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Mananui Mineral Sands

713 Ruatapu Road
Westland

Geotechnical Effects Assessment

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Contents

1 Introduction

1.1 Purpose of this report

This assessment addresses the off-site effects of proposed sand mining operations for the Mananui Mineral Sands project. Offsite effects are primarily expected to occur at the site boundaries. Specifically:

- South boundary shared with a DOC reserve
- East boundary with Tūwharewhare
- West boundary shared with State Highway 6

All of these effects are informed by an assessment of pit wall stability as discussed in Section 2.

The report is intended to be used in support of a resource consent application.

1.2 Site

713 Ruatapu Road is a 140.2 hectare farm on the east side of State Highway 6, 7.5 km south of Hokitika. The land parcel is around 1.67 km long (parallel to the highway) and 0.7 to 1.1 km wide. The legal description is Lot 1 DP 3854.

The terrain comprises a succession of beach ridges, running sub-parallel to the highway. There are six ridges across the site with crest elevations ranging from 7 to 13 m and with troughs typically at around 6 m. The highest ridge is furthest east and has localised 'peaks' up to 15.5 m elevation. From this ridge the land drops away steeply to Tūwharewhare (Mahinapua Creek) where the lowest ground level of 2.5 m is found.

1.3 Project description

The Mananui Mineral Sands project is fully described in the AEE but for the purposes of this assessment involves:

- Excavation with a floating sand dredge that sits within a mine pond. As the dredge mining face advances the void created behind is progressively backfilled with tailings sand, re contoured and top soiled. The proposed dredge path is on average 50m wide and predominantly in a North - South direction (Figure 1-1)
- Mining will target the layered heavy mineral bearing sands within the proposed mining footprint above RL 0.0 m.
- Dredged ore is screened at the mine pond to remove oversize particles, then the sand fraction is slurry pumped to a 4.4 Ha plant site situated adjacent to SH6.
- Three main products will be generated; Garnet concentrate, Ilmenite concentrate and gold. Up to 300,000 tons of heavy mineral concentrate will be removed from site per year.
- The tailings sand is slurry pumped back to the mine pond for back filling.
- Consent is sought for a total of 16 years with anticipated 10-year mining at maximum production.
- The mining methodology involves a continuously moving mine pond and progressive rehabilitation and revegetation.

