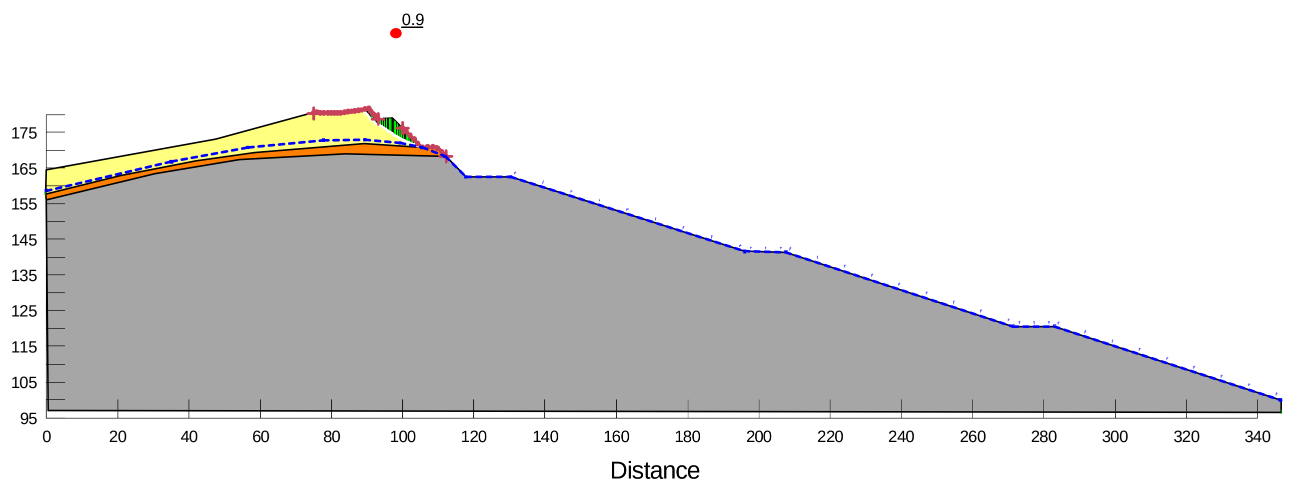


Title: Section 4 Southern slope.gsz		DWP Number:
Analysis: Static (elevated groundwater) Southern Slope		Analysed by: ALNA
Comments:	Scale: 1:1,500 @ A4	Checked by: TIMC

Directory: C:\Users\alna\Documents\Polaris\Appendix C\

Analysis Notes:
1. Name: Seismic ULS Southern Slope
2. Method: Morgenstern-Price
3. Direction of movement: Left to Right
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load: 0.19

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	HW-MW rock	Mohr-Coulomb	21	60	35	0	0	0	1
	Residual Soil	Mohr-Coulomb	17	7	30	0	0	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	0	0	1

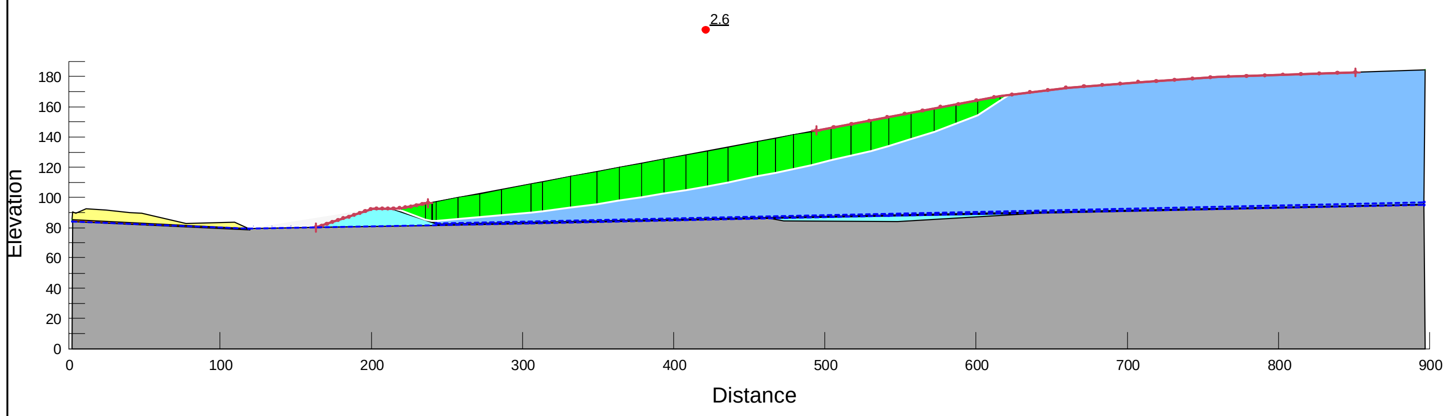


Title: Section 4 Southern slope.gsz		DWP Number:
Analysis: Seismic ULS Southern Slope		Analysed by: ALNA
Comments:	Scale: 1:1,500 @ A4	Checked by: TIMC

Directory: C:\Users\alna\Documents\Polaris\Appendix C\

Analysis Notes:
1. Name: Static long term
2. Method: Morgenstern-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	HW/MW rock	Mohr-Coulomb	21	60	35	0	1
	Liner interface (peak)	Mohr-Coulomb	17	0	25	0	1
	Liner interface (post peak)	Mohr-Coulomb	17	0	16	0	2
	Refuse fill	Mohr-Coulomb	14	5	25	0	2
	Residual Soil	Mohr-Coulomb	17	7	30	0	1
	Structural 120 fill	Mohr-Coulomb	18	6	29	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	1

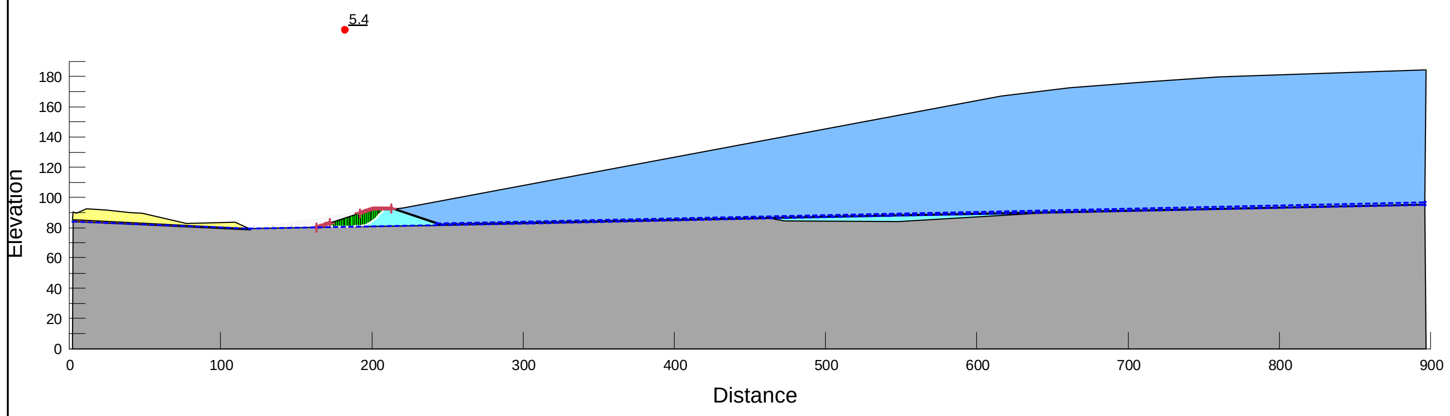


Tonkin+Taylor / Waste Management	Title: Landfill Stability longsection.gsz	DWP Number:
	Analysis: Static long term	Analysed by: ALNA
	Comments: Scale: 1:2,500 @ A3	Checked by:

Directory: C:\Users\alna\Documents\Polaris\slope W\

Analysis Notes:
1. Name: Static short term
2. Method: Morgenstern-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion (kPa)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	HW/MW rock	Mohr-Coulomb	21		60	35	0	1
	Liner interface (peak)	Mohr-Coulomb	17		0	25	0	1
	Refuse fill	Mohr-Coulomb	14		5	25	0	2
	Residual Soil	Mohr-Coulomb	17		7	30	0	1
	Structural 120 fill	Undrained (Phi=0)	18	120				1
	SW-UW rock	Mohr-Coulomb	22		150	40	0	1

