Appendix C

Slope Stability Report

Waitakere Coastal Communities Landslide Risk Assessment

Appendix C – Muriwai Slope Stability Assessment Report

Auckland Council

30 April 2024

→ **The Power of Commitment**

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C1. Introduction

C1.1 Purpose of this Report

GHD has been engaged by Auckland Council (AC)[1] to carry out landslide risk assessments as well as to provide associated landslide risk management advice and geotechnical investigations in the Waitakere area, specifically for the residential areas of Muriwai, Piha and Karekare.

The purpose of this report is to present a slope stability and back analysis assessment of one of the large, failed slopes at the escarpment to the east of Muriwai township. The objective of the analyses was to estimate rock or soil strength parameters that could be used to inform conceptual remediation options to demonstrate the engineering measures required to stabilise the escarpment.

A two-dimensional Limit Equilibrium Analysis was carried out to estimate material parameters applicable to the failed zone. The analysis has also assessed the influence of changing pore pressure levels. The seismic performance of the slopes was also assessed, considering factors such as design life, site soil class, peak ground acceleration, and compliance with the NZ Building Code. We have considered commonly acceptable mitigation approaches and provided rough cost estimates for implementing the options investigated.

Figure C-1 shows the site location and the mapped landslides in the area.

[1] Under Contract CW198379, Master Services Agreement CCCS: CW74240 dated 7/09/2019

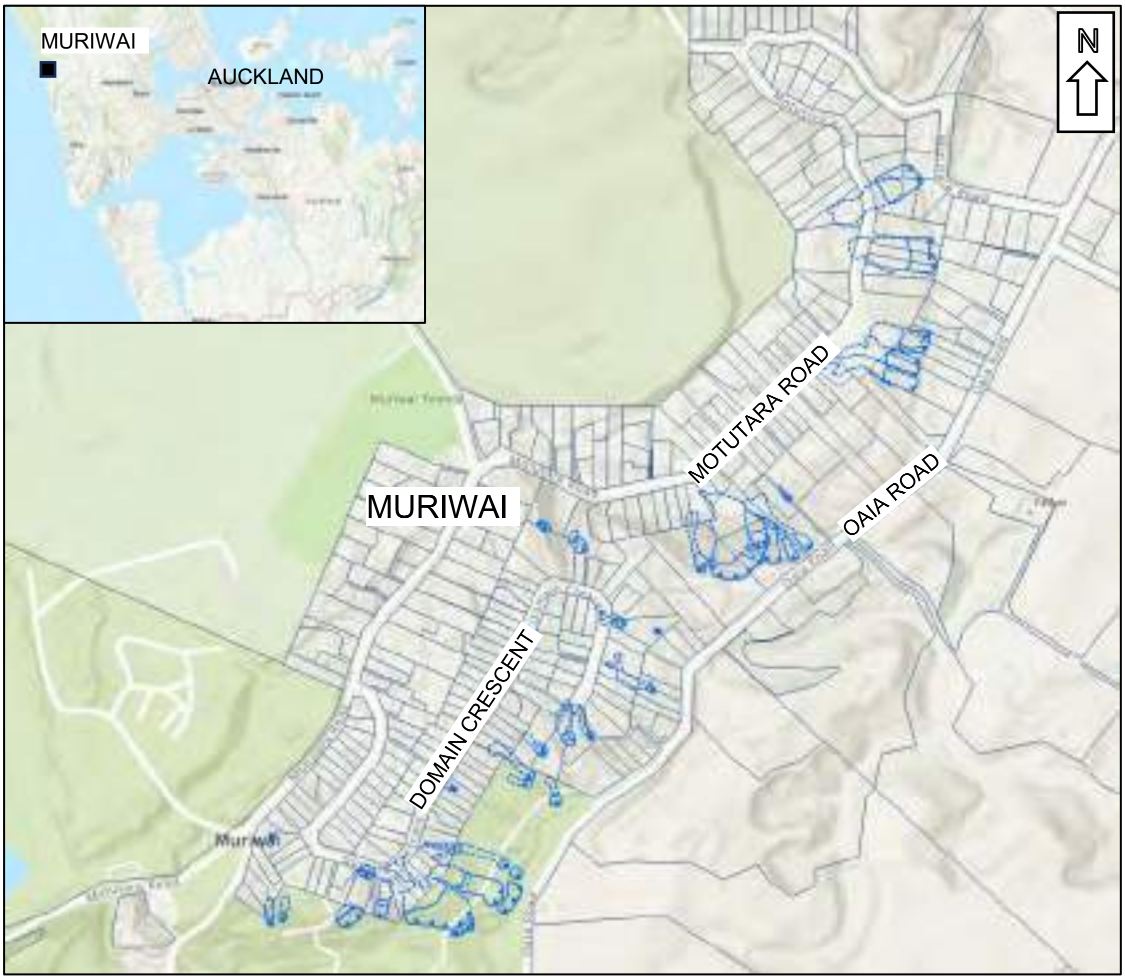

Figure C-1 Muriwai location showing the February 2023 landslides mapped by GHD (blue lines)

C1.2 Scope

The following scope of works has been undertaken:

- Conduct back analyses using Slope/W on recently failed slopes in Muriwai to derive material parameters for the assessment of remedial options.
- Quantify the sectional area of landslides by overlaying pre- and post-failure ground profiles.
- Perform a sensitivity analysis on moisture levels within the slope to assess their impact on slope stability, comparing with rainfall data.
- Explore mitigation options for the escarpment below Oaia Road based on the stability analysis results, targeting a Factor of Safety of 1.5 or greater for static analyses and a target Factor of Safety of unity for Damage Control Limit State seismic cases.
- Assess the seismic performance by determining the Importance Level, assigning Site Soil Class, deriving a Peak Ground Acceleration, and ensuring compliance with the NZ Building Code.
- Consider the NZ National Seismic Hazard Model for updated guidance.
- Exclude simultaneous occurrence of extreme weather events and large earthquakes. The coinciding of two such low probability events is not required by design codes.
- Examine a flatter benched profile and soil nailing with inclined drains as remedial options.
- Provide hand sketches and cost estimates for the proposed mitigation options.

This technical memorandum has been prepared by GHD for Auckland Council. This memo should be read in conjunction with all other GHD design documentation for the project.

C2. Report Structure

The accompanying GHD Engineering Geology report provides a detailed description of the site as well as discussion of site geology and geomorphology, historical landsliding, landslide mapping, landslide classification and slope processes. The reader is advised to consult the accompanying GHD reports for further information not contained herein.

Table C-1 presents a summary of the figures referred to in this report (see Appendix A of the overall report).

Table C-1 Summary of accompanying Muriwai landslide risk assessment reports

Report Section	Description
Overall Report	Waitakere Coastal Communities Landslide Risk Assessment (Muriwai)
Appendix A	Figures
Appendix B	Engineering Geological Report
Appendix C	*Slope Stability Assessment Report (this report)*
Appendix D	RAMMS debris flow analysis
Appendix E	Landslide Risk Assessment
Appendix F	Geotechnical Investigations Report
Appendix G	Geotechnical Laboratory Testing Report

C3. Geotechnical Information

C3.1 Representative Cross Section

The cross section shown in Figure C-2 was selected for assessment purposes as being a conservative (i.e. steep) example of the range of failure geometries. The cross-section extends for approximately 260 m horizontally approximately 100 m vertically. A comparison of landslide cross section profiles is presented in Figure C-3.

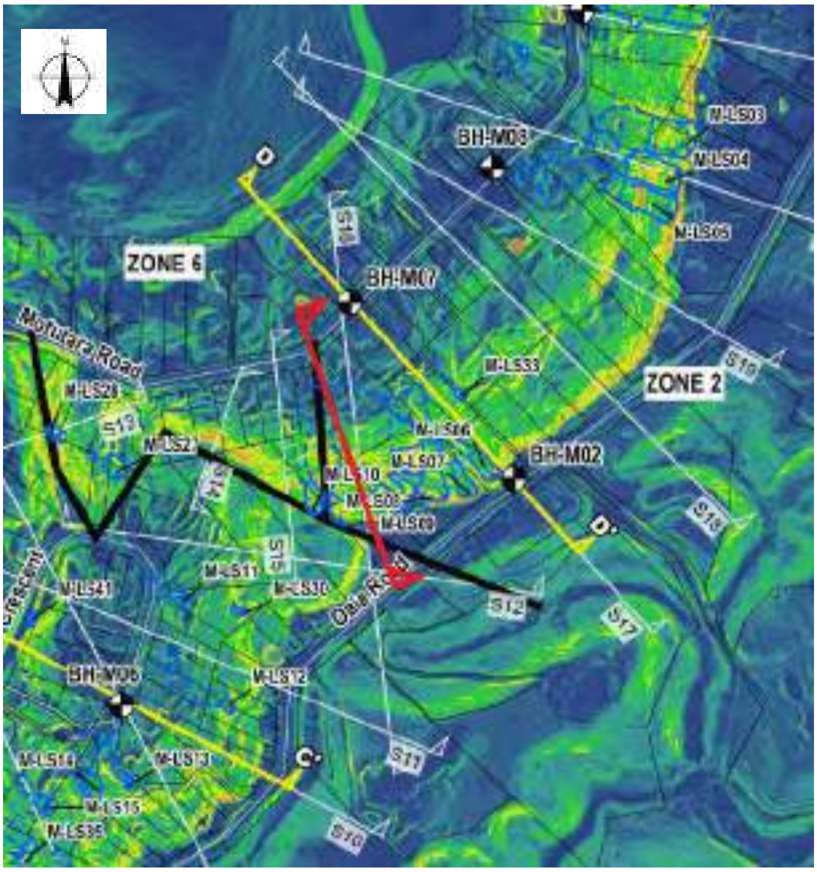

Figure C-2 ***Assessment area plan indicating the major February 2023 slip areas and the analysed cross-section A-A (retrieved from Google Earth Pro and GHD Atlas). A close-up view of Figure A125 (see Appendix A) showing the analysed cross section 'A' relative to other slope profiles, cross sections and boreholes.***

