Waste Futures – Green Island Landfill Closure

Liquefaction and Stability Assessment

Dunedin City Council

20 January 2023

→ **The Power of Commitment**

Project name	GILF Closure Consents
Document title	Waste Futures – Green Island Landfill Closure | Liquefaction and Stability Assessment
Project number	12547621
File name	Green Island Landfill Liquefaction and Stability Assessment revA.docx

Status Code	**Revision**	**Author**	**Reviewer**		**Approved for issue**		
			Name	**Signature**	**Name**	**Signature**	**Date**
S3	Rev01	K Tang	Debbie Fellows		Stephen Douglass		21/12/22
S4	Rev02	K Tang	Debbie Fellows		Stephen Douglass		20/01/23
S4	Rev03	K Tang	Debbie Fellows		Stephen Douglass		20/02/23

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1. Introduction

1.1 Project overview

As part of Dunedin's wider commitment to reducing carbon emissions and reducing waste going to landfill, the Dunedin City Council (Council) has embarked on the Waste Futures Programme to develop an improved comprehensive waste management and diverted material system for Ōtepoti Dunedin. The Waste Futures Programme includes the roll out of an enhanced kerbside recycling and waste collection service for the city from July 2024. The new service will include collection of food and green waste.

To support the implementation of the new kerbside collection service, the DCC are planning to make changes to the use of Green Island landfill site (Figure 1) in coming years.

The proposed changes include:

- planning for the closure of the Green Island landfill, which is coming to the end of its operational life
- developing an improved Resource Recovery Park (RRPP) to process recycling, and food and green waste
- providing new waste transfer facilities to service a new Class 1 landfill currently planned for a site south of Dunedin, at Smooth Hill.

The resource consents for the new Smooth Hill landfill are subject to appeal. Depending on the outcome of this appeal process, and the time needed to undertake baseline monitoring, preparation of management plans, landfill and supporting infrastructure design and construction, DCC anticipate that the new Class I landfill facility, won't be able to accept waste until 2027/2028 at the earliest.

In the interim, DCC therefore plans to continue to use Green Island landfill for waste disposal. Based on Dunedin's current waste disposal rates, it is likely that that the Green Island landfill can keep accepting waste for another six years (until about 2029). Between now and then, and as it continues to fill up, the landfill will be closed and capped in stages. When the landfill closes completely, there will be opportunities for environmental enhancements and public recreational use around the edge of the site. Examples could be planting restoration projects and new walking and biking tracks beside the Kaikorai Estuary. Long term use and public access to the landfill site post closure will be determined in consultation with Te Rūnanga o Ōtākou, the local community and key stakeholders.

As current Otago Regional Council resource consents needed to operate a landfill at Green Island expire in October 2023, the DCC are now applying to ORC for replacement resource consents to continue to use the landfill until it closes completely, and waste disposal can be transferred to a new landfill facility. The replacement consents relate to ground disturbance, flood defence and discharges to land, water, and air. The site is subject to an operative designation (D658) in the Proposed Second-Generation Dunedin City District Plan (2GP) for the purpose of Landfilling and Associated Refuse Processing Operations and Activities.

The development of the new RRPP and waste transfer facilities at Green Island does not form part of the replacement consent applications. Resource consents for the development and operation of the RRPP will be applied for following the completion of design work and technical assessments later in 2023.

GHD Limited (GHD) has been engaged by the Council to provide engineering services to support the closure of the Landfill. This includes providing geotechnical services to advance the understanding of ground conditions and provide a geotechnical assessment of the slope responses to earthquake. The scope of assessment is presented in Section 1.2. As part of the scope, GHD has completed a geotechnical investigation, which consists of seven Cone Penetration Test (CPTs) and 12 machine boreholes. The information collected from the investigation is presented in (GHD Limited, 2023b), which shall be read in conjunction with this report.

A site plan showing the extent of the Landfill is presented in Figure 1.

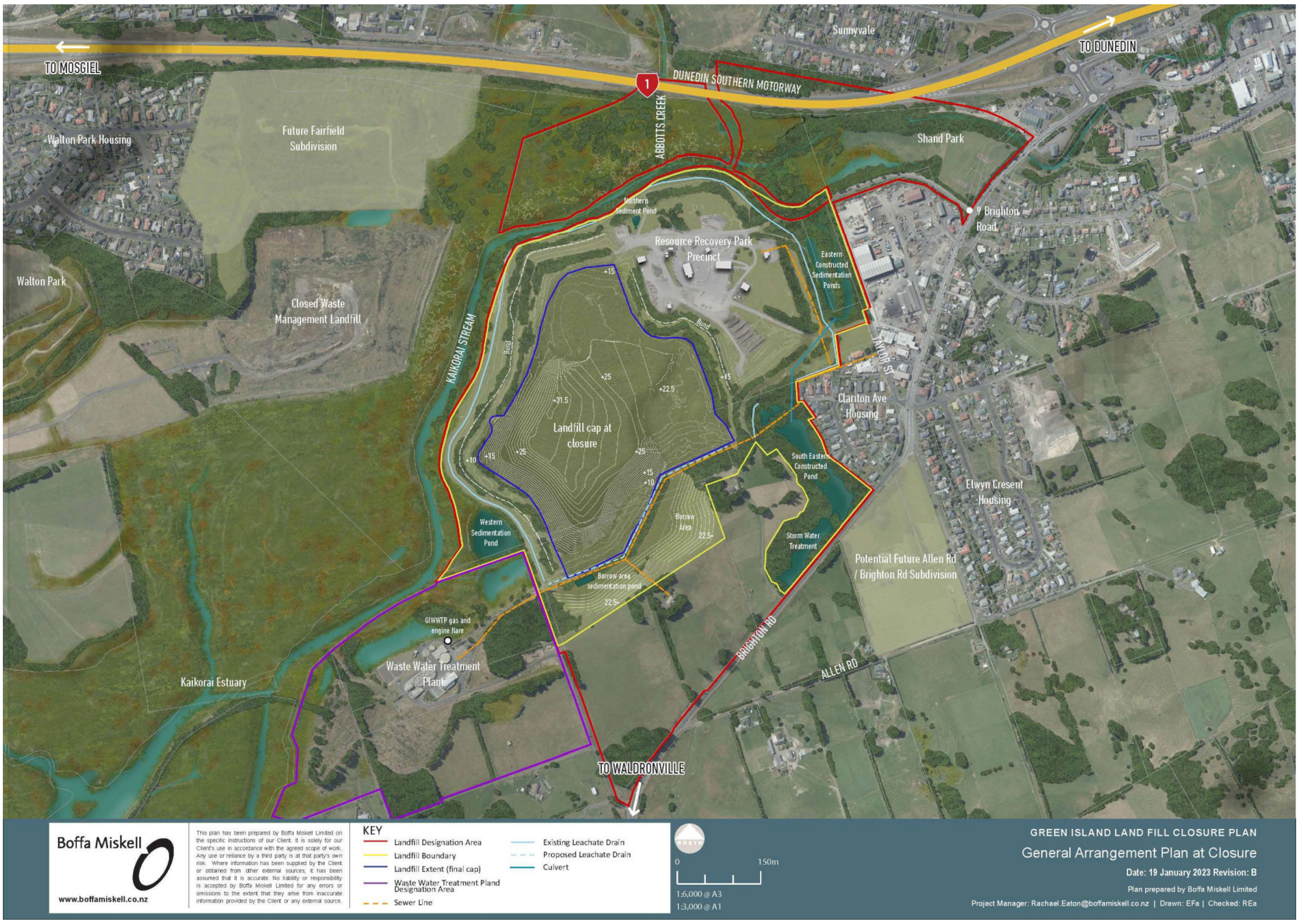

Figure 1 Landfill Layout and Boundaries

1.2 Scope of assessment

The scope of this assessment was limited to:

- Seismic hazard assessment to provide site specific consideration of the seismic hazard at the Landfill site through development of a Probabilistic Seismic Hazard Assessment (PSHA). This work was undertaken by Professor Mark Stirling and Govinda Niroula from the University of Otago on behalf of Dunedin City Council.
- Liquefaction susceptibility assessment of the underlying natural soils using the 2022 geotechnical investigation data.
- Slope stability and lateral spreading assessment of the preferred closure design . Five cross sections were developed and analysed for static, elevated groundwater, Serviceability Limit State (SLS) and Ultimate Limit State (ULS) seismic and post-earthquake conditions.
- Size of likely displacements if slopes are unstable.
- Sludge area assessment – assessing the influence of areas of sludge within the landfill on stability at a single cross section.

1.3 Purpose of this report

The purpose of this report is to:

- provide an assessment of the liquefaction potential of the natural soils underlying the Green Island landfill;
- provide an assessment of the long-term slope stability (post closure) of the preferred closure option landfill footprint; and

- document the potential and quantum of slope displacements (if appropriate).

This is a Technical Assessment Report designed to provide assessment for the design of the preferred closure option and to support the March 2023 resource consent application.

1.4 Limitations

This reporthas been prepared by GHD for Dunedin City Council and may only be used and relied on by Dunedin City Council for the purpose agreed between GHD and Dunedin City Council as set out in Section 1 of this report. GHD otherwise disclaims responsibility to any person other than Dunedin City Council and Council officers, consultants, the hearings panel and submitters associated with the resource consent and notice of requirement process for the Green Island Landfill Closure Project arising in connection with this report.

GHD also excludes implied warranties and conditions, to the extent legally permissible. The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report. The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report.

GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared. The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report. GHD disclaims liability arising from any of the assumptions being incorrect.

GHD has prepared this report on the basis of information provided by Dunedin City Council and others who provided information to GHD (including Government authorities)], which GHD has not independently verified or checked beyond the agreed scope of work. GHD does not accept liability in connection with such unverified information, including errors and omissions in the report which were caused by errors or omissions in that information. The opinions, conclusions and any recommendations in this report are based on information obtained from, and testing undertaken at or in connection with, specific sample points. Site conditions at other parts of the site may be different from the site conditions found at the specific sample points.

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

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

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

1.5 Assumptions

- Soil, rock and waste design parameters have been derived based on the available geotechnical information for the waste at the site from the recent GHD investigation and data supplied by the Council, available laboratory testing results and published literatures and papers including empirical correlations which are referenced throughout the report.
- For the development of the ground models for this technical assessment we have assumed the following:-
  - Design parameters based on the available information on the waste at the site, available in situ testing and published literatures and papers including empirical correlations.
  - A linear soil layer profile in the slope models..

2. Site description

2.1 Site location and environs

The Green Island Landfill site is located approximately 8.8 km by road from Central Dunedin. The landfill site comprises a total area of 75.6 ha, being the total area of the landholding owned by Council and designated in the Proposed Dunedin City District Plan (2GP), as shown outlined in. Primary access to the site is via Brighton Road..

The site is generally bound by State Highway 1 to the north, the Kaikorai Stream and Estuary to the west, the Green Island Wastewater Treatment Plant (GIWWTP) to the south-west, and Brighton Road to the south. The Clariton Avenue residential area and Green Island industrial area are to the east.

The Clariton Avenue residential area comprises the closest residential properties to the landfill, being approximately 200 m southeast of the existing waste transfer and resource recovery facilities, and 140 m east of the current landfill footprint. The Council is also proposing to rezone a block land between Weir Street and Brighton Road, south of Clariton Avenue, to a General Residential Zone enabling low-medium density residential living.

Other residential properties are located to the southeast at Elwyn Crescent, and to the north and west within Sunnyvale and Fairfield. Those residential properties are located at greater distances and separated from the landfill site by a combination of the State Highway 1 corridor, the Kaikorai Stream and Estuary, and rural and open space land. An area of undeveloped land zoned General Residential exists within Fairfield, accessed from Walton Park Avenue.

The margins of the Kaikorai Stream and Estuary bordering the landfill to the north and west are identified as a Regionally Significant Wetland in the Regional Plan: Water; and an Area of Significant Biodiversity Value, and a Wāhi Tupuna of cultural significance to mana whenua in the 2GP. Low lying areas around the stream and estuary are also identified as being within a Hazard 2 Flood overlay at risk of flooding in the 2GP.

2.2 Topography and geomorphology

The landfill and associated operations are located in the upper (northeast) part of the Kaikorai Estuary, immediately to the east of Kaikorai Stream. Kaikorai Stream flows into the estuary approximately 400 m southwest of the site. Prior to landfill development the site would have been characterised by low lying (1 -2 m above sea level) estuary flats and wetland. Immediately to the east of the landfill the land rises gently to a series of low hills.

2.3 Geology

The geology underlying the landfill area comprises sediments of estuarine origin underlain by Abbotsford Formation mudstone. The estuarine sediments, described as Kaikorai Estuary Formation (KEF), are likely to be approximately 11 m thick in the landfill area according to (Barry J Douglas Geological Consultants, 2002). The KEF was divided into upper and lower layers (members), with the upper member being further divided into two subgroups (refer Figure 2).

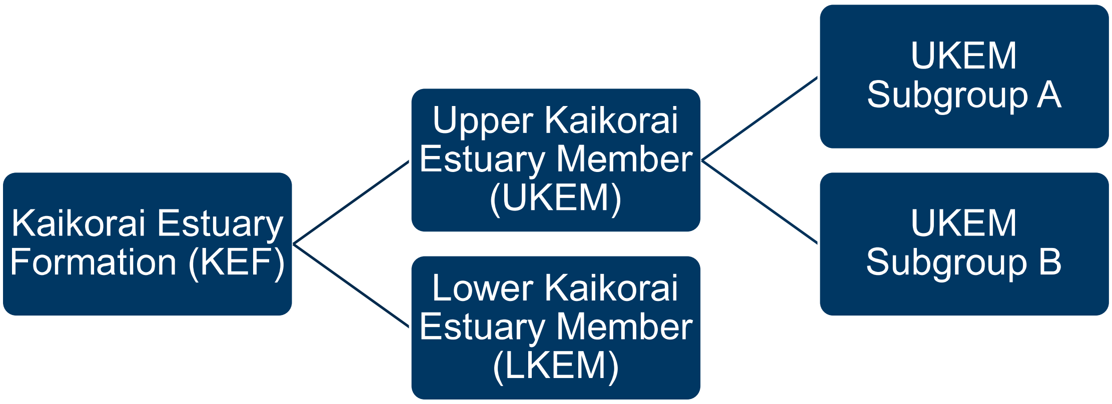

Figure 2 Lithological sequence mapping schematic

Table 1 provides a summary of the lithological units as described in (Barry J Douglas Geological Consultants, 2002).

Table 1 Description of lithological units (Barry J Douglas Geological Consultants, 2002)

Member	Description	Subgroup	Thickness
Upper Kaikorai Estuary Member (UKEM)	Variable thin beds of sand, silty sand, sandy silt, silt, clayey silt and silty clay	Subgroup A -mostly homogeneous fine grained	Approximately 4.5 m
		Subgroup B – heterogeneous, coarser grain size	
Lower Kaikorai Estuary Member (LKEM)	Massive homogeneous beds of clayey silt, silty clay and silt, and minor (possibly localised) beds of clay, very fine sandy silt and silty very fine sand.	-	Approximately 6.5 m

The recent intrusive investigations (GHD 2023B) did not provide any clear geotechnical differentiation to support the subdivision of the UKEM into two subgroups. As a result, for this assessment, the subgroups have not been utilised in the ground model development. The investigations did allow for clear geotechnical differentiation between the UKEM and the LKEM units.

2.4 Current leachate collection and environmental monitoring system

In 1995, a leachate interception trench and collection system consisting of nine pump stations interconnected via a gravel-filled trench with an inbuilt perforated collector drain located on the perimeter of the landfill was retrofitted around the majority of the perimeter of the landfill. A network of groundwater/leachate monitoring wells was installed in a series of lines crossing perpendicular to the interception trench. The location of the leachate interception trench, collection system and monitoring wells are shown on Figure 3. The interception trench allows for the leachate to be collected and discharged to the GIWWTP, which is located to the southwest of the landfill. There is currently a gap in the leachate collection trench along the south-eastern boundary of the landfill.

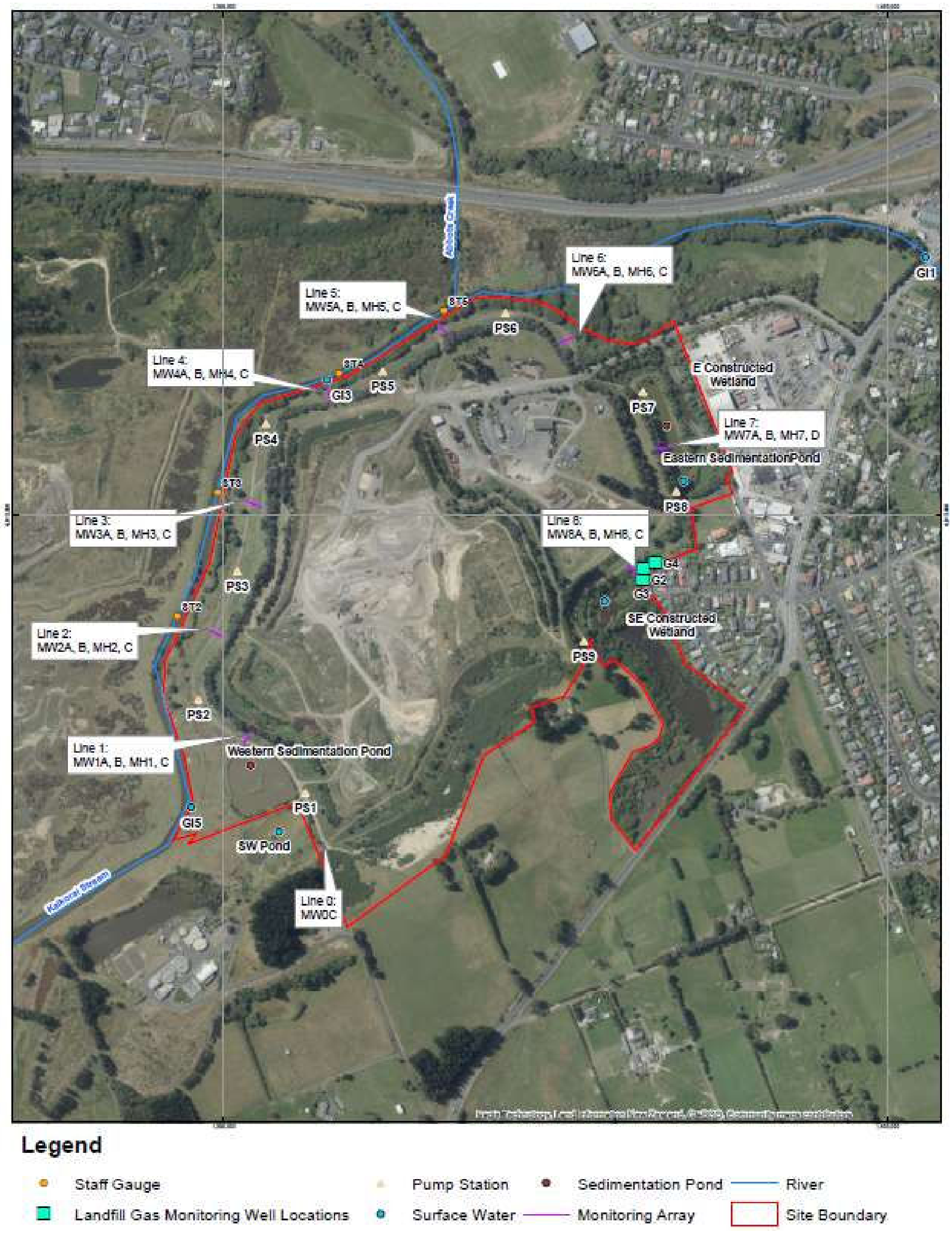

Figure 3 ***Monitoring well locations within the KEF***

A schematic figure of the leachate interception trench is provided in Figure 4.

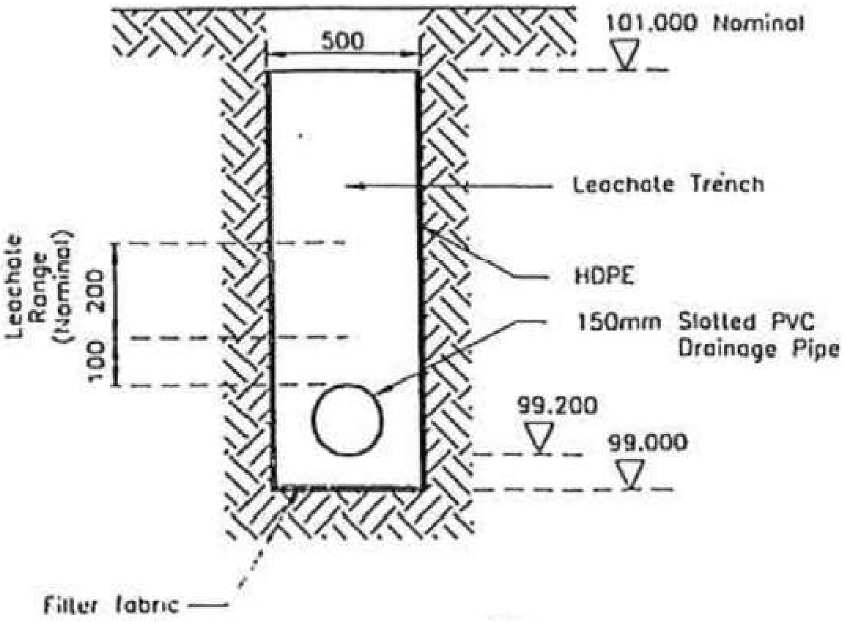

Figure 4 Leachate interception trench schematic figure (City Consultants Engineers and Surveyors, 1997)

In addition, leachate collection drains have been installed within the landfill, with additional drains proposed for future stages, to manage the mounding of leachate within the main body of the landfill. The locations of the current and future leachate drains are documented in the *Green Island Landfill Closure Design Report* (GHD Limited, 2023c).

Further works are proposed as part of this consent application include:

- extension of the existing leachate collection trench along the southern boundary;
- continued installation of lateral drains within the landfill waste; and
- extraction of leachate directly from waste via a series of pumps retro-fitted to the Landfill Gas extraction system.

These proposed additional measures are also described in detail in the *Green Island Landfill Closure Design Report* (GHD Limited, 2023c).

An understanding of the current and proposed leachate levels is fully documented in the *Groundwater Technical Assessment* (GHD Limited, 2023a). These leachate levels have been used in the stability models.

2.5 Current landfill gas collection system

(GHD Limited, 2021a) states that a landfill gas extraction system has been installed within the landfill. Figure 5 shows the layout of the gas-field network in July 2021, with some of these wells intercepting leachate. Several gas wells were dipped to measure leachate levels in August 2022. Table 2 summarises these measurement results which are presented as metres above mean sea level (m amsl).

Table 2 August 2022 gas well leachate level

Wells	Level at base of well (m amsl)	Level of Leachate (m amsl)
GW22	8.0	9.6
GW21	8.5	10.4
GW17	6.4	16.7
GW13	11.7	17.7
GW14	12.2	22.2
GW15	-8.3	21.8
Leach Riser	15.4	16.3
GW9	14.1	14.1
GW36	12.2	14.2
GW37	16.1	19.2
GW26	15.4	16.7
GW11	-2.0	9.2
GW7	3.5	8.8
GW1	8.7	10.5
Piez1	2.5	6.6
Gas Compound	4.3	6.3
Piez2	2.1	5.4
Piez3	5.5	8.7
Piez4	3.8	8.2

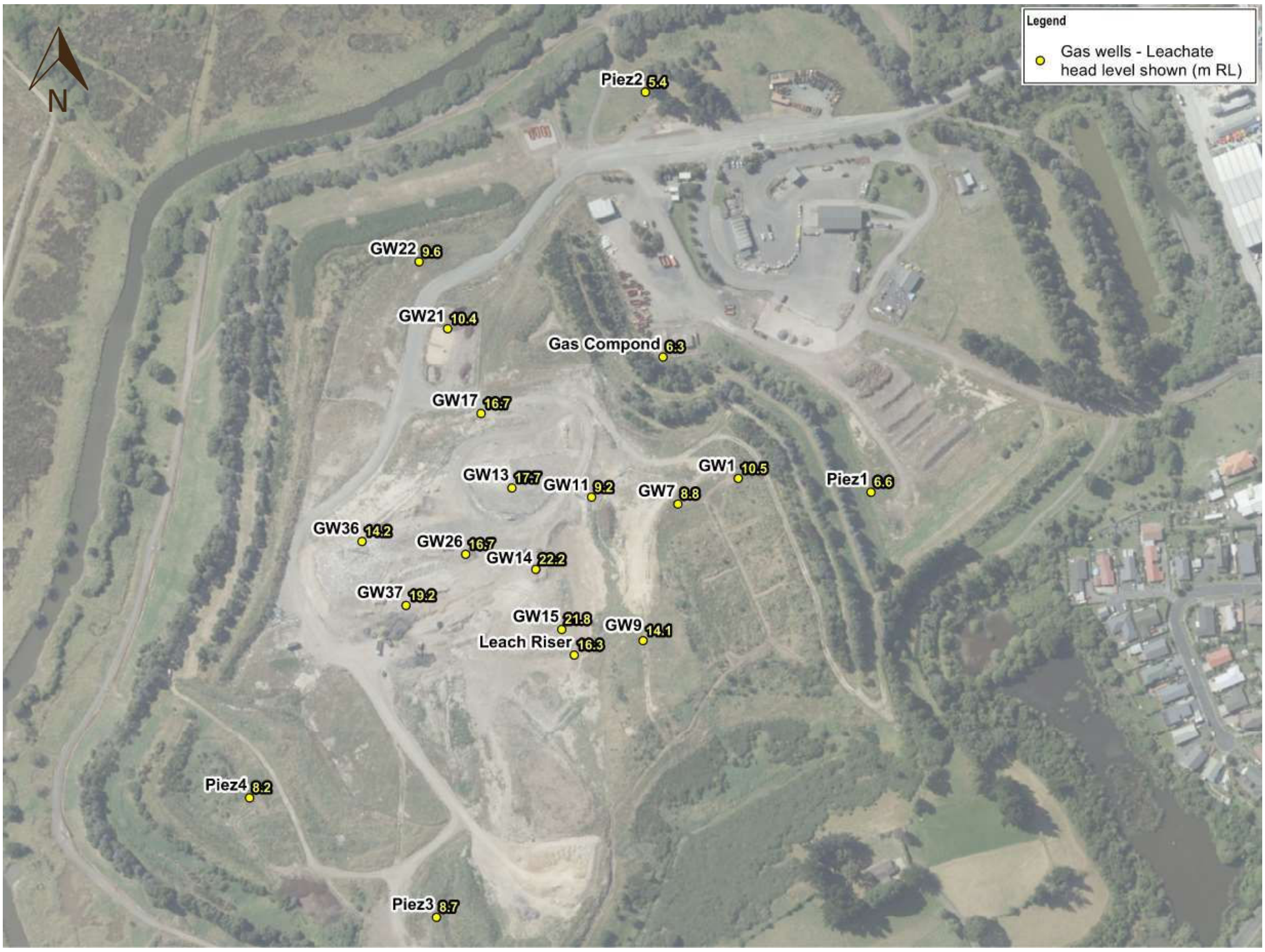

Figure 5 *Landfill gas well plan*

2.6 Hydrogeology

The KEF forms a shallow water bearing strata under the landfill and surrounding area with groundwater levels close to the ground surface. Permeability testing has been completed during the recent GHD site investigation and is reported in the *Groundwater Technical Assessment* Report (GHD Limited, 2023a). The permeability testing results reflect the heterogeneity created by the depositional environment. The testing results also indicate that the shallower deposits exhibited a higher permeability than the wells screened across the lower formation. These findings are consistent with findings from testing of three bore holes at various depths by (Beca Steven Consulting Environmental Engineers, 1992).