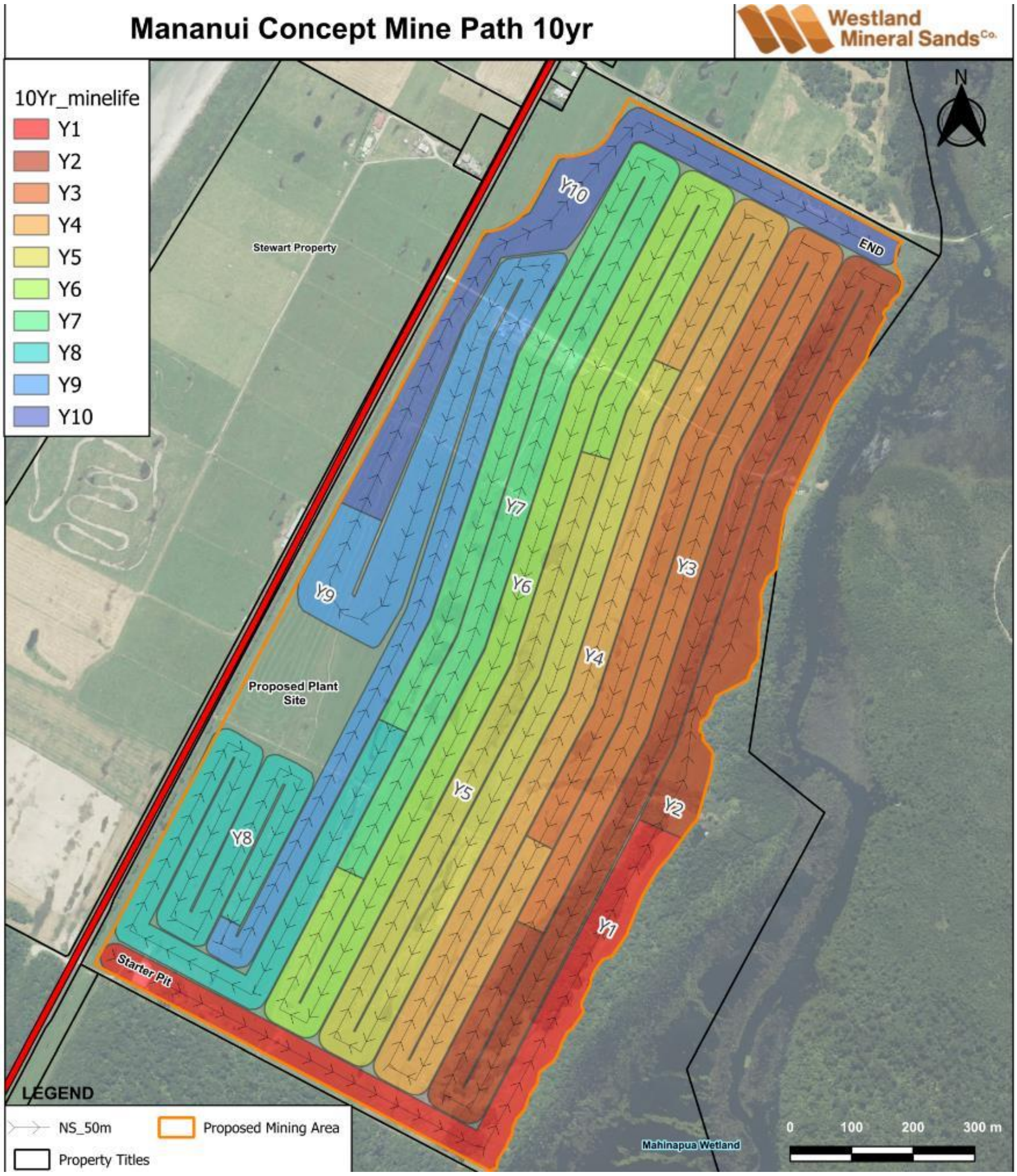

Figure 1-1 Schematic of proposed 10 year mine path

2 Pit Wall Stability

2.1 Overview

Conditions that influence pit wall stability include:

a) Geotechnical properties of the mineral sand deposits.
b) Groundwater elevation.
c) Terrain elevation and depth of excavation.
d) Regional seismicity and earthquake effects.

These aspects are discussed in the following sections.

Analysis has been carried out using Spencer's method[1], with circular failure surfaces, constrained to fail at the toe of the slope. No factoring of load or resistance forces has been included in the analysis, that is, a traditional Limit Equilibrium design approach has been adopted, with stability represented by Factors of Safety.

For long term slopes a Factor of Safety of 1.5 would typically be accepted as 'stable'. In the mine, steeper batters will allow more efficient operations but will mean a lower factor of safety for the slopes. A reduced factor of safety is justifiable on the basis that these batters are not permanent and are expected to be fully backfilled within weeks to months, this reduces exposure to earthquakes and extreme weather events.

2.2 Ground Conditions

2.2.1 Site geology

GNS Geological Map 12 (Nathan, Rattenbury, & Suggate, 2002) shows the site as being located on post-glacial beach deposits consisting "mainly of well sorted beach sand and nearshore gravel and sand". The coastline is advancing in this location with the shore of 6,000 years ago mapped along the line of Tūwharewhare (Māhinapua Creek), with Lake Māhinapua an estuary at that time.

Pleistocene age glaciations (20,000 to 400,000 years b.p.) will have extended out beyond the current coast with their moraine's and outwash plains submerged by rising sea levels since the last major glaciation.

On the site, geomorphology and sub-surface investigations have identified a series of beach ridges comprising sands and gravelly sands with variability of particle size distribution and density, but generally becoming denser with depth. This beach sequence overlies dense alluvial gravels, that are interpreted as glacial age outwash, comprising gravels (sometimes silt bound) with cobbles and boulders. The outwash surface is typically at around 0.0 m elevation, this is effectively the base of the economic resource.

2.2.2 Geotechnical properties

Geotechnical properties have been interpreted from Cone Penetration Testing (CPT), drill core logs and selected laboratory samples, all as described in the Geotechnical Factual Report (Geotech Consulting (NZ) Ltd, 2019).

[1] Implemented in Geo-stru Slope 2023 software

For the purposes of mine pit wall stability a 'typical' soil profile can be described as:

Layer Code	Soil description	Depth	Top surface level	Dry density (kN/m^3)	angle of friction, ϕ	Cohesion C_u (kPa)
NSS	Near surface sands (loose, brown medium sand)	0 to 2 m	6.5 m	17	36°	0
MGS	Grey medium to coarse sands (medium dense to dense)	2 to 6 m	4.5 m	18.5	38°	0
VDSG	Very dense sandy gravels (or "Muddy Gravels"	6.0+m	0.5 m	20.5	42°	0

Table 2-1 Typical soil profile

Stability is sensitive to the frictional properties assumed for the materials. Analysis has been based on a conservative assessment of soil parameters from the available data, with, for example, the MGS layer assigned a friction angle of 38°. This is intended to be a 'characteristic value' with 80 to 90% of slopes on the site expected to have higher strength.