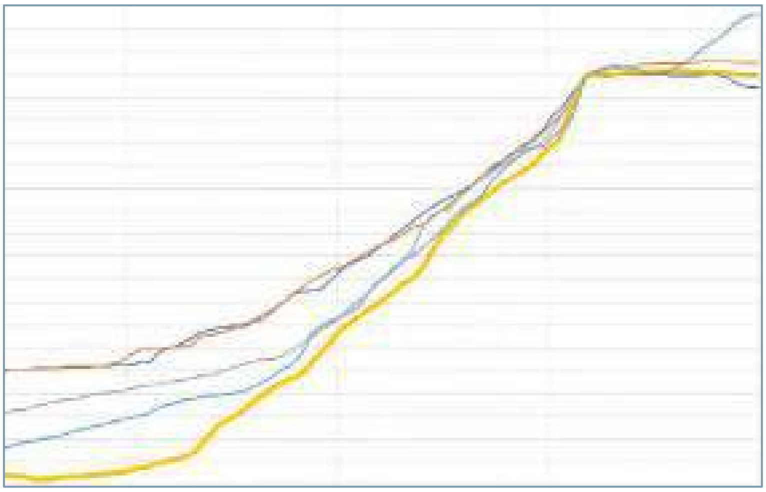

Figure C-3 Indicative comparison of cross section slope profiles at February 2023 landslide sites. The slope profiles at the upper slope have a similar geometry. The thick orange line is the analysed profile.

C3.2 Ground Model

This analysis preceded detailed site mapping and subsurface investigations by GHD (see Appendix B of the GHD landslide risk assessment). Based on initial site observations and the published literature, we have modelled the slope as Awhitu Group comprising the following:

– Weakly cemented sand (sandstone) with localised silt/clay. This comprises the upper three-quarters of the slope profile. This area was the initiation point of the landslides.

– A lower, relatively stronger sandstone. Field observations indicate that the lower slope was relatively stable.

This is consistent with the companion Engineering Geological report (Appendix B) and the 1:250,000 published geological map (Edbrooke 2001).

For the purposes of our analysis, we have made a nominal subdivision with weaker sandstone overlying a higher strength sandstone.

We employed Ru values to model varying degrees of saturation as an alternative to groundwater level, which was unknown at the time of landsliding in February 2023.

The ground model is shown in Figure C-4.

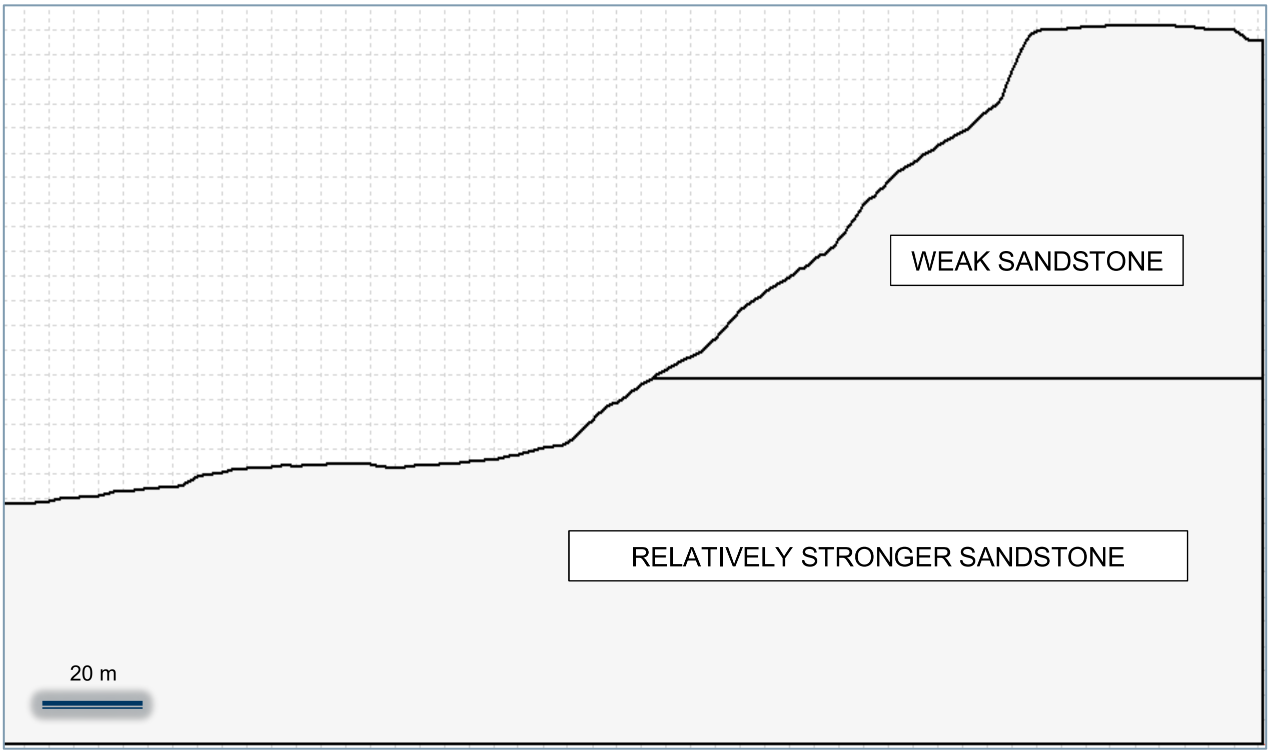

Figure C-4 Simplified ground model used for the analyses

C4. Analysis

C4.1 Introduction

The February 2023 landslides occurred during heavy rainfall that caused elevated, destabilising pore-water pressures. The failure surface is characterised by its shallow depth, yet it spans a significant horizontal distance and reaches high elevations, as shown in the Appendix.

Slope stability analyses were carried out using Slope W version 2021.3 (a GeoStudio Package). As part of the back analyses and feasibility assessments, we examined circular, non-circular, shallow, and irregular user-defined slip surfaces. We also employed Ru values to model varying degrees of saturation. The Morgenstern-Price method was chosen as it is suitable for analysing slope stability problems with these features.

C4.2 Back Analysis and Pore Pressure Sensitivity Assessments

The methodology adopted for the back analysis was as follows:

1. The slip to be examined was assumed to have occurred solely within the upper, weaker Kahu Sand weakly cemented sandstone.
2. A single set of c-phi effective stress parameters was assumed to be applicable.
3. No foliation or structural anisotropy was considered applicable.
4. As the failure occurred following heavy rain, we varied Ru within the slope stability model from what is considered to be a "steady state / typical long term" value of 0.125 to relatively highly saturated values of 0.25 to 0.35. We assumed these latter values to be representative of pore pressure levels that may exist after a period of prolonged and heavy rain.
5. In order to derive the geotechnical parameters, we fixed the phi value at 39 degrees. This is considered to represent an upper bound value that may be assigned to a naturally occurring granular material.
6. With the phi value fixed, the effective cohesion was varied until Factors of Safety were derived indicating a progression from marginal stability to "failure" as the Ru value was increased.

Table C-2 shows the results of the sensitivity analysis as Ru was increased. It will be seen that at an Ru value of 0.3, failure of the slope effectively occurs. This is an indication that the parameters utilised are providing reasonable agreement with observed behaviour.

Table C-2 Slope stability sensitivity analysis by modification of Ru for Φ' = 39 degrees and c' of 21 kPa

Ru adopted	FoS derived
0.125	1.24
0.150	1.20
0.200	1.12
0.250	1.03
0.300	**0.95 (slope failure)**
0.350	**0.87**

Table C-3 confirms the parameters derived for the Awhitu Sand from the back analysis. These are considered acceptable for a cemented sand. They align well with the ranges provided in publications (e.g., Collins & Sitar 2009). These then, were the geotechnical parameters carried forward to the analyses examining remedial works. Results from the back analyses are included in Appendix C1.

Table C-3 Geotechnical parameters

Material description	Unit weight (kN/m³)	Effective cohesion c' (kPa)	Angle of internal friction Φ' (°)
Weak sandstone	18	21	39

C4.3 Seismic Analysis

The seismic demand was derived in accordance with the New Zealand Bridge Manual (2022). The peak ground acceleration (PGA) for slope stability analysis was determined using the equation below:

$$PGA = C_{0,1000}\frac{R_u}{1.3}fg$$

NZTA Bridge Manual (2022), Section 6.2.2

Where the coefficients are provided in Table C-4 and g is 9.81 m/s2.

Table C-4 Peak ground acceleration (PGA) derivation parameters for slope stability seismic case

Parameter / Variable	Value	Source
Design life	50 years	Client-specified
Importance Level (IL)	3	NZS1170.0:2002, Table 3.2.
Annual probability of exceedance for the ultimate limit state for earthquake actions (DCLS)	1/1000	NZTA Bridge Manual, Table 2.3
Subsoil class	Likely D as the sand is ~100 m deep. Intrusive site investigation is being employed to help with ground profiling.	NZS1170.5:2004, Clause 3.1.3.2 – 3.1.3.6
1000-year return period PGA coefficient (C0,1000)	0.19	NZTA Bridge Manual, Table C6.1
Return Period Factor (Ru)	1.3	NZS1170.5:2004, Table 3.5
Site subsoil class factor (f)	1.0	NZTA Bridge Manual, Clause 6.2.2

Based on the above, the PGA is determined to be 0.19 g, which aligns with the PGA values for the ULS case in Auckland for a range of return periods of 500 to 2500 years given in Appendix A of Module 1 (MBIE, November 2021).