Table 3 provides a summary of water levels measured on 13 July 2022 within the KEF from monitoring wells in the vicinity of the landfill (see Figure 3). The water levels were measured shortly after a heavy rainfall event and as such, represented a conservative scenario. A schematic of the monitoring well lines is shown in Figure 6. Note that the water levels in the wells are influenced by leachate trench pumping. The data demonstrates variability with depth below ground level, but the general pattern is an increasing water level with increasing depth below ground level, including three of the deepest wells (screened between 5 and 8 m below ground level) exhibiting artesian conditions and an upward hydraulic vertical gradient. The groundwater system is described in more details in the *Groundwater Technical Assessment* (GHD 2023A).

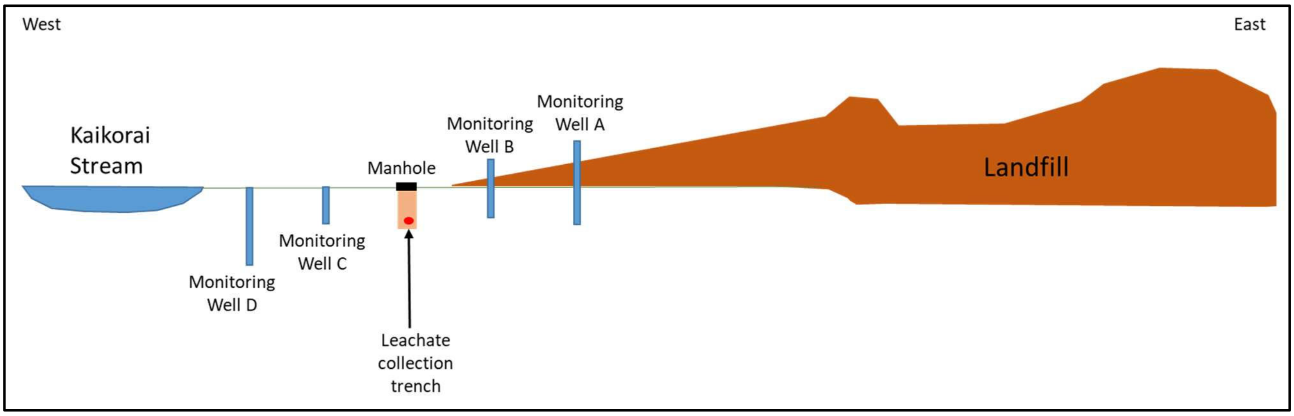

Figure 6 *Schematic of monitoring well transect*

Table 3 *Summary of water levels within the KEF measured on 13 July 2022*

Well name	Depth of well (m)	Base of well (m amsl) – 2004	Upstand height (m above ground level)	SWL 13 July 2022 (m below top of PVC casing)	Static water level (m) – adjusted for upstand height	Aquifer
MW0C	3.87	-2.35	-0.03	0.97	0.94 (below ground)	Upper
MW1C	5.70	-2.17	1.50	0.94	0.56 (above ground)	Upper
MW2C	5.11	-2.31	0.23	0.62	0.39 (below ground)	Upper
MW2D	10.67	-7.49	0.35	0.52	0.18 (below ground)	Lower
MW3C	4.07	-1.58	0.37	1.61	1.24 (below ground)	Upper
MW4C	4.86	-1.72	0.21	1.17	0.97 (below ground)	Upper
MW4D	8.18	-7.51	2.09	1.74	0.35 (above ground)	Lower
MW5C	4.82	-1.45	0.62	1.10	0.47 (below ground)	Upper
MW6C	5.04	-1.64	0.47	0.47	0.0 (ground level)	Upper
MW7D	5.23	-3.84	0.16	0.97	0.81 (below ground)	Unknown
MW8C	3.97	-1.66	0.02	1.70	1.68 (below ground)	Upper

3. Ground model development

3.1 Available geological and geotechnical data

3.1.1 2022 GHD investigation data

GHD completed a geotechnical investigation between 17 October 2022 and 11 November 2022, comprising:

- Seven Cone Penetration Tests (CPTs) to a depth of 15 m below ground level (bgl) or refusal.
- Six machine boreholes, with standpipe piezometer installed, to a depth of 15 mbgl or 1 m into the underlying mudstone.
- Six machine boreholes to a depth of 20 mbgl.

A location plan showing the machine borehole and CPT locations is shown in Appendix A. Various soil samples from the machine boreholes were collected and tested for particle size distribution and Atterberg Limit. The laboratory report and investigation data are presented in *2022 Geotechnical Investigation Factual Report* (GHD Limited, 2023b).

3.1.2 Previous investigation data

A review of the following reports was undertaken to obtain the available existing investigation data:

- Beca Steven Consulting Environmental Engineers (1992). *Environmental Impact Assessment of the Green Island Sanitary Landfill.*
- Barry J Douglas Geological Consultants (2002). *Green Island Landfill Leachate Collection Trench Geological Report. Vol 1 and 2*
- Tonkin & Taylor Limited (2019). *Green Island Landfill – Bund Stability Assessment.*
- GHD Limited (2021). *Green Island Landfill Capping Design – Design Report.*
- Tonkin & Taylor Limited (2021). *Green Island Landfill – Perimeter Bund Assessment.*

Locations of the selected investigation data are shown in Appendix A.

3.2 Ground models

The ground encountered in the geotechnical investigation around the landfill perimeter comprised soil fill (internal perimeter bund) over KEF (UKEM and LKEM), underlain by Abbotsford Mudstone. The encountered geology is generally consistent with the reported geological information (Barry J Douglas Geological Consultants, 2002). Table 4 shows a summary of the encountered ground model at the toe of the landfill, which is derived from the geotechnical investigation data available near the landfill toe. Plans showing interpreted elevation contours for the top of the KEF and Abbotsford Formation mudstone are presented in Appendix B. These plans have been developed based on the available geotechnical data, which are at discrete locations. There is some uncertainly around the top of KEF and mudstone due to core loss. As such, these plans are indicative only and should not be relied on for design without further validation where relevant.

Table 4 ***Encountered engineering geological units (excludes BH105, BH106, BH107, BH109, and BH110 as they are not located within the landfill toe)***

Geological unit	General description	Depth to top of unit (mbgl)	Thickness (m)	SPT N_{60}	Shear vane peak reading (kPa)	Cone resistance, q_c (MPa)
Bund	Silty, sand, and clay with MSW and wood fragments	0.00	1.3 – 13.5	0 – 50+ (average = 14)	N/A	0.1 – 22.1

Geological unit	General description	Depth to top of unit (mbgl)	Thickness (m)	SPT N_{60}	Shear vane peak reading (kPa)	Cone resistance, q_c (MPa)
UKEM	Sandy silt with minor to some clay and trace to some organics	1.3 – 5.5 (not encountered in BH104 and BH108)	0 – 3.2	1 – 5 (average = 2)	9 – 49 (average = 27)	0.1 – 3.2
LKEM	Silt, sand, and clay with pockets of organic, trace seashell	3.95 – 13.5 (not encountered in BH104)	0 – 8.55	0 – 9 (average = 2)	10 – 63 (average = 35)	0.1 – 1.9
Abbotsford mudstone	Weathered mudstone or siltstone extremely to very weak	4.5 – 16.2	unproven	8 – 50+ (average = 34)	123 (single reading only)	2.3 – 49.1

Six geological cross sections were generated around the perimeter of the landfill and used for the slope stability assessment. The cross sections were selected as they represented a range of internal landfill structures, which vary across the site and include general changes in fill characteristics and final fill height, and account for the different thickness of underlying estuary sediments. The alignment of the cross sections is presented in Figure 7.

Figure 7 Cross section locations

A summary of the conditions at each section location is presented below:

- Section 1 – Located on the North-western margin of the landfill adjacent to the Kaikorai Stream. The landfill has three levels of internal perimeter bunding on the front face. A leachate collection trench is present at the toe of the fill.
- Section 2 and 3 – Located on the West and southwestern margin of the landfill adjacent to the Kaikorai Stream. The landfill has two levels of internal perimeter bunding on the front face. Section 2 also includes the extent of the existing sludge disposal area. Section 3 is adjacent to the stormwater detention pond. A leachate collection trench is present at the toe of the fill for both sections.
- Section 4 - Located on the southern margin of the landfill. The landfill has no known internal perimeter bunding on the front face. There is currently no leachate collection trench at the toe of the fill. Only limited investigation data (BH 108) exists to support the ground model at this location. It is assumed that there are minimal estuarine sediments in this area and the weathered Abbotsford Mudstone is present to the immediate south (see Appendix B contours).
- Section 5 - Located on the eastern margin of the landfill. The landfill has three levels of internal perimeter bunding on the front face of the fill. A leachate collection trench is present at the toe of the fill and a stormwater pond is immediately adjacent to the toe of the fill. We have assumed the estuary sediment is relatively thin and we have estimated the pond depth.
- Section 6 - Located on the southern margin of the landfill which intercepts the existing asbestos disposal area. We have conservatively applied the sludge design properties to the asbestos disposal area. There is no known internal perimeter bunding on the front face of the existing fill. There is currently no leachate collection trench at the toe of the fill. The ground model has been developed based on extrapolated data from nearby investigations. It is assumed that there are minimal estuarine sediments in this area and the weathered Abbotsford Mudstone is present to the immediate south.

The geological cross sections are shown in Slope/W outputs presented in Appendix E. The dimensions of the internal perimeter bunds were derived based on the bund details provided previously in (City Consultants Engineers and Surveyors, 1999 and Tonkin & Taylor Ltd, 2021). The dimensions of the existing sludge disposal area and asbestos disposal area were derived based on the information provided by the Council. We note that future sludge and biosolids disposal will be integrated within layers of the normal landfill waste and no (or very limited) further development of the sludge pits are likely. In future it will be lime stabilised, mixed with the general waste (i.e. 10% sludge; 90% of general waste). The existing sludge placement areas have been modelled to understand the sensitivity of the stability to the existing sludge placement to date.

The geotechnical design parameters were adopted in the slope stability assessment are described in Table 5. These parameters were derived based on the available geotechnical investigation data, laboratory test results, literature review and/or our past local experiences.

Table 5 Geotechnical design parameters

Unit	Unit weight (kN/m^3)	Effective friction angle (°)	Effective cohesion (kPa)	Undrained shear strength (kPa)	Liquefied shear strength ratio[2]
Bund	17	27	1	75	-
UKEM	16	26	0	-	0.08
LKEM	15.5	24	0	15kPa + 0.23*effective overburden stress[1]	-
Weathered Abbotsford mudstone	18	32	10	200	-
Waste (Fill)	14.5	25	3	-	-
Final capping	17	29	2	100	-
Sludge/biosolids	13	24	0	-	-

Notes:

[1] Undrained shear strength for the LKEM was derived based on (Skempton, 1957) where $S_u/\sigma'_v = 0.11 + 0.0037PI$. PI is the plasticity index which was derived from the laboratory test results.

[2] The liquefied shear strength ratio was derived from the software CLiq as part of the liquefaction assessment. The details of the liquefaction assessment are documented in Section 5.

3.3 Groundwater and leachate levels

The groundwater and leachate levels shown on the ground models were estimated based on the available data collected from the monitoring wells around the landfill toe (as documented in Section 2.6) and the gas wells installed within the landfill (as documented in Section 2.5). The details on both monitoring systems are documented in (GHD Limited, 2023a).

Two groundwater/leachate level design scenarios have been considered: long term groundwater/leachate level and elevated groundwater/leachate level. For each groundwater/leachate level design scenario, three piezometric surface (piezo) lines have been modelled. The definition and description for each piezo line is provided in Table 6.

Table 6 Piezo line definition and description

Piezo line number	Applicable unit	Description
1	Bund, fill, final capping, sludge	For the long-term leachate level, we assume that both the current and future drains installed within the landfill will be functioning, capping of the landfill will reduce infiltration and pumping from LFG wells will contribute to managing leachate within the landfill. With these changes it has been assumed that the leachate level within the landfill will be lowered to approximately the level of the internal drains. . The drain levels adopted in the model vary from 10 m amsl to 13.5 m amsl. The drain levels to be constructed will vary across the site from 11 m amsl to 14 m amsl. On average a long-term leachate level of 12 m amsl has been assumed. For the elevated leachate case, we assume that the drains within the landfill will be out of action temporarily, which allows the build-up of the leachate within the landfill. An elevated leachate level of 16 m amsl is adopted in the slope assessment, which is considered to be conservative compared to the groundwater models documented in the *Groundwater Technical Assessment Report* (GHD Limited, 2022a).
2	UKEM	To simplify the assessment, we assume that the groundwater level within the natural deposit (i.e. KEF) is constant for both the long term and elevated design cases. This is considered to be appropriate as the groundwater level measured within the KEF has been relatively consistent based on the monitoring data available to date. The KEF groundwater readings shown in Table 3 were adopted in the ground model.
3	LKEM	

4. Seismic hazard

The design ground motion for both SLS and Ultimate Limit State ULS have been derived in accordance with the Earthquake Geotechnical Engineering Practice Module 1 (Ministry of Business, Innovation, and Employment & New Zealand Geotechnical Society, 2021).

Method 1 stated in Ministry of Business, Innovation and Employment & New Zealand Geotechnical Society, 2021 provides estimates of hazard parameters based on the generic probabilistic seismic hazard analysis (PSHA). It is stated that the values provided in the module are applicable only for routine geotechnical engineering projects until a compressive update of the National Seismic Hazard Model (NSHM) is completed.

Given that the revised NSHM was not available at the commencement of this project and the potential environmental impact if the site fails, Method 2 (site-specific PSHA) was recommended and adopted. Professor Mark Stirling was engaged to undertake a site specific PSHA, which considered the nearby known faults in the derivation of the hazard parameters. The PSHA report is included in Appendix C.

In summary, the fault source model includes all seismic sources within 200 km of the site. The faults in the more immediate area of Green Island Landfill are shown on Figure 8. The Akatore is the closest, most active fault to the site and the Green Island Fault is the continuation of the Akatore Fault. While the Kaikorai Fault is shown as the closest fault to the site, it is not considered a potential seismic source of great importance to the project because of its very long estimated recurrence interval. Likewise, the Titri Fault shows considerably lower rates of activity then the Akatore Fault. The results of the PHSA are summarised in Table 7.

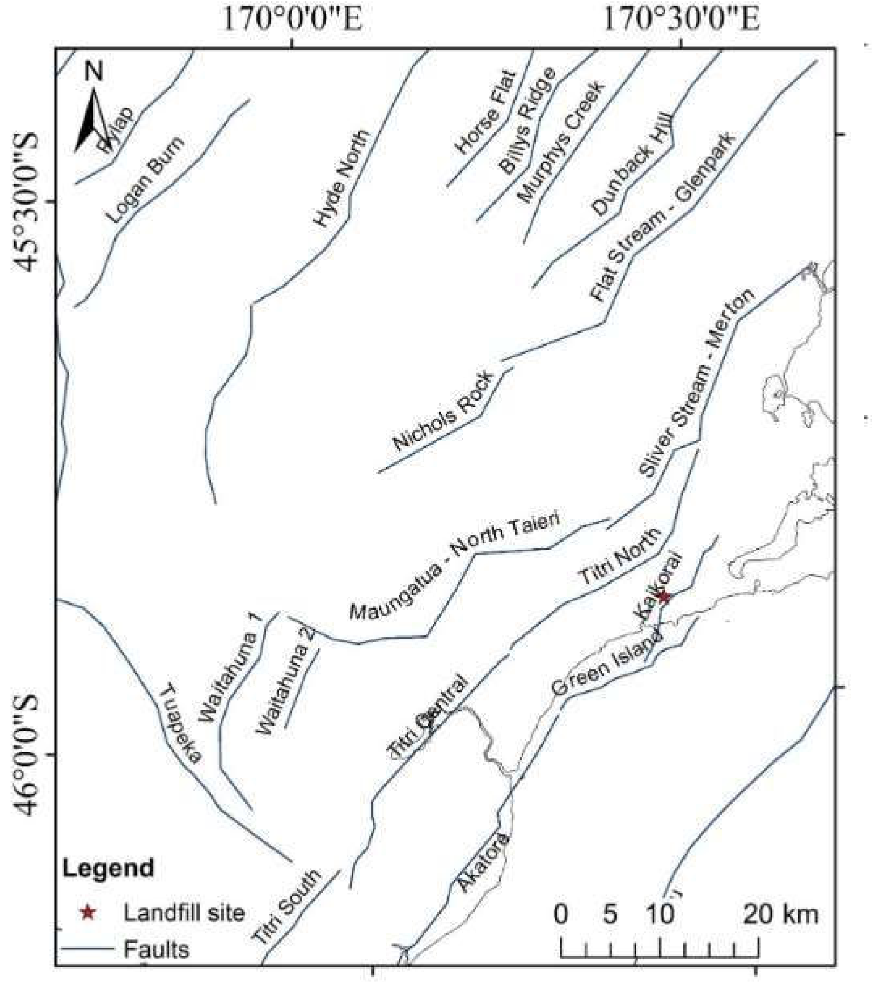

Figure 8 *Nearby faults*

The new NSHM was announced part way through this project by GNS in October 2022 (GNS Science, 2022), which gives slightly higher values for both SLS and ULS design seismic parameters. As such, the project has chosen to take a conservative approach and adopted these in the design to account for the hazard uncertainty. It is noted that current interim industry guidance on the NSHM indicates that the NSHM has been developed to inform design standards and actions. It is not a design standard or design action standard and should not be used as such. A review of the current relevant NZ design standards is underway by the industry, and it is not expected that advice for engineers, in a format that can be applied to engineering projects, will be available for at least another year. A summary of the seismic design inputs and design criteria are summarised in Table 7.

Table 7 Seismic criteria

Item	Description	Reference
Design life	100 years	Council's requirement
Importance level	IL3	NZS1170.0 Table 3.2 (Standards Australia and Standards New Zealand, 2002)
Return period	SLS – 1/25 ULS – 1/2500	NZS1170.0 Table 3.3 (Standards Australia and Standards New Zealand, 2002)
Site subsoil class	Class C	Derived based on the available geological information
Design PGA and Magnitude	**NSHM (adopted parameters)** **SLS – 0.06g; 6.0 (M_w)** **ULS – 0.51g; 7.3 (M_w)** PSHA SLS – 0.01g; 6.0 (M_w) ULS – 0.47g; 7.3 (M_w)	**(GNS Science, 2022)** (Stirling & Niroula, 2022)

5. Liquefaction assessment

5.1 Methodology

An assessment of the likelihood of liquefaction has been conducted for the site in accordance with the recommendations provided in the Earthquake Geotechnical Engineering Practice Module 3 (Ministry of Business, Innovation, and Employment & New Zealand Geotechnical Society, 2021). The module recommends the adoption of the Boulanger and Idriss (2014) CPT and SPT based liquefaction triggering assessment procedure. The CPT and SPT based liquefaction assessment was carried out with CLiq v.3.4.1.4 ((Geologismiki, 2022).

A review of the laboratory test results (particle size distribution and Atterberg limit) was also undertaken to quantify the liquefaction potential of each geological unit.

5.2 Results

The liquefaction triggering assessment was completed using the 2022 GHD investigation data, which consists of laboratory results, SPT and CPT data.

The liquefaction assessment results indicate that liquefaction is not anticipated at the site under SLS seismic event for all soil layers. However, under ULS seismic event, liquefaction is likely to occur within the UKEM layer which demonstrates sand like behaviour. The results are summarised in Table 8 with CLiq output summary plots provided in Appendix D.

Table 8 Liquefaction triggering

Unit	Soil behaviour	SLS liquefaction triggering risk	ULS Liquefaction triggering risk
Bund	Clay like (PI>12)	Low	Low
UKEM	Generally sand like (PI<12) with occasional thin lenses of clay like layers (PI>12)	Low	High
LKEM	Generally clay like (PI>12) with occasional thin lenses of sand like layers (PI<12)	Low	Low
Weathered Abbotsford Mudstone	Clay like (PI>12)	Low	Low

Free field settlements as a result of liquefaction are not anticipated to be uniform across the site and along the existing leachate drain. Anticipated free field settlements based on each CPT data is presented in Table 9. Differential settlements of drains and other infrastructure within the site may occur, particularly where the liquefied layers are located within the foundation zone of influence. The proposed approach to mitigate these and other seismic related stability issues is discussed in the *Design Report* (2023c). Potential remedial options are provided in the design report (GHD Limited, 2023c).

Table 9 Estimated liquefaction induced free field settlement

CPT No.	CPT termination depth (m)	SLS free field settlement (mm)	ULS free field settlement (mm)
CPT100	14.80	Negligible	25
CPT101	15.04	Negligible	30
CPT102	13.84	Negligible	15
CPT103	16.53	Negligible	35
CPT104	11.81	Negligible	10
CPT105	13.17	Negligible	5
CPT108	15.00	Negligible	35

6. Slope stability

6.1 Design criteria

The recommended minimum slope stability factors of safety (FoS - defined as the available shear strength dividing by the net driving force) for limited equilibrium stability analysis are provided in Table 10. There are no NZ guidelines that document the required load cases and target factors of safety for landfill stability but the target factors of safety are consistent with the New Zealand current practice. Similar design targets were adopted for the Smooth Hill Landfill and other consented landfill sites. Where the calculated seismic FoS is less than 1.0, slope displacement criteria apply.

(Seed & Bonaparte, 1992) report a permanent seismic displacement no greater than 150 to 300 mm is typically used in practice for the design of geosynthetic liner systems in the US. For cover systems, where permanent seismic deformations may be observed in post-earthquake inspections and damage to components can be repaired, larger permanent deformations may be considered acceptable. As the landfill is unlined but capped, the acceptable displacements for SLS and ULS events are considered to be <0.3 m and <1.0 m, respectively. These values have been selected based on GHD's past experience from similar sites and they are consistent with the current New Zealand design practice. Slope displacements are estimated using the four methods detailed in the NZ transport Authority Bridge Manual (Waka Kotahi NZ Transport Agency, 2022) based on 50th percentile:

Methods valid for estimates of displacement without liquefaction and lateral spreading:

- Ambraseys and Menu
- Ambraseys and Srbulov
- Jibson

The method valid for estimates of displacement where liquefaction and lateral spreading is present is:

- Bray and Travasarou (Superseded by Bray, Macedo and Travasarou 2018 for subduction zone earthquakes and Bray and Macedo for shallow crustal earthquakes)

Table 10 Slope stability load cases and design criteria

Load case	Description	Target FoS
Static – long term groundwater level – local	Static case with long term groundwater and leachate levels modelled. Drained soil parameters to be adopted. Slip zones to be limited within the landfill.	≥1.5
Static – long term groundwater level – global	Static case with long term groundwater and leachate levels modelled. Drained soil parameters to be adopted. Slip zones to be extended to the leachate drain or to the nearest free face, which is further.	≥1.5
Static – elevated groundwater level – local	Static case with elevated groundwater and leachate levels modelled. Drained soil parameters to be adopted. Slip zones to be limited within the landfill.	≥1.2
Static – elevated groundwater level – global	Static case with elevated groundwater and leachate levels modelled. Drained soil parameters to be adopted. Slip zones to be extended to the leachate drain or to the nearest free face, which is further.	≥1.2
Seismic – SLS – non liquefied	Seismic SLS (0.06g; 6.0 M_w) case with long term groundwater and leachate levels modelled. Bund, final capping, LKEM and weathered mudstone units to adopt the undrained parameters.	≥1.0 or displacement based (<0.3 m displacement). Displacement shall be estimated based on Ambraseys and Menu, Ambraseys and Srbulov, and Jibson methods.