The near surface sands (NSS) comprise an orangey brown sand layer of about 1 to 1.5m thickness and a grey medium sand layer of similar density down to about 2 m depth. This layer is evident over most of the site, at a similar thickness, despite variations in ground level.

The MGS layer is the most critical for batter stability because it occupies most of the potential failure zone for any slope. It is also more variable and is often interlayered with bands of denser sands and gravelly sands which can be expected to have better shear strength. There are a number of CPT's that show the MGS layer continuing down to the outwash gravels and this has been taken as the more critical case for analysis.

The very-dense sandy gravels (VDSG) represent the base of mineralisation. The properties of this layer are not especially relevant in the analysis as all critical failure surfaces are assumed to occur above it.

For most of the site there is a denser sand layer (DGS) above the outwash gravels. This was included for modelling along the higher parts on the Tūwharewhare boundary where it is a more persistent layer, and as expected, improved the stability of the slope.

There are a range of reasons why slopes may perform better than expected from the assessed properties. The most obvious of these is a light cementation that occurs in some layers within the sands but has the effect of reinforcing a slope such that trial excavations were observed to stand vertically, over limited height and limited time. Another factor is soil suction (or a 'sand-castle' effect) that is effective at holding sands of a certain size range, at a certain moisture content, until they become too wet or too dry.

2.3 Groundwater

Groundwater modelling (Sephira Environmental, 2019), (Rekker & Etheridge, 2023) indicates a moderate water surface at an RL of 3 m aMSL[2] at the groundwater divide near the east side of the site, Falling to creek level at the east and to a similar elevation at the south-west corner. Adjusting these levels to NZVD2016 gives 2.7 m at the divide and 2 m at either side.

[2] Above Mean Sea Level or, to the 1937 Local Vertical Datum (LVD37)

A variation of water levels of +0.8m and -0.7m (elevation up to 3.5 m) is considered representative of the likely range of groundwater fluctuation. This may not represent the full range but is considered sufficient to test the effects of groundwater variation.

Mounding of groundwater is likely with the return of process water to the mine pit. Modelling by Komanawa (pers comm.) indicates a typical increase in groundwater level of around 0.5 m based on a ‘best estimate’ of hydraulic conductivity.

2.4 Terrain

The existing terrain[3] is taken as the top of cut level. A cross-section (Figure 2-1), near to the north boundary shows that a ‘top of cut’ of 6.5 m would capture most of the terrain in this cross section. The maximum ground elevation is around 8.5 m.

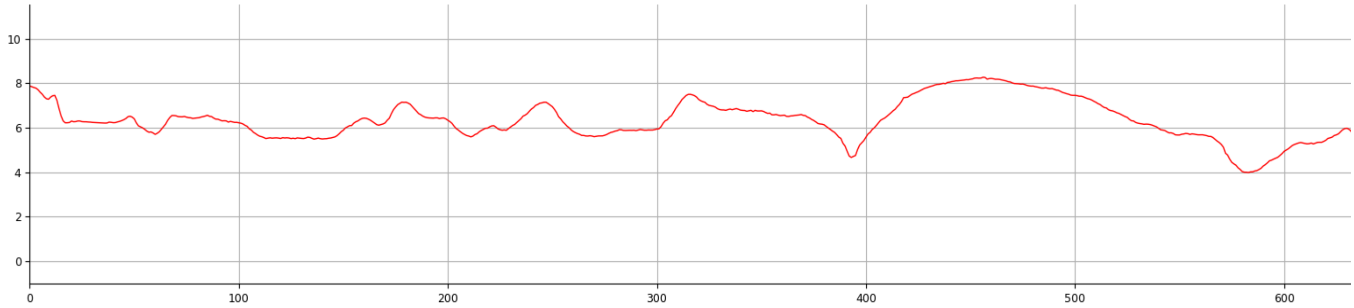

Figure 2-1 - terrain profile near north boundary (distance from west to east, height in 2m intervals)

Considering the wider site, Figure 2-2 shows terrain relief in bands of <6.5 m (yellow), 6.5 to 8.5 m (green), 8.5 to 10.5 m (orange) and >10.5 m (red). It can easily be seen that much of the site is less than 6.5 m in elevation and only a small proportion is higher than 8.5 m.