C5. Examination of Remediation Options

C5.1 Purpose of Options Assessment

The purpose of conceptual remedial measures is to provide the indicative effort required to inform discussion of whether mitigating debris flow would be practical. It is intended to provide information on two options that may be used to increase slope stability. Other engineering options are available, but exploring these is outside the scope of our work.

C5.2 Proposed Remedial Measures

Two potential remediation options have been examined. These are:

a) Strengthening the slope by using soil nails and inclined drains

b) Excavating the slope to a flatter, more stable, benched profile that satisfies New Zealand code-based stability criteria.

Details of the work carried out to examine their feasibility follows.

C5.3 Options Examined - Discussion

C5.3.1 Soil nails

The following design elements have been considered:

– No shotcrete facing.
– Installation of drains to reduce the pore water pressure within the slope.
– Nails to be installed by roped access method involving small, portable rigs similar to that shown in Figure C-5.
– Surface erosion prevention matting is installed to prevent loss of material at the surface from the space between the nails (see Figure C-6).

Figure C-5 Example of typical small, roped access rig used to install anchors or nails on steep slopes.

Figure C-6 ***Example of surface erosion prevention matting and mesh for use with soil nails on relatively steep slopes.***

Soil nail feasibility calculations were carried out in accordance with the following guidance document:

- FHWA. Geotechnical Engineering Circular No. 7 - Soil Nail Walls. 2015 (FHWA-NHI-14-007)

The recommended minimum Factors of Safety for “Overall Stability” for this option requires the following Factor of Safety:

- For Static Loading = 1.5
- For Seismic Loading = 1.1

The soil nail array derived satisfying the above criteria is shown in the Figure C-7 below. To facilitate initial pricing of the option, details of the key materials, quantities and spacings proposed are shown in the Table C-5.

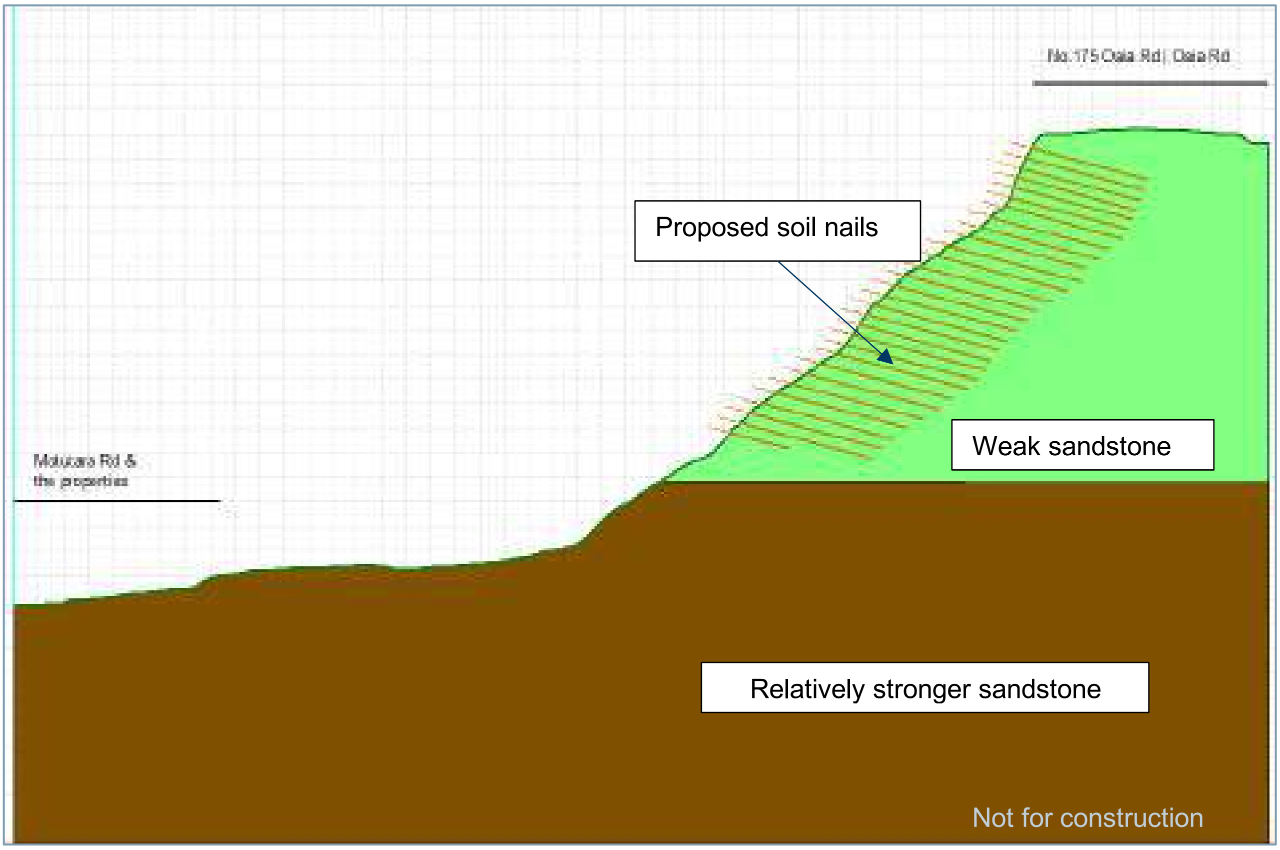

Figure C-7 *Soil nail option*

Table C-5 *Soil nail option – materials, quantities and spacings proposed*

Soil nail length (m)	Number of nail rows	Grid (m x m)	Steel bar diameter (mm)	Grade (MPa)	Bond diameter (mm)
28	30	2.0 x 2.0	25	500	120

Construction of the soil nailed option would need to include, but not be limited to, the following processes:

– Obtaining of any necessary easements if the nails were to extend over private property boundaries
– Vegetation removal and/or trimming as required
– Clearing the slope face of any debris or loose material
– Installation of the soil nails via the roped access method chosen
– Installing inclined drains to alleviate the soil pore pressure

C5.3.2 Benched profile

The solution examined has assumed the use of benches nominally 5 m high, each with a horizontal platform of approximately 4 m width. It is assumed that the face of each bench will be sloped back at 2 vertical to 1 horizontal. Although not an essential requirement of the design, inclined drains may also be employed with this option to increase stability.

Conventional, relatively large plant could be used to construct this option employing a “top-down” methodology. For safety reasons, the plant used would be set back at a suitably safe distance from the edge of the slope edge as the works progress downwards.

The option derived involves cutting the crest of the slope back by a horizontal distance of 20 m.

The volume of material to be removed under this option would be approximately 320 m3 per linear metre of remediated slip. The benching layout is shown in Figure C-8.

The stability calculations for this option were carried in accordance with the NZTA Bridge Manual (2022), which also satisfies the Auckland Code of Practice for Land Development and Subdivision (2023) requirements.

The recommended minimum Factors of Safety for “Overall Stability” for this option requires the following Factor of Safety:

– For Static Loading = 1.5
– For Seismic Loading = 1.0

Analysis results are included in Appendix C3.

<u>Note on displacement-based acceptance criteria:</u>

For seismic analyses, the NZTA Bridge Manual allows for displacement-based acceptance criteria for infrastructure such as filled embankments and other earthworks. For the landslip examined, and in particular the benching option, we considered that displacement-based acceptance criteria would not be suitable. This is because, unlike an embankment constructed from engineered granular material, there is more uncertainty regarding the geology and behaviour in a slope cut in natural materials. If such an approach were allowed, there could be a risk that movements lead to crest tension cracks which could then fill with water and unacceptably compromise stability. Given the consequences of a significant failure of such a high cut face in proximity to residential areas, it was decided that a displacement-based approach was not acceptable for the seismic cases for this project. For these reasons, a factor of safety of unity was targeted for the DCLS seismic case.