Load case	Description	Target FoS
	Drained parameters for the remaining units.	
Seismic – ULS – non liquefied	Seismic ULS (0.51g; 7.3 M_w) case with long term groundwater and leachate levels modelled. Bund, final capping, LKEM and weathered mudstone units to adopt the undrained parameters. Drained parameters for the remaining units. This load case is only valid when liquefaction is not anticipated.	≥1.0 or displacement based (<1.0 m displacement). Displacement shall be estimated based on Ambraseys and Menu, Ambraseys and Srbulov, and Jibson methods.
Post-earthquake – flow failure	Immediately post-earthquake – static case with long term groundwater and leachate levels modelled. Bund, final capping, LKEM and weathered mudstone units to adopt the undrained parameters. UKEM unit to adopt the liquefied soil strength. Drained parameters for the remaining units. This load case is only valid when liquefaction is anticipated.	≥1.05
Seismic – ULS – liquefied	Seismic ULS (0.51g; 7.3 M_w) case with long term groundwater and leachate levels modelled. Bund, final capping, LKEM and weathered mudstone units to adopt the undrained parameters. Drained parameters for the remaining units. This load case is only valid when liquefaction and lateral spreading are anticipated and when the FoS for post-earthquake – flow failure is greater than 1.05.	≥1.0 or displacement based (<1.0 m displacement). Displacement shall be estimated based on Bray and Macedo method.

6.2 Stability modelling – Preferred Closure Design

Slope stability assessment have been undertaken using limit equilibrium slope stability analysis based on Morgenstern-Price (1967) method of slices' approach which considers both force and moment equilibrium. Commercial software, GeoStudio Slope/W (Geoslope International Ltd, 2021) was used for this analysis.

Six cross sections have been generated around the perimeter of the landfill to understand the stability of site based on the landfill closure landform. Locations of the cross sections are shown in Figure 7.

The outcomes from the stability assessment are summarised in Table 11. Slope/W outputs are provided in Appendix E.

Table 11 Slope Stability Summary – Critical FoS

Load case	Target FoS	Critical slope stability FoS					
		Section 1	Section 2	Section 3	Section 4	Section 5	Section 6
Static – long term groundwater level – local (run 01)	≥1.5	3.7	2.1	2.1	2.0	4.3	1.9
Static – long term groundwater level – global (run 02)	≥1.5	1.5	1.8	1.8	1.5	1.4	1.5
Static – elevated groundwater level – local (run 03)	≥1.2	3.2	1.5	1.6	1.6	3.5	1.6

Load case	Target FoS	Critical slope stability FoS					
		Section 1	Section 2	Section 3	Section 4	Section 5	Section 6
Static – elevated groundwater level – global (run 04)	≥1.2	1.4	1.3	1.3	1.3	1.4	1.3
Seismic – SLS – non liquefied (run 06)	≥1.0[1]	<1.0	<1.0	1.0	1.1	1.6	<1.0
Seismic – ULS – non liquefied (run 08)	≥1.0[1]	<1.0	<1.0	<1.0	<1.0	<1.0	<1.0
Post-earthquake – flow failure (run 10)	≥1.05	1.1	1.2	1.1	1.4[2]	2.1[2]	1.1

Notes:

[1] For the seismic design cases, if FoS < 1.0, displacement-based assessment shall be relied on, the estimated displacements are presented in Section 6.3.

[2] Liquefiable layers not anticipated along the section.

6.3 Seismic slope displacement

In accordance with the design criteria set for this project, when the seismic FoS is less than 1.0, displacement analysis based on the four methods documented in Section 6.1 was carried out. When the FoS is greater than 1.0, slope displacement is not anticipated. Based on the stability assessment, Sections 1, 2, and 6 do not meet the required FOS for the SLS and all sections do not meet the required FOS for the ULS. As stated in Table 10, where the results that do not meet the required FoS a displacement based design approach has been adopted. Table 12 summarises the calculated slope displacements for the SLS and ULS. As shown in the table below, although the FoS is less than 1.0, the anticipated slope displacements do not exceed the maximum allowable displacement provided in Table 10. Further discussion is provided in Section 8.

Table 12 Calculated slope displacement

Load case	Slope displacement limit (mm)	Anticipated slope displacement (mm)					
		Section 1	Section 2	Section 3	Section 4	Section 5	Section 6
Seismic – SLS – non liquefied	<300	<5	<5	-	-	-	<5
Seismic – ULS – non liquefied	<1,000	N/A[1]	N/A[1]	N/A[1]	205 – 325	35 – 75	N/A[1]

Note:

[1] Liquefaction likely triggered during ULS seismic design event, which may result in lateral spreading. Refer to Table 13 for lateral spreading assessment outcomes.

7. Lateral spreading

Lateral spreading is the permanent ground displacements (i.e. cracks, fissures, vertical offsets and slumping of the ground) due to the presence of liquefied soils. It typically occurs in sloping ground and are especially prevalent near to free faces, such as waterways.

Post-earthquake flow failure type movement is not anticipated based on slope assessment.

Lateral spreading assessment has been conducted in accordance with NZ Bridge Manual (Waka Kotahi NZ Transport Agency, 2022). Yield accelerations were estimated from the limit equilibrium software, Slope/W (Geoslope International Ltd, 2021), assuming peak soil strengths, the lateral spread movement was then calculated using the Bray and Macedo method based on the 50th percentile displacement. A shear wave velocity of 120 m/s has been adopted for the sliding mass based on our interpretation from the CPT data and a review of the available literatures on MSW shear wave velocity estimation for a multilayer soil profile.

Table 13 Estimated lateral spread based on earthquake size.

Load case	Slope displacement limit (mm)	Anticipated slope displacement (mm)					
		Section 1	Section 2	Section 3	Section 4[1]	Section 5[1]	Section 6
Seismic – ULS – liquefied	<1,000	680	630	270	N/A	N/A	930
Note: [1] No liquefiable unit identified along the cross section.							

The extent of the zone affected by lateral spreading could be impacted by various factors, such as liquefaction characteristics of the critical layer, location of the liquefiable soils within the profile in relation to the free face, lateral continuity of critical layers, overall deposit characteristics, river geometry, site topography, ground motion characteristics, and other. Therefore, uncertainty exists regarding the possible extent of any zone. Based on the observations from the 2010-2011 Canterbury earthquakes, lateral spreading typically affects a zone of up to approximately 150 m to 200 m from the riverbank edge (Ministry of Business, Innovation, and Employment & New Zealand Geotechnical Society, 2021). It is also noted that the displacement is typically spread over a large area and does not always occur along one or two discrete failure surfaces.

8. Discussion

A series of geotechnical assessments have been carried out to estimate the performance of the Green Island Landfill for the proposed final landform under both static and seismic conditions. This includes liquefaction, slope stability and lateral spreading assessment. The key finding is that under the ULS seismic event, portions of the landfill will deform by differing amounts. The predicted deformations are presented in Table 14 and summarised below. Note the implications for the landfill from these findings and the proposed remedial measures are described in the *Design Report (GHD 2023c)*:

Table 14 Summary of total expected seismic deformations for the ULS event (excluding liquefaction settlement)

Description	Section 1	Section 2	Section 3	Section 4	Section 5	Section 6
Anticipated slope displacement (mm)	680	630	270	205 – 325	35 – 75	930

- **Liquefaction:** Results from the liquefaction assessment indicate that majority of natural soils underlying the Green Island landfill are not liquefiable under both SLS and ULS seismic events. However, some layers in the UKEM geological unit that exhibit sand-like behaviour are likely to undergo liquefaction under ULS seismic event. Liquefaction induced free field settlement was estimated for both SLS and ULS seismic design cases. No free field settlement is anticipated under SLS; up to 35 mm of free field settlement is likely under ULS. Differential settlements of drains and other infrastructure within the site may occur, particularly where the liquefied layers are located within the foundation zone of influence. Given that the reported free field settlement is reasonably small, the impact on the landfill and other infrastructure at the site is likely to be minimal.
- **Slope stability and lateral spreading:** Six cross sections for the proposed final landform have been analysed. Based on the slope stability assessment, all six cross sections met the FoS stability criteria for all static load cases. For seismic SLS load case, only Sections 3, 4, and 5 met the FoS requirement. None of the sections met the FoS requirement under the design ULS seismic event. When the seismic FoS is less than 1.0, displacement analysis was carried out based on the methodology laid out in the NZ Bridge Manual (Waka Kotahi NZ Transport Agency, 2022). Post seismic flow failure is not anticipated.

  During a SLS seismic event, landfill is expected to remain stable with negligible deformation (i.e., ≤ 5 mm Sections 1,2,and 6).

  During an ULS seismic event, the landfill is likely to variably deform around the landfill perimeter. The magnitude of the slope deformation is dependent on various factors with two major factors being underlying ground conditions and the presence of internal perimeter bunding. The geology of the site is variable due to the nature of the estuary sediments beneath the site and variations in the depth to the underlying mudstones. Based on the available geotechnical investigation data, some areas at the site appear to have no UKEM sand like layer (as observed in TP15, TP09, TP27, TP32, and TP33 from Tonkin and Taylor, 2021 and BH108 and BH104 (GHD 2023B). Two cross sections modelled (sections 4 and 5) show no UKEM sand like layer, thus no liquefaction or lateral spreading is anticipated along these sections. The total seismic induced slope displacement is likely to be in an order of 35 to 325 mm in areas where there is no liquefiable layer present.

  In areas where liquefaction is expected to occur (sections 1, 2, 3, and 6) under ULS seismic event, lateral spreading is anticipated. Based on the adopted displacement assessment method, the total seismic induced slope displacement is likely to be in an order of 270 to 930 mm when a liquefiable layer is present. Depending on the continuity of the liquefiable layer and other factors listed in Section 7, the zone between the free face (i.e., Kaikorai Stream, sedimentary pond, etc.) and up to 200 m from the free face could experience the ground distortion as a result of lateral spreading.

  During an ULS equivalent seismic event, the northern, western and southwestern perimeter of the landfill (sections 1,2,3) are likely to move towards the nearest free face (i.e., Kaikorai Stream or western sedimentation pond) as a result of lateral spreading, in an order of 270 to 930 mm. It is likely that multiple cracks will form near riverbanks, at the toe of landfill and within landfill and cap. The lateral movement and cracking are expected to be generally parallel to the Kaikorai Stream. Due to the predicted deformations, damage to the existing leachate collection trench is likely and will vary along its length. Such deformation could include failure of the pipe joints and between the pipe connections to the pump stations.

The difference in expected deformation from Sections 1 to 3 is that Section 3 has the smallest height of sliding mass for the predicted yield acceleration and this height has a strong influence on the predicted deformations. The larger the landfill height, the greater the deformations. Around the southern perimeter (section 6), around 930 mm of lateral displacement is predicted. The ground distortion is likely to be in the forms of cracks and local slumping of ground. There is no leachate collection trench currently installed in this area. Future drain systems should be designed to accommodate these levels of movement.

Section 6 has an adverse landfill geometry, i.e. no internal perimeter bunds coupled with the full thickness of estuary sediments including a liquefiable layer. As a result, calculated lateral spread is higher than at other sections. Section 6 has assumed a geological profile which has not been validated by any investigations along the section and is conservative. It is noted that the predicted stability or deformations are not significantly impacted by the amount of sludge or asbestos/silty mud tank materials.

Around the eastern perimeter (sections 4 and 5), liquefaction and lateral spreading is not anticipated due to the absence of the liquefiable unit. Smaller ground movement, in the order of 75 to 325 mm, is anticipated. Similar to other portions of the fill, cracks and local slumping of landfill and capping is likely. Leachate collection trench may experience deformation from the ground movement.

– **Deformation assessment sensitivity:** During this assessment, the stability and deformation predicted were tested with sensitivity checks (i.e. continuity and thickness of liquefiable layer, shear wave velocity of the soils, dimensions of internal perimeter bunding on the landfill face, material properties). These showed that the assumed continuity of the liquefiable layers, location and dimension of the internal bunding, landfill height and the leachate levels are the key influences on the stability. The higher the landfill the greater the deformations. Similarly, any sections with no or reduced internal bunding also have higher predicted deformations.

– **Leachate levels:** To account for the uncertainty in ground behaviour and leachate level, sensitivity checks were undertaken on various leachate levels. The results show that the slope performance is sensitive to the leachate level adopted in the model. Appropriate measures are planned and reported in the *Design report* (GHD 2023c) to address management of leachate levels within the landfill, especially around the edge/perimeter of the landfill. The leachate level between the toe and 40 m back from the top edge of the landfill should generally be controlled at near or below 12 m amsl as illustrated in the schematic sketch shown in Figure 9. Under an extreme event, leachate level within the immediate vicinity from landfill shoulder shall not be higher than 16 m amsl, as illustrated in Figure 10 (GHD Limited 2023c).

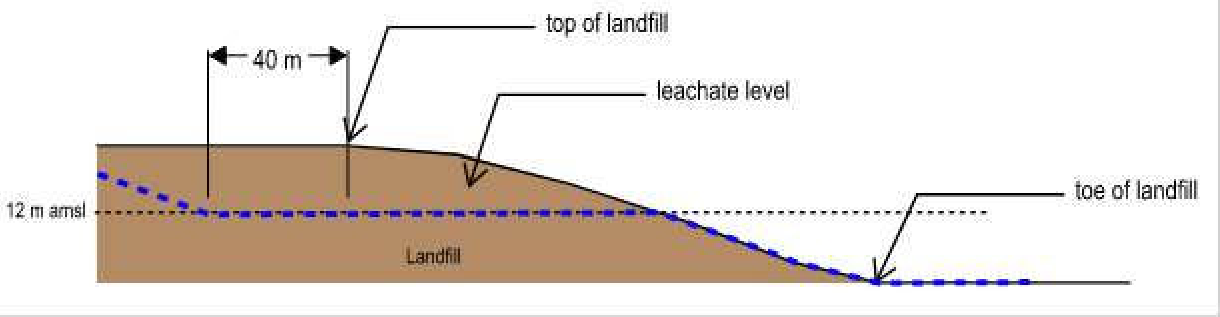

Figure 9 Schematic sketch – long term leachate level requirement

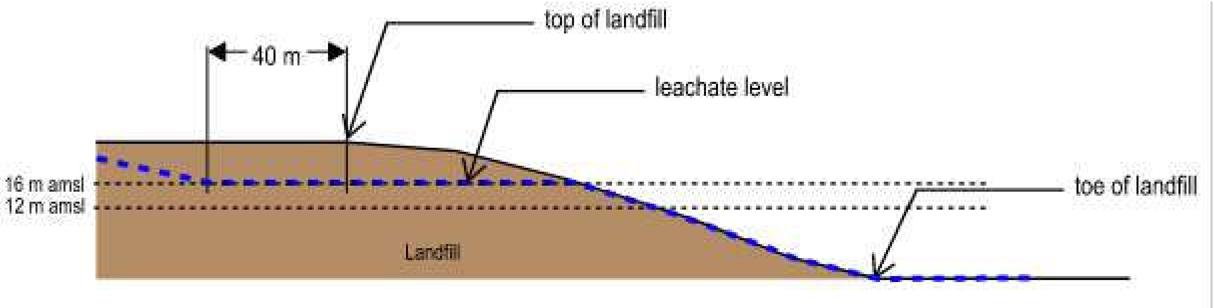

Figure 10 Schematic sketch – extreme (maximum) leachate level

9. Conclusion

As stated above, the landfill is likely to undergo deformation under ULS seismic event due to liquefaction, lateral spreading and slope movement. To reduce the severity of the impact, remedial work are proposed. Remedial options are documented in the *Design Report* (GHD Limited, 2023c).As discussed in Section 8 of this report, proposed measures documented in the Design Report include management of leachate levels within the landfill.

10. References

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Appendices

Appendix A

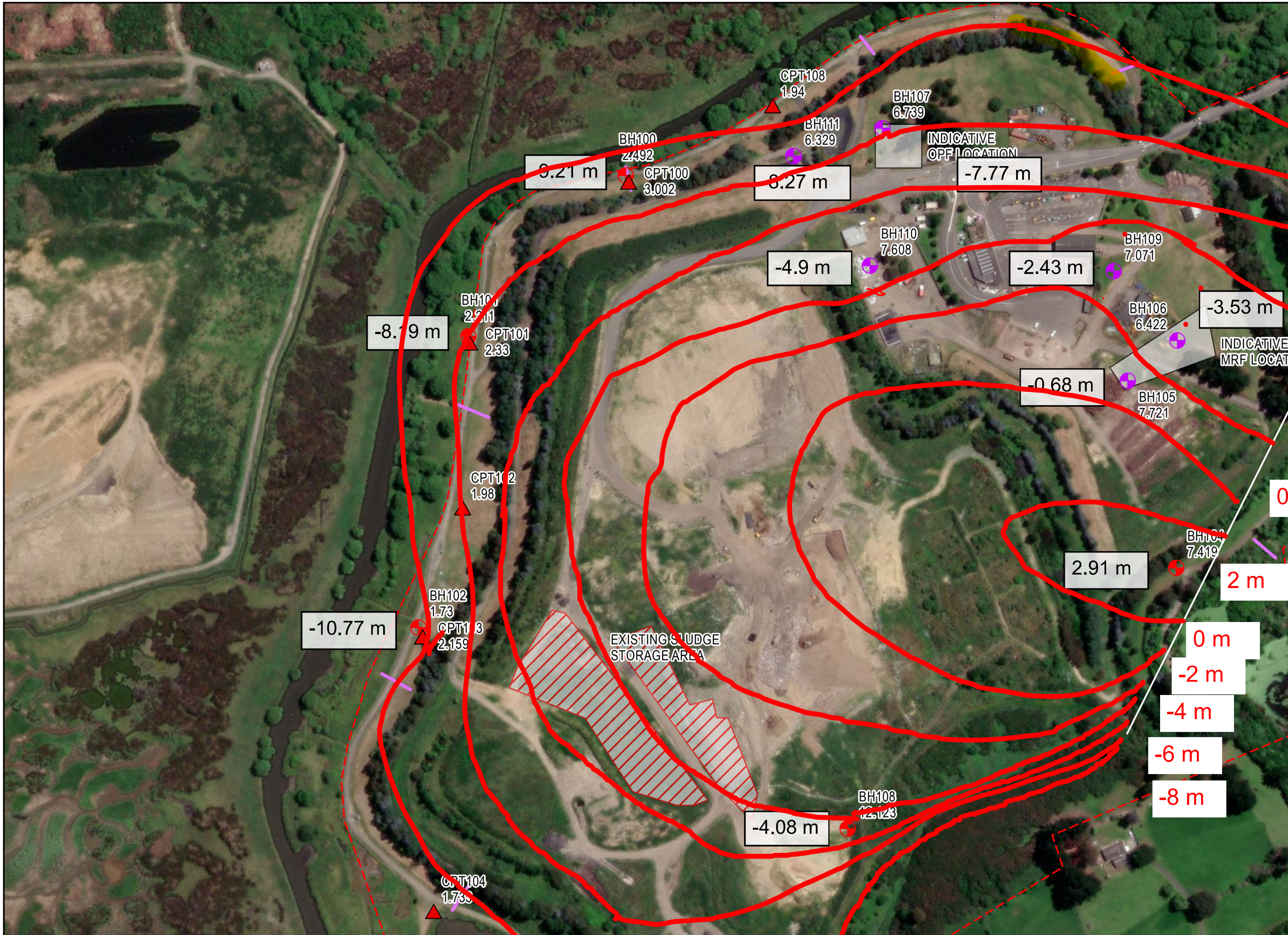
Investigation Plan


CPT108
1.94
BH107
6.739
BH111
6.329
INDICATIVE
OPF LOCATION
BH100
2.492
CPT100
3.002
BH110
7.608
BH109
7.071
BH106
6.422
INDICATIVE
MRF LOCAT
BH105
7.721
BH101
2.211
CPT101
2.33
Section 1
CPT102
1.98
BH104
7.419
BH102
1.73
CPT103
2.159
Section 2
Section 5
EXISTING SLUDGE
STORAGE AREA
BH108
12.123
Section 4
Section 3
Section 6
CPT104
1.733

Appendix B

Geological Contour Plans


-1.0 m
-0.5 m
0 m
0.5 m
-0.77 m
CPT108
1.94
BH107
6.739
BH111
6.929
INDICATIVE
OPF LOCATION
0.79 m
BH100
2.492
CPT100
3.002
0.83 m
BH110
7.608
BH109
7.071
-0.1 m
0.27 m
0.9* m
BH106
6.422
-1.08 m
INDICATIVE
MRF LOCAT
0.92 m
BH105
7.721
BH101
2.211
0.91 m
CPT101
2.33
1.0 m
CPT102
1.98
0.47 m
BH104
-0.5 m
0 m
0.5 m
0.5 m
BH102
1.73
0.5 m
2.159
0 m
0.13 m
EXISTING SLUDGE
STORAGE AREA
0.9 m
BH108
12.123
0.6* m
CPT104
1.733


CPT108
1.94
BH107
6.739
BH111
6.329
INDICATIVE
OPF LOCATION
BH100
2.492
9.21 m
CPT100
3.002
8.27 m
-7.77 m
BH110
7.608
BH109
7.071
-4.9 m
-2.43 m
BH106
6.422
-3.53 m
INDICATIVE
MRF LOCAT
BH101
2.211
-8.19 m
CPT101
2.33
-0.68 m
BH105
7.721
CPT102
1.98
BH104
7.419
2.91 m
0
2 m
BH102
1.73
-10.77 m
CPT103
2.159
EXISTING SLUDGE
STORAGE AREA
0 m
-2 m
-4 m
-6 m
-8 m
BH108
12.123
-4.08 m
CPT104
1.735

Appendix C

Site Specific PSHA Report

Probabilistic seismic hazard analysis of the Green Island Landfill site

Mark Stirling* and Govinda Niroula*

November 2022

* Otago Earthquake Science Group, University of Otago

Executive summary

We undertake a probabilistic seismic hazard analysis (PSHA) for the Green Island landfill site. This includes the development of a fault source model from the newly-completed national Community Fault Model, and a distributed seismicity model from the New Zealand earthquake catalogue. For ground motion modelling, five ground motion models are selected for the PSHA. Uniform hazard acceleration spectra produced from the PSHA show peak ground accelerations of 0.05 g, 0.15 g and 0.47 g for the 150 year, 475 year and 2500 year return periods, respectively. Disaggregations show the local Akatore Fault (c. 15 km from the site) to be the main contributor to hazard at the 475 year, and longer, return periods. The deterministic 84th percentile spectrum for the Akatore Fault is virtually identical to our 2500 year return period spectrum. The newly completed NSHM shows considerably higher spectra for the 150 and 475 year return periods, which is likely due to the comprehensive treatment of epistemic uncertainty in the distributed seismicity model for the NSHM. In contrast, the impact of the new distributed seismicity model is considerably less for the 2500 year return period due to the dominance of the Akatore Fault source at that return period. We recommend the NSHM spectra be used for the 150 year and 475 year return periods, and our spectrum is used for the 2500 year return period based on its very close match to the Akatore Fault 84th percentile deterministic spectrum.

Contents

Introduction

At the request of GHD Ltd, we undertake a probabilistic seismic hazard analysis (PSHA) for the Green Island landfill site (Fig. 1). The PSHA will be used to inform the process of landfill decommissioning. Our PSHA incorporates state of the art datasets, namely: (1) the recently completed national Community Fault Model (CFM; Seebeck et al. 2022); (2) a seismicity catalogue developed as part of a major update of the New Zealand national seismic hazard model (NSHM; Christophersen et al. 2022; Gerstenberger et al. 2022); and (3) a selection of state-of-the-art ground motion models (GMMs). Epistemic (model) uncertainty in (1)-(3) above is incorporated by way of logic trees.

Fault Source Model

The fault source model incorporates the newly released CFM (Seebeck, et al. 2022), in which each source is assigned a range of earthquake magnitudes based on recently-developed moment magnitude (Mw) – fault area scaling relations (Stirling et al. 2021). Ranges of recurrence intervals (RIs) are also taken from the CFM. The fault source model includes all sources within 200 km of the site (Figs. 1 and 2), and the logic tree structure for the fault sources is shown in Figure 3. The fault source parameterisation is simplistic, in that the magnitude-frequency distribution of earthquakes is limited to the range of Mw expected to be produced by rupture of the full area of the fault source (length x width of the fault plane). No allowance is made for rupture of parts of the fault, or for multi-fault ruptures. In this simplistic approach we rely on the distributed seismicity model to account for earthquakes less than or greater than the Mw estimated from rupture of the full fault plane. Parameters for recently-studied proximal sources (Akatore and Green Island faults) are developed according to methods described in the following paragraphs. These faults are highlighted due to the Akatore Fault being the closest, most active fault to the site (and in fact Otago's most active fault), and the Green Island Fault being the along-strike continuation of the Akatore Fault (e.g. Holt, 2017).