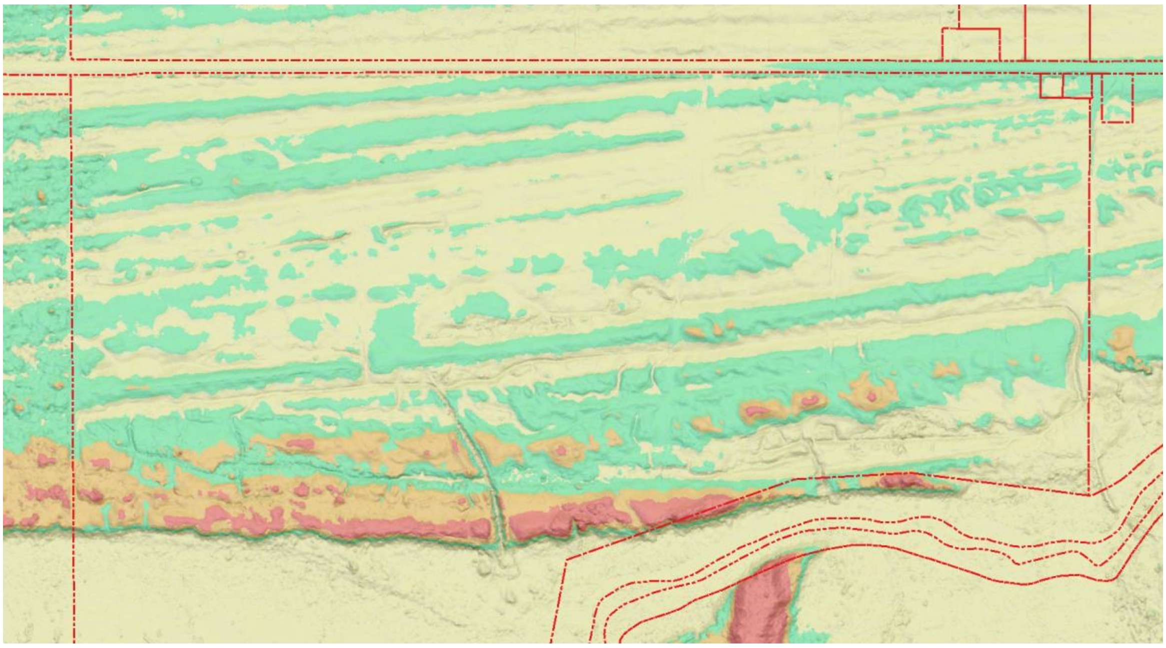

Figure 2-2 Mananui site with coloured relief (yellow <6.5m, orange >8.5m)

[3] Terrain data obtained from Landpro October 2019 LiDAR survey with elevation in terms of NZVD2016

2.5 Seismic Considerations

2.5.1 Load cases for assessment

Earthquake shaking will reduce stability of cut slopes and with sufficient intensity will cause liquefaction of sands on the site.

Seismic load parameters for geotechnical assessments are provided in Appendix A of NZGS Module 1 (NZGS, 2021). For analysis of temporary slopes, with exposure time measured in weeks to months, we consider that a 1 in 25 year earthquake with Magnitude 6.5 and Peak Ground Acceleration (PGA) of 0.13g is appropriate. In our slope modelling the PGA of 0.13g is applied with a factor of 0.54[4] to give an earthquake coefficient $k_h = 0.07$. A reduced horizontal coefficient is normal practice for slope design and accounts for a small, acceptable degree of slope displacement.

Recent research has highlighted an increased probability of rupture on the Alpine Fault, estimated at 75% within the next 50 years (Howarth, et al., 2021). The Alpine Fault is capable of generating a range of earthquake magnitudes depending on how much of the Fault ruptures in any particular earthquake. A Magnitude 8.2 (AF8) event (Orchiston, et al., 2016) with PGA of 0.59g[5] is possible at the site.

2.5.2 Liquefaction in natural sands

The sands at the Mananui site are reasonably dense and are typically dilative. That is, when sheared they tend to expand as the sand grains try to roll past one another. Dilative sands have some initial resistance to liquefaction because as they dilate the pore pressure drops. However, with continued shaking in large earthquakes, pressure will eventually increase, effective stress will reduce and liquefaction can occur.

Liquefaction susceptibility and predicted settlements have been assessed using cLiq[6] software with analysis showing that liquefaction is not expected in the 1 in 25 year earthquake but is likely in larger earthquakes. Liquefaction Susceptibility Number (LSN) is an indicator of likely ground damage or overall site performance including sand ejection and is plotted in Figure 2-3. We assess the liquefaction effects as likely to be ‘insignificant to mild’ (with reference to Table 5.1 of NZGS Module 3) over most of the site. But with the north-east corner expected to suffer from ‘moderate to high’ liquefaction with the reason for this poor performance being a lower ground level and hence relatively high groundwater.