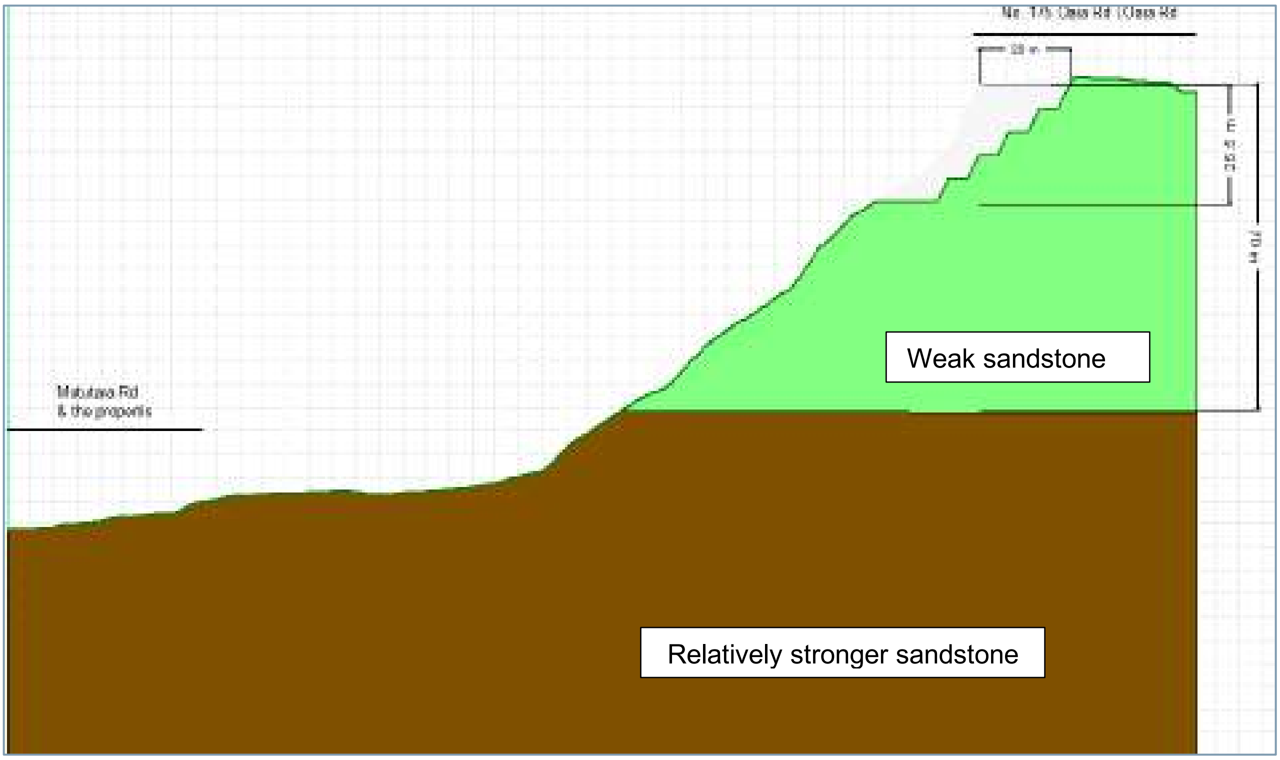

Figure C-8 Illustration of benching option

Construction of the benching option would need to include, but not be limited to, the following processes:

– Obtain any necessary easements if the benches were to extend over private property boundaries
– Preparation of a stable platform for plant
– Vegetation removal and/or trimming as required
– Apply erosion protection measures
– Consider suitable drainage system
– Top-down excavation processes

C5.4 Mitigation Cost Consideration and Cost Indication

Construction considerations of the two remedial options are included in Table C-6 to highlight the challenges with both options.

Costs associated with remediation is not possible without a more advanced design, paired with a detailed methodology. No consideration has been given to legal or consenting costs of either option.

We propose the following nominal order of magnitude constructed costs for either option:

– **Several hundred thousand dollars** per lineal metre of remediated slope.
– **Millions of dollars to repair damage** to existing February 2023 slips.
– **Tens of millions of dollars** to include the slope that did not experience landslide.

Table C-6 Remedial option construction consideration comparison (more X symbols equals less desirable)

Construction Consideration	Soil Nails		Benched Profile	
Site access (considers plant size and truck movements)	XX	Smaller plant	XXXXX	Large trucks. Damage to local roads from heavy truck movement.
Duration (machinery efficiency)	XXXXX	Smaller plant and hand work	XXX	Large machinery
Machinery size	X	Small rigs	XXXXX	Large trucks
Environmental (dust and erosion)	X	Drilling dust	XXXXX	Large exposed area
Environmental (noise)	XXXX	Loud drilling	XXXX	Loud large machinery
Safety risk to contractors	XXX	Many hours working at height on abseil	XXX	Fall from slope edge
Earthworks soil disposal	X	No disposal	XXXXX	Off site disposal
Change to existing slope profile	X	Strengthens in situ slope	XXXXX	Requires removal of slope, including buildings and infrastructure to the east of Muriwai. Requires bespoke stormwater design and infrastructure.
Post construction maintenance	XXXXX	Regular and expensive maintenance	XX	Minor earthworks and ongoing control of stormwater

C6. Limitations

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

GHD otherwise disclaims responsibility to any person other than Auckland Council arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

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

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

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An understanding of the geotechnical site conditions depends on the integration of many pieces of information, some regional, some site specific, some structure specific and some experienced based. Hence this report should not be altered, amended, abbreviated, or issued in part in any way without prior written approval by GHD. GHD does not accept liability in connection with the issuing of an unapproved or modified version of this report.

Verification of the geotechnical assumptions and/or model is an integral part of the design process - investigation, construction verification, and performance monitoring. If the revealed ground or groundwater conditions vary from those assumed or described in this report the matter should be referred back to GHD.

C7. References

- Edbrooke, S.W. (compiler), 2001. Geology of the Auckland Area. Institute of Geological and Nuclear Sciences 1:250,000 Geological Map 3. Institute of Geological and Nuclear Sciences Ltd, Lower Hutt.
- New Zealand Transport Agency (NZTA), Bridge Manual (SP/M/022) (Third edition, Amendment 4).
- Collins, B. D., & Sitar, N. (2009). Geotechnical properties of cemented sands in steep slopes. *Journal of geotechnical and geoenvironmental engineering*, *135*(10), 1359-1366.
- Federal Highway Administration (FHWA), (2015). Soil nail walls reference manual (FHWA-NHI-14-007). Washington, DC: FHWA.
- Auckland Council (2023). The Auckland Code of Practice for Lad Development and Subdivision (Version 2.0) (Chapter 2).
- MBIE and NZGS (2021). “Geotechnical Earthquake Engineering Practice: Module 1 – Overview of the Guidelines”. Ministry of Business, Innovation and Employment (MBIE) and New Zealand Geotechnical Society (NZGS).

Appendix

Slope stability analysis output figures

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru
	Sandstone - No failure	High Strength	22			0
	Weakly Cemented Sand - L1 - c21,phi'39	Mohr-Coulomb	18	21	39	0.125

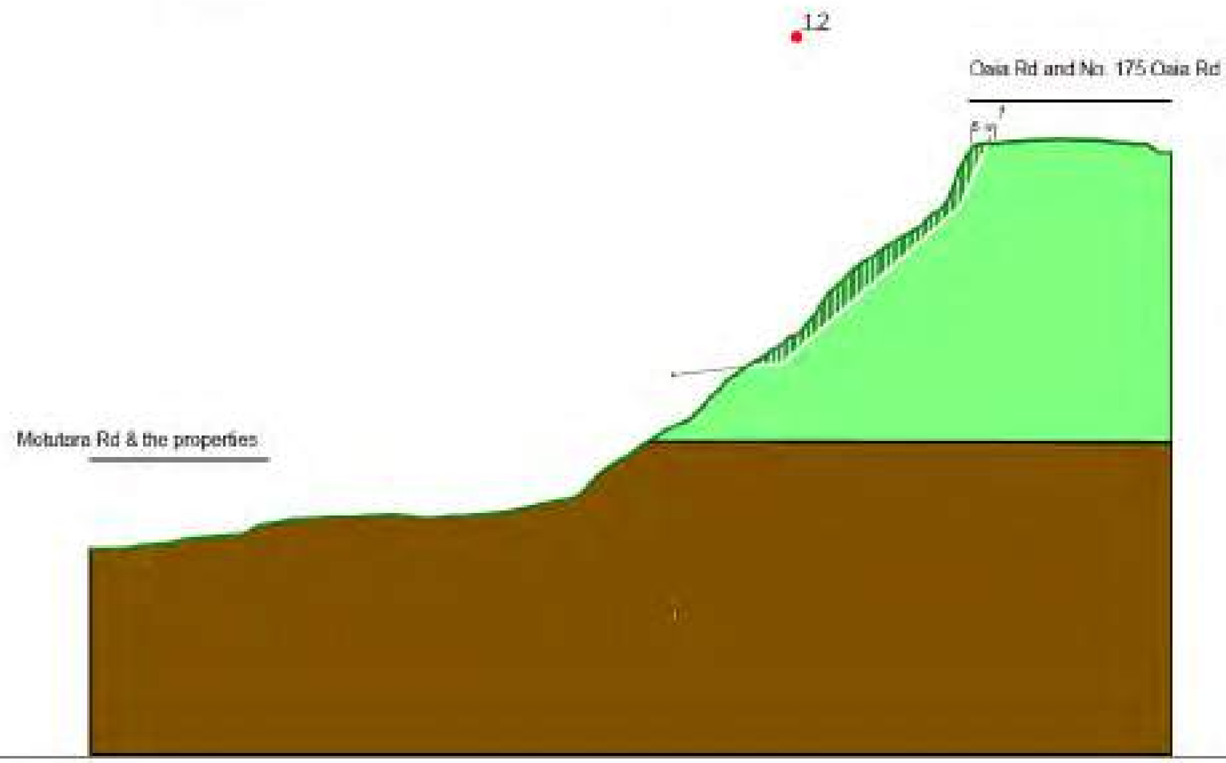

Figure C(A)-1. Back Analysis - Normal Groundwater

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru
	Sandstone - No failure	High Strength	22			0
	Weakly Cemented Sand - L1 - c21,phi'39	Mohr-Coulomb	18	21	39	0.3