The Akatore Fault has been the focus of a recent paleoseismic study (Taylor-Silva et al., 2020), and has also been reviewed in a regional study for Otago Regional Council (Barrell, 2021). The fault appears to show aperiodic behaviour, with a minimum 100,000 year period of quiescence being followed by three earthquakes in the latest Pleistocene to Holocene. The following recurrence intervals are assigned to the Akatore Fault:

- Preferred RI = 1700 years. This is the preferred RI from Barrell (2021), who suggested that the three Holocene earthquakes identified by Taylor Silva et al (2020) occurred in the

last 5000-7000 years, based on observations of Holocene shore platforms uplifted by the fault.
- Minimum RI = 450 years. This is the minimum RI from Taylor Silva et al. (2020)
- Maximum RI = 118,000 years. This is obtained by subtracting 7000 years from the 125,000 year age of marine terraces at Taieri Mouth, which only show evidence for the three earthquakes identified by Taylor Silva et al. (2020).

The magnitude estimates for the Akatore Fault are Mw 7.3 as preferred, Mw 7.2 as minimum, and Mw 7.5 as maximum. These Mws are estimated from Mw – area scaling relations developed for the NSHM update (Stirling et al. 2021).

The Green Island Fault lies totally offshore from the site, and strikes northeast from the north end of the Akatore Fault (Figs. 1 and 2). It shows a relatively sharp scarp on the seabed (Holt, 2017), but for obvious reasons does not have any paleoseismic data to constrain a RI. The preferred RI is based on Barrell (2021), but we also assign the Akatore Fault maximum and preferred RIs to the Green Island Fault. This is to acknowledge that the Green Island Fault is considered to be the along-strike continuation of the Akatore Fault (e.g. Holt, 2017), and may therefore show somewhat similar recurrence behaviour. Clearly, these assumptions are based on expert opinion, and are not constrained by data. The following recurrence intervals are estimated for the Green Island Fault on the basis of the expert opinion:

- Preferred RI = 22,000 years. This is estimated by Barrell (2021) using the method of Stirling et al. (2012).
- Minimum RI = 1700 years. This is equal to the preferred value of RI for the Akatore Fault (Barrell 2021).
- Maximum RI = 118,000 years. This is equal to the maximum value of RI for the Akatore Fault (see above).

The Mw estimates for the Green Island Fault are Mw 6.9 as preferred, Mw 6.8 as minimum, and Mw 7.1 as maximum. These Mw estimates come from the use of estimates of the fault area, together with recently-developed magnitude-area scaling relations (Stirling et al., 2021).

For these faults (and all other faults), a simple 0.5, 0.25 and 0.25 weighting is applied to preferred, minimum, and maximum RIs, and equal weighting is given to the three Mws. The RI weights are chosen to represent a simplistic triangular distribution, in which the confidence in the preferred RI is higher than for the minimum and maximum RIs. The equal weightings of the three Mws reflect the equal viability of the three scaling relations developed for the NSHM by Stirling et al. (2021).

While the Kaikorai Fault is shown as the closest fault to the site, it is not a source of great importance to the PSHA. This is because it has been demonstrated not to represent the onshore continuation of the Akatore Fault (Sangster, 2019), and because of its very long estimated recurrence interval (Seebeck et al. 2022). Likewise, the Titri Fault shows considerably lower rates of activity than the Akatore Fault. While all of these faults are included in the fault source model, only the Akatore Fault is used to develop scenario earthquake spectra for the PSHA. It is the closest, most active source to the site.

The fault sources used in the PSHA are shown in Figure 1 and listed in Appendix 1. The sources lie within 200 km of the Green Island landfill site. Appendix 1 also shows the parameters used in the fault source model, but the reader is also encouraged to view the CFM publication (Seebeck et al. 2022) and spreadsheet (google “New Zealand community fault model”) to gain a fuller appreciation of the CFM.

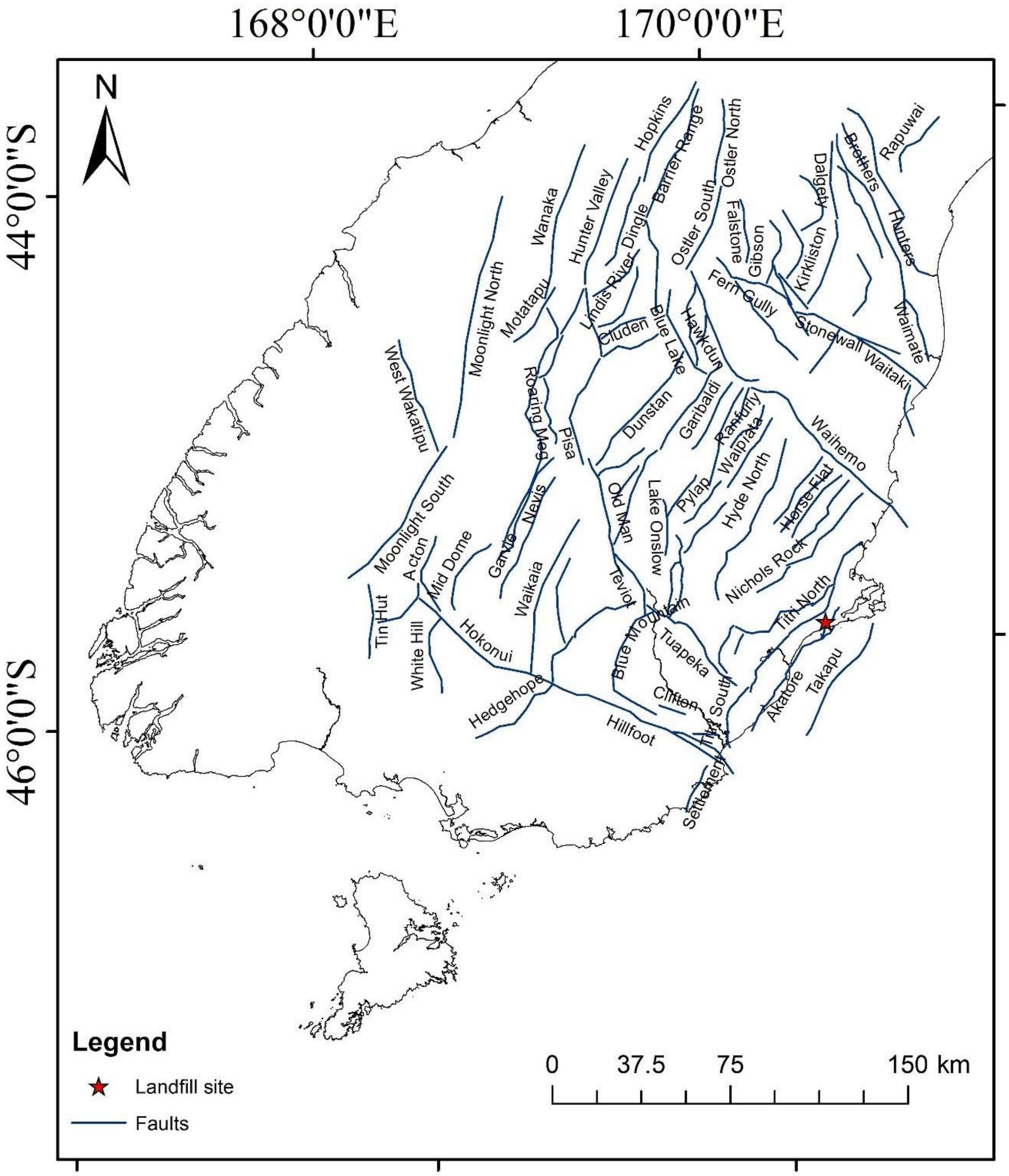


Figure 1: Fault sources used in the PSHA for the Green Island landfill site.

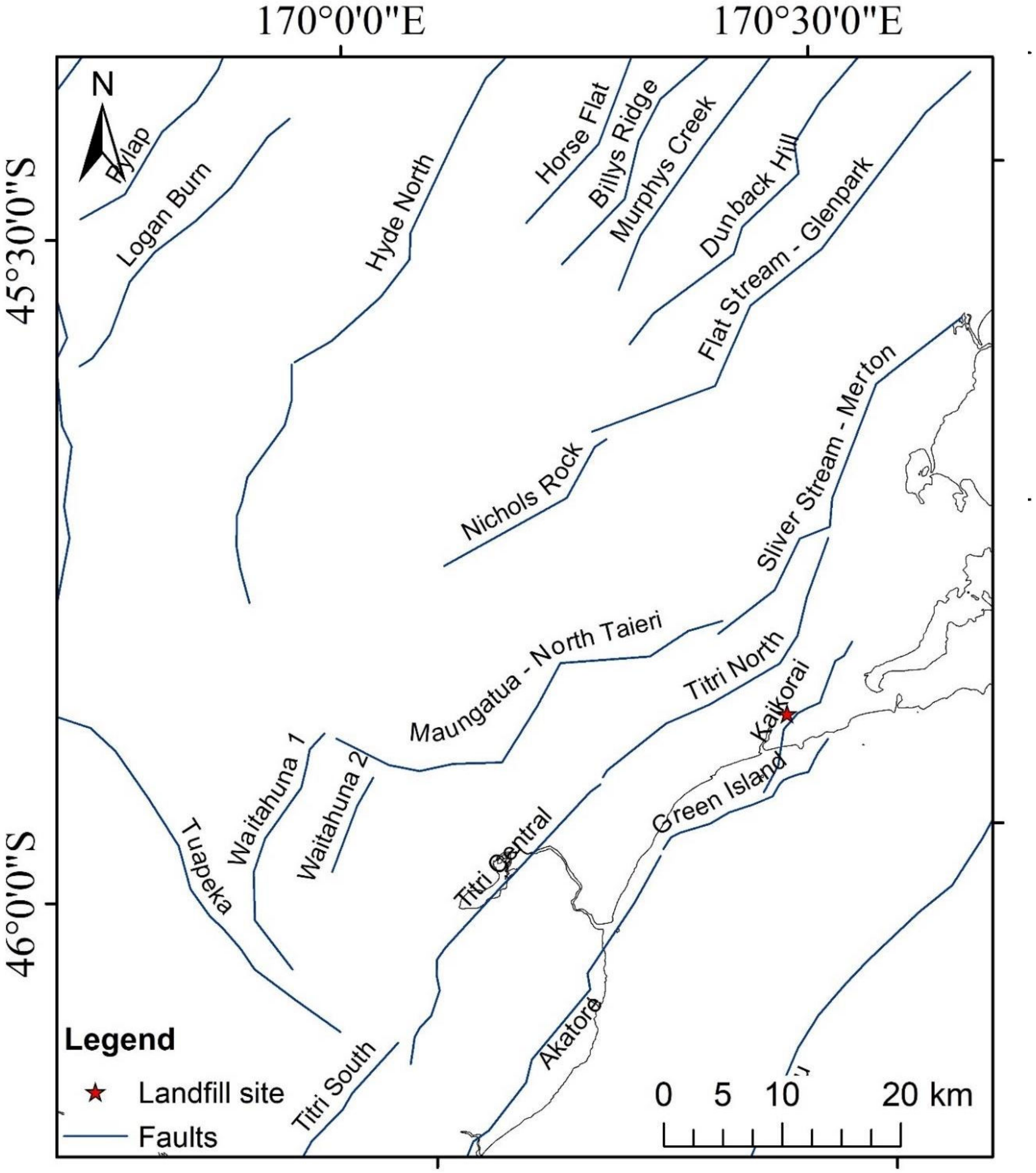


Figure 2: Faults in the more immediate area of the Green Island site than shown in Figure 1.

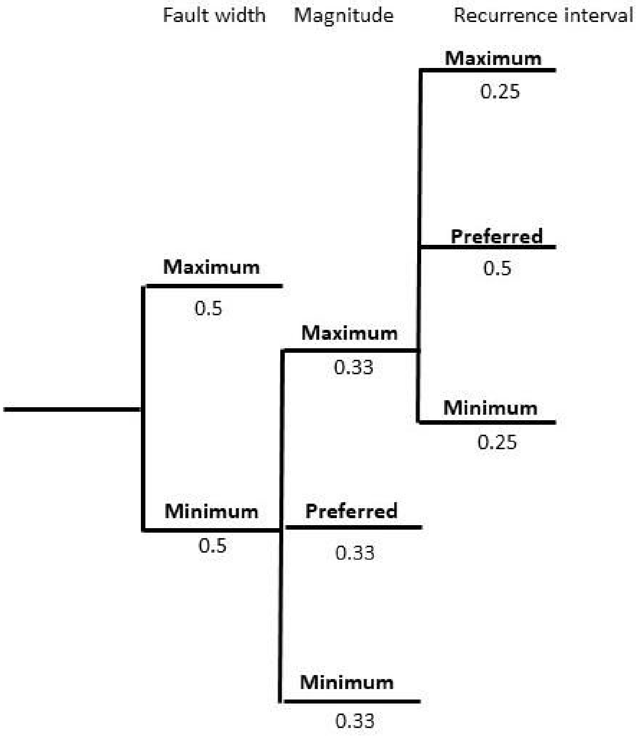


Figure 3: Logic tree structure for fault sources.

Distributed seismicity model

Distributed seismicity models allow for the occurrence of earthquakes away from the fault sources, and also allow for earthquakes less than the maximum-size earthquakes on or close to the faults. The distributed seismicity model for the PSHA is developed from the New Zealand catalogue of Christophersen et al (2022), and is limited to Mw $\geq$ 4 for the period 1964-2020 (see Stirling et al. 1998, 2002, 2012 for explanations regarding this magnitude-time subset of the catalogue). The seismicity for this time period and Mw range is shown in Figures 4 and 5. The distributed seismicity model is actually made up of two mutually exclusive models: A gridded smoothed seismicity model; and a uniform area source model. The gridded smoothed seismicity model follows the method of Frankel (1995), in which a gaussian smoothing kernel with a correlation distance of 50 km is applied to the seismicity across a grid of sites positioned at three depth layers (0-5 km, 5-12 km and 12-40 km; Figs. 4 and 5). The depth control of earthquakes in most of the New Zealand catalogue is very poor, and most events are randomly assigned restricted depths of 5, 12 and 33 km. The uniform area source model is developed by counting the earthquakes within a regional polygon, and then uniformly distributing the rates across a grid of sites at the three depth layers. Each of the distributed seismicity models is assigned a weight of 0.5 as neither is seen as preferable in a zone of low seismicity with no major earthquake sequences in the time period 1964-2020. The Gutenberg-Richter relation (Gutenberg and Richter, 1944) is then applied to the gridded seismicity rates in the two distributed models to estimate the recurrence of Mw 5 to 8 events. Mw 8 is a default upper bound Mw used for New Zealand in the recent NSHM update, and is a simple alternative to an attempted regionalisation of the estimates of the upper-bound Mw. The smoothed seismicity and uniform seismicity models are shown in Figures 6 and 7, and the logic tree structure for combining the two models is shown in Figure 8.

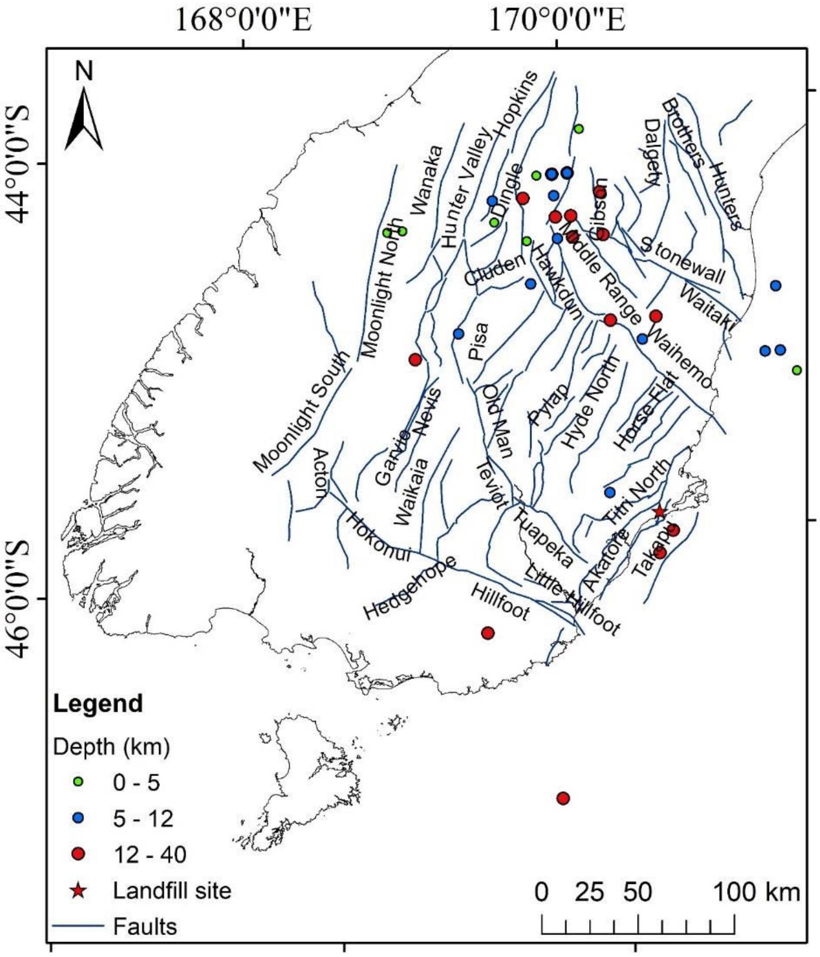


Figure 4: Seismicity of M≥4 for the period 1964-2020, and colour-coded according to depth.

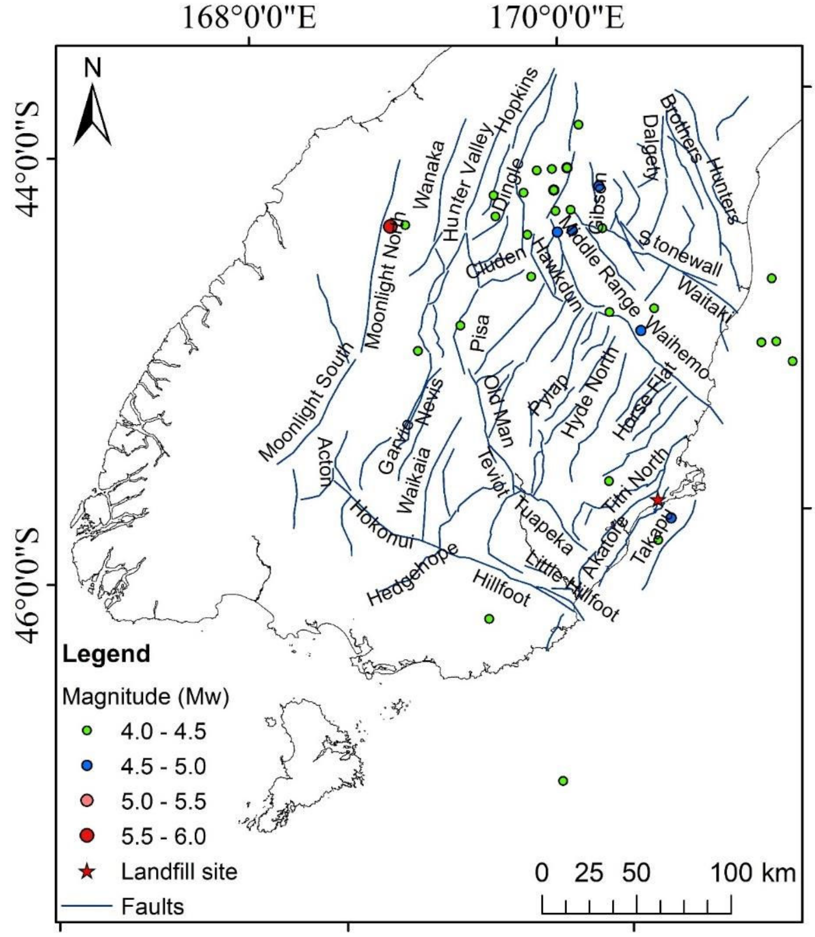


Figure 5: Seismicity as for Figure 4, but instead colour-coded according to Mw.

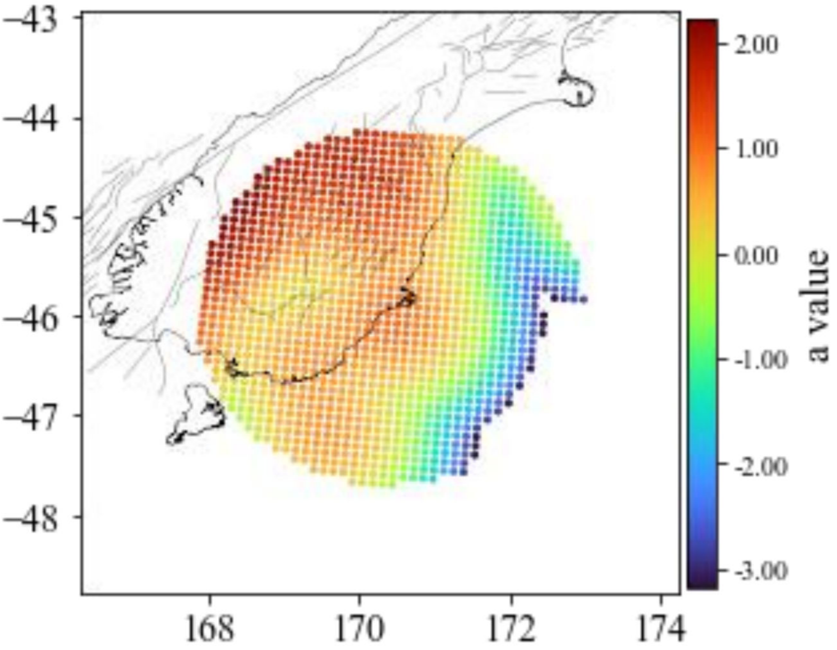


Figure 6: Smoothed seismicity model for seismicity at 0-5 km depth (shallowest of three depth levels in the model), showing Gutenberg-Richter a-values across a grid with a resolution of 10km^2.

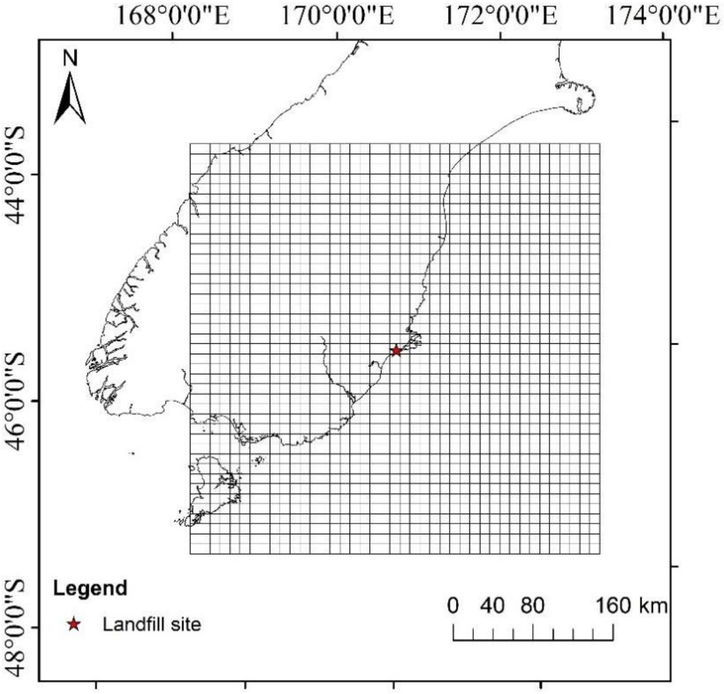


Figure 7: Uniform area source model grid coverage.

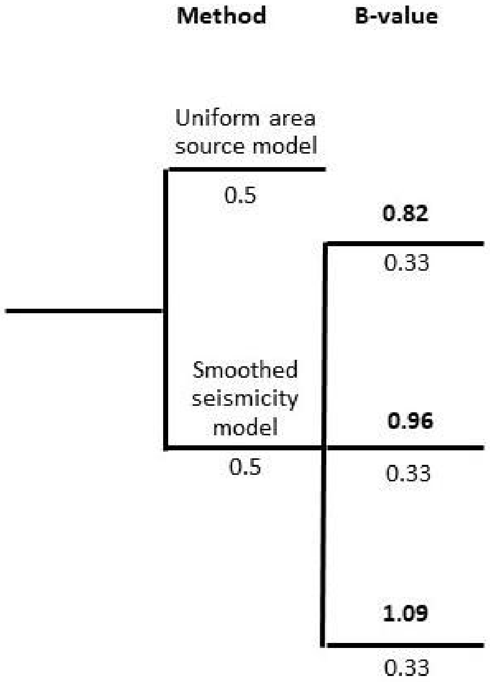


Figure 8: Logic tree structure for combining the distributed seismicity models.