[4] Ref DMG Special Publication 117: Guidelines for Analyzing and Mitigating Landslide Hazards in California, Chapter 11

[5] PGA for Hokitika, 23 km from the fault (Holden & Kaiser, 2016)

[6] CLiq v.3.3.3.4 by Geologismiki https://geologismiki.gr/products/cliq/

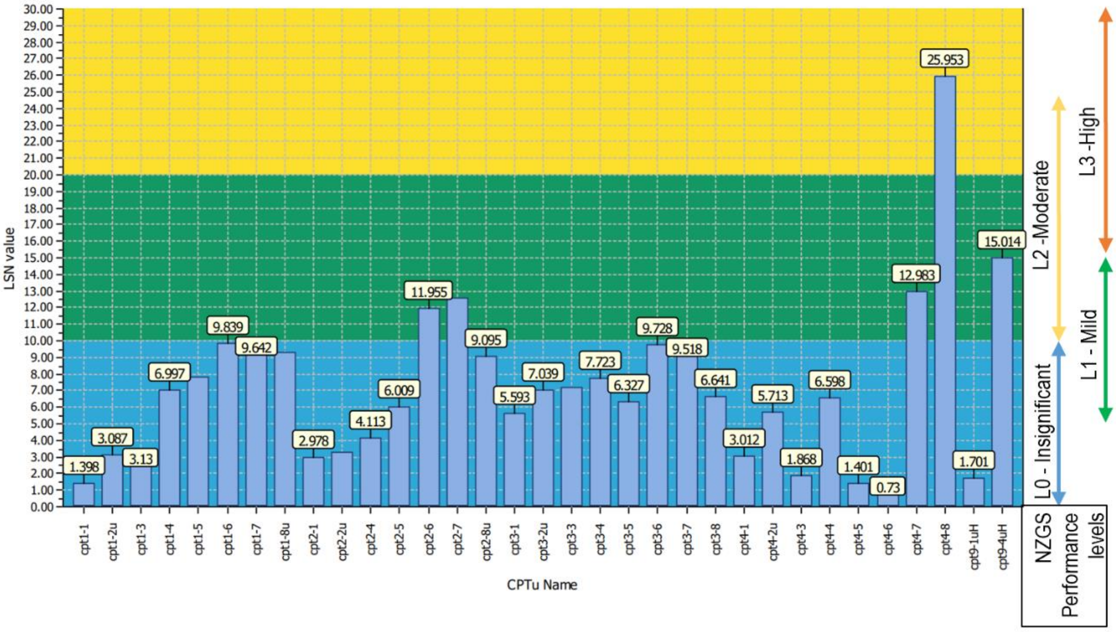

Figure 2-3 – Liquefaction Susceptibility Number by test location with AF8 earthquake loads

2.5.3 Effect on slopes

Liquefaction occurrence at high levels of earthquake shaking is likely to result in slope failure and lateral spread around the working mine pit. However, we note that this not expected at the 25 year return period and in more extreme earthquakes will be happening in the context of widespread damage throughout the affected part of the West Coast (McCahon, Elms, & Dewhirst, 2017).

2.5.4 Liquefaction in pit backfill

Pit backfill will be a hydraulic fill of process plant tailings. Some minor compactive effort will be applied by way of an excavator bucket to help settle the backfill and assist in draining the water. However, the inevitable result of this process is that the sand below the water table will be deposited in a loose state and therefore be more prone to liquefaction than the site is currently.

In this condition, sands will liquefy at lower levels of shaking and will consolidate (settle) more than the natural sands. For example, a loose sand layer of 2 m thickness (from groundwater to pit base) could consolidate about 100 mm. Analysis of a synthetic ‘backfill’ soil profile indicates that liquefaction will initiate at around 0.1g and increase rapidly to become fully liquefied by 0.15g. However, with the depth of crust there is still only a moderate/high surface effect from liquefaction (LSN ≈ 18 typical) even in the AF8 load case.

2.6 General Stability Assessment

An assessment of pit wall stability has been carried out, with key inputs discussed above, and with consideration of the general form of mine progression as shown in Figure 1-1. An excavation level of 0.0 m has been adopted and a nominal side wall batter of 2h:1v, ≈27° with a target Factor of Safety of 1.2 for static conditions and 1.0 in the 1 in 25 year earthquake.