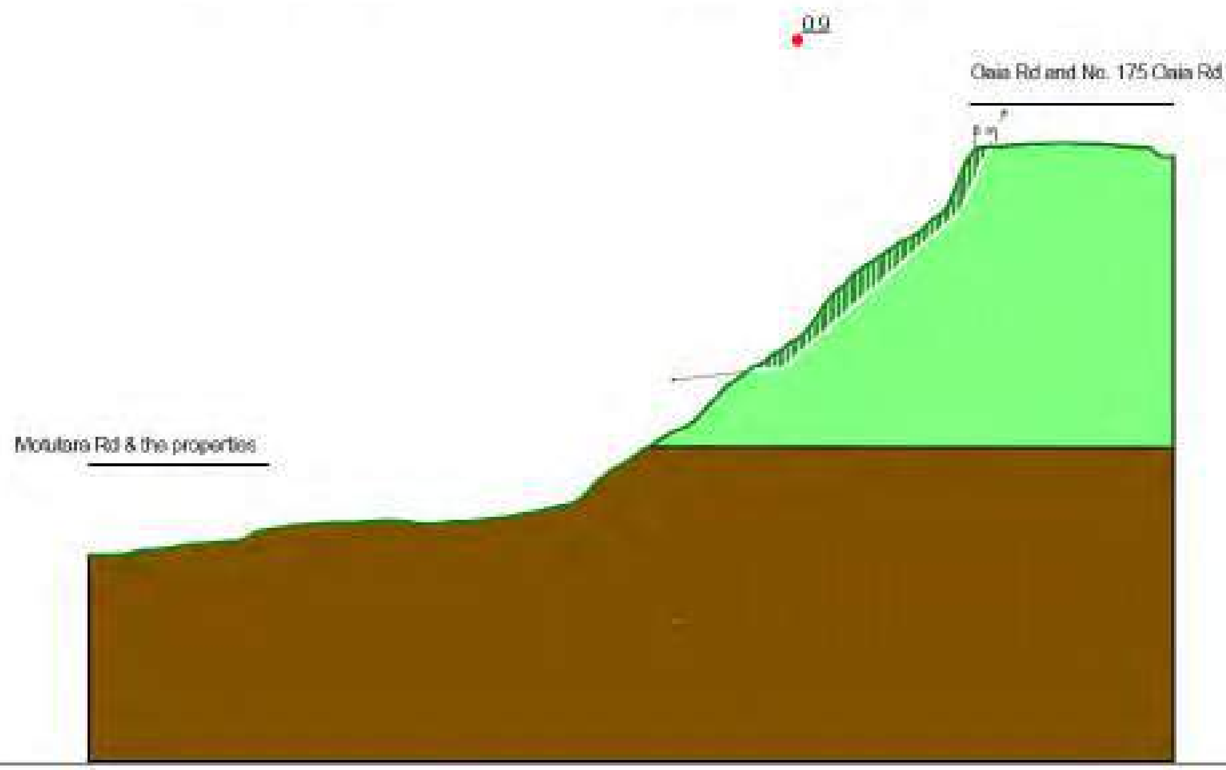

Figure C(A)-2. Back Analysis - Extreme Groundwater

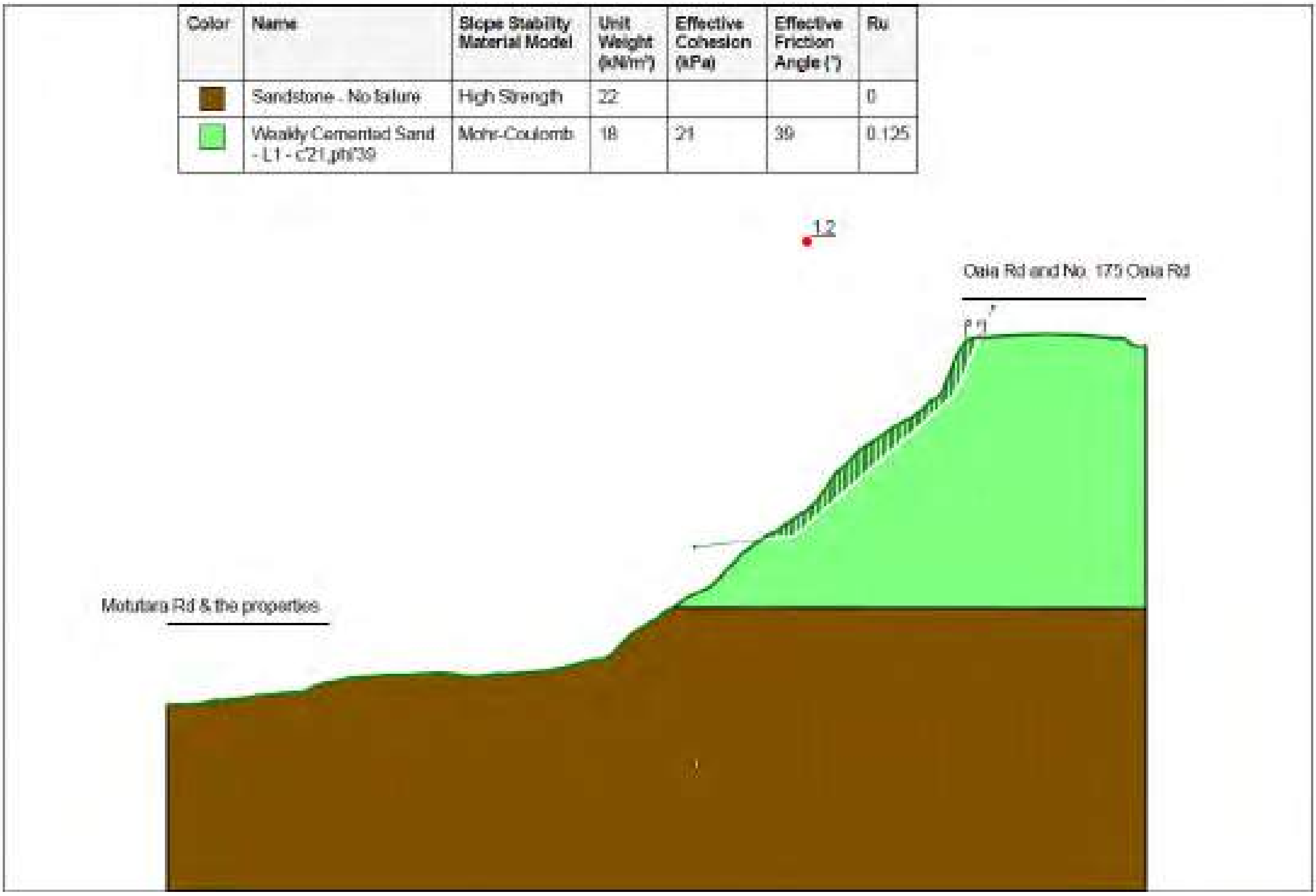

Figure C(A)-3. Nail Option - Normal Groundwater

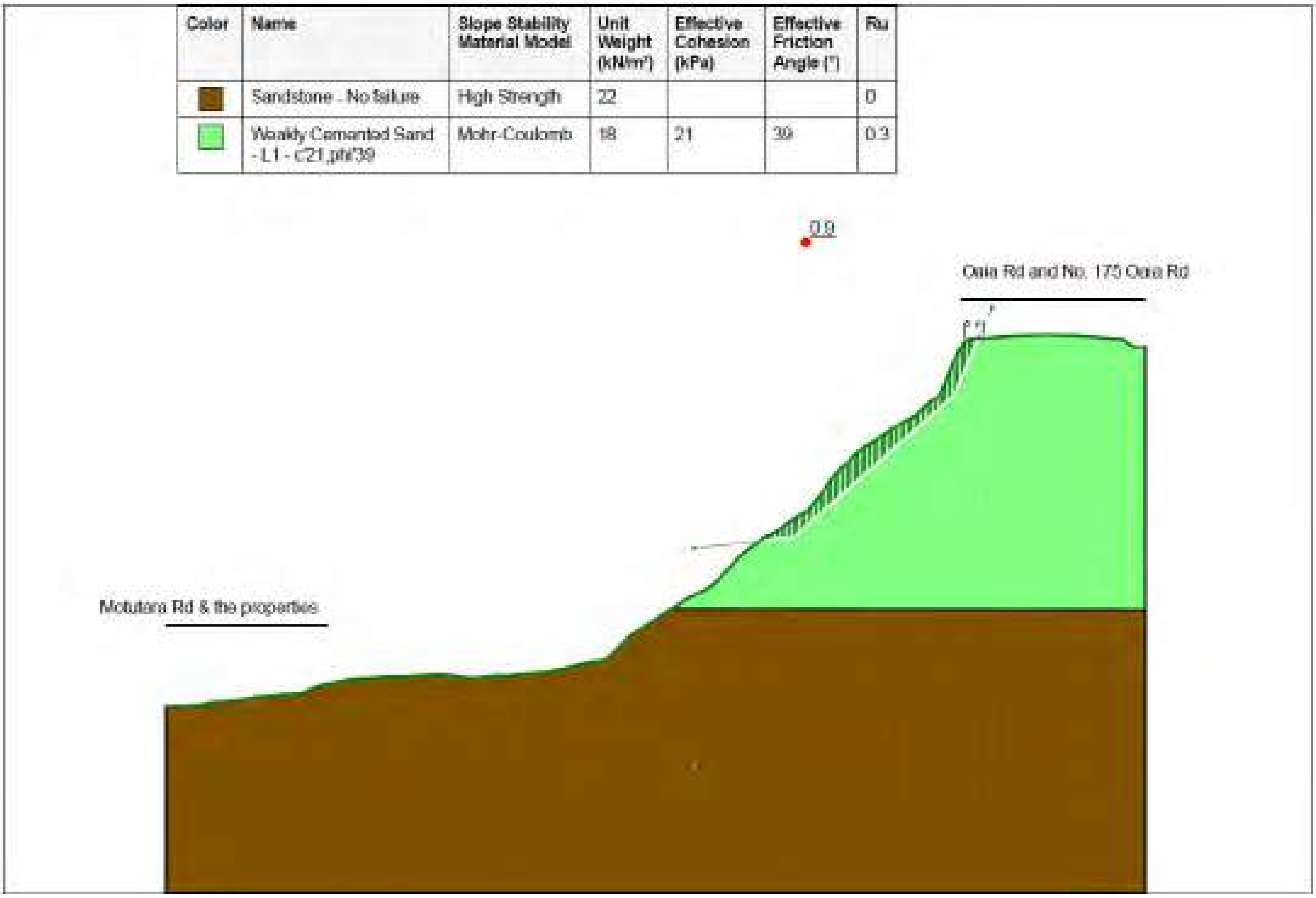

Figure C(A)-4. Nail Option - Extreme Groundwater

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru
	Sandstone - No failure	High Strength	22			0
	Weakly Cemented Sand - L1 - c'21.phi'39	Mohr-Coulomb	18	21	39	0.1