Ground motion models and site conditions

Five ground motion models (GMMs) are used in the Green Island PSH model. These are: Boore and Atkinson (2008); Abrahamson and Silva (2008); Bradley (2013); Campbell and Bozorgnia (2014); and Chiou and Youngs (2014). These GMMs are routinely used in PSHA, and all use Vs30 (average shear wave velocity for the top 30 m of site geology) as the ground condition metric. A Vs30 of 360 m/s is assumed for the Green Island landfill site, as Tertiary Abbotsford Mudstone is encountered only 8 m below the ground surface (Douglas, 2002). The top 8 m is composed of fine sands and silts (0-4 m), and silty clay marine sediments (4-8 m). Assignment of Vs30 = 360 m/s is made according to site geology on the basis of the following references: Building Seismic Safety Council (1997); and Standards New Zealand (2004). Our assigned Vs30 is broadly equivalent to Site Class C of NZS1170.5 (Standards New Zealand, 2004).

Weights of 0.2 are assigned to each of the five GMMs in the PSHA (Fig. 9), as all are considered state-of-the-art GMMs of relevance to New Zealand conditions. RotD50 (50th percentile values of response spectra of the two horizontal components projected onto all non-redundant azimuths; Boore and Kishida, 2017) is used for Chiou and Youngs (2014) and Campbell and Bozorgnia (2014) GMMs. Rotl50 (Median value of the geometric mean of the two horizontal components rotated through all non-redundant period independent angles; Boore and Kishida, 2017) is used for Boore and Atkinson (2008) and Abrahamson and Silva (2008) GMMs. Finally, the largest horizontal component is used for the Bradley (2013) GMM.

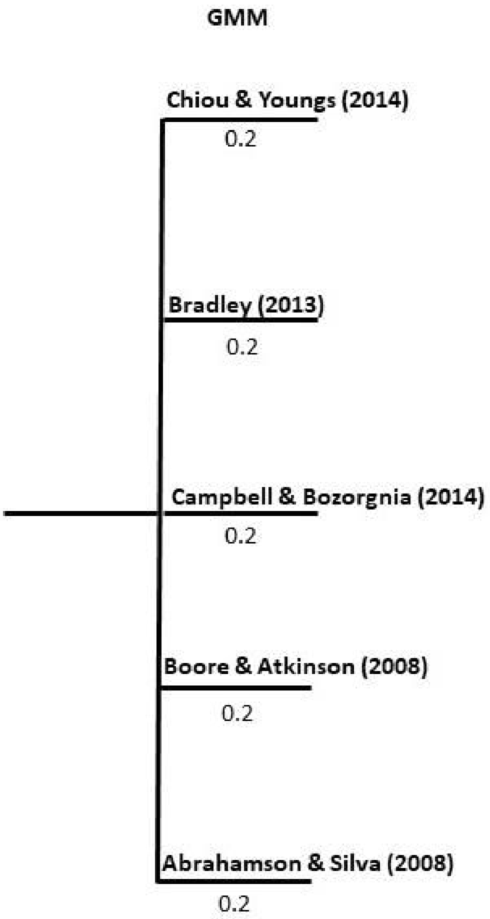


Figure 9: Logic tree structure for the GMMs used in the PSHA.

Results

Uniform hazard acceleration spectra are shown for the 150, 475, and 2500 year return periods in Figure 10, and are tabulated in Table 1. The mean spectra are shown for all three return periods. PGAs for these return periods are plotted at the 0.01 second period on the graph, given that 0 second period cannot be plotted on a log-plot. The spectra show PGAs of 0.05 g, 0.15 g and 0.47 g for the 150 year, 475 year and 2500 year return periods, respectively. A disaggregation in Figure 11 shows the dominant contributions to PGA at the 475 year return period (and longer) to be from the Akatore Fault, which is located c. 15 km to the site. The 84th percentile deterministic spectrum for the Akatore Fault (Fig. 10, Table 1; Mw7.3 reverse fault earthquake) is virtually identical to the 2500 year spectrum. The Akatore Fault spectrum should be used for the site if a scenario or maximum considered earthquake is required. More conservative approaches could be to use the Kaikorai Fault to estimate the maximum considered earthquake for the site based on their much closer distance to the site (< 5 km). However, it would be difficult to justify using the Kaikorai source in this respect due to the absence of data and very long estimated recurrence interval for the fault (1,000,000 years; Appendix 1).

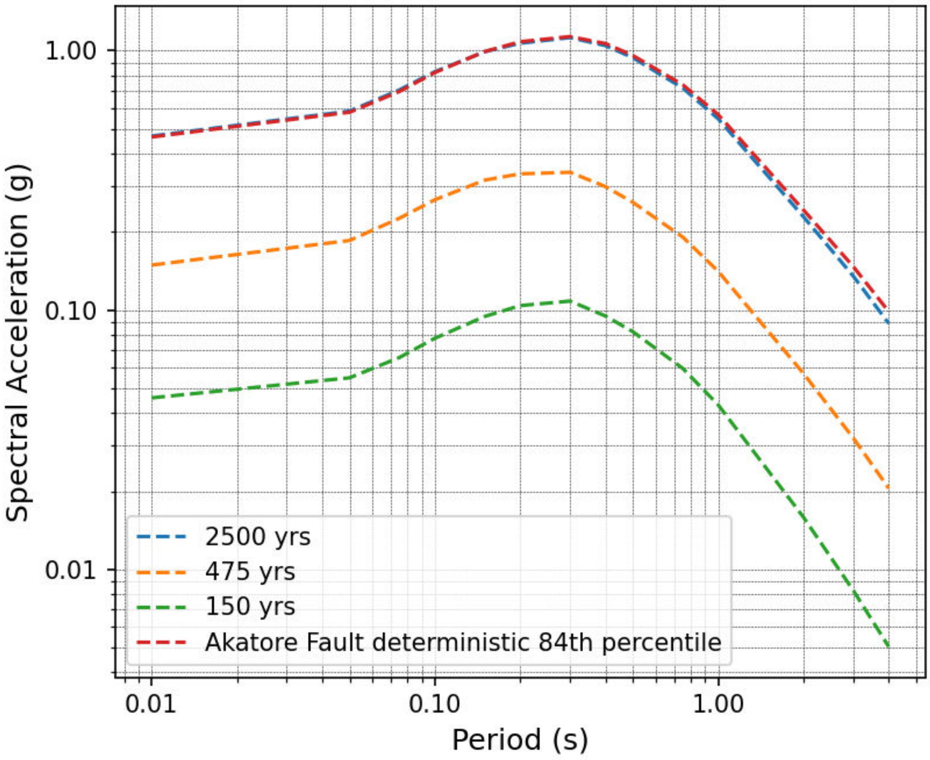


Figure 10: Uniform hazard acceleration spectra for the Green Island landfill site, for Vs30=360 m/s. Mean spectra are shown for all return periods, and the 84th percentile deterministic spectrum is shown for the Akatore Fault (Mw7.3 reverse fault earthquake at 15 km distance).

Table 1: Tabulated spectra for 150 year, 475 year, and 2500 year return periods, and for the 84th percentile Akatore Fault deterministic earthquake. Vs30=360 m/s.

Period (sec)	**SA(g), 150 years**	**SA(g), 475 years**	**SA(g), 2500 years**	**Akatore Fault deterministic 84th percentile**
PGA	0.046	0.149	0.469	0.465
0.050	0.055	0.185	0.585	0.579
0.075	0.066	0.226	0.706	0.696
0.100	0.078	0.266	0.830	0.824
0.150	0.094	0.317	0.991	0.993
0.200	0.104	0.335	1.069	1.083
0.300	0.108	0.340	1.124	1.134
0.400	0.095	0.299	1.047	1.066
0.500	0.082	0.260	0.940	0.958
0.750	0.060	0.191	0.718	0.739
1.000	0.043	0.141	0.546	0.567
2.000	0.016	0.057	0.229	0.243
3.000	0.008	0.032	0.135	0.147
4.000	0.005	0.021	0.089	0.099

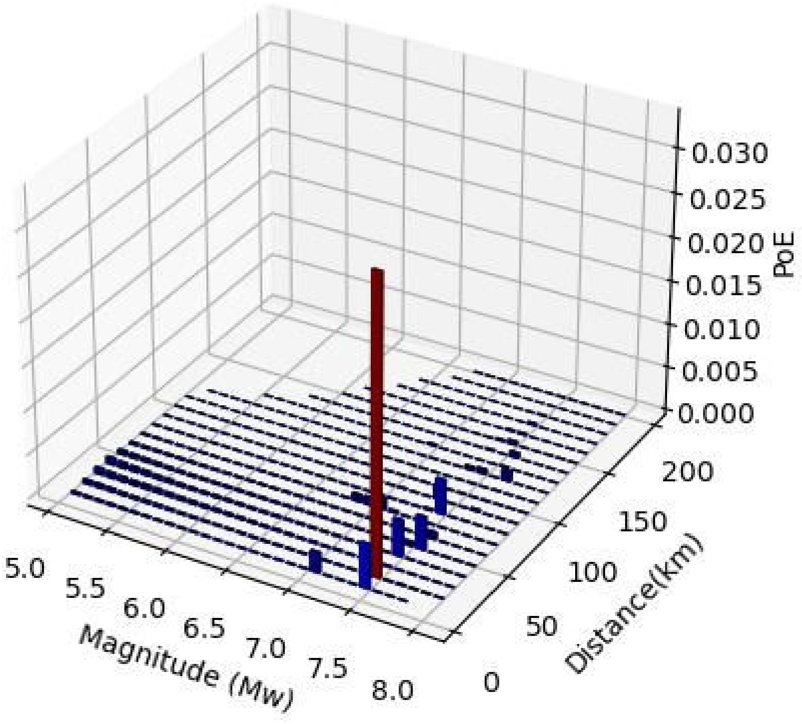


Figure 11: Disaggregation for PGA at the 475 year return period. The plots show the probability of exceedance for the 475 year PGA (z-axis) as a function of Mw and distance (x and y-axes). The prominent peak is the Akatore Fault source.

Comparison to NSHM

A major update of the NSHM was completed and released in early October 2022, making it possible to do a very late-stage comparison of our spectra to those of the NSHM. The NSHM mean spectra are shown in Figure 12 and tabulated in Table 2, and we include the Akatore 84th percentile deterministic spectrum from Figure 10 in the Figure for comparison. The NSHM spectra are taken from the public website https://nshm.gns.cri.nz/Hazardcurves, which do not allow the same detail of spectral periods and associated accelerations as those of our spectra in Figure 10 and Table 1. Furthermore, the Vs30 of 350 m/s used for the NSHM spectra is the closest available on the website to our Vs30 of 360 m/s. However, we consider the two sets of spectra to be broadly comparable, except in the 0.02 to 0.1 sec period range where the NSHM website does not provide accelerations, resulting in the spectra having simplified straight-line approximations.

The NSHM spectra clearly exceed our spectra by 50% or more at the 150 year and 475 year return periods, whereas the 2500 year spectra are very similar. The large differences at the shorter return periods are likely due to the comprehensive treatment of epistemic uncertainty in the distributed seismicity model for the NSHM. A new GMM developed specifically for the NSHM (Stafford, 2022) is also probably responsible for a small part of the differences, but the similarity of the 2500 year spectra show that this influence is not great.

The complexity of the NSHM makes it impossible to export to other machines at the present time. Simplification of the NSHM for export is on the list of things to do in NSHM follow up work over the next year or so. Comparison of our model to the NSHM and explanation of the differences is therefore the best that can be done at this time. Figure 12: Uniform hazard acceleration spectra for the Green Island landfill site, from the recently-developed NSHM, for Vs30=350 m/s. Mean spectra are shown, and our 84th percentile Akatore Fault deterministic spectrum is also shown for comparison.

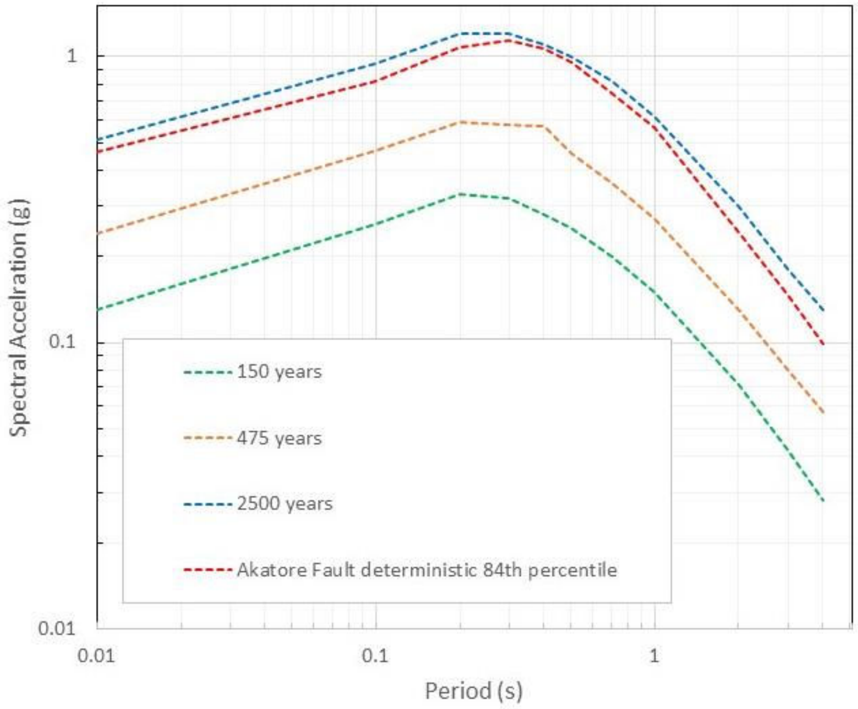


Figure 12: Uniform hazard acceleration spectra for the Green Island landfill site, from the recently-developed NSHM, for Vs30=350 m/s. Mean spectra are shown, and our 84th percentile Akatore Fault deterministic spectrum is also shown for comparison.

Table 2: Tabulated spectra for 150 year, 475 year, and 2500 year return periods from the recently-updated NSHM, for Vs30=350 m/s. The 84th percentile Akatore Fault deterministic spectrum is also tabulated.

Period (sec)	SA(g), 150 years	SA(g), 475 years	SA(g), 2500 years	Akatore Fault deterministic 84th percentile
PGA	0.13	0.24	0.51	0.465
0.100	0.26	0.47	0.95	0.824
0.200	0.33	0.59	1.2	1.083
0.300	0.32	0.58	1.2	1.134
0.400	0.28	0.57	1.1	1.066
0.500	0.25	0.46	1.0	0.958
0.700	0.20	0.36	0.82	0.739
1.000	0.15	0.27	0.62	0.567
2.000	0.071	0.13	0.30	0.243
3.000	0.042	0.08	0.18	0.147
4.000	0.028	0.057	0.13	0.099

Conclusions and recommendations

We have undertaken a PSHA for the Green Island landfill site. The PSHA has involved the development of a fault source model from the newly-completed CFM (Seebeck et al 2022), and a distributed seismicity model from an earthquake catalogue recently developed for the ongoing national seismic hazard model update (Christophersen et al. 2022; Gerstenberger et al. 2022). A selection of five GMMs have been used in the PSH model. Uniform hazard acceleration spectra produced from the model show PGAs of 0.05 g, 0.15 g and 0.47 g for the 150 year, 475 year and 2500 year return periods, respectively. Disaggregations show the local Akatore Fault (c. 15 km from the site) to be the main contributor to hazard for the 475 year return periods, with lesser contributions from the Green Island Fault and other east Otago faults. A deterministic 84th percentile spectrum for the Akatore Fault is virtually identical to the 2500 year spectrum. The newly completed NSHM shows considerably higher spectra for the 150 and 475 year return periods, which is likely due to the comprehensive treatment of epistemic uncertainty in the distributed seismicity model for the NSHM. In contrast, the impact of the new distributed seismicity model is considerably less for the 2500 year return period due to the dominance of the Akatore Fault source at that return period. We recommend the NSHM spectra be used for the 150 year and 475 year return periods, and our spectrum is used for the 2500 year return period based on its very close match to the Akatore Fault 84th percentile deterministic spectrum.

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Appendix 1: Fault source parameters. See the key at the base of the table for explanation of the columns.

Name	Dippf	Dipmx	Dipmn	Slip type	Rake_pref	Tpf	Tmn	Tmx	Tgen	D1	D2	Lm	W1	W2	A1	A2	Mw1mn	Mw1pf	Mw1max	Mw2mn	Mw2pf	Mw2mx
Acton	60	60	50	reverse	110				10,000	33.9	28.5	34864	39	33	1365	1147	7.1	7.2	7.4	7.0	7.2	7.4
Akatore	45	45	30	reverse	110	1700	450	118000		28.1	25.4	40364	40	36	1604	1450	7.2	7.3	7.5	7.1	7.3	7.5
Albury	60	60	50	reverse	110				100,000	26	20.6	20679	30	24	621	492	6.7	6.9	7.1	6.6	6.8	7.0
Awahokomo	60	60	40	reverse	110				10,000	24	19.2	17660	28	22	489	392	6.6	6.8	7.0	6.5	6.7	6.9
Awamoa	45	45	30	reverse	90				100,000	20.7	17.4	22254	29	25	651	548	6.8	6.9	7.1	6.7	6.8	7.0
Backbone	60	60	40	reverse	70				1,000,000	34.1	28.3	23740	39	33	935	776	6.9	7.1	7.3	6.8	7.0	7.2
Barrier Range	60	60	40	reverse	110				1,000,000	25.7	22.5	58713	30	26	1742	1525	7.2	7.3	7.5	7.1	7.3	7.5
Beacon Hill	70	70	55	reverse	70				1,000,000	33.4	28.8	20337	36	31	723	623	6.8	7.0	7.2	6.7	6.9	7.1
Beaumont River	60	60	40	reverse	70				1,000,000	29.5	24.8	36468	34	29	1242	1044	7.0	7.2	7.4	7.0	7.1	7.3
Billys Ridge	45	45	30	reverse	110				1,000,000	24.4	20.4	34114	35	29	1177	984	7.0	7.2	7.4	6.9	7.1	7.3
Black Hill	45	45	30	reverse	110				1,000,000	20.9	17.4	22150	30	25	655	545	6.8	6.9	7.1	6.7	6.8	7.0
Blue Lake	70	70	60	reverse	110				1,000,000	25.8	21.2	40950	27	23	1124	924	7.0	7.2	7.4	6.9	7.1	7.3
Blue Mountain	45	45	30	reverse	90	100 000	10 000	100 000	100,000	32	27.8	39547	45	39	1790	1555	7.2	7.4	7.6	7.1	7.3	7.5
Brothers	60	60	50	reverse	110				100,000	26.7	21.2	52019	31	24	1604	1273	7.2	7.3	7.5	7.1	7.2	7.4
Campbell Hills	45	45	30	reverse	90				1,000,000	23.8	18	29428	34	25	990	749	6.9	7.1	7.3	6.8	7.0	7.2
Cardrona - Hawea	60	60	40	reverse	110				10,000	26.9	23.2	20864	31	27	648	559	6.8	6.9	7.1	6.7	6.8	7.0
Chamberlain	60	60	50	reverse	110				100,000	25.4	19.3	43688	29	22	1281	974	7.1	7.2	7.4	6.9	7.1	7.3
Clifton	70	70	55	reverse	110				10,000	32.6	28	11423	35	30	396	340	6.5	6.7	6.9	6.5	6.6	6.8
Cluden	60	60	40	reverse	110				100,000	27.1	22.5	25868	31	26	809	672	6.9	7.0	7.2	6.8	6.9	7.1
Cross Eden	60	60	40	reverse	110				1,000,000	23.9	20.2	18889	28	23	521	441	6.7	6.8	7.0	6.6	6.7	6.9
Dalgety	60	60	50	reverse	110				10,000	26	20	32580	30	23	978	752	6.9	7.1	7.3	6.8	7.0	7.2
Dingle	60	60	40	reverse	110				1,000,000	26	22.5	40343	30	26	1211	1048	7.0	7.2	7.4	7.0	7.1	7.3
Dromedary	60	60	40	reverse	110				10,000	27.1	22.6	39321	31	26	1230	1026	7.0	7.2	7.4	7.0	7.1	7.3
Dryburgh	60	60	40	reverse	70				10,000	24.2	18.1	18798	28	21	525	393	6.7	6.8	7.0	6.5	6.7	6.9
Dunback Hill	45	45	30	reverse	110				1,000,000	24.2	20.3	37812	34	29	1294	1086	7.1	7.2	7.4	7.0	7.1	7.3
Dunstan	45	45	30	reverse	90				10,000	28	22.9	60854	40	32	2410	1971	7.3	7.5	7.7	7.2	7.4	7.6
Falstone	60	60	40	reverse	110				1,000,000	25.6	19.7	26834	30	23	793	610	6.8	7.0	7.2	6.7	6.9	7.1
Fern Gully	75	75	60	sinistral	20				10,000	24.8	19.7	35348	26	20	908	721	6.9	7.1	7.3	6.8	7.0	7.2
Flat Stream - Glenpark	45	45	30	reverse	110				1,000,000	24.2	20.6	45655	34	29	1562	1330	7.1	7.3	7.5	7.1	7.2	7.4
Galloway	45	45	30	reverse	110				10,000	30.2	24.2	23099	43	34	987	791	6.9	7.1	7.3	6.8	7.0	7.2
Galloway Shanty	45	45	30	reverse	110				1,000,000	29.1	23.6	17234	41	33	709	575	6.8	7.0	7.2	6.7	6.9	7.1
Garibaldi	45	45	30	reverse	110				1,000,000	26.2	21.5	33021	37	30	1224	1004	7.0	7.2	7.4	7.0	7.1	7.3
Garvie	45	45	30	reverse	110				1,000,000	33	27.1	56240	47	38	2625	2155	7.4	7.5	7.7	7.3	7.4	7.6
Gibson	45	45	30	reverse	90				1,000,000	25.3	19.4	29203	36	27	1045	801	7.0	7.1	7.3	6.9	7.0	7.2
Gimmerburn	45	45	30	reverse	110	18 000	18 000	500 000	10,000	26.1	21.3	43900	37	30	1620	1322	7.2	7.3	7.5	7.1	7.2	7.4
Goodger	60	60	40	reverse	110				1,000,000	26.6	22.1	20644	31	26	634	527	6.8	6.9	7.1	6.7	6.8	7.0
Grampian	45	45	30	reverse	90				1,000,000	25.9	19.6	31181	37	28	1142	864	7.0	7.2	7.4	6.9	7.0	7.2
Grandview	60	60	40	reverse	70				100,000	26.9	22.4	29415	31	26	914	761	6.9	7.1	7.3	6.8	7.0	7.2
Green Island	45	45	30	reverse	110	22000	1700	118000		27.7	24.6	17554	39	35	688	611	6.8	6.9	7.1	6.7	6.9	7.1