Slope stability results are summarised in Table 2-2 with a typical model shown in Figure 2-4. Assessment has been carried out for typical excavation profiles that will remain open for a “medium-term” (weeks to months), this is expected to be the side walls of the mining strips inside the boundary.

Factor of Safety for high wall		Static case	Static case	Seismic (1in25yr)
Cut into	**Design batter**	**water at RL2.0**	**water at RL3.5**	**water at RL3.5**
natural ground	2h:1v - 27°	1.3	1.3	1.1
Highway boundary	2h:1v - 27°	1.3	1.3	1.1
Tūwharewhare bdy	2h:1v - 27°		1.36	1.16
Tailings/backfill	2.5h:1v – 22°	1.2	1.2	1

Table 2-2 Side Wall stability – expressed as factors of safety

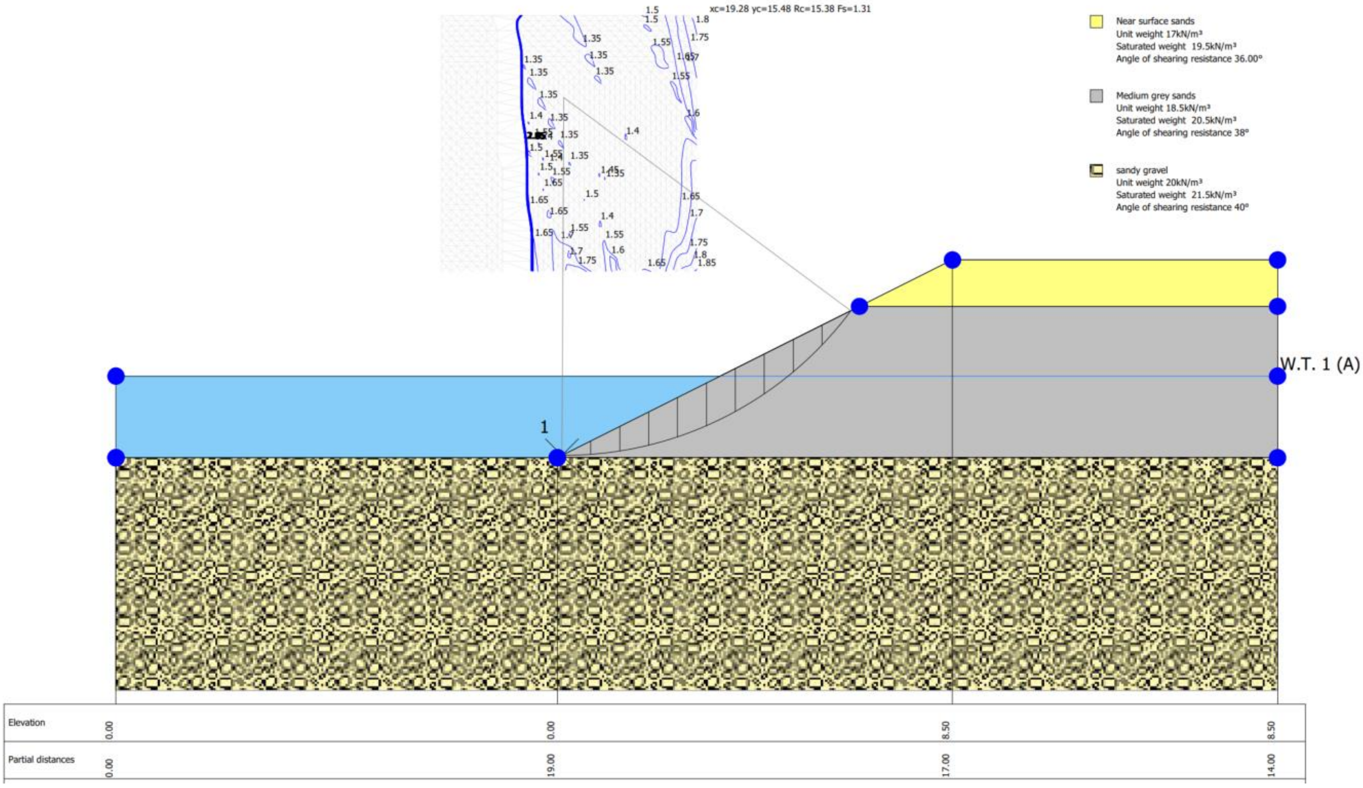

Figure 2-4 Slope model in natural ground, 2h:1v batter, GWL at RL3.5

No surcharge loads have been provided for, except the visual bund on the highway boundary. The bund was analysed with a 7 m wide berm from the edge of the excavation and was found to have no influence on stability.