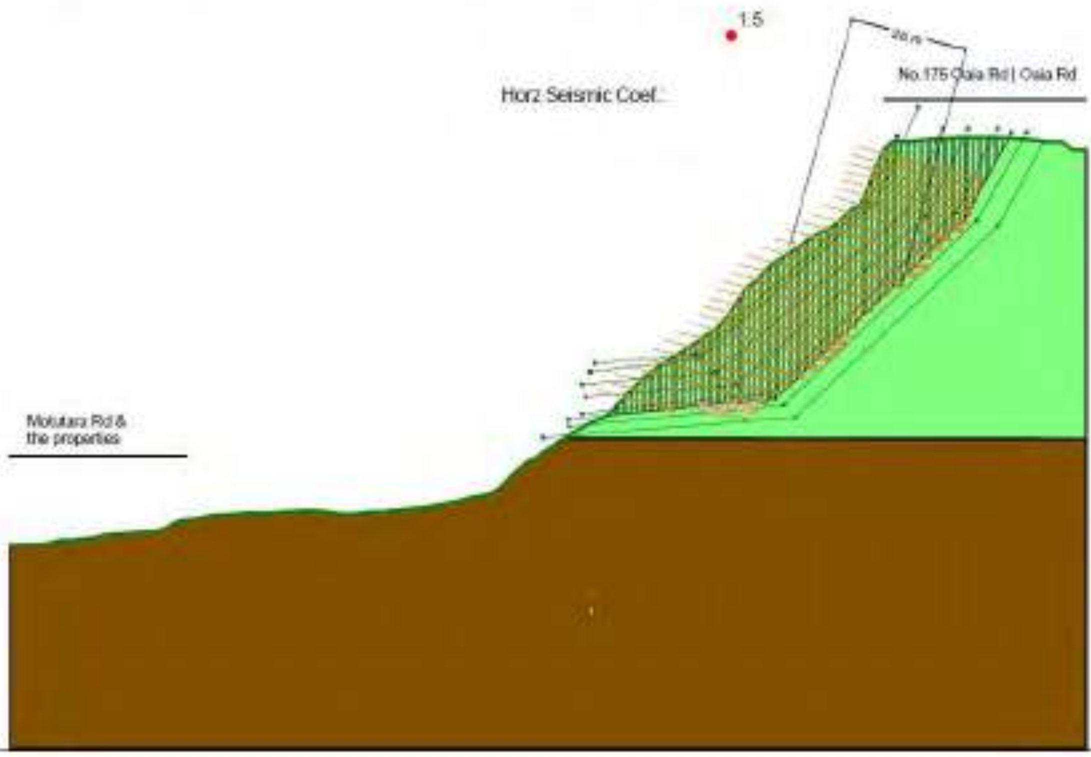

Figure C(A)-5. Nail Option - Nail Length (Static Case)

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru
	Sandstone - No failure	High Strength	22			0
	Weakly Cemented Sand - L1 - c'21.phi'39	Mohr-Coulomb	18	21	39	0.1

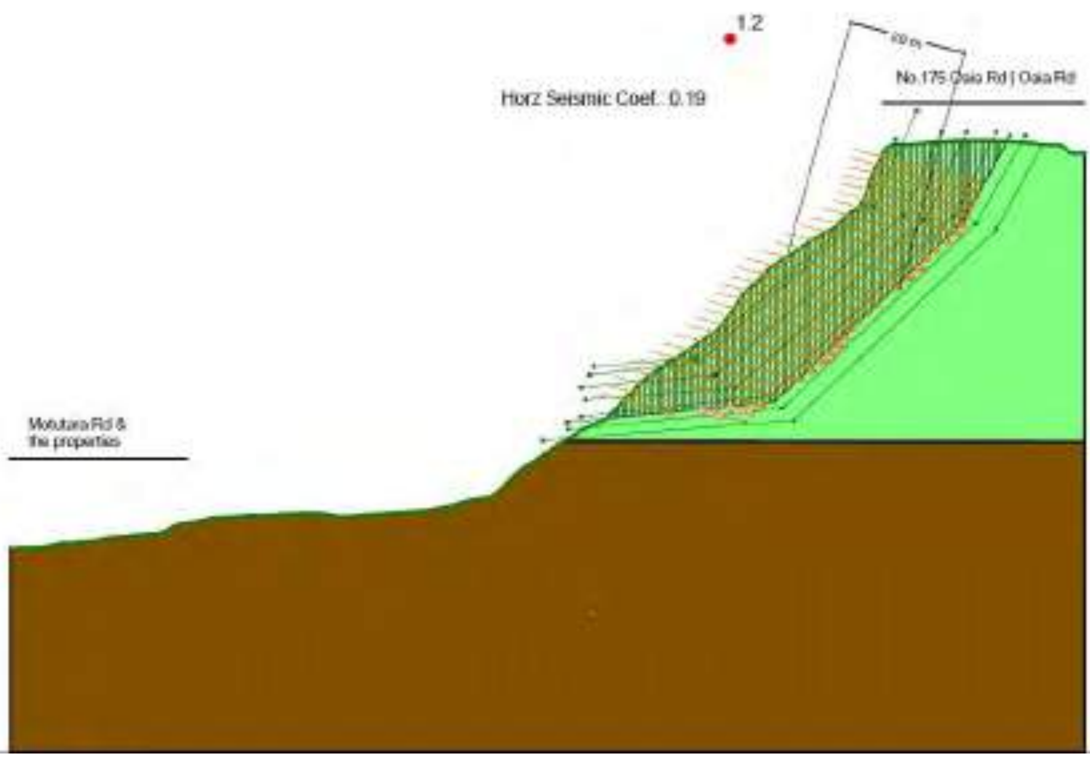

Figure C(A)-6. Nail Option - Nail Length (Seismic Case)

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru
	Sandstone - No failure	High Strength	22			0
	Weakly Cemented Sand - L1 - c'21 phi'39	Mohr-Coulomb	18	21	39	0.03

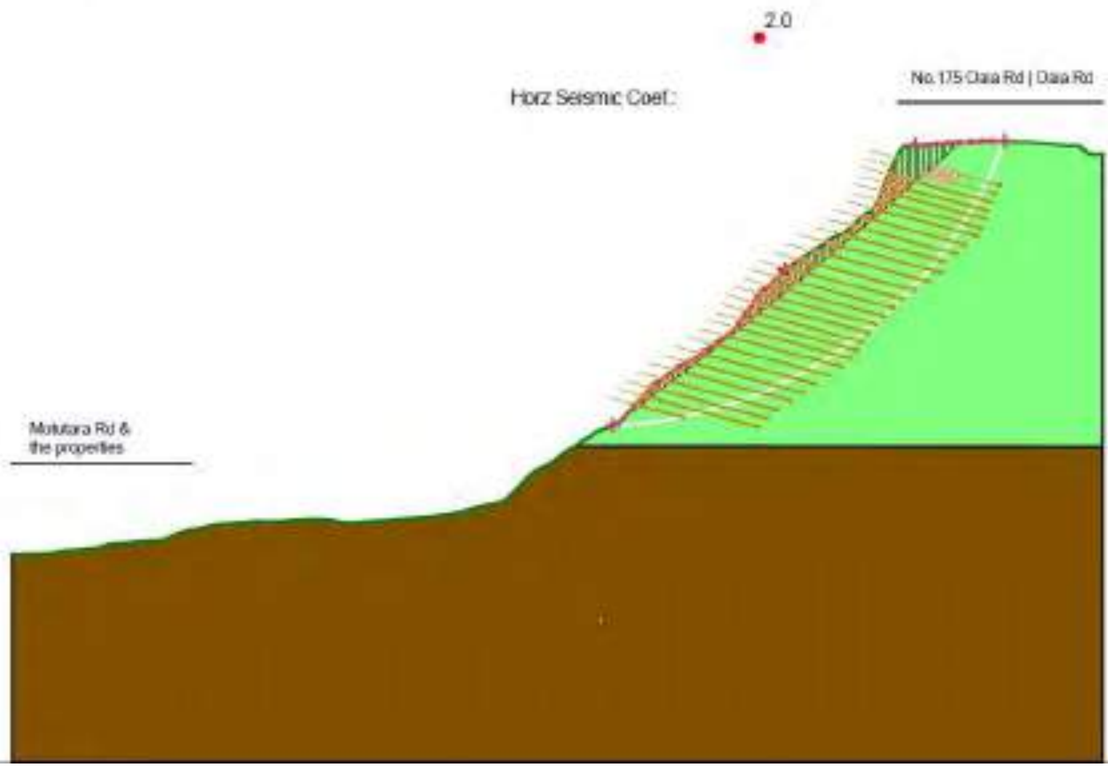

Figure C(A)-7. Nail Option - Shallow Failure Surface (Static Case)