Name	Dippf	Dipmx	Dipmn	Slip type	Rake_pref	Tpf	Tmn	Tmx	Tgen	D1	D2	Lm	W1	W2	A1	A2	Mw1mn	Mw1pf	Mw1max	Mw2mn	Mw2pf	Mw2mx
Hawkdun	70	70	60	reverse	110				1,000,000	24.9	20.2	57804	26	21	1532	1243	7.1	7.3	7.5	7.0	7.2	7.4
Hedgehope	60	60	50	reverse	110				1,000,000	33.1	28.4	40973	38	33	1566	1344	7.1	7.3	7.5	7.1	7.2	7.4
Hillfoot	75	75	60	reverse	70				1,000,000	33.2	28.3	80704	34	29	2774	2364	7.4	7.5	7.7	7.3	7.5	7.7
Hokonui	75	75	60	reverse	70				1,000,000	33.3	28.6	66468	34	30	2291	1968	7.3	7.5	7.7	7.2	7.4	7.6
Horse Flat	45	45	30	reverse	110				1,000,000	24.5	20.5	35139	35	29	1218	1019	7.0	7.2	7.4	7.0	7.1	7.3
Hunter Valley	60	60	40	reverse	110				10,000	25.2	21.4	55931	29	25	1628	1382	7.2	7.3	7.5	7.1	7.2	7.4
Hunters	60	60	50	reverse	110				100,000	24.3	18.4	63749	28	21	1789	1354	7.2	7.4	7.6	7.1	7.2	7.4
Hyde North	45	45	30	reverse	110				10,000	26.3	21.6	55218	37	31	2054	1687	7.3	7.4	7.6	7.2	7.3	7.5
Hyde South - The Twins	45	45	30	reverse	110				1,000,000	31.9	27	21377	45	38	964	816	6.9	7.1	7.3	6.9	7.0	7.2
Kaikorai	45	45	30	reverse	110				1,000,000	26.8	23.7	15461	38	34	586	518	6.7	6.9	7.1	6.7	6.8	7.0
Kirkliston	50	50	40	reverse	110				1,000,000	25.1	18.9	37820	33	25	1239	933	7.0	7.2	7.4	6.9	7.1	7.3
Lake Onslow	60	60	40	reverse	70				1,000,000	30.4	25.2	23411	35	29	822	681	6.9	7.0	7.2	6.8	6.9	7.1
Lindis River	60	60	40	reverse	110				1,000,000	26.6	21.9	32696	31	25	1004	827	7.0	7.1	7.3	6.9	7.0	7.2
Little Hillfoot	75	75	60	reverse	70				1,000,000	32.5	28	31814	34	29	1070	922	7.0	7.1	7.3	6.9	7.1	7.3
Little Valley	45	45	30	reverse	110				1,000,000	27	24.2	44021	38	34	1681	1507	7.2	7.3	7.5	7.1	7.3	7.5
Logan Burn	60	60	40	reverse	110				1,000,000	28.2	23.6	27871	33	27	908	760	6.9	7.1	7.3	6.8	7.0	7.2
Long Valley	60	60	40	reverse	110				10,000	27.7	22.7	21338	32	26	682	559	6.8	6.9	7.1	6.7	6.8	7.0
Maungati	60	60	50	reverse	110				100,000	25.6	16.8	31751	30	19	939	616	6.9	7.1	7.3	6.7	6.9	7.1
Maungatua - North Taie	45	45	30	reverse	110				1,000,000	27.9	23.7	38707	39	34	1527	1297	7.1	7.3	7.5	7.1	7.2	7.4
Mid Dome	60	60	50	reverse	110				1,000,000	33.9	28.6	34473	39	33	1349	1138	7.1	7.2	7.4	7.0	7.2	7.4
Middle Range	60	60	40	reverse	70				1,000,000	23.5	18.6	42463	27	21	1152	912	7.0	7.2	7.4	6.9	7.1	7.3
Moonlight North	60	60	50	reverse	110				100,000	25.4	21.4	103613	29	25	3039	2560	7.4	7.6	7.8	7.4	7.5	7.7
Moonlight South	60	60	50	reverse	110				100,000	33	27.7	69873	38	32	2663	2235	7.4	7.5	7.7	7.3	7.4	7.6
Mossburn	60	60	50	reverse	110					32.8	28.8	26544	38	33	1005	883	7.0	7.1	7.3	6.9	7.0	7.2
Motatapu	60	60	40	reverse	110				10,000	28.3	23.8	28928	33	27	945	795	6.9	7.1	7.3	6.9	7.0	7.2
Mt Sutton	45	45	30	reverse	90				1,000,000	24.8	19.2	32599	35	27	1143	885	7.0	7.2	7.4	6.9	7.0	7.2
Murphys Creek	45	45	30	reverse	110				1,000,000	24.2	20.3	33782	34	29	1156	970	7.0	7.2	7.4	6.9	7.1	7.3
Nevis	45	45	30	reverse	110				10,000	30.5	25.2	48882	43	36	2108	1742	7.3	7.4	7.6	7.2	7.3	7.5
Nichols Rock	45	45	30	reverse	110				1,000,000	27.4	23.2	17876	39	33	693	587	6.8	6.9	7.1	6.7	6.9	7.1
NW Cardrona North	60	60	40	reverse	110				10,000	28.6	24.2	31421	33	28	1038	878	7.0	7.1	7.3	6.9	7.0	7.2
NW Cardrona South	60	60	40	reverse	110				10,000	28.3	23.7	28067	33	27	917	768	6.9	7.1	7.3	6.8	7.0	7.2
Old Man	45	45	35	reverse	70				100,000	31.4	25.7	52064	44	36	2312	1892	7.3	7.5	7.7	7.2	7.4	7.6
Omarama Saddle	60	60	40	reverse	110				10,000	25.2	20.5	35495	29	24	1033	840	7.0	7.1	7.3	6.9	7.0	7.2
Opawa	60	60	50	reverse	110				100,000	26.5	21.2	19484	31	24	596	477	6.7	6.9	7.1	6.6	6.8	7.0
Ostler North	45	45	30	reverse	90				10,000	26.4	23.1	36325	37	33	1356	1187	7.1	7.2	7.4	7.0	7.2	7.4
Ostler South	45	45	30	reverse	90				10,000	27.4	23.2	38111	39	33	1477	1250	7.1	7.3	7.5	7.0	7.2	7.4
Otanomomo	70	70	55	reverse	110				10,000	31.8	27.5	10289	34	29	348	301	6.5	6.6	6.8	6.4	6.6	6.8
Paddys Ridge	60	60	40	reverse	110				1,000,000	28	23.3	20857	32	27	674	561	6.8	6.9	7.1	6.7	6.8	7.0
Pisa	45	45	30	reverse	110				100,000	28.9	24.2	48515	41	34	1983	1660	7.2	7.4	7.6	7.2	7.3	7.5
Pylap	60	60	40	reverse	110				1,000,000	27.3	22.9	23778	32	26	750	629	6.8	7.0	7.2	6.7	6.9	7.1

Name	Dippf	Dipmx	Dipmn	Slip type	Rake_pref	Tpf	Tmn	Tmx	Tgen	D1	D2	Lm	W1	W2	A1	A2	Mw1mn	Mw1pf	Mw1max	Mw2mn	Mw2pf	Mw2mx
Raggedy - Blackstone	45	45	30	reverse	110				1,000,000	26.7	21.9	45004	38	31	1699	1394	7.2	7.3	7.5	7.1	7.2	7.4
Ranfurly	45	45	30	reverse	110				100,000	23.4	19.9	24425	33	28	808	687	6.9	7.0	7.2	6.8	6.9	7.1
Rapuwai	45	45	30	reverse	90				100,000	27.1	21	30230	38	30	1159	898	7.0	7.2	7.4	6.9	7.1	7.3
Roaring Lion	45	45	30	reverse	110				1,000,000	32.1	26	50184	45	37	2278	1845	7.3	7.5	7.7	7.2	7.4	7.6
Roaring Meg	60	60	40	reverse	110				1,000,000	28.7	24	29824	33	28	988	826	6.9	7.1	7.3	6.9	7.0	7.2
Rocks Creek	60	60	40	reverse	110				10,000	26.5	21.8	13108	31	25	401	330	6.6	6.7	6.9	6.5	6.6	6.8
Settlement	45	45	30	reverse	90	125 000	125 000	3 600	100,000	30	24.1	21501	42	34	912	733	6.9	7.1	7.3	6.8	7.0	7.2
Sliver Stream - Merton	45	45	30	reverse	110				1,000,000	26.2	22.4	35452	37	32	1314	1123	7.1	7.2	7.4	7.0	7.2	7.4
Spylaw	45	45	30	reverse	90				10,000	33.4	28.7	31415	47	41	1484	1275	7.1	7.3	7.5	7.1	7.2	7.4
Stonewall	60	60	40	reverse	70				10,000	22.5	17.4	47440	26	20	1233	953	7.0	7.2	7.4	6.9	7.1	7.3
Takapu	45	45	35	reverse	90					30	21.3	55659	42	30	2361	1677	7.3	7.5	7.7	7.2	7.3	7.5
Teviot	45	45	30	reverse	70				100,000	34.4	29	29314	49	41	1426	1202	7.1	7.3	7.5	7.0	7.2	7.4
Timaru Creek	60	60	40	reverse	110				10,000	26.8	22.8	28533	31	26	883	751	6.9	7.0	7.2	6.8	7.0	7.2
Tin Hut	60	60	50	reverse	110					30.4	26.5	31102	35	31	1092	952	7.0	7.1	7.3	6.9	7.1	7.3
Titri Central	45	45	30	reverse	110				10,000	28.5	25.3	29621	40	36	1194	1060	7.0	7.2	7.4	7.0	7.1	7.3
Titri North	45	45	30	reverse	110				10,000	27.2	24.1	29294	38	34	1127	998	7.0	7.2	7.4	6.9	7.1	7.3
Titri South	45	45	30	reverse	110				10,000	29.5	26.3	28564	42	37	1192	1062	7.0	7.2	7.4	7.0	7.1	7.3
Tuapeka	75	75	60	reverse	70				100,000	32.3	27.5	53143	33	28	1777	1513	7.2	7.3	7.5	7.1	7.3	7.5
Waihemo	70	70	60	reverse	70				100,000	22.3	19	91120	24	20	2162	1842	7.3	7.4	7.6	7.2	7.4	7.6
Waikaia	60	60	50	reverse	110				1,000,000	33.8	28.2	69236	39	33	2702	2254	7.4	7.5	7.7	7.3	7.5	7.7
Waikaka	60	60	50	reverse	110				1,000,000	34	28.9	22813	39	33	896	761	6.9	7.1	7.3	6.8	7.0	7.2
Waimate	60	60	50	reverse	110				100,000	21.2	16.8	27973	24	19	685	543	6.8	6.9	7.1	6.7	6.8	7.0
Waipiata	45	45	30	reverse	110				1,000,000	23.7	20.2	34770	34	29	1165	993	7.0	7.2	7.4	6.9	7.1	7.3
Waitahuna 1	45	45	30	reverse	110				1,000,000	28.7	25	22617	41	35	918	800	6.9	7.1	7.3	6.9	7.0	7.2
Waitahuna 2	45	45	30	reverse	110				1,000,000	28.7	25	8674	41	35	352	307	6.5	6.6	6.8	6.4	6.6	6.8
Waitaki	60	60	40	reverse	70				1,000,000	20.8	17.2	35771	24	20	859	710	6.9	7.0	7.2	6.8	7.0	7.2
Waitangi	70	70	60	reverse	110				10,000	24.4	19.2	18115	26	20	470	370	6.6	6.8	7.0	6.5	6.7	6.9
Wanaka	60	60	40	reverse	110				1,000,000	26.5	22.5	67073	31	26	2052	1743	7.3	7.4	7.6	7.2	7.3	7.5
Wendon Valley	60	60	50	reverse	110				1,000,000	34.5	28.9	36999	40	33	1474	1235	7.1	7.3	7.5	7.0	7.2	7.4
West Wakatipu	60	60	45	reverse	110				10,000	28.3	24.2	49967	33	28	1633	1396	7.2	7.3	7.5	7.1	7.2	7.4
White Hill	60	60	50	reverse	110				10,000	33.2	28.9	34073	38	33	1306	1137	7.1	7.2	7.4	7.0	7.2	7.4

Key to parameters

Name = Fault source name
Dippf = Preferred fault dip angle (deg)
Dipmn = Minimum fault dip angle (deg)
Dipmx = Maximum fault dip angle (deg)
Slip typ = dominant sense of slip
Tpf = Preferred recurrence interval (yrs)
Tmn = Minimum recurrence interval (yrs)
Tmx = Maximum recurrence interval (yrs)
Tgen = Generalised recurrence interval (yrs)
D1 = Maximum depth to base of fault source (km)
D2 = Minimum depth to base of fault source (km)
Lm = Length of fault source (m)
W1 = Maximum width of fault source (km)
W1 = Minimum width of fault source (km)
A1 = Maximum area of fault source (km^2)
A2 = Minimum area of fault source (km^2)
Mw1mn = Magnitude from minimum scaling relation Mw = log(A1) + 3.95
Mw1pf = Magnitude from preferred scaling relation Mw = log(A1) + 4.1
Mw1mx = Magnitude from maximum scaling relation Mw = log(A1) + 4.3
Mw2mn = Magnitude from minimum scaling relation Mw = log(A2) + 3.95
Mw2pf = Magnitude from preferred scaling relation Mw = log(A2) + 4.1
Mw2mx = Magnitude from maximum scaling relation Mw = log(A2) + 4.3

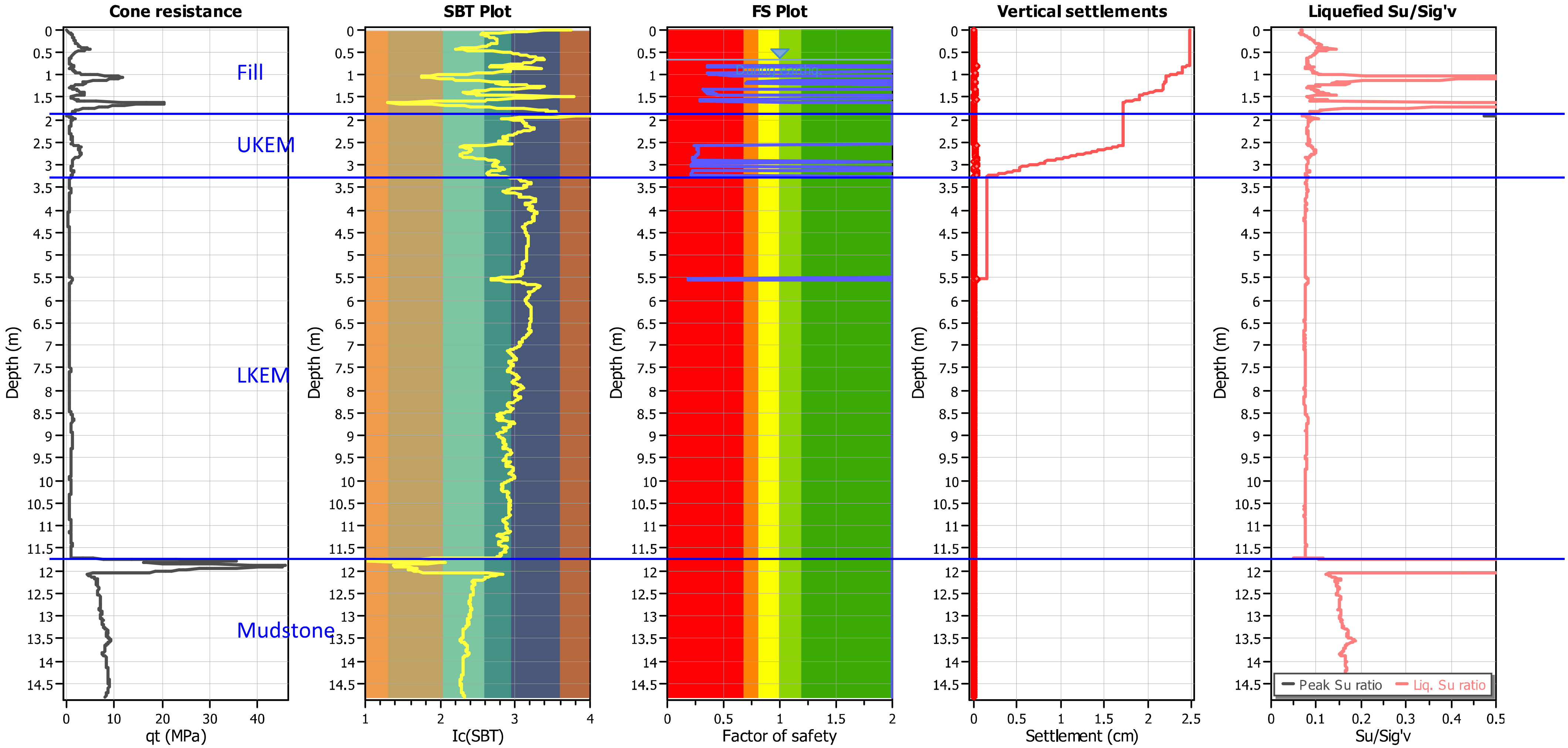
Appendix D

CLiq Outputs

GHD Limited
138 Victora Street, Level 3
Christchurch Central, Canterbury 8013, New Zealand
https://www.ghd.com

Project: **GILF Closure Consent**
Location: **Dunedin**

CPT: CPT100
Total depth: 14.80 m

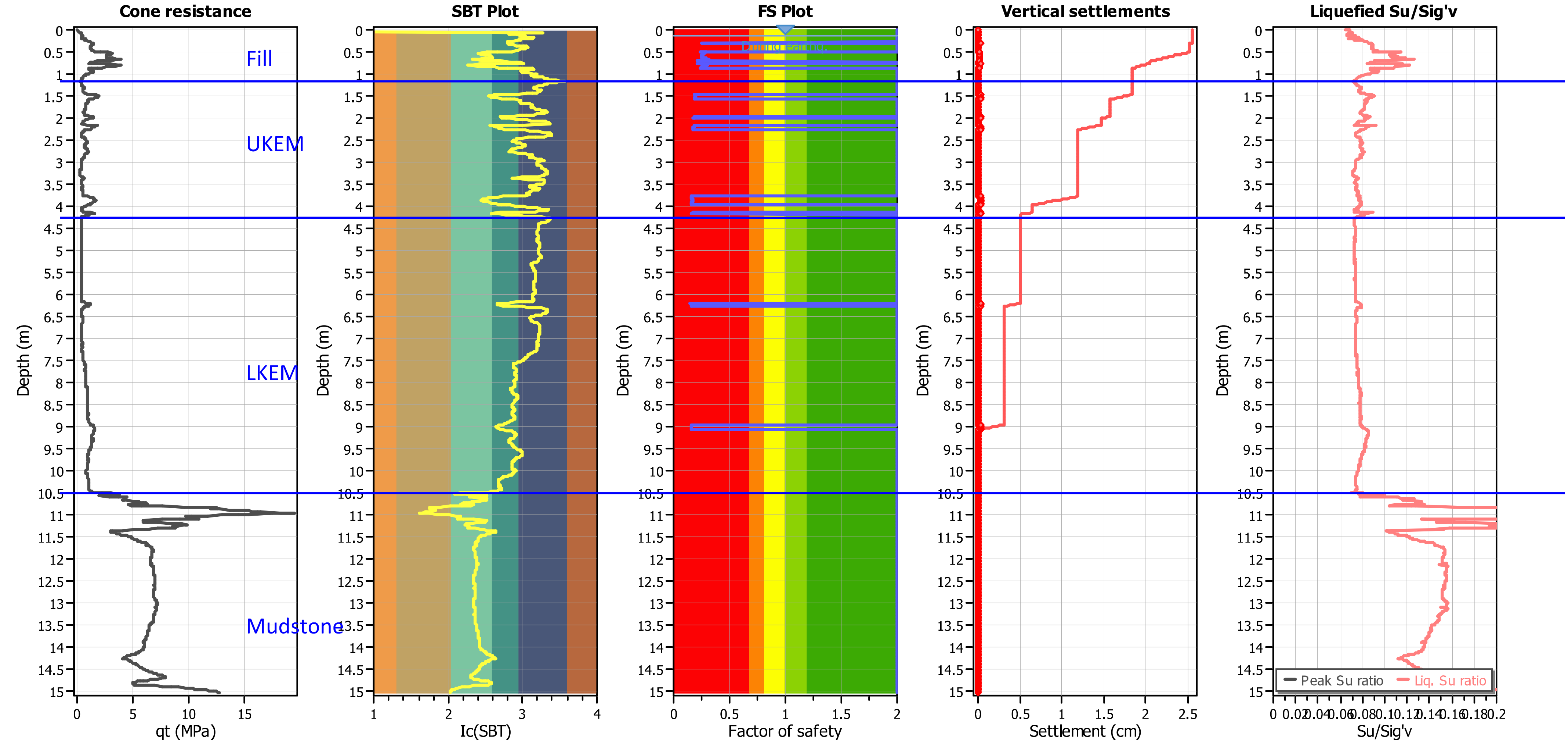


Analysis method:	B&I (2014)	G.W.T. (in-situ):	0.68 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.68 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

GHD Limited
138 Victora Street, Level 3
Christchurch Central, Canterbury 8013, New Zealand
https://www.ghd.com

Project: **GILF Closure Consent**
Location: **Dunedin**

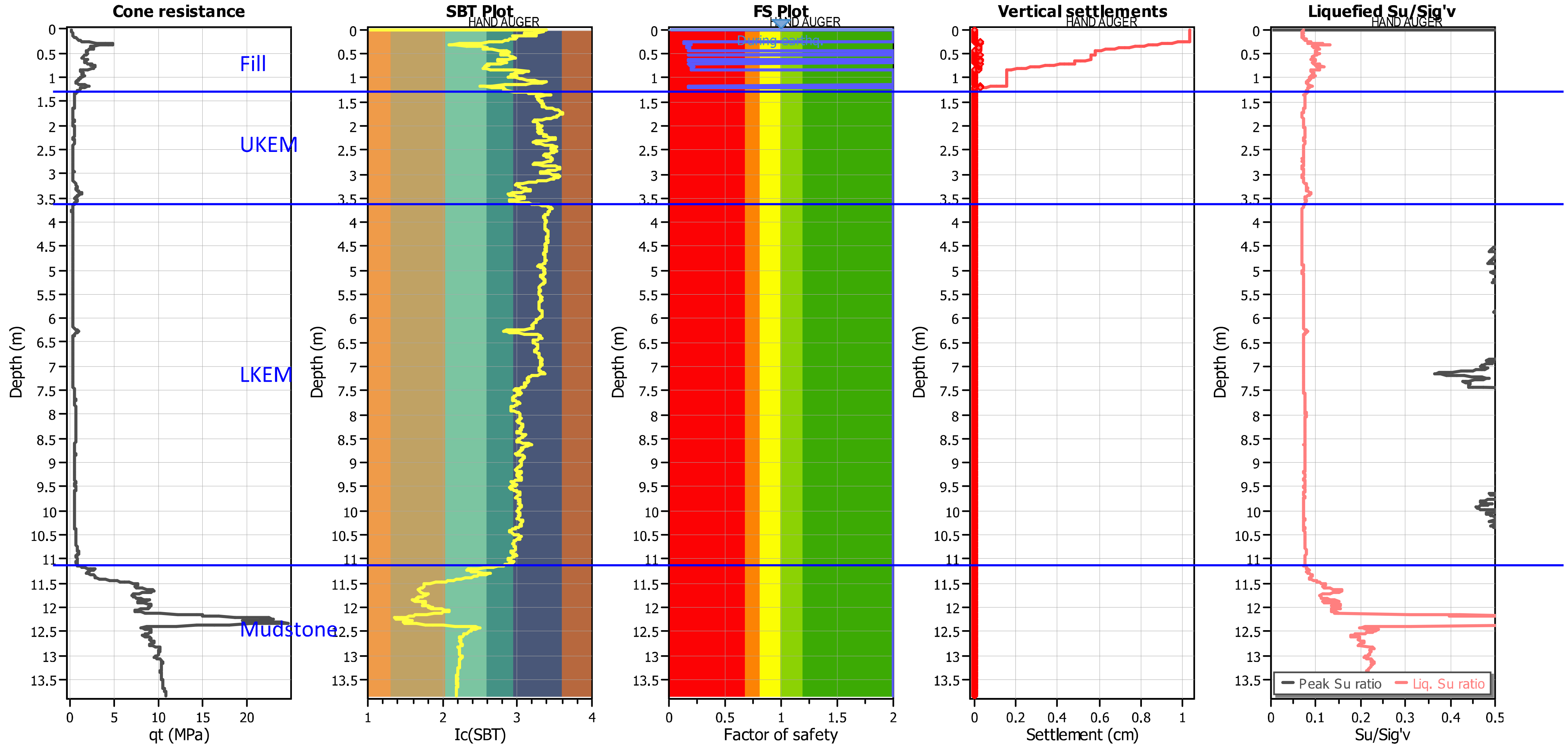
CPT: CPT101
Total depth: 15.04 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	0.13 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.13 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project: GILF Closure Consent
Location: Dunedin

CPT: CPT102
Total depth: 13.84 m



Analysis method: B&I (2014)
Fines correction method: B&I (2014)
Points to test: Based on Ic value
Earthquake magnitude M_w: 7.30
Peak ground acceleration: 0.51

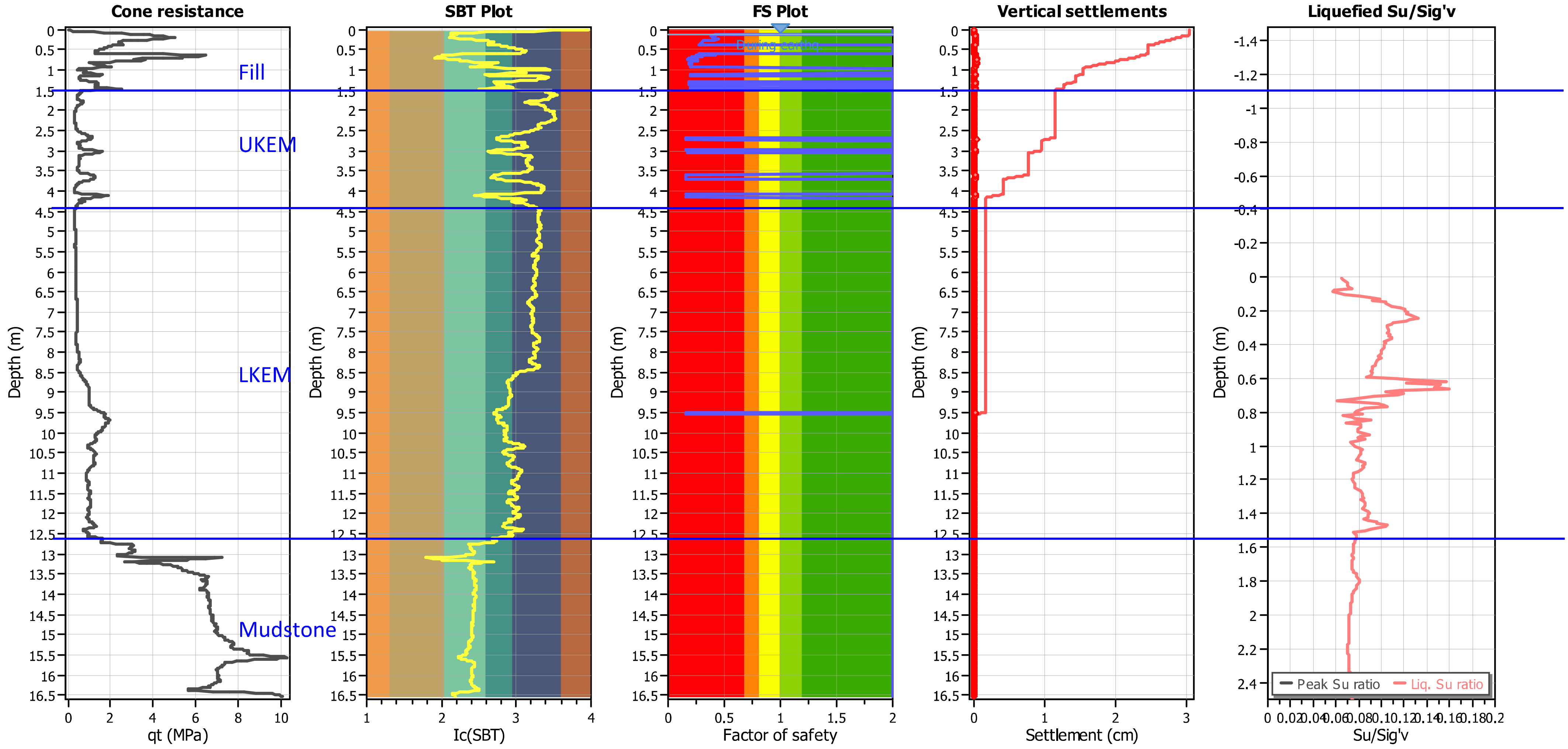
G.W.T. (in-situ): 0.00 m
G.W.T. (earthq.): 0.00 m
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT

Use fill: No
Fill height: N/A
Fill weight: N/A
Trans. detect. applied: No
K_σ applied: Yes

Clay like behavior applied: .
Limit depth applied: No
Limit depth: N/A
MSF method: Method based

Project: **GILF Closure Consent**
Location: **Dunedin**

CPT: CPT103
Total depth: 16.53 m



Analysis method: B&I (2014)
Fines correction method: B&I (2014)
Points to test: Based on Ic value
Earthquake magnitude M_w: 7.30
Peak ground acceleration: 0.51

G.W.T. (in-situ): 0.12 m
G.W.T. (earthq.): 0.12 m
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT

Use fill: No
Fill height: N/A
Fill weight: N/A
Trans. detect. applied: No
K_σ applied: Yes

Clay like behavior applied: .
Limit depth applied: No
Limit depth: N/A
MSF method: Method based

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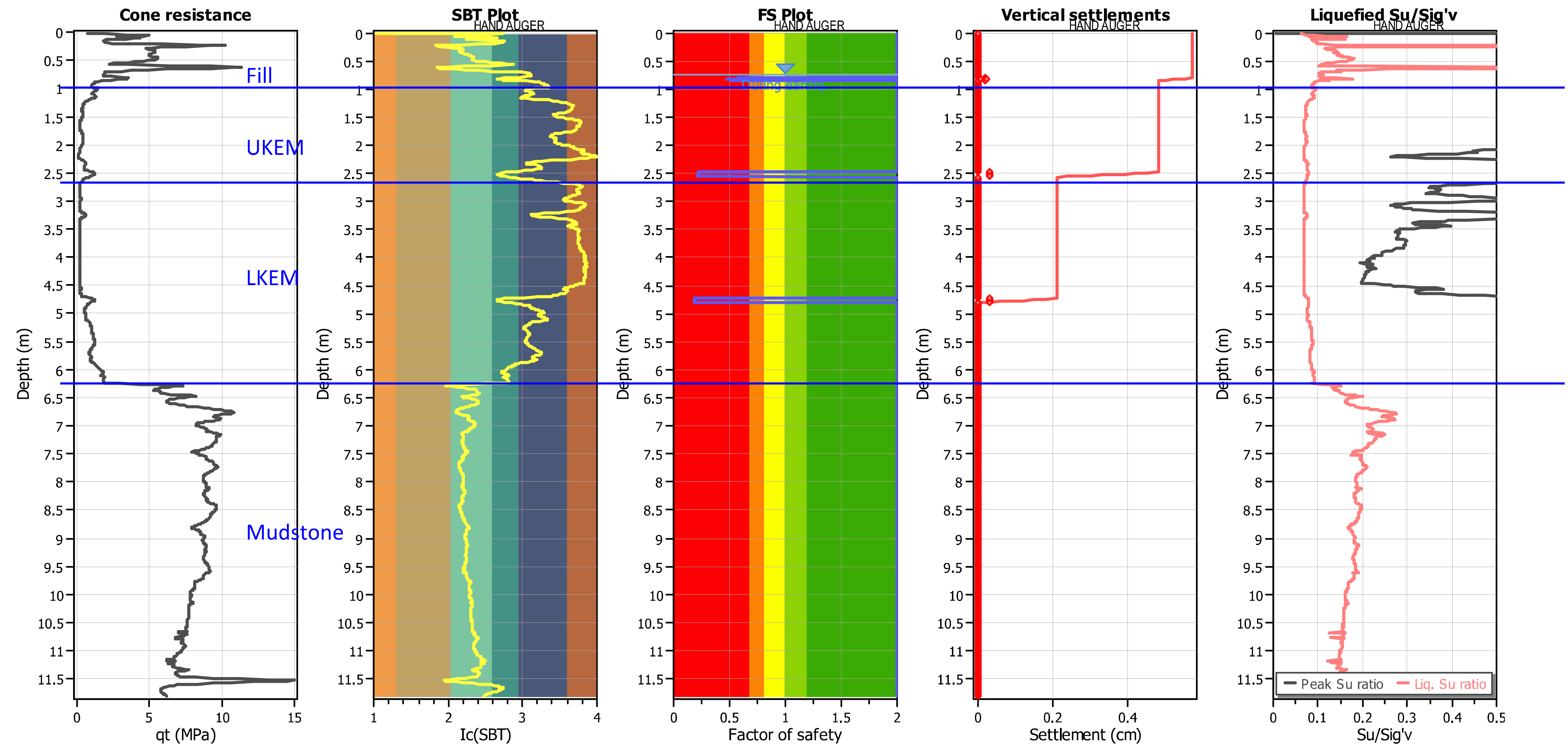
https://www.ghd.com

Project: **GILF Closure Consent**
Location: **Dunedin**

CPT: CPT104
Total depth: 11.81 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	0.73 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.73 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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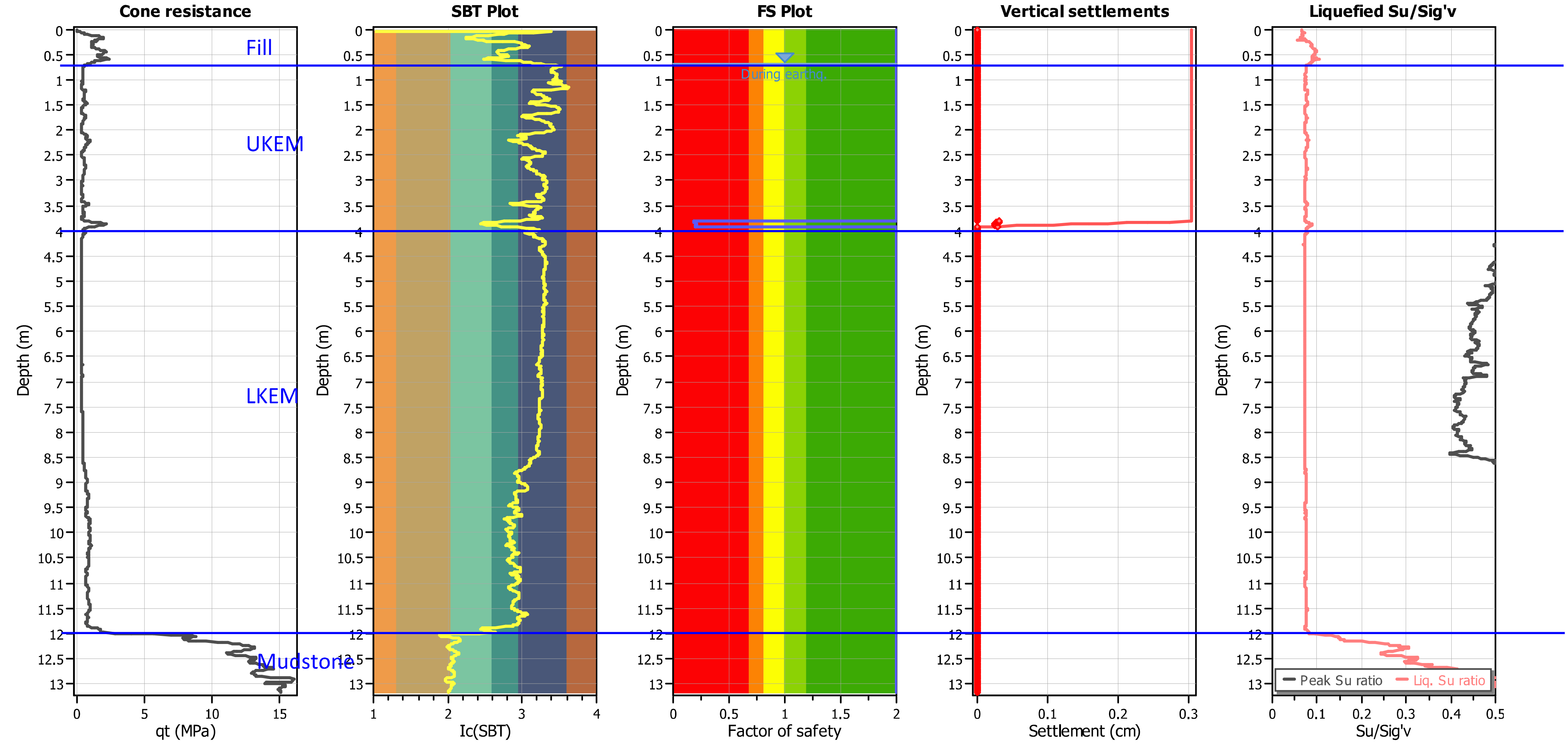
Project: **GILF Closure Consent**
Location: **Dunedin**

CPT: CPT105
Total depth: 13.17 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	0.68 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.68 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

GHD Limited
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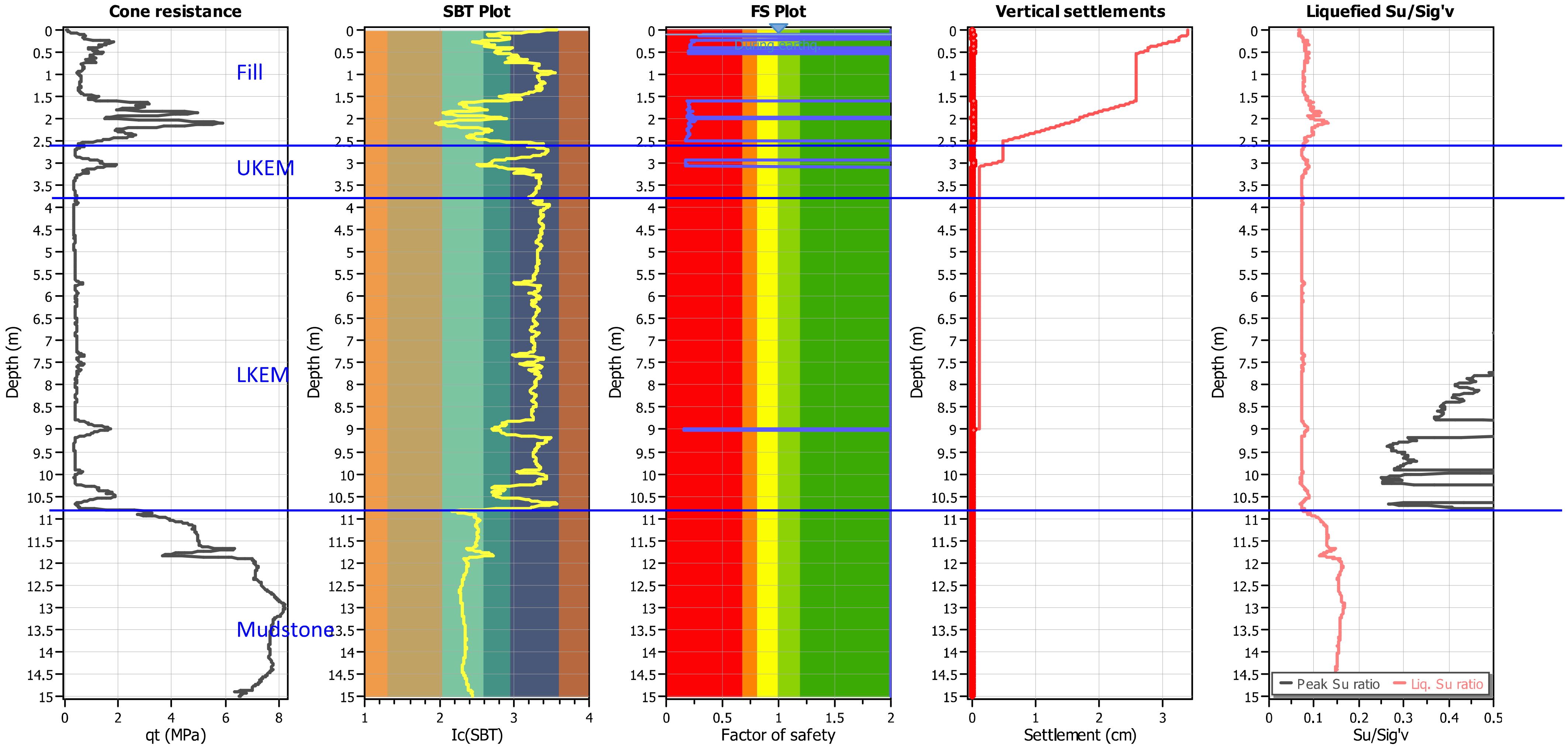
Project: **GILF Closure Consent**
Location: **Dunedin**

CPT: CPT108
Total depth: 15.00 m



Analysis method:	B&I (2014)	G.W.T. (in-situ):	0.11 m	Use fill:	No	Clay like behavior applied:	.
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	0.11 m	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

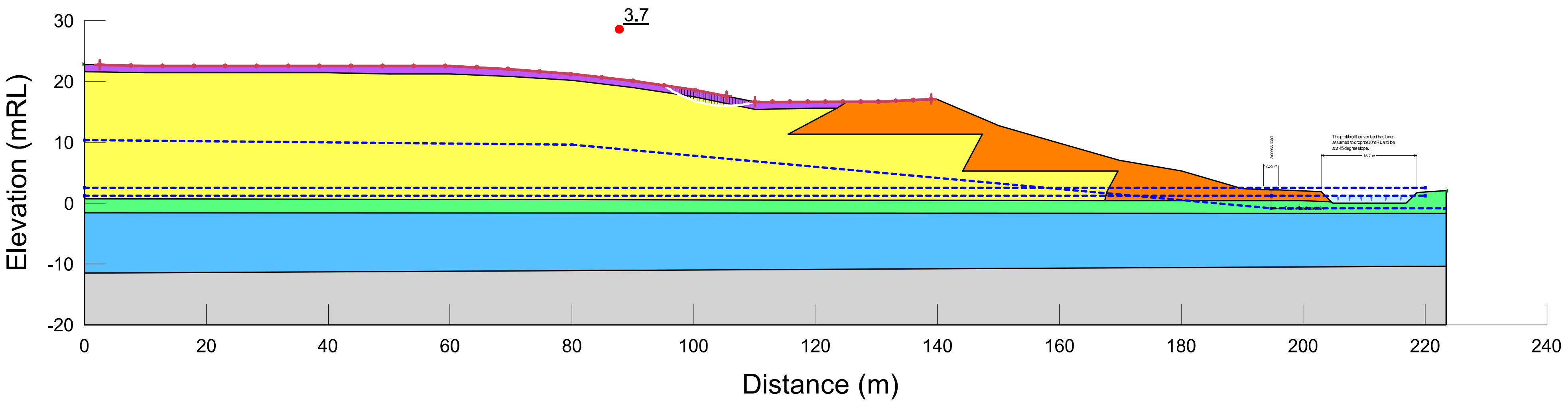
Appendix E

Slope/W Outputs

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟧	Bund	Mohr-Coulomb	17	1	27	1
🟨	Fill	Mohr-Coulomb	14.5	3	25	1
🟪	Final capping	Mohr-Coulomb	17	2	29	1
🟦	LKEM	Mohr-Coulomb	15.5	0	24	3
🟩	UKEM	Mohr-Coulomb	16	0	26	2
⬜	Weathered Mudstone	Mohr-Coulomb	18	10	32	

Horz Seismic Coef.:

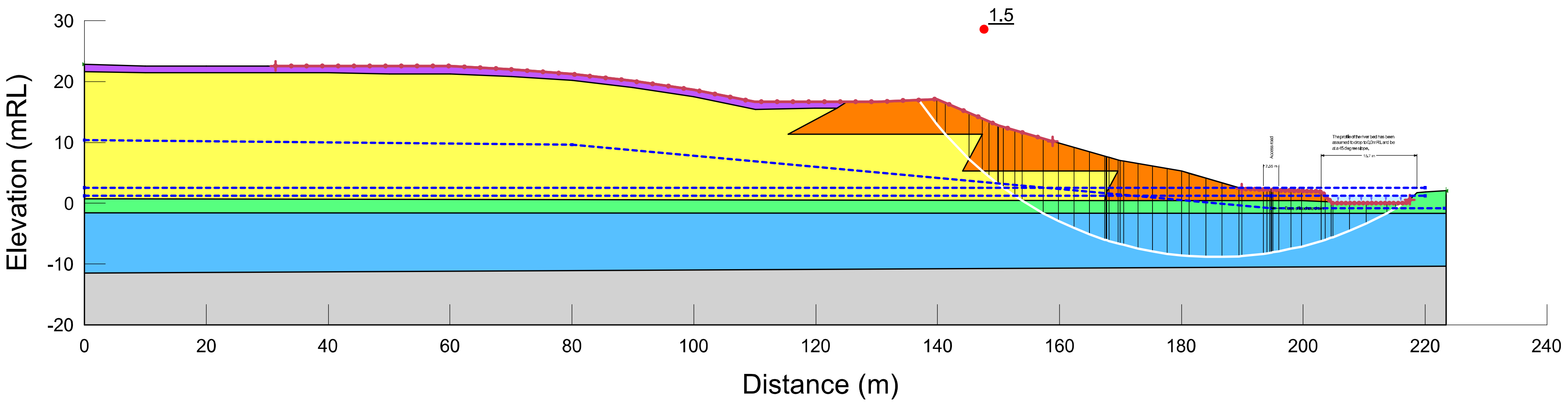


01. Static long term groundwater level - local
GILF Section 1 revC.gsz
18/01/2023 1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟧	Bund	Mohr-Coulomb	17	1	27	1
🟨	Fill	Mohr-Coulomb	14.5	3	25	1
🟪	Final capping	Mohr-Coulomb	17	2	29	1
🟦	LKEM	Mohr-Coulomb	15.5	0	24	3
🟩	UKEM	Mohr-Coulomb	16	0	26	2
⬜	Weathered Mudstone	Mohr-Coulomb	18	10	32	

Horz Seismic Coef.:



02. Static long term groundwater level - global

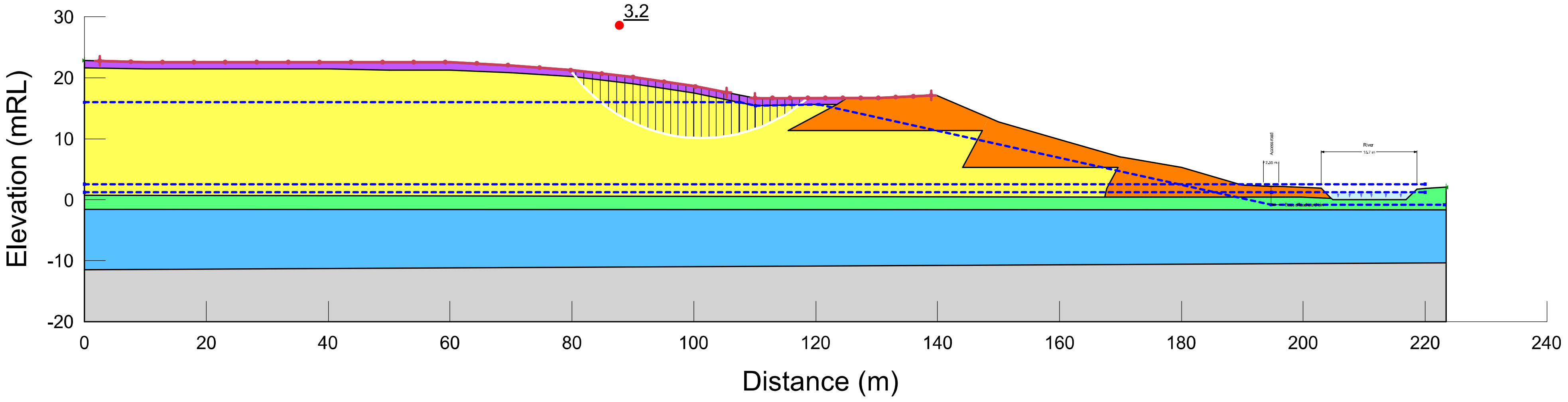
GILF Section 1 revC.gsz

18/01/2023 1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟧	Bund	Mohr-Coulomb	17	1	27	1
🟨	Fill	Mohr-Coulomb	14.5	3	25	1
🟪	Final capping	Mohr-Coulomb	17	2	29	1
🟦	LKEM	Mohr-Coulomb	15.5	0	24	3
🟩	UKEM	Mohr-Coulomb	16	0	26	2
⬜	Weathered Mudstone	Mohr-Coulomb	18	10	32	

Horz Seismic Coef.:



03. Static elevated groundwater level - local

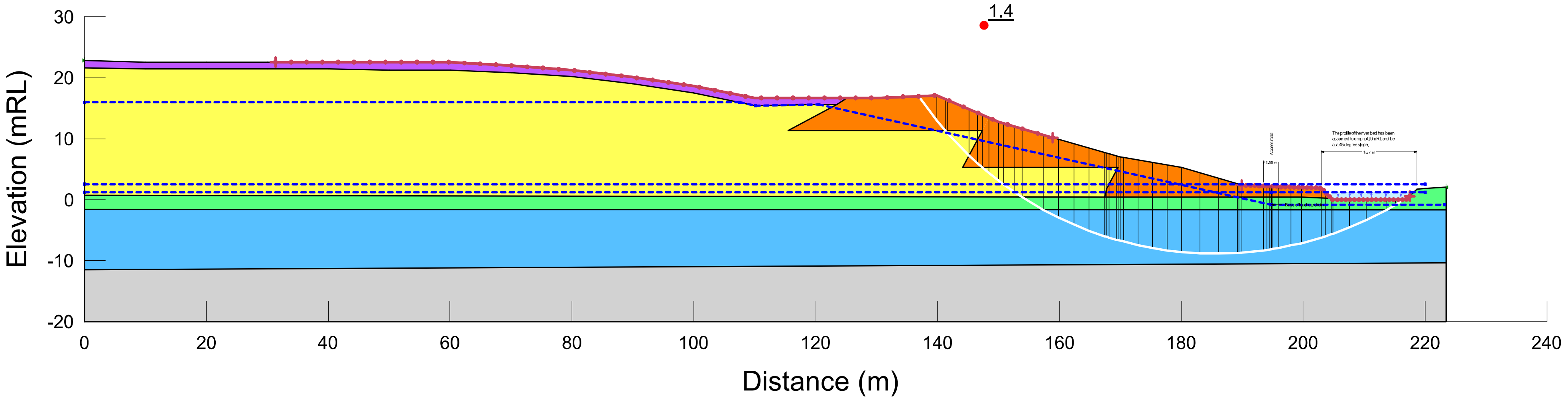
GILF Section 1 revC.gsz

18/01/2023 1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟧	Bund	Mohr-Coulomb	17	1	27	1
🟨	Fill	Mohr-Coulomb	14.5	3	25	1
🟪	Final capping	Mohr-Coulomb	17	2	29	1
🟦	LKEM	Mohr-Coulomb	15.5	0	24	3
🟩	UKEM	Mohr-Coulomb	16	0	26	2
⬜	Weathered Mudstone	Mohr-Coulomb	18	10	32	

Horz Seismic Coef.:



04. Static elevated groundwater level - global

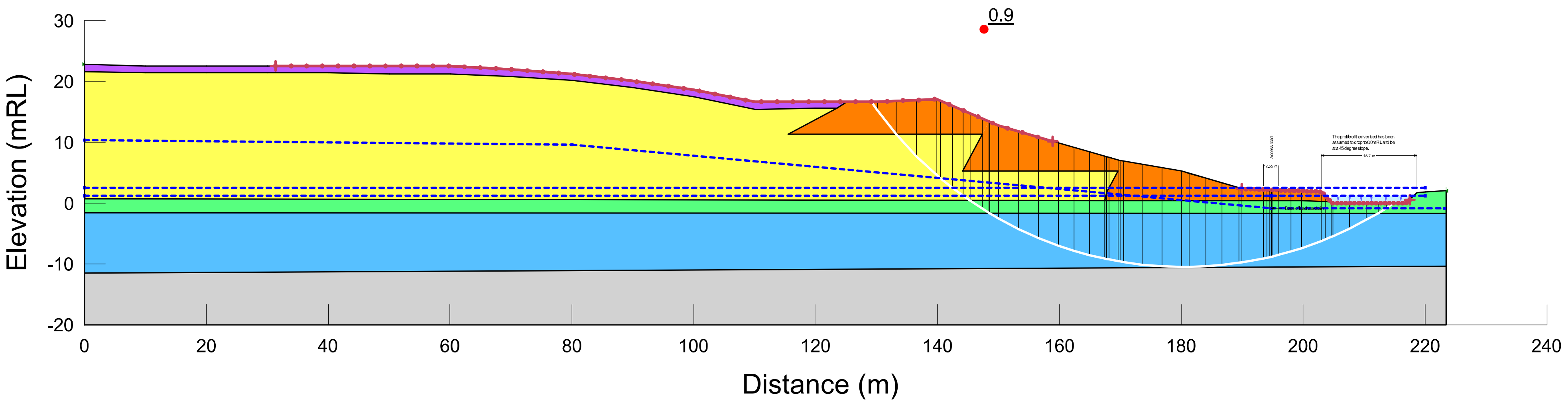
GILF Section 1 revC.gsz

18/01/2023 1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Minimum Strength	Tau/Sigma Ratio	Effective Cohesion	Effective Friction Angle	Total Cohesion	Piezometric Line
🟧	Bund (undrained)	Undrained (Phi=0)	17					75	1
🟨	Fill	Mohr-Coulomb	14.5			3	25		1
🟪	Final capping (undrained)	Undrained (Phi=0)	17					100	1
🟦	LKEM (undrained)	SHANSEP	15.5	15	0.23				3
🟩	UKEM	Mohr-Coulomb	16			0	26		2
⬜	Weathered Mudstone (undrained)	Undrained (Phi=0)	18					200	