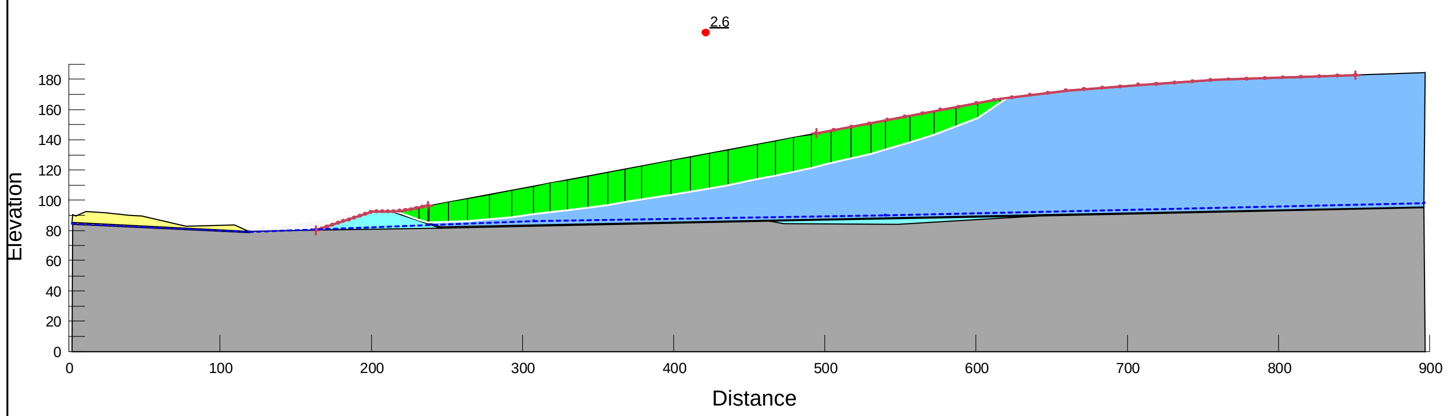


Tonkin+Taylor / Waste Management	Title: Landfill Stability longsection.gsz	DWP Number:
	Analysis: Static short term	Analysed by: ALNA
	Comments: Scale: 1:2,500 @ A3	Checked by:

Directory: C:\Users\alna\Documents\Polaris\slope W\

Analysis Notes:
1. Name: Static elevated groundwater
2. Method: Morgenstern-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	HW/MW rock	Mohr-Coulomb	21	60	35	0	1
	Liner interface (peak)	Mohr-Coulomb	17	0	25	0	1
	Liner interface (post peak)	Mohr-Coulomb	17	0	16	0	1
	Refuse fill	Mohr-Coulomb	14	5	25	0	1
	Residual Soil	Mohr-Coulomb	17	7	30	0	1
	Structural 120 fill	Mohr-Coulomb	18	6	29	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	1

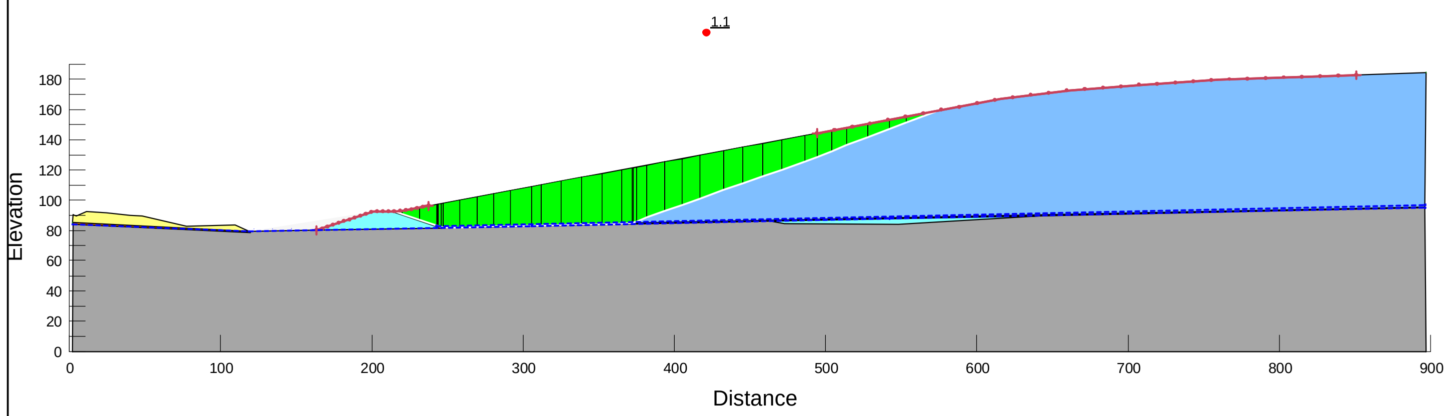


Tonkin+Taylor / Waste Management	Title: Landfill Stability longsection.gsz	DWP Number:
	Analysis: Static elevated groundwater	Analysed by: ALNA
	Comments: Scale: 1:2,500 @ A3	Checked by:

Directory: C:\Users\alna\Documents\Polaris\slope W\

Analysis Notes:
1. Name: Seismic ULS
2. Method: Morgenstern-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load: 0.19

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	HW/MW rock	Mohr-Coulomb	21	60	35	0	0	0	1
	Liner interface (post peak)	Mohr-Coulomb	17	0	16	0	0	0	2
	Refuse fill	Mohr-Coulomb	14	5	25	0	0	0	2
	Residual Soil	Mohr-Coulomb	17	7	30	0	0	0	1
	Structural 120 fill	Mohr-Coulomb	18	6	29	0	0	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	0	0	1



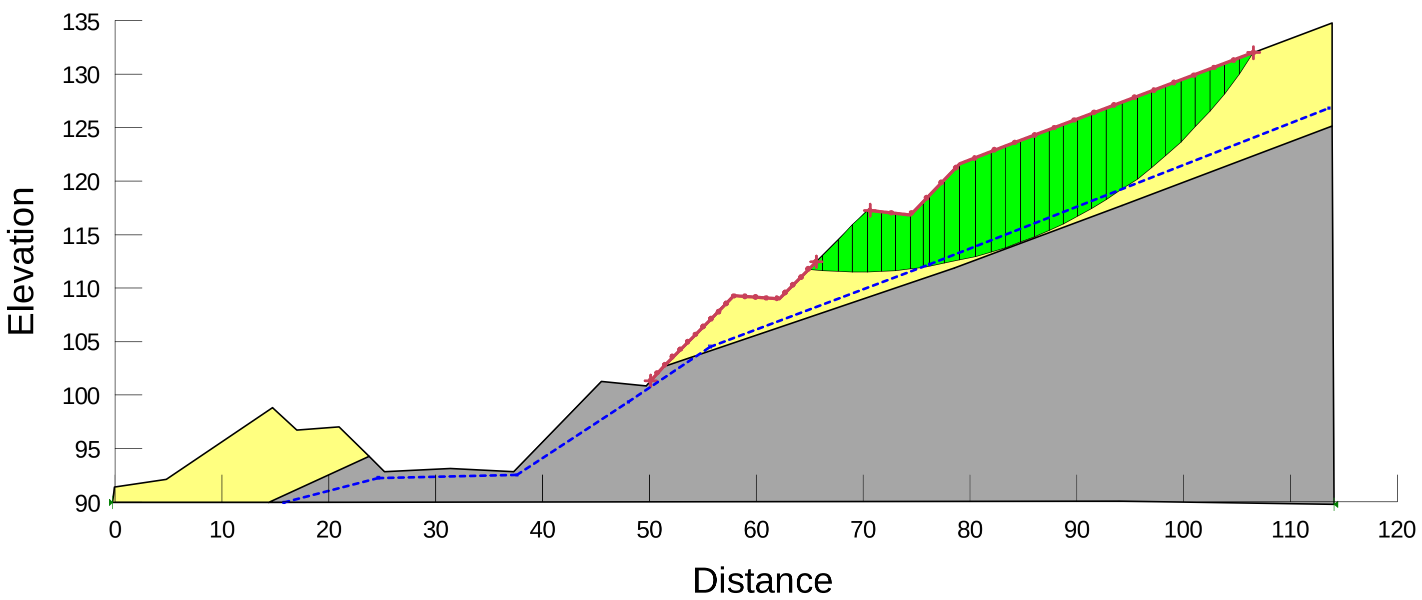
Tonkin+Taylor / Waste Management	Title: Landfill Stability longsection.gsz	DWP Number:
	Analysis: Seismic ULS	Analysed by: ALNA
	Comments: Scale: 1:2,500 @ A3	Checked by:

Directory: C:\Users\alna\Documents\Polaris\slope W\

Analysis Notes:
1. Name: Static long term
2. Method: Morgenstern-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Residual Soil	Mohr-Coulomb	17	7	30	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	1

1.6



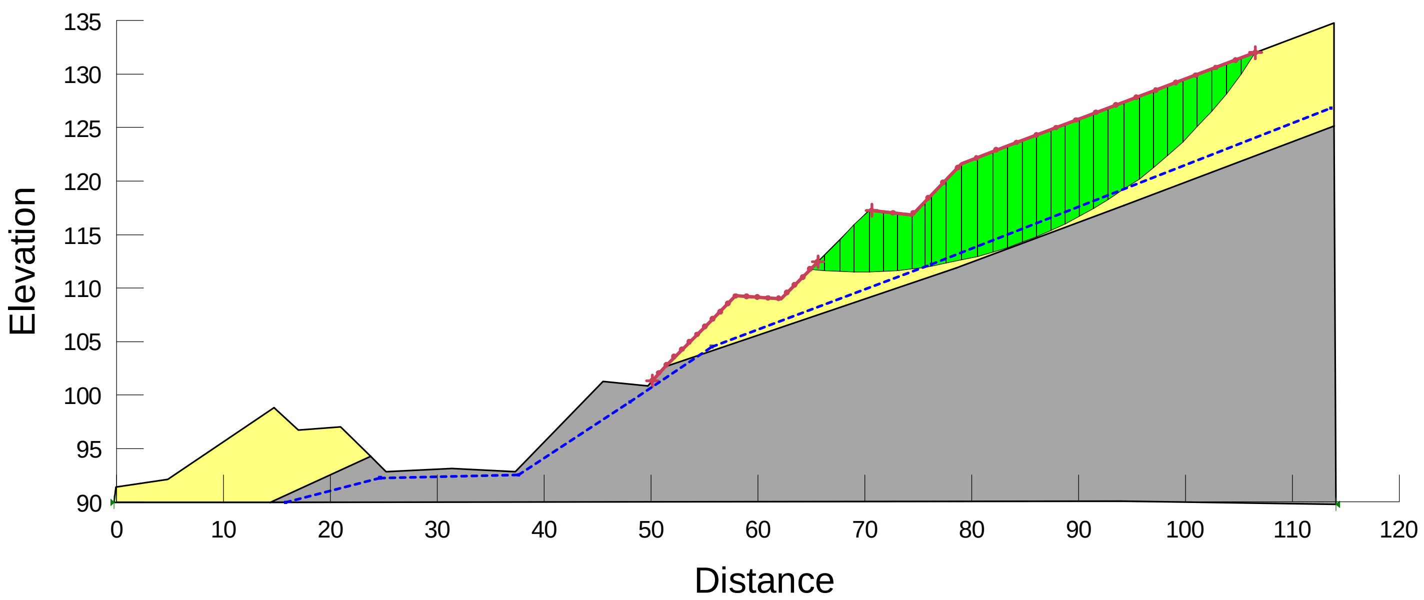
Tonkin+Taylor	Title: Access Road CH1300 cut slope stability 1H to 1V.gsz	DWP Number:
Waste Management	Analysis: Static long term	Analysed by: ALNA
	Comments: / Scale: 1:500 @ A4	Checked by: TIMC

Directory: C:\Users\alna\Documents\Polaris\Appendix C\

Analysis Notes:
1. Name: Static short term
2. Method: Morgenstern-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Cohesion (kPa)	Piezometric Line
	Residual Soil	Undrained (Phi=0)	17				100	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0		1

2.8



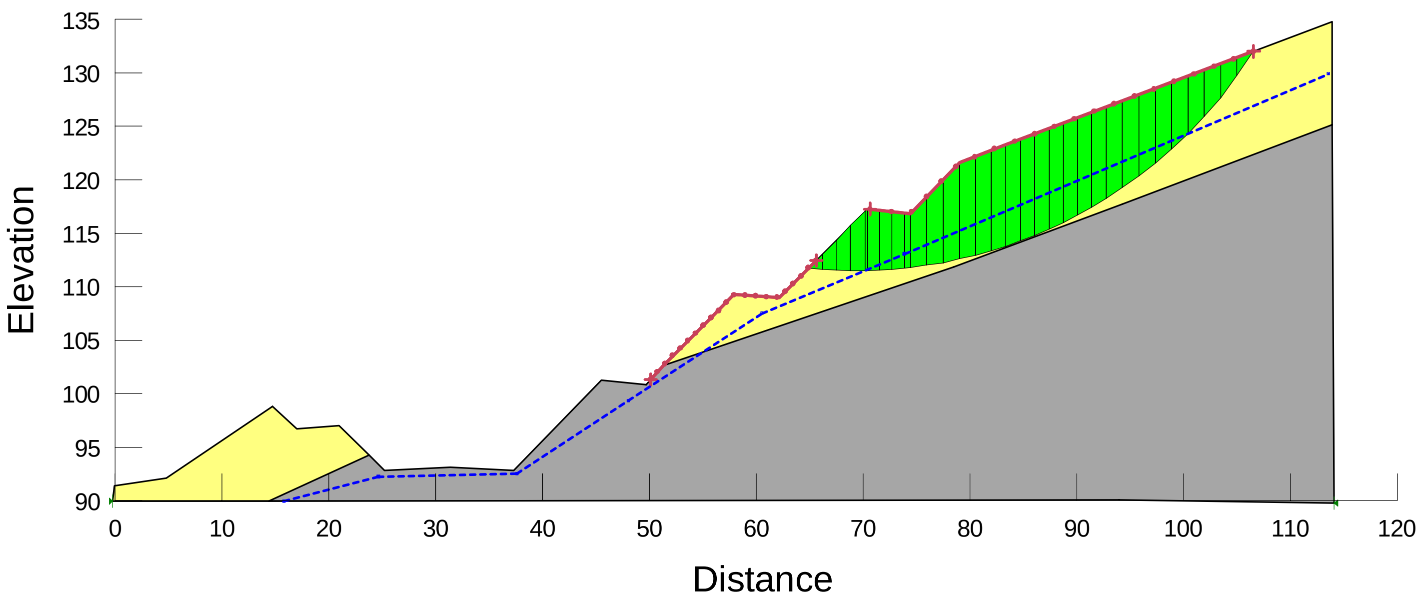
Tonkin+Taylor	Title: Access Road CH1300 cut slope stability 1H to 1V.gsz	DWP Number:
Waste Management	Analysis: Static short term	Analysed by: ALNA
	Comments: / Scale: 1:500 @ A4	Checked by: TIMC

Directory: C:\Users\alna\Documents\Polaris\Appendix C\

Analysis Notes:
1. Name: Static elevated groundwater
2. Method: Morgenstern-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Residual Soil	Mohr-Coulomb	17	7	30	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	1

1.4



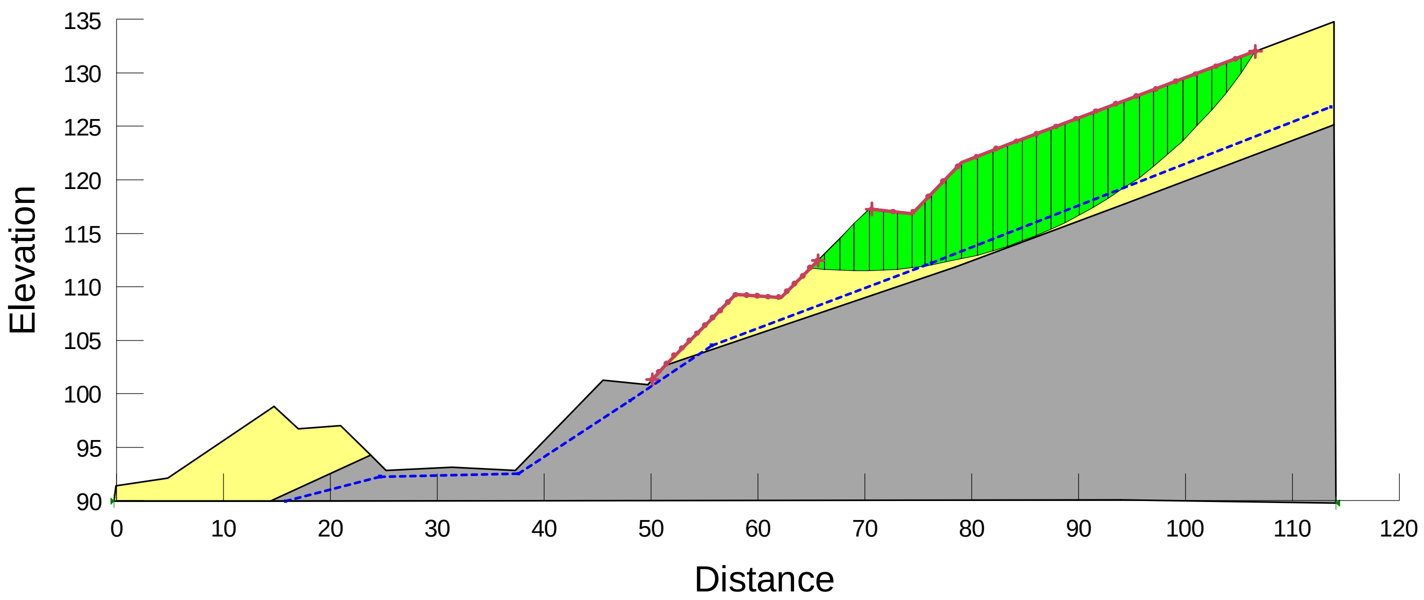
Tonkin+Taylor	Title: Access Road CH1300 cut slope stability 1H to 1V.gsz	DWP Number:
Waste Management	Analysis: Static elevated groundwater	Analysed by: ALNA
	Comments: / Scale: 1:500 @ A4	Checked by: TIMC

Directory: C:\Users\alna\Documents\Polaris\Appendix C\

Analysis Notes:
1. Name: Seismic ULS
2. Method: Morgenstern-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load: 0.19

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Residual Soil	Mohr-Coulomb	17	7	30	0	0	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	0	0	1

1.1

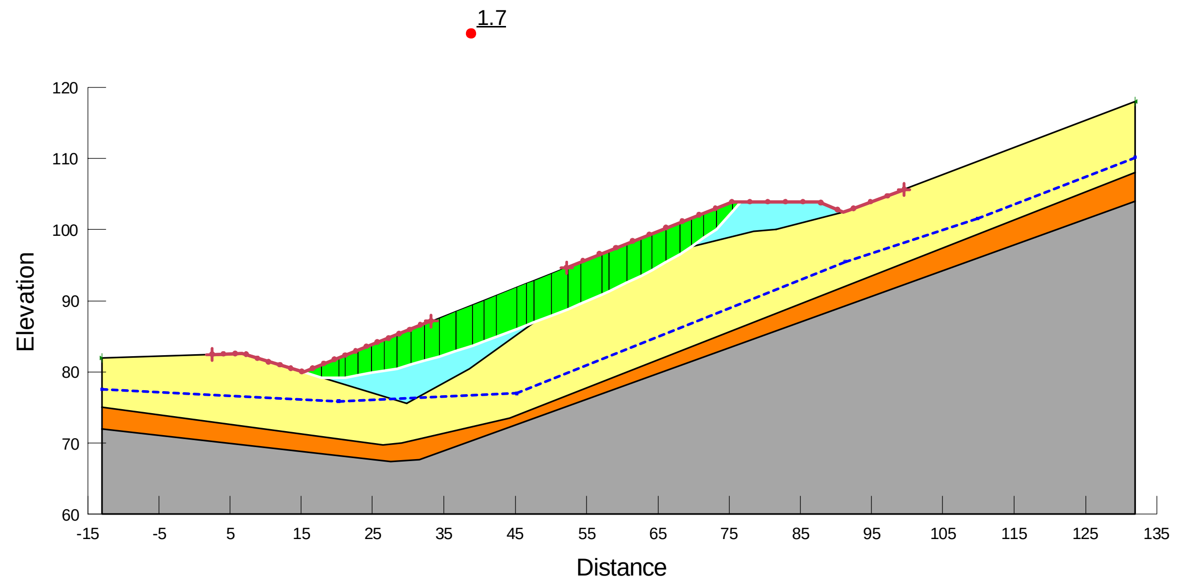


Tonkin+Taylor	Title: Access Road CH1300 cut slope stability 1H to 1V.gsz	DWP Number:
Waste Management	Analysis: Seismic ULS	Analysed by: ALNA
	Comments: / Scale: 1:500 @ A4	Checked by: TIMC

Directory: C:\Users\alna\Documents\Polaris\Appendix C\

Analysis Notes:
1. Name: Static long term
2. Method: Morgenstem-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	HW-MW rock	Mohr-Coulomb	21	60	35	0	1
	Residual Soil	Mohr-Coulomb	17	7	30	0	1
	Structural fill	Mohr-Coulomb	18	6	29	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	1



Title: Access Road CH1450 fill slope stability.gsz | DWP Number:
Analysis: Static long term | Analysed by: ALNA
Comments: | Scale: 1:750 @ A4 | Checked by: TIMC

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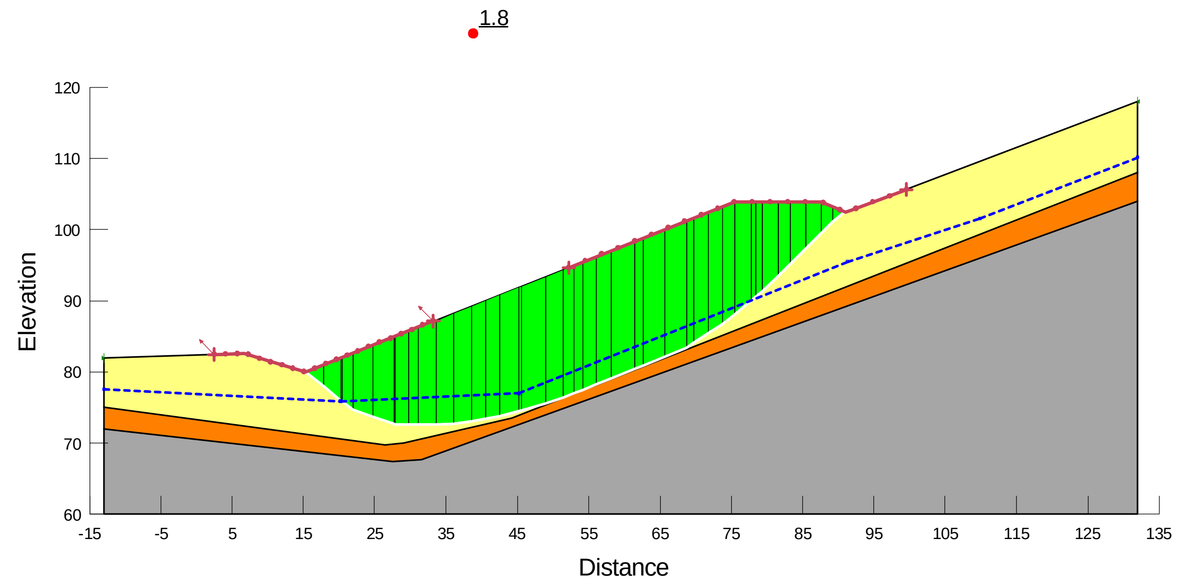
Analysis Notes:
1. Name: Static short term
2. Method: Morgenstem-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Cohesion (kPa)	Piezometric Line
	Bulk fill	Undrained (Phi=0)	18				120	1
	HW-MW rock	Mohr-Coulomb	21	60	35	0		1
	Residual Soil	Undrained (Phi=0)	17				100	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0		1

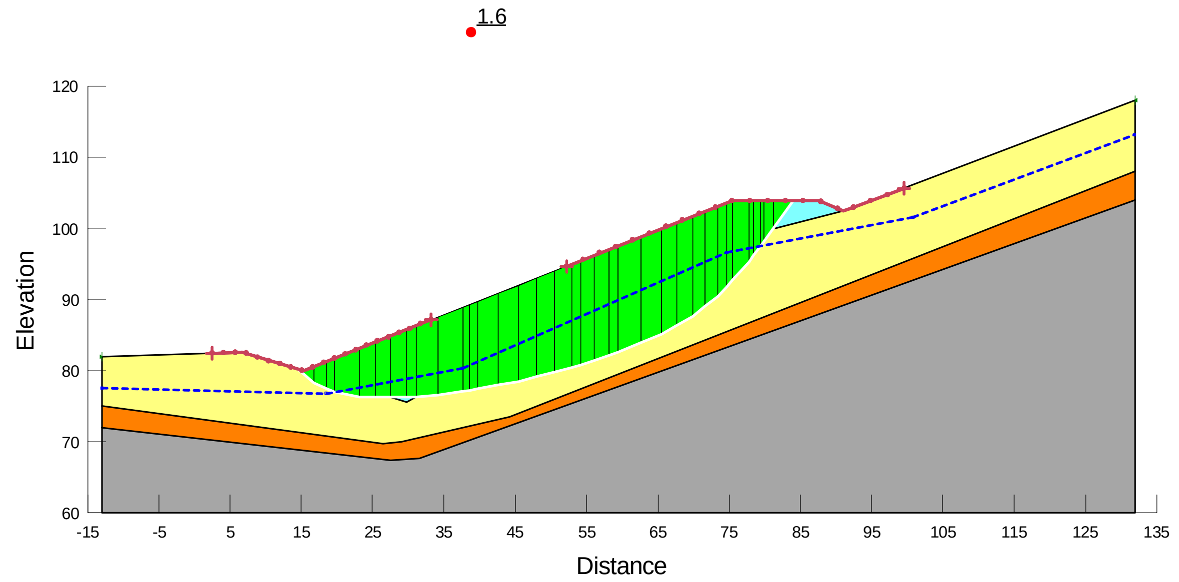


Title: Access Road CH1450 fill slope stability.gsz | DWP Number:
Analysis: Static short term | Analysed by: ALNA
Comments: | Scale: 1:750 @ A4 | Checked by: TIMC

Directory: C:\Users\alna\Documents\Polaris\Appendix C\

Analysis Notes:
1. Name: static elevated groundwater
2. Method: Morgenstem-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	HW-MW rock	Mohr-Coulomb	21	60	35	0	1
	Residual Soil	Mohr-Coulomb	17	7	30	0	1
	Structural fill	Mohr-Coulomb	18	6	29	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	1

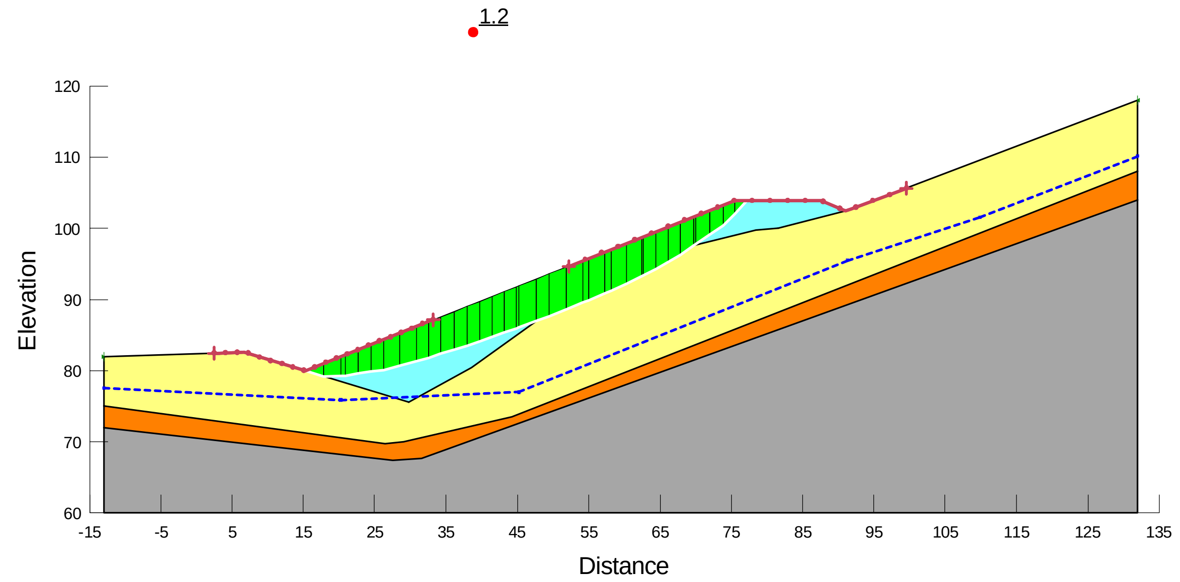


Title: Access Road CH1450 fill slope stability.gsz | DWP Number:
Analysis: static elevated groundwater | Analysed by: ALNA
Comments: | Scale: 1:750 @ A4 | Checked by: TIMC

Directory: C:\Users\alna\Documents\Polaris\Appendix C\

Analysis Notes:
1. Name: Seismic ULS
2. Method: Morgenstem-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load: 0.19

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	HW-MW rock	Mohr-Coulomb	21	60	35	0	0	0	1
	Residual Soil	Mohr-Coulomb	17	7	30	0	0	0	1
	Structural fill	Mohr-Coulomb	18	6	29	0	0	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	0	0	1

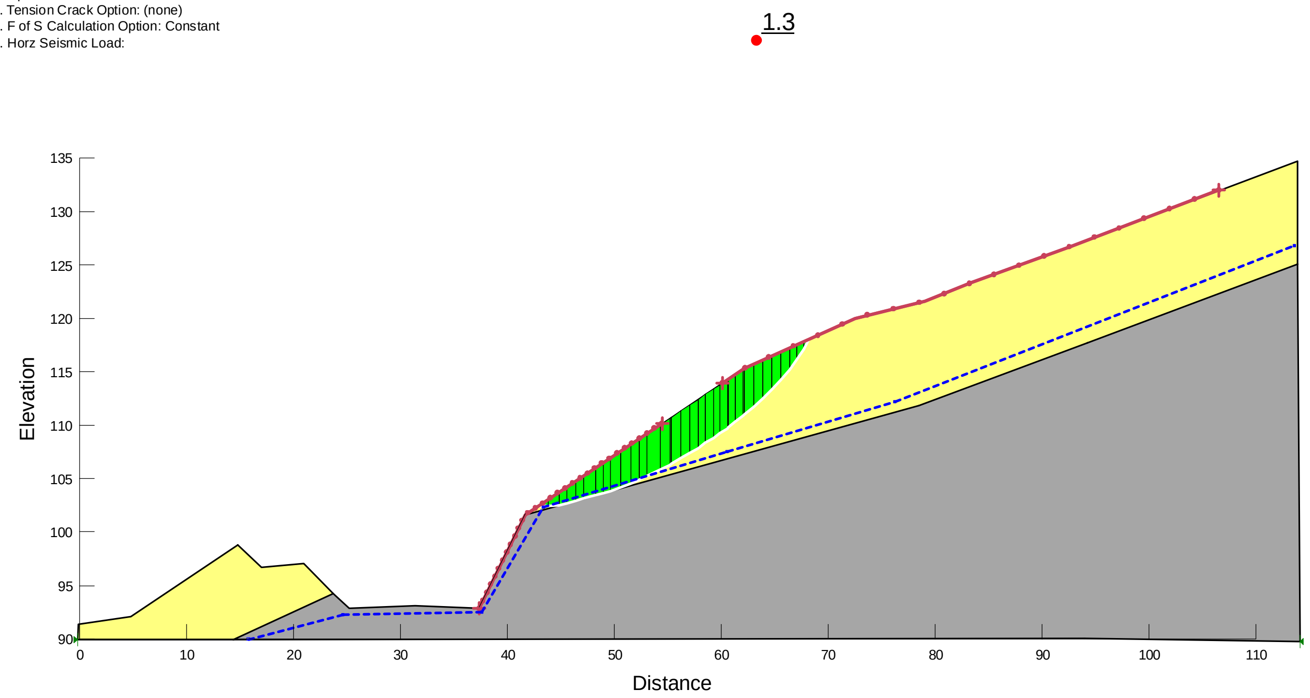


Title: Access Road CH1450 fill slope stability.gsz | DWP Number:
Analysis: Seismic ULS | Analysed by: ALNA
Comments: | Scale: 1:750 @ A4 | Checked by: TIMC

Directory: C:\Users\alna\Documents\Polaris\Appendix C\

Analysis Notes:
1. Name: Static long term
2. Method: Morgenstern-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Residual Soil	Mohr-Coulomb	17	7	30	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	1

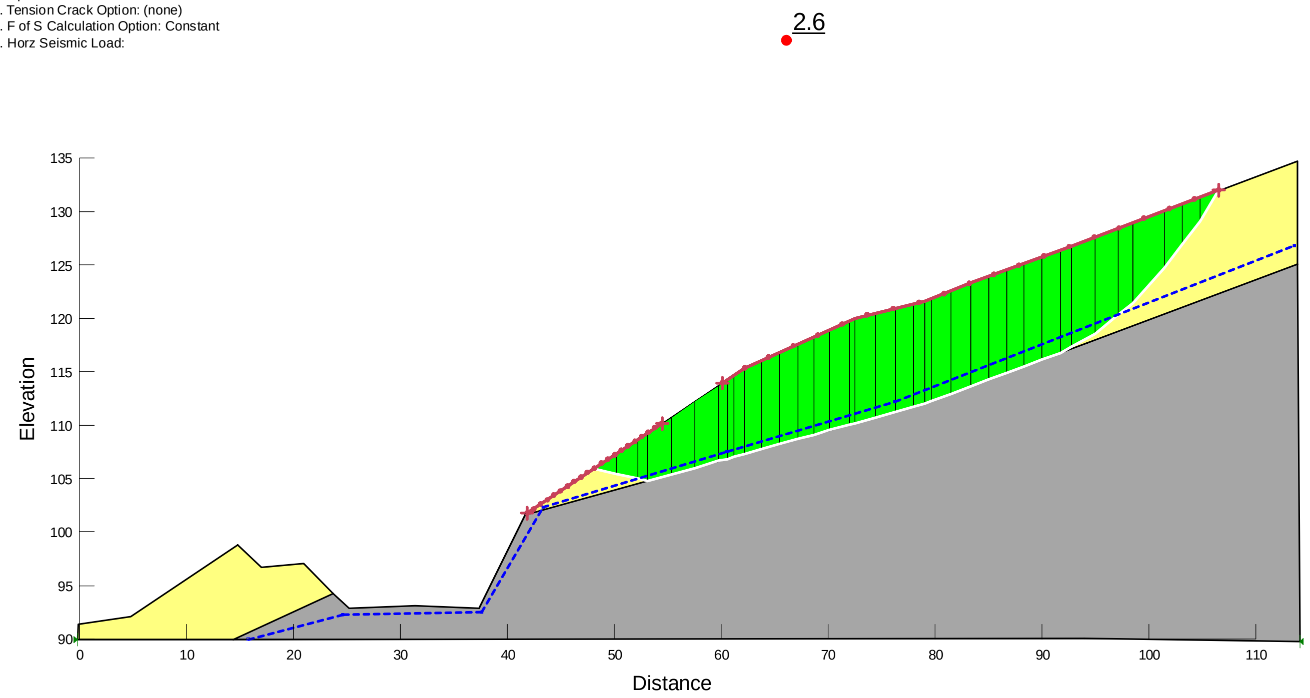


Tonkin+Taylor	Title: Access Road CH1300 cut slope stability 1.5H to 1V.gsz	DWP Number:
	Analysis: Static long term	Analysed by: ALNA
	Comments: Scale: 1:500 @ A4	Checked by: TIMC

Directory: C:\Users\alna\Documents\Polaris\Appendix C\

Analysis Notes:
1. Name: Static short term
2. Method: Morgenstern-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Cohesion (kPa)	Piezometric Line
	Residual Soil	Undrained (Phi=0)	17				100	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0		1

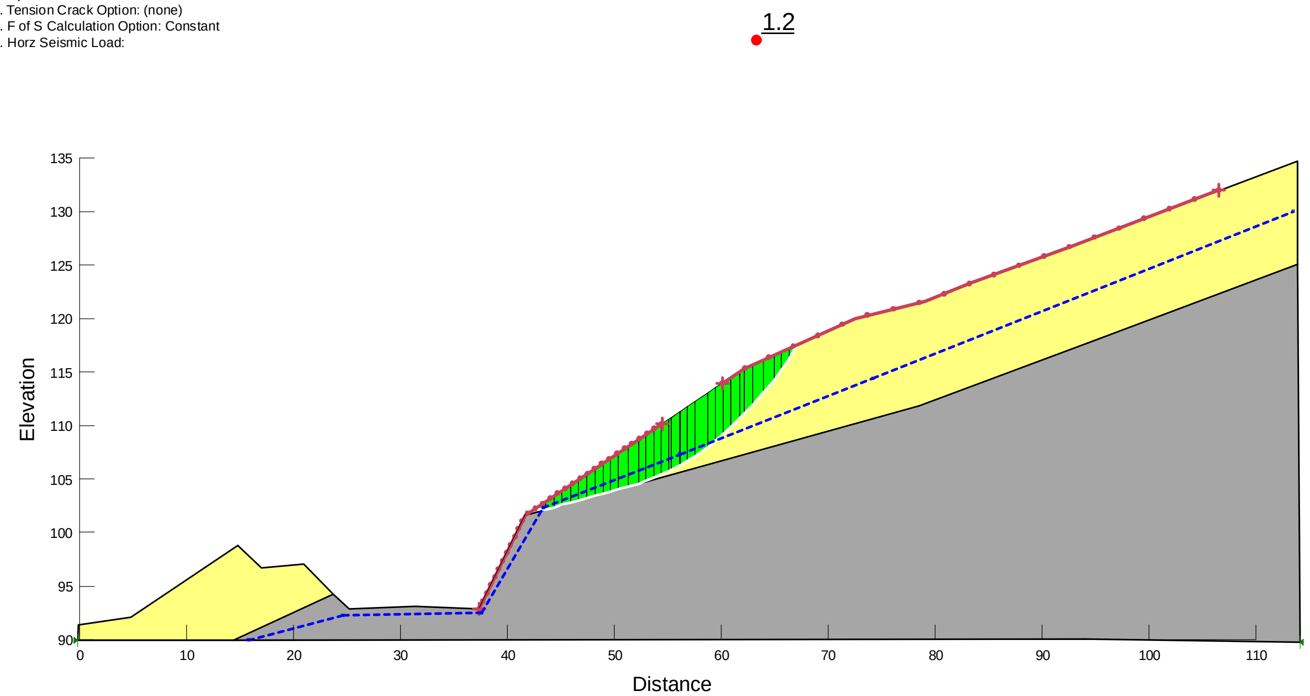


Tonkin+Taylor	Title: Access Road CH1300 cut slope stability 1.5H to 1V.gsz	DWP Number:
	Analysis: Static short term	Analysed by: ALNA
	Comments: Scale: 1:500 @ A4	Checked by: TIMC

Directory: C:\Users\alna\Documents\Polaris\Appendix C\

Analysis Notes:
1. Name: Static elevated groundwater
2. Method: Morgenstern-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Residual Soil	Mohr-Coulomb	17	7	30	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	1



Tonkin+Taylor	Title: Access Road CH1300 cut slope stability 1.5H to 1V.gsz	DWP Number:
	Analysis: Static elevated groundwater	Analysed by: ALNA
	Comments: Scale: 1:500 @ A4	Checked by: TIMC

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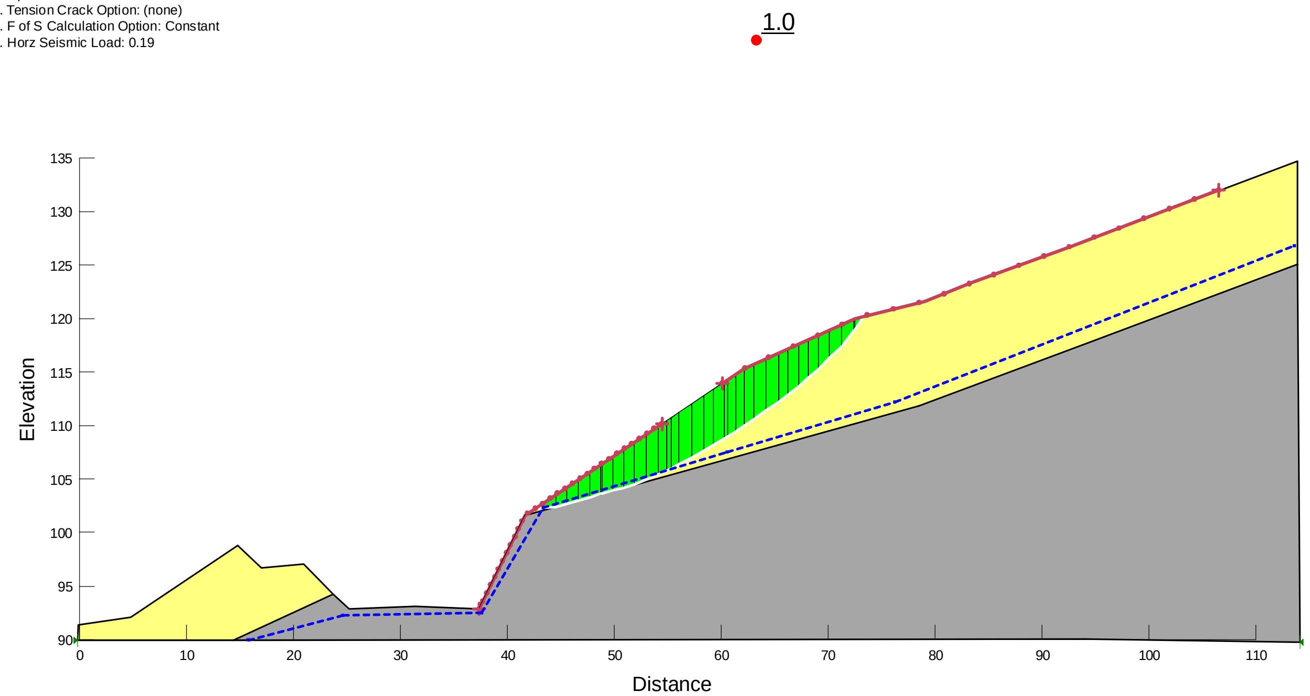
Analysis Notes:
1. Name: Seismic ULS
2. Method: Morgenstern-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load: 0.19

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Residual Soil	Mohr-Coulomb	17	7	30	0	0	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	0	0	1

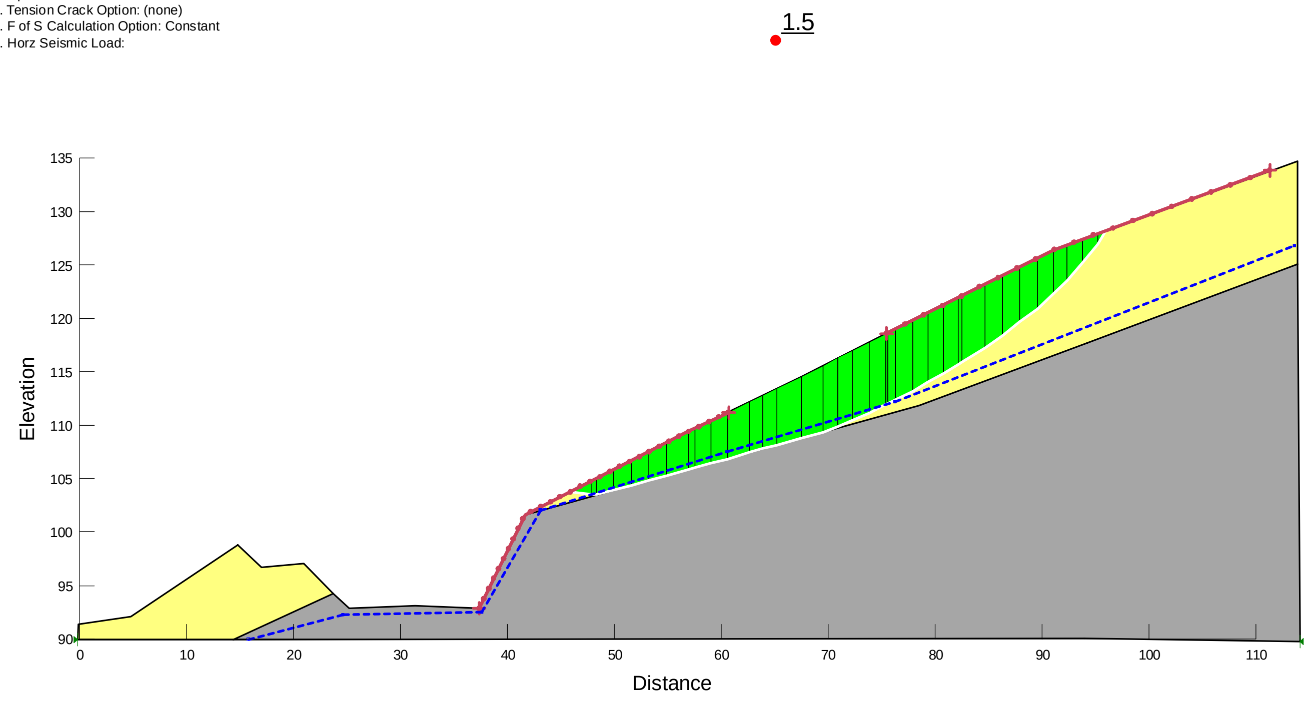


Tonkin+Taylor	Title: Access Road CH1300 cut slope stability 1.5H to 1V.gsz	DWP Number:
	Analysis: Seismic ULS	Analysed by: ALNA
	Comments: Scale: 1:500 @ A4	Checked by: TIMC

Directory: C:\Users\alna\Documents\Polaris\Appendix C\

Analysis Notes:
1. Name: Static long term
2. Method: Morgenstern-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Residual Soil	Mohr-Coulomb	17	7	30	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	1

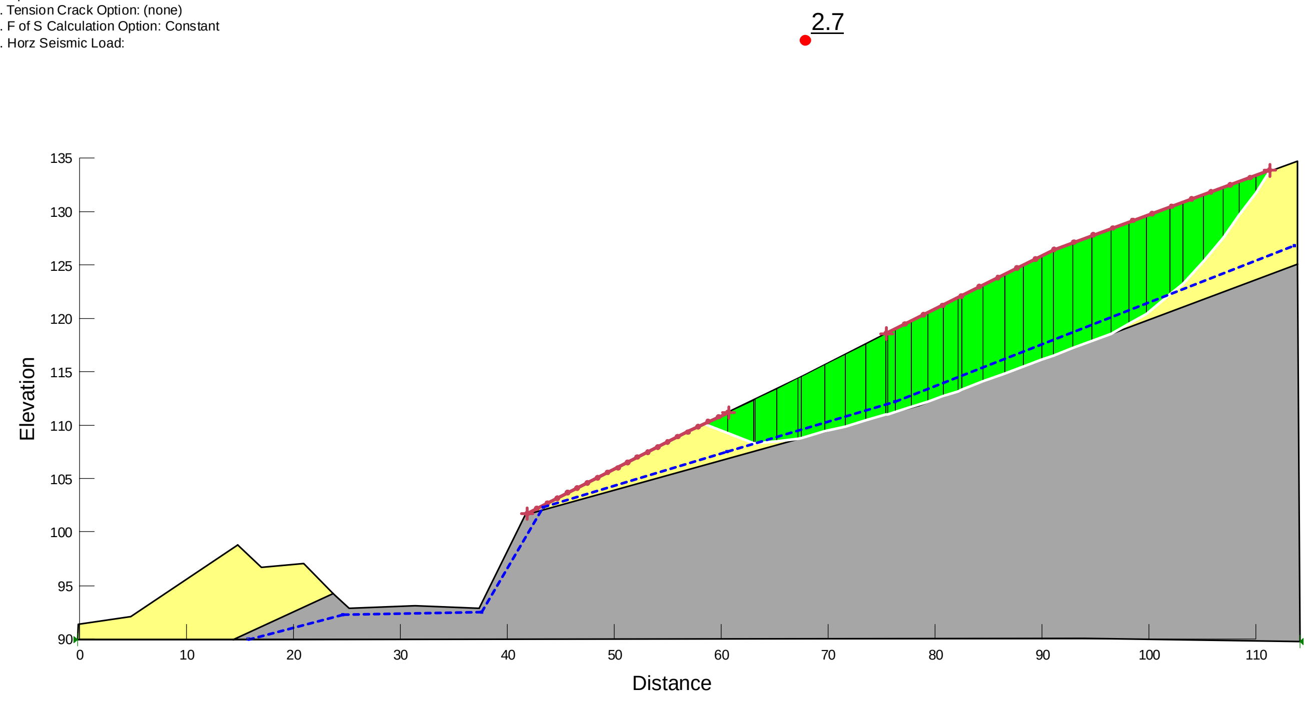


Tonkin+Taylor	Title: Access Road CH1300 cut slope stability 2H to 1V.gsz	DWP Number:
Waste Management	Analysis: Static long term	Analysed by: ALNA
	Comments: Scale: 1:500 @ A4	Checked by: TIMC

Directory: C:\Users\alna\Documents\Polaris\Appendix C\

Analysis Notes:
1. Name: Static short term
2. Method: Morgenstern-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Cohesion (kPa)	Piezometric Line
	Residual Soil	Undrained (Phi=0)	17				100	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0		1

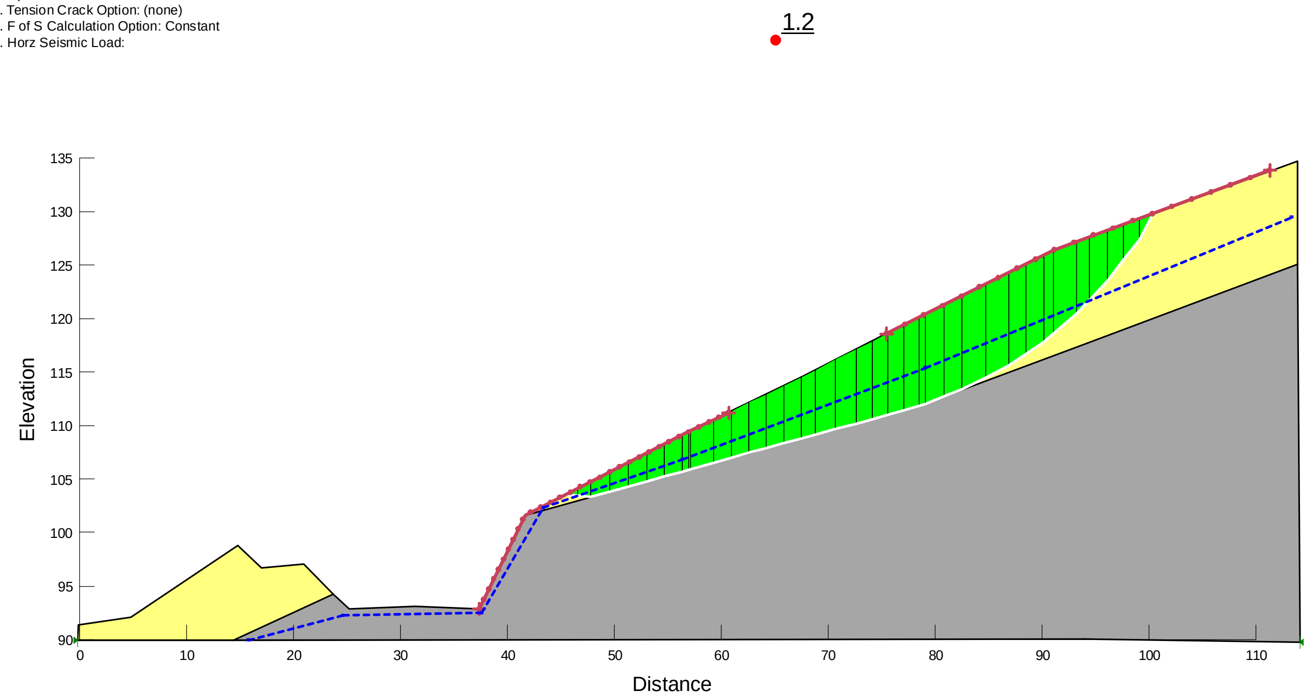


Tonkin+Taylor	Title: Access Road CH1300 cut slope stability 2H to 1V.gsz	DWP Number:
Waste Management	Analysis: Static short term	Analysed by: ALNA
	Comments: Scale: 1:500 @ A4	Checked by: TIMC

Directory: C:\Users\alna\Documents\Polaris\Appendix C\

Analysis Notes:
1. Name: Static elevated groundwater
2. Method: Morgenstern-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load:

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	Residual Soil	Mohr-Coulomb	17	7	30	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	1



Tonkin+Taylor	Title: Access Road CH1300 cut slope stability 2H to 1V.gsz	DWP Number:
Waste Management	Analysis: Static elevated groundwater	Analysed by: ALNA
	Comments: Scale: 1:500 @ A4	Checked by: TIMC

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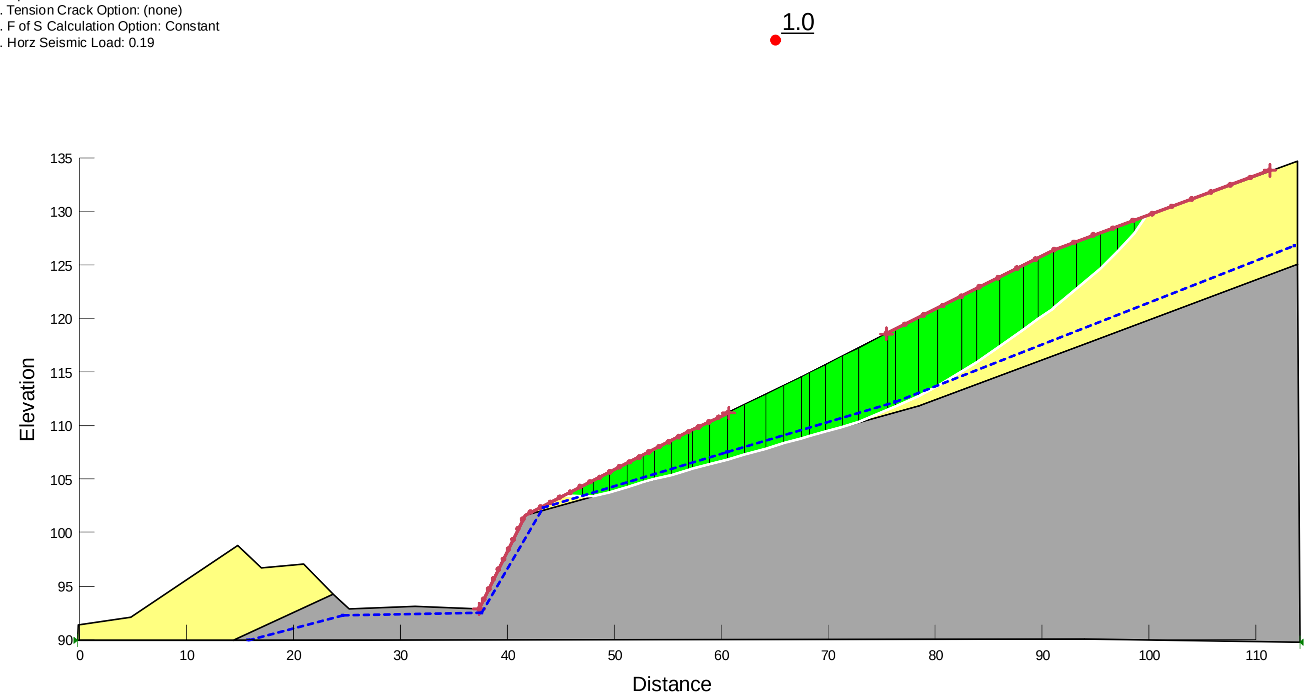
Analysis Notes:
1. Name: Seismic ULS
2. Method: Morgenstern-Price
3. Direction of movement: Right to Left
4. Slip Surface Option: Entry and Exit
5. PWP Conditions Source: Piezometric Line
6. Optimization: Yes
7. Tension Crack Option: (none)
8. F of S Calculation Option: Constant
9. Horz Seismic Load: 0.19

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Cohesion R (kPa)	Phi R (°)	Piezometric Line
	Residual Soil	Mohr-Coulomb	17	7	30	0	0	0	1
	SW-UW rock	Mohr-Coulomb	22	150	40	0	0	0	1



Tonkin+Taylor	Title: Access Road CH1300 cut slope stability 2H to 1V.gsz	DWP Number:
Waste Management	Analysis: Seismic ULS	Analysed by: ALNA
	Comments: Scale: 1:500 @ A4	Checked by: TIMC

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