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru
	Sandstone - No failure	High Strength	22			0
	Weakly Cemented Sand - L1 - c'21 phi'39	Mohr-Coulomb	18	21	39	0.03

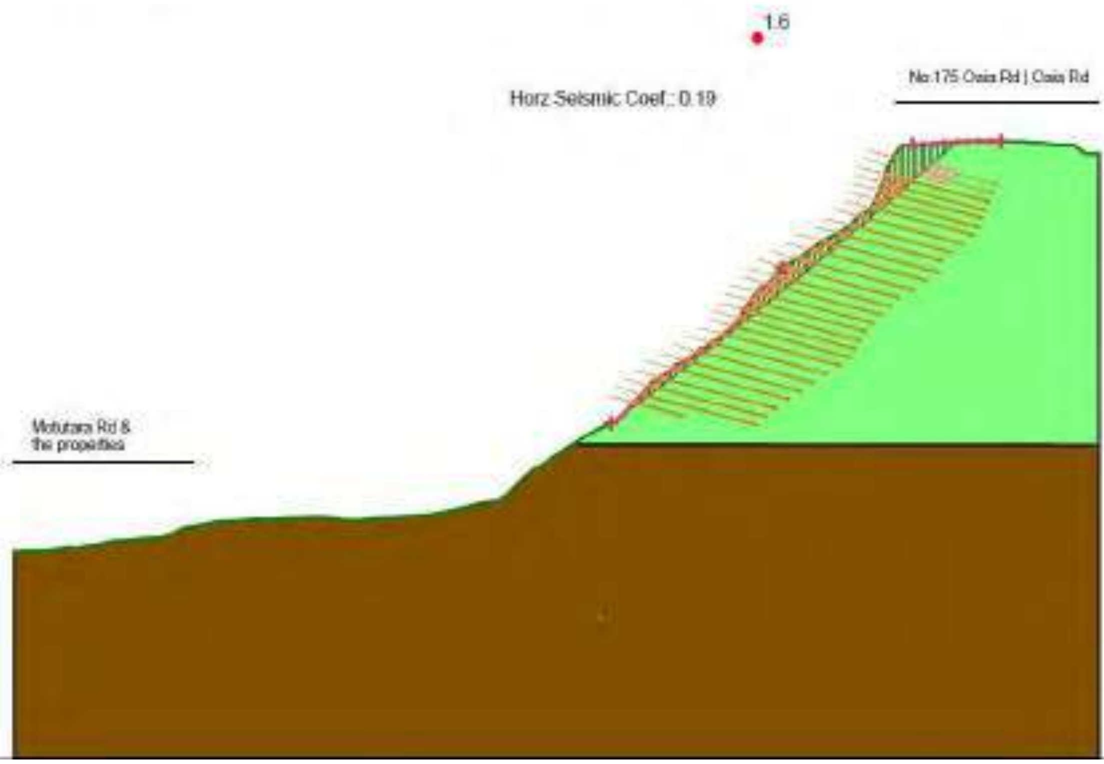

Figure C(A)-8. Nail Option - Shallow Failure Surface (Seismic Case)

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru
	Sandstone - No failure	High Strength	22			0
	Weakly Cemented Sand - L1 - c'21 phi'39	Mohr-Coulomb	18	21	39	0.03

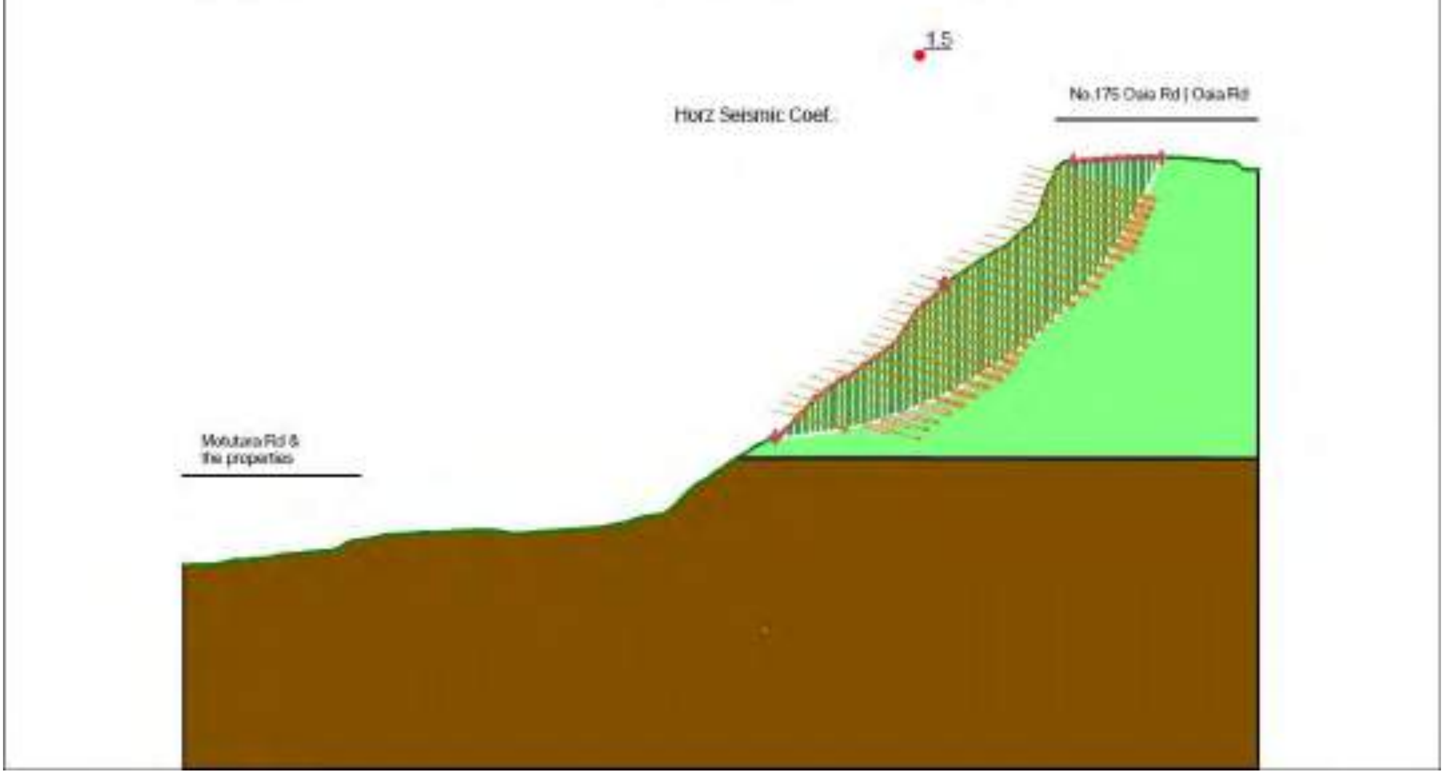

Figure C(A)-9. Nail Option - Deep Failure Surface (Static Case)

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru
	Sandstone - No failure	High Strength	22			0
	Weakly Cemented Sand - L1 - c'21 phi'39	Mohr-Coulomb	18	21	39	0.03

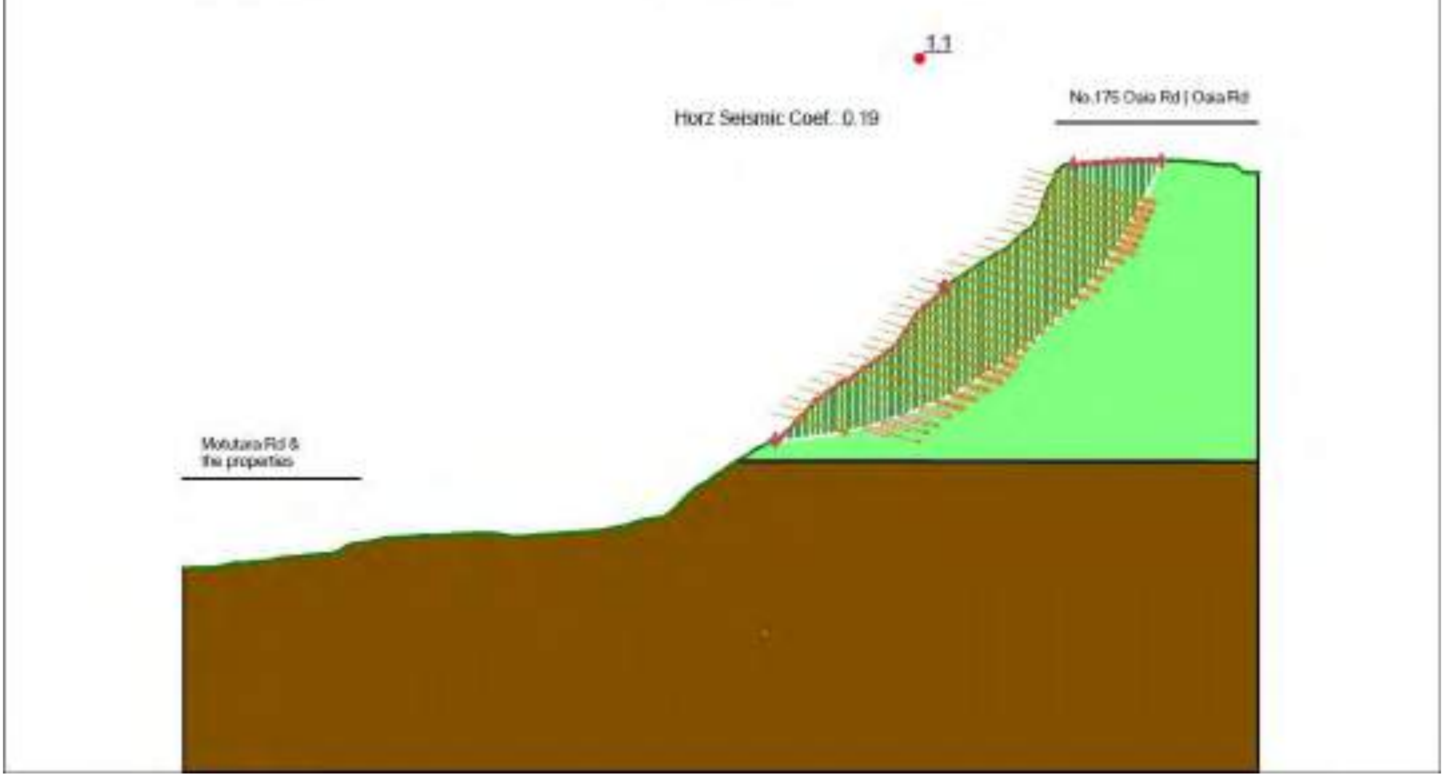

Figure C(A)-10. Nail Option - Deep Failure Surface (Seismic Case)

Color	Name	Slope Stability Material Model	Unit Weight (kN/m^3)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru
	Sandstone - No failure	High Strength	22			0
	Weakly Cemented Sand - L1 - c21,phi'39	Mohr-Coulomb	18	21	39	0.025

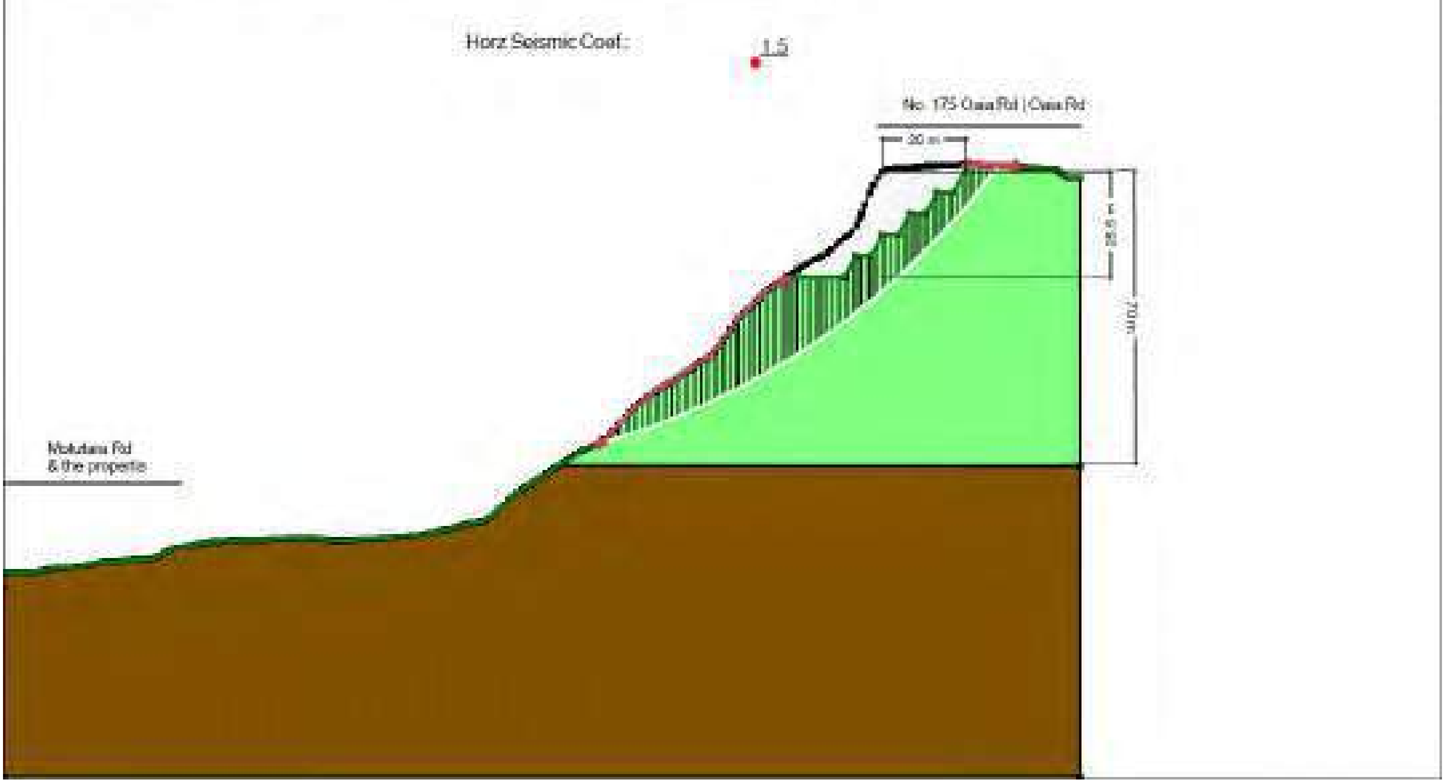

Figure C(A)-11. Bench Option - Static Case

Color	Name	Slope Stability Material Model	Unit Weight (kN/m^3)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Ru
	Sandstone - No failure	High Strength	22			0
	Weakly Cemented Sand - L1 - c21,phi'39	Mohr-Coulomb	18	21	39	0.025

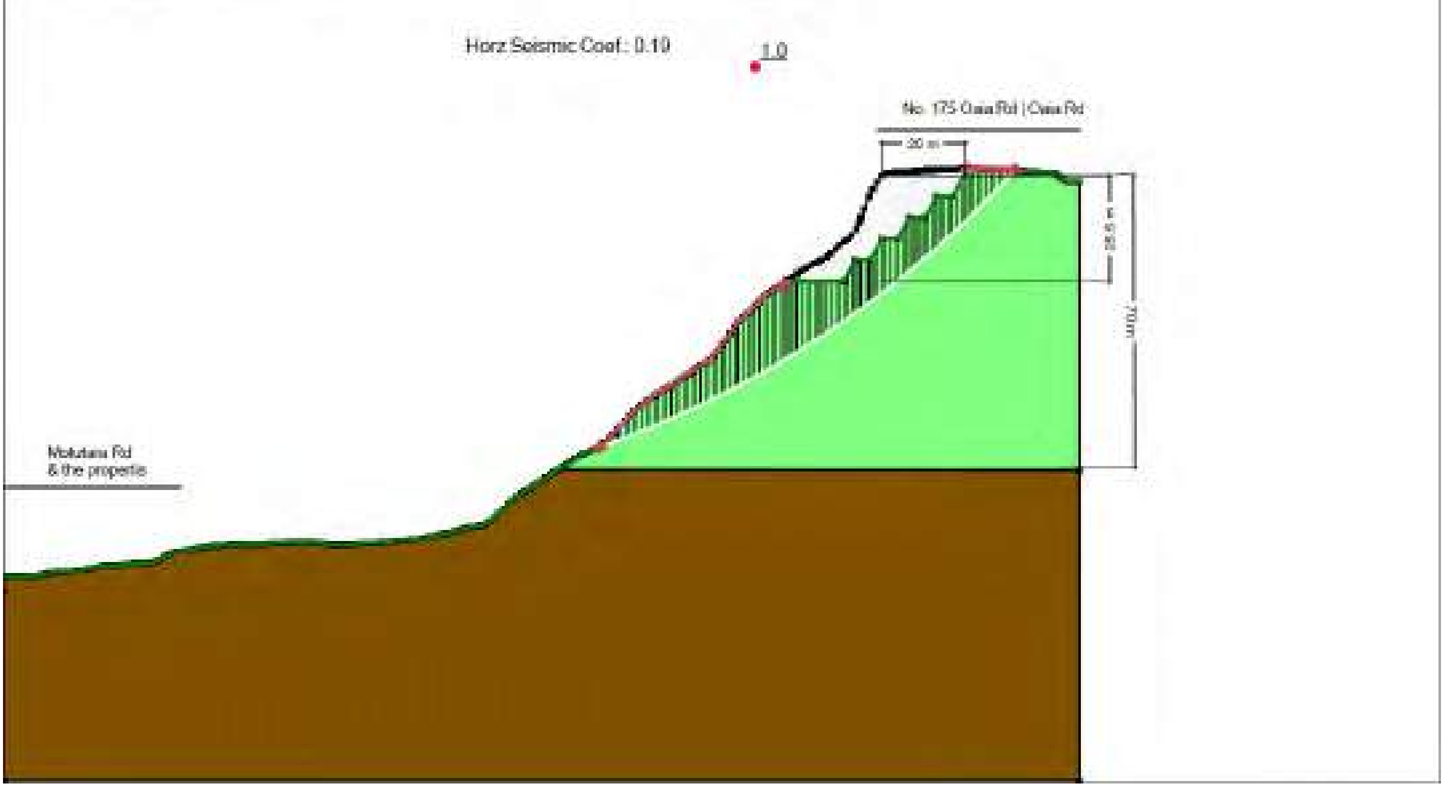

Figure C(A)-12. Bench Option - Seismic Case ghd.com

→ The Power of Commitment