Horz Seismic Coef.: 0.06



06. Seismic SLS (undrained)

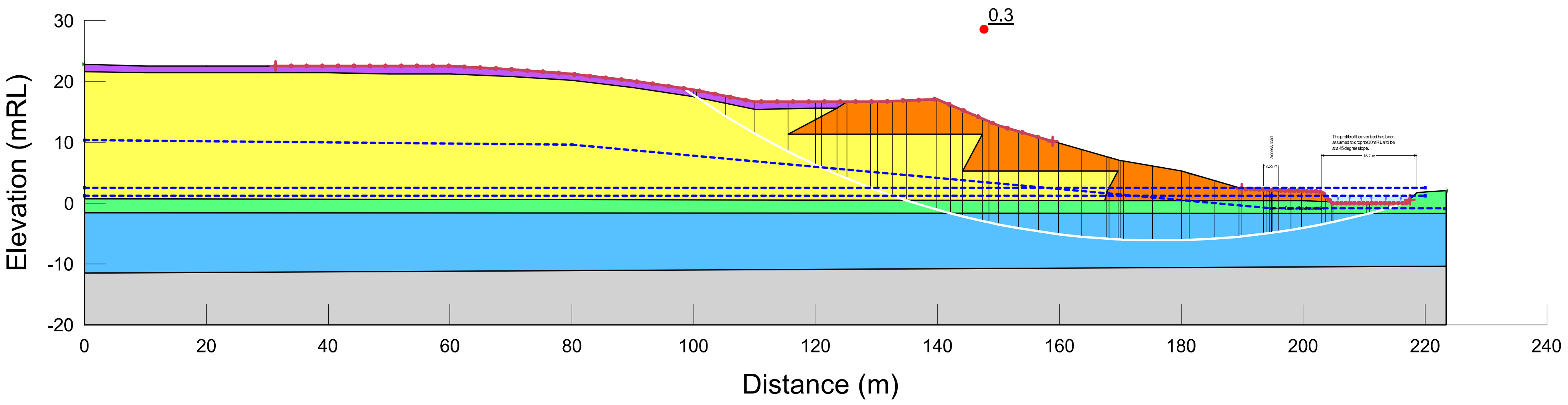
GILF Section 1 revC.gsz

18/01/2023 1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Minimum Strength	Tau/Sigma Ratio	Effective Cohesion	Effective Friction Angle	Total Cohesion	Piezometric Line
🟧	Bund (undrained)	Undrained (Phi=0)	17					75	1
🟨	Fill	Mohr-Coulomb	14.5			3	25		1
🟪	Final capping (undrained)	Undrained (Phi=0)	17					100	1
🟦	LKEM (undrained)	SHANSEP	15.5	15	0.23				3
🟩	UKEM	Mohr-Coulomb	16			0	26		2
⬜	Weathered Mudstone (undrained)	Undrained (Phi=0)	18					200	

Horz Seismic Coef.: 0.51

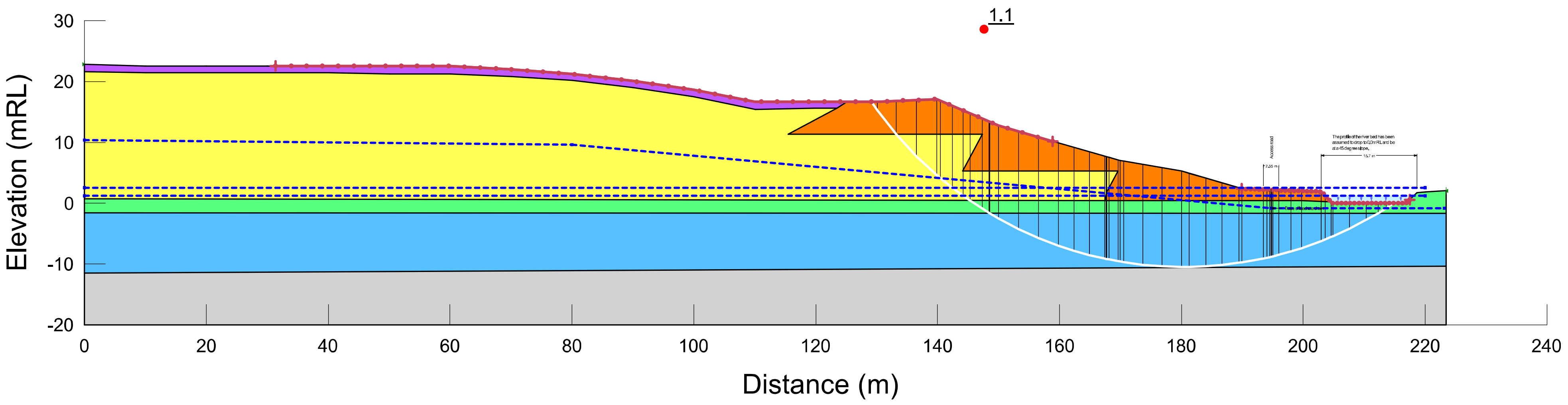


08. Seismic ULS (undrained)	
GILF Section 1 revC.gsz	
18/01/2023	1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Minimum Strength	Tau/Sigma Ratio	Effective Cohesion	Effective Friction Angle	Total Cohesion	Piezometric Line
🟧	Bund (undrained)	Undrained (Phi=0)	17					75	1
🟨	Fill	Mohr-Coulomb	14.5			3	25		1
🟪	Final capping (undrained)	Undrained (Phi=0)	17					100	1
🟦	LKEM (undrained)	SHANSEP	15.5	15	0.23				3
🟩	UKEM (liq)	SHANSEP	16	0	0.08				2
⬜	Weathered Mudstone (undrained)	Undrained (Phi=0)	18					200	

Horz Seismic Coef.: 0



10. Post seismic

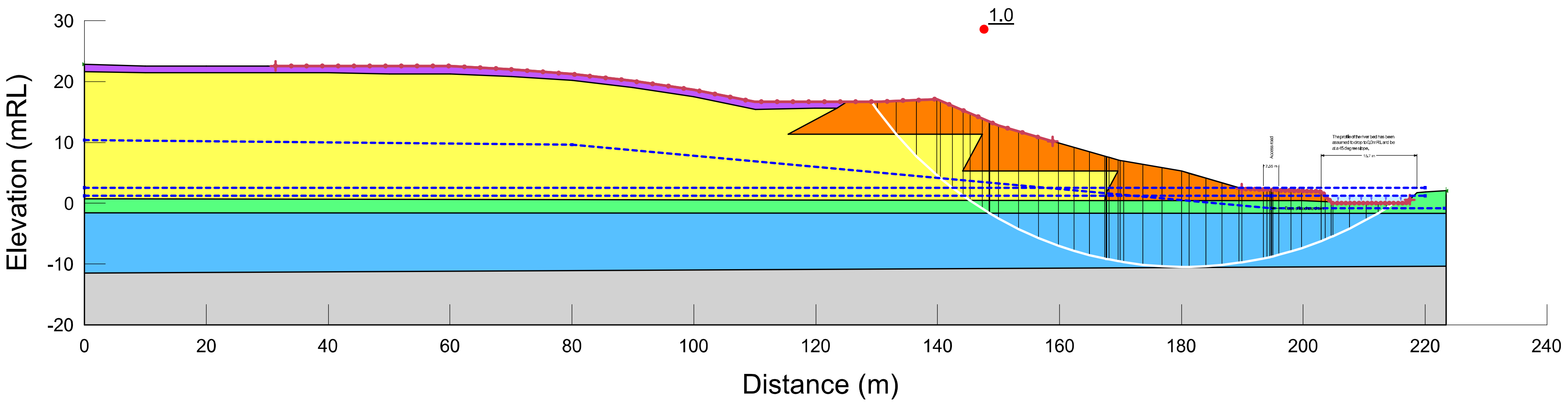
GILF Section 1 revC.gsz

18/01/2023 1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Minimum Strength	Tau/Sigma Ratio	Effective Cohesion	Effective Friction Angle	Total Cohesion	Piezometric Line
🟧	Bund (undrained)	Undrained (Phi=0)	17					75	1
🟨	Fill	Mohr-Coulomb	14.5			3	25		1
🟪	Final capping (undrained)	Undrained (Phi=0)	17					100	1
🟦	LKEM (undrained)	SHANSEP	15.5	15	0.23				3
🟩	UKEM	Mohr-Coulomb	16			0	26		2
⬜	Weathered Mudstone (undrained)	Undrained (Phi=0)	18					200	

Horz Seismic Coef.: 0.041

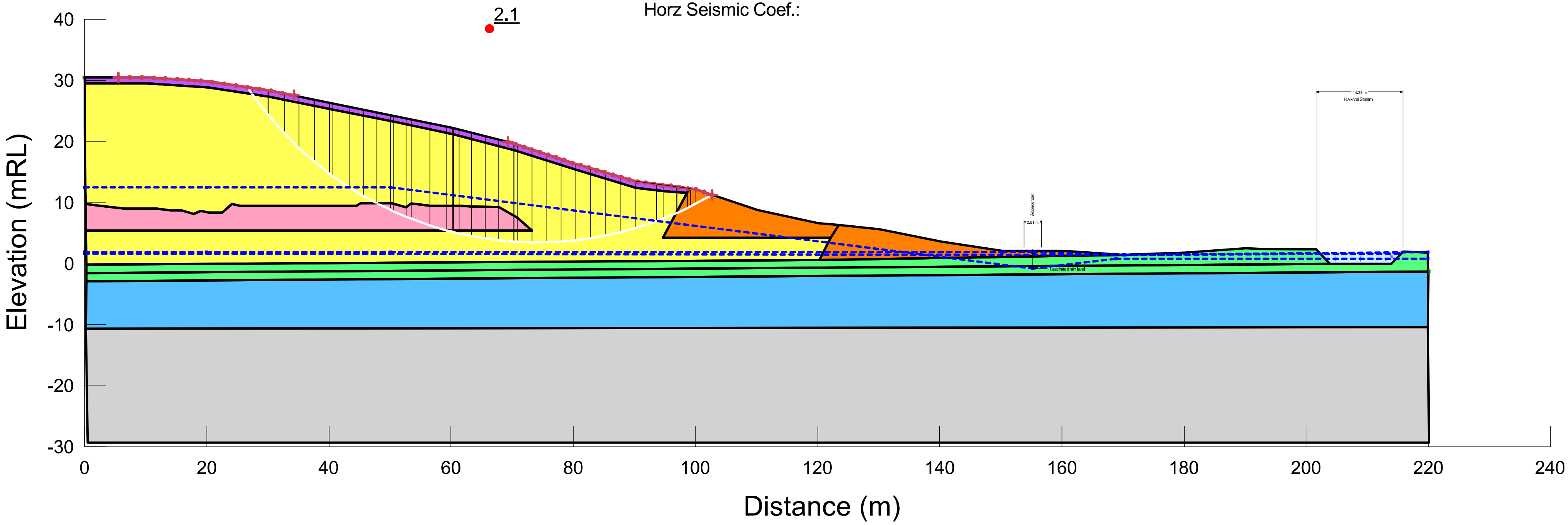


11. Seismic yield (undrained)
GILF Section 1 revC.gsz
18/01/2023
1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟧	Bund	Mohr-Coulomb	17	1	27	1
🟨	Fill	Mohr-Coulomb	14.5	3	25	1
🟪	Final capping	Mohr-Coulomb	17	2	29	1
🟦	LKEM	Mohr-Coulomb	15.5	0	24	3
🟥	Sludge	Mohr-Coulomb	13	0	24	1
🟩	UKEM	Mohr-Coulomb	16	0	26	2
⬜	Weathered Mudstone	Mohr-Coulomb	18	10	32	

Horz Seismic Coef.:

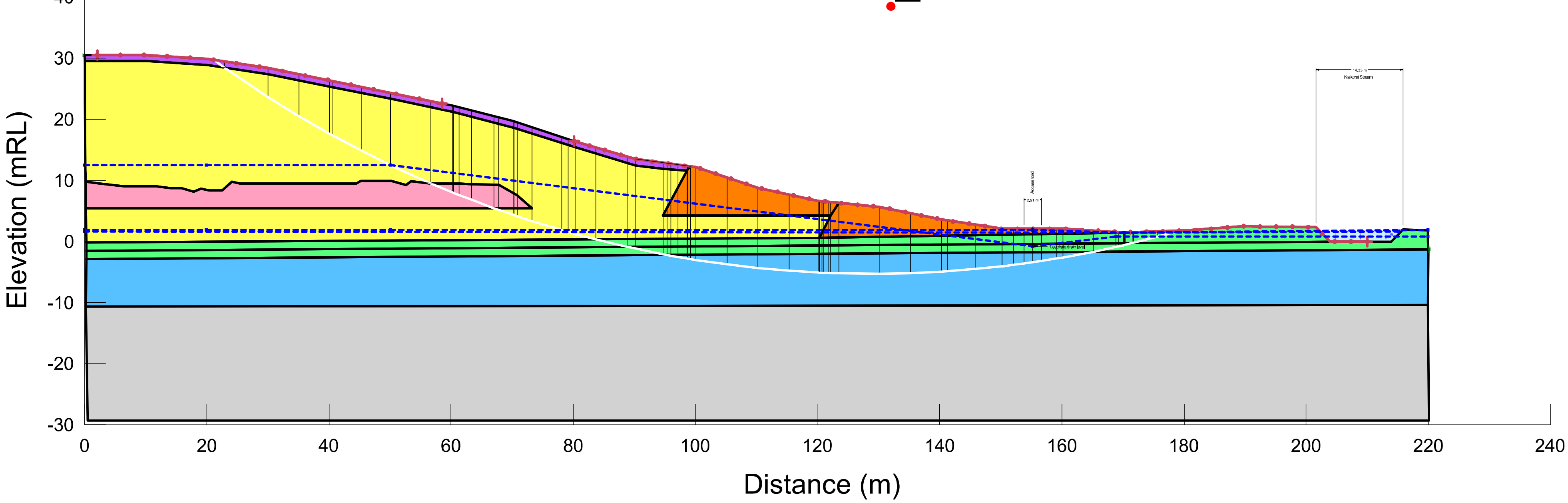


01. Static long term groundwater level - local
GILF Section 2 revC.gsz
07/12/2022 1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟧	Bund	Mohr-Coulomb	17	1	27	1
🟨	Fill	Mohr-Coulomb	14.5	3	25	1
🟪	Final capping	Mohr-Coulomb	17	2	29	1
🟦	LKEM	Mohr-Coulomb	15.5	0	24	3
▨	Sludge	Mohr-Coulomb	13	0	24	1
🟩	UKEM	Mohr-Coulomb	16	0	26	2
⬜	Weathered Mudstone	Mohr-Coulomb	18	10	32	

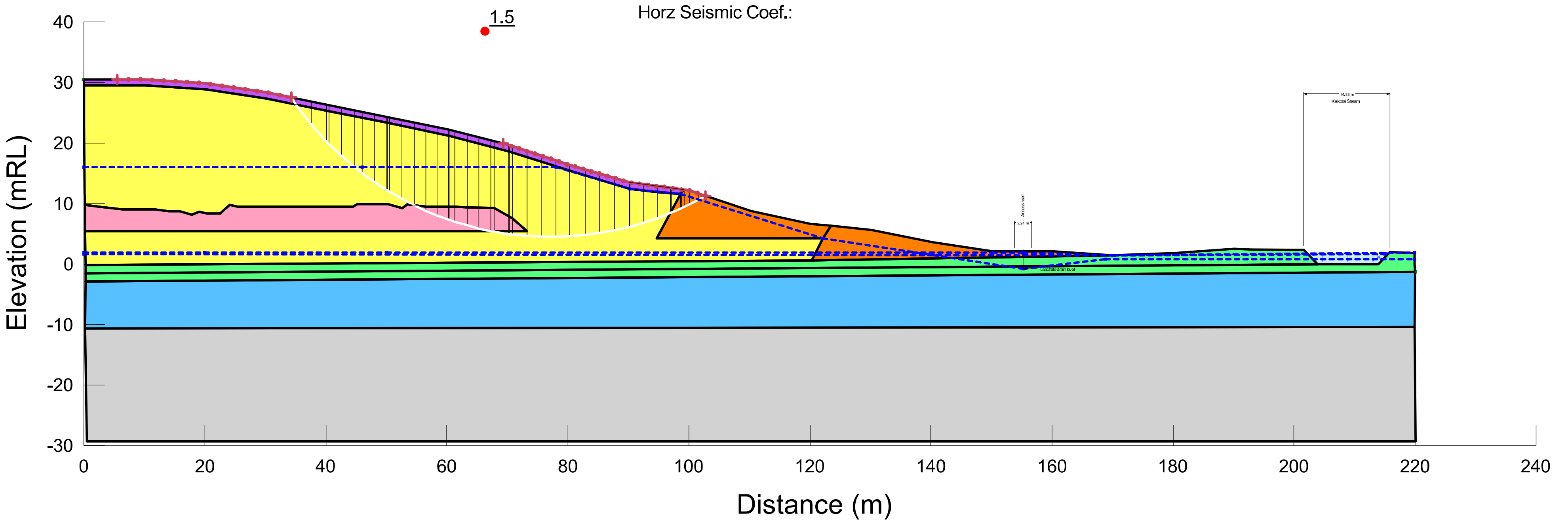
Horz Seismic Coef.: 1.8



02. Static long term groundwater level - global
GILF Section 2 revC.gsz
07/12/2022 1:700

Analysis Type: Morgenstern-Price

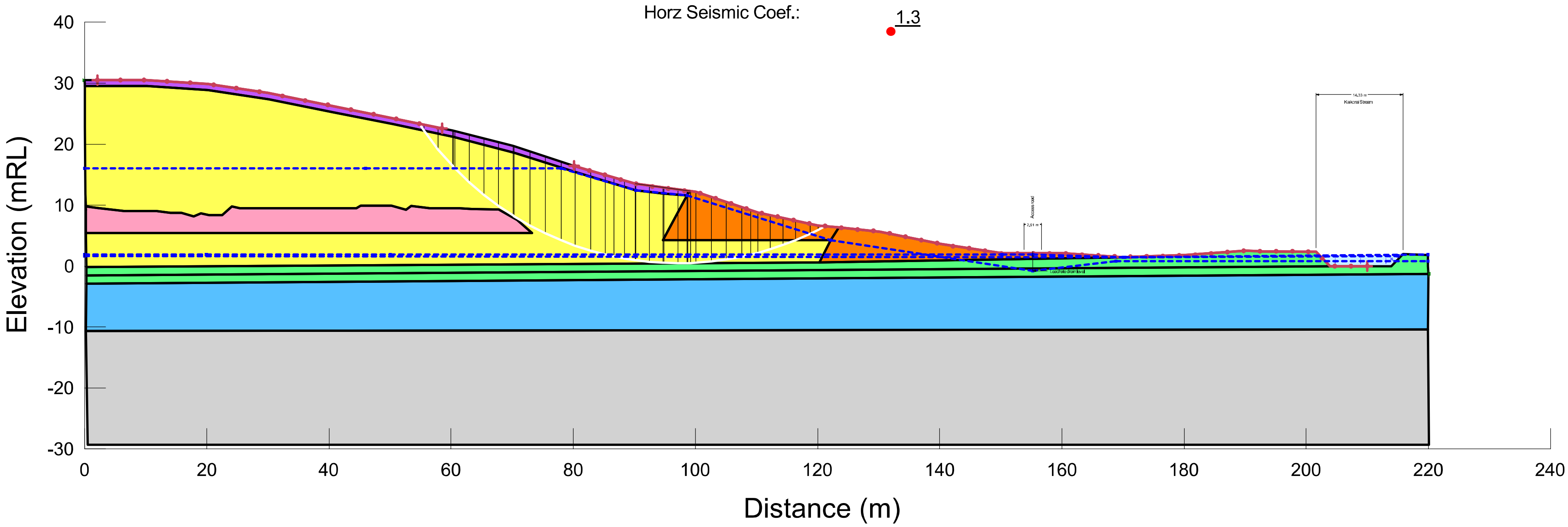
Color	Name	Slope Stability Material Model	Unit Weight	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟧	Bund	Mohr-Coulomb	17	1	27	1
🟨	Fill	Mohr-Coulomb	14.5	3	25	1
🟪	Final capping	Mohr-Coulomb	17	2	29	1
🟦	LKEM	Mohr-Coulomb	15.5	0	24	3
▨	Sludge	Mohr-Coulomb	13	0	24	1
🟩	UKEM	Mohr-Coulomb	16	0	26	2
⬜	Weathered Mudstone	Mohr-Coulomb	18	10	32	



03. Static elevated groundwater level - local
GILF Section 2 revC.gsz
07/12/2022 1:700

Analysis Type: Morgenstern-Price

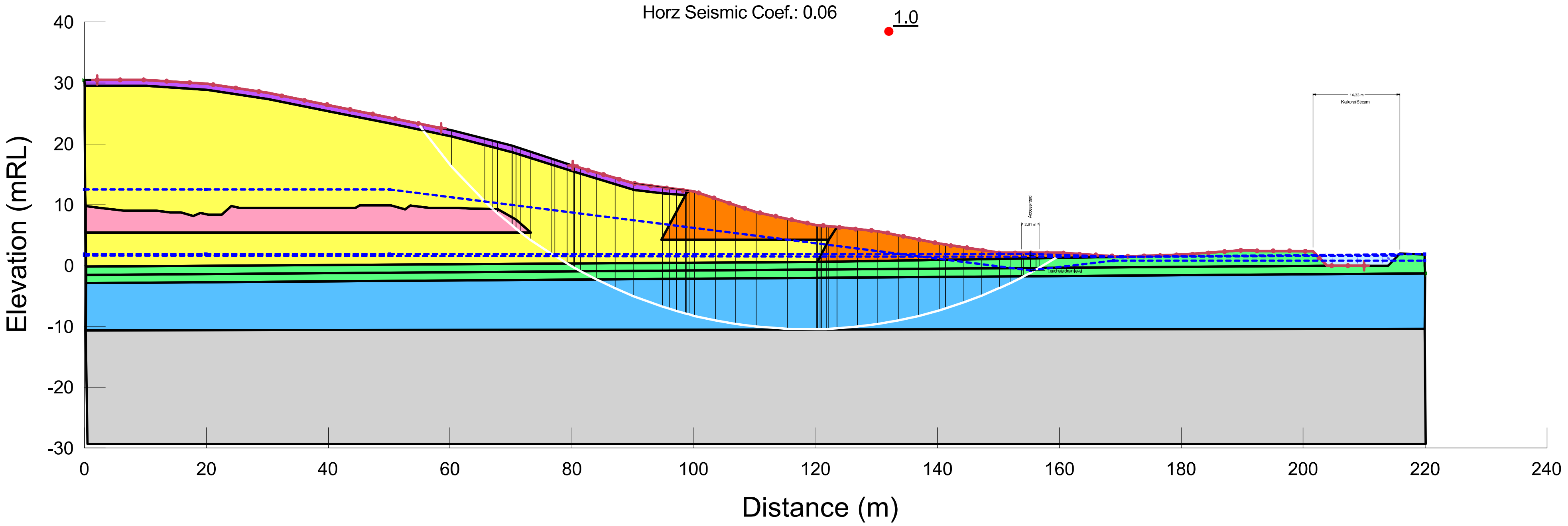
Color	Name	Slope Stability Material Model	Unit Weight	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟧	Bund	Mohr-Coulomb	17	1	27	1
🟨	Fill	Mohr-Coulomb	14.5	3	25	1
🟪	Final capping	Mohr-Coulomb	17	2	29	1
🟦	LKEM	Mohr-Coulomb	15.5	0	24	3
🟥	Sludge	Mohr-Coulomb	13	0	24	1
🟩	UKEM	Mohr-Coulomb	16	0	26	2
⬜	Weathered Mudstone	Mohr-Coulomb	18	10	32	



04. Static elevated groundwater level - global
GILF Section 2 revC.gsz
07/12/2022 1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Minimum Strength	Tau/Sigma Ratio	Effective Cohesion	Effective Friction Angle	Total Cohesion	Piezomet Line
🟧	Bund (undrained)	Undrained