The Tūwharewhare boundary was modelled with the existing ground level up to 14 m elevation. An improved stability result here is attributed to the presence of the dense sand layer below 4 m elevation.

After the first pass of the mining operation along the site boundary, each subsequent pass will have an excavation face in the natural ground on one side and in loosely deposited tailings (pit backfill) on the other. The backfill face is assessed as having reduced stability and a flatter slope 2.5h:1v may be necessary to achieve an acceptable factor of safety against slope failure.

Stability is not sensitive to slope height (which is to be expected for a non-cohesive soil) and critical failure surfaces typically 'daylight' below the top of the cut. Failure surfaces are generally shallow and indicate the most likely failure mode as being a sloughing of the face (especially around the water level) which is expected to progress until a stable, flatter, slope is attained.

Stability also does not appear to be especially sensitive to groundwater elevation. Mounding of groundwater within the pit was found to slightly improve stability, as a result of the gradient of water away from the pit.

Rapid drawdown will reduce stability but we understand the dredge operation won't result in drawdown. There is an analogous situation when heavy rainfall saturates the slope. Analysis indicates the slopes will be marginally stable (FoS $\approx$ 1) in these conditions. We consider this acceptable because it is an extreme event and unlikely to happen in reality, given the high hydraulic conductivity of the sands. We also anticipate that there are some management options that can mitigate this risk, if it turns out to be a limiting factor in mine operations.

3 Assessment of effects

3.1 South boundary – DoC Reserve

Mining is planned to commence along the south boundary with a 10 m setback to the adjoining DoC reserve which has forest vegetation growing up to the boundary. The proposed mine path length is 700 m along the boundary and the duration of exposure is expected to be six months.

Existing ground levels vary from $\approx$6 m at the south-west corner, up to near 11 m on the highest ridge before dropping back to $\approx$9 m at the south-east corner of the mine area.

Analysis of the range of slope heights indicates a low probability of slope failure affecting the forest edge at the proposed 2h:1v cut batter. Accordingly, a 10 m wide setback is assessed as appropriate along this boundary.

3.2 East boundary Tūwharewhare

At the south-east corner the mine path turns through 90 degrees and tracks along the edge of the native vegetation that indicates the extent of the wetland area. Ground levels vary from 9 m at the south-east corner, up to 15 m and down to 5 m in the north-east. The mine path is 1.6 km long with critical sections (where the ground elevation exceeds 5 m) only 1.4 km long and expected to be traversed in 10 months. The proposed edge of excavation is the more restrictive of either the site boundary, or the bush edge.

Excavation up to the boundary or bush edge requires a high level of confidence in the suitability of a 2h:1v batter. This is proposed to be confirmed by a formal review of slope performance to be carried out before the excavation comes within 100 m of the east boundary. Matters to be considered include:

1) Alignment of top edge of cut at time of excavation and at time of assessment;
2) Observation of exposed materials prior to backfill;
3) Pond water levels;
4) Rainfall records and any earthquake event records;
5) Thorough review of any notable slope failure (where more than 1m of berm is lost).

Potential effects to be considered on the east boundary are bi-directional because of the steep bank down from site to the Tūwharewhare wetland. Failure into the mine pond could result in loss of native vegetation into the pit, conversely, failure of the stream terrace riser could result in a loss of pit backfill into the wetland area.

3.2.1 Failure toward the mine pond

Slope modelling of the cut batter indicates a Factor of Safety (Fos) of 1.36 under static conditions and 1.16 under seismic conditions (1 in 25 year event). The 1 in 25 year earthquake has an approximate 4% chance of occurring during the period of exposure and the Alpine Fault earthquake has ≈1.2% chance.

In our assessment, and subject to the review of slope performance discussed above, it is unlikely that there will be a slope failure affecting neighbouring land or native vegetation during excavation along the eastern boundary.

In the low probability event of an Alpine Fault earthquake while the pit is alongside this boundary it is possible that there will be some loss of vegetation toward the mine pit. The area of disturbance in this situation is not expected to be more than 0.1 to 0.2 hectares.

3.2.2 Failure toward the wetland

An examination of the topography indicates that there will be a substantial natural embankment left in place between the mine pit and the toe of the terrace riser in the wetland. The width of this embankment varies with the height of terrain from around 20 m at the narrowest point (measured between the toe of terrace riser and the base of mine pond) to 60 m where the terrain is at 14 m elevation. The embankment will retain mine pond water in the short term and mine backfill (tailings) in the long term. Tailings at Mananui are typically a coarse sand with good strength properties, albeit deposited in a loose state.

Field experiments and laboratory testing, to assess the permeability of the insitu sands and the backfill (Sephira Environmental, 2019) shows that the backfill has much the same permeability as the parent sands and can be expected to drain quickly such that it regains most of its in-situ strength. This can be confirmed, during early stages of mine work, by installation of a standpipe, or by leaving a pressure transducer, in the mine pond to measure the drop in pore pressure and assess the speed of strength gain. Alternatively, a light tracked CPT rig could test the backfill soon after placement. This would serve to confirm dissipation of excess pore pressure and provide data for liquefaction and slope stability assessment of the backfill.

With the presence of a natural embankment along the east side of the mine workings and with the composition of the mine backfill, we consider it unlikely that the mine workings will have a material influence on slope failure into the wetland.

This does not mean that slope failure will not occur. The bank is steep and can be expected to be of marginal stability in extreme events (eg an Alpine Fault earthquake). It is likely that slope failures have occurred in the past. However, in our assessment the risk of failure is unlikely to be worsened by the presence of the mine workings and in the case of the mine pond we would not expect rapid discharge of pond water into the wetland.

3.3 West boundary and SH6

The potential effect to be considered is failure of the cut batter to such an extent that it extends onto the road reserve with resulting damage to the State Highway.

A 20 m setback is proposed from the west boundary of the site, with the edge of seal of the State Highway about 10 m beyond the boundary. Within the 20 m width a visual and noise bund will be constructed, about 12 m wide and up to 3 m high, with a 3h:1v landscape planted outer slope and a 2h:1v grassed inner slope. The typical ground elevation along the highway boundary is 7 m and with a base of pit at 0 m and a 2h:1v batter the toe of the excavation is about 34 m from the boundary and 44 m from the edge of seal.

Slope modelling indicates a Factor of Safety (Fos) of 1.3 under static conditions and 1.1 under seismic conditions (1 in 25 year event). There is no reduction in FoS cause by the earth bund.

The total length of SH6 boundary is 1.15 km and with the expected rate of mining progress will be traversed in about 6 months. The 1 in 25 year earthquake has an approximate 2% chance of occurring during the period of exposure and the Alpine Fault earthquake has <1% chance.

In our assessment it is very unlikely that there will be any effect beyond the site boundary.

In the low probability event of an AF8 earthquake occurring while the mine pit is alongside the boundary it is possible that flow failure of the batter will result in ground cracks extending onto or across the highway

3.4 North boundary and residential properties in north-west corner

A 10 m setback is proposed for the north boundary, the same as for the south. An 85 m setback is proposed for the three residential properties near the north-west corner of the site.

By comparison with the south boundary assessment, we do not foresee any geotechnical issues affecting these residential properties or the Mananui Tramline walkway to the north.

4 Conclusions

This report assesses the geotechnical issues involved in developing a mineral sands mine at Mananui, Westland.

The Mananui site is a series of sandy beach ridges stranded by an advancing coastline along this part of the West Coast. The existing ground level is between 5 m and 15 m and the mineral rich beach sands are underlain by an alluvial outwash plain at about RL 0.0 m.

The location is in a seismically active region and is close to the Alpine Fault (22 km away).

Pit-wall stability is one of the main considerations for planning of mine operations and assessing mineral recovery rates, it is also the main factor to consider in assessing off-site effects. The proposed pit batter of 27° is expected to be adequately stable for short term exposure in the natural sand deposits.

The sands on site are resistant to seismic effects up to around the 1 in 25 year (SLS) earthquake and liquefaction settlements are typically no more than moderate even in the extreme Alpine Fault event.

We assess a low risk that the mine workings will have a material effect on any adjacent property, State Highway 6, or the Tūwharewhare wetland. This assessment is subject to a formal review of slope performance prior to approaching within 100 m of the native vegetation in the south-east corner.

5 Limitations

This report has been prepared for the benefit of Westland Mineral Sands Limited as our client, with respect to the brief. The reliance by other parties on the information or opinions contained in the report shall be at such parties' sole risk.

Recommendations and opinions (not to be construed as guarantees) in this report are based on a discrete number of on-site tests, and data derived from other sources. The nature and continuity of subsoil conditions away from the test locations are inferred and it must be appreciated that actual conditions could vary from the assumed model.

During any excavation works, the site should be examined by an Engineer or Engineering Geologist competent to judge whether the exposed subsoils are compatible with the inferred conditions on which the report has been based. It is possible that the nature of the exposed subsoils may require further investigation, and the subsequent modification of design work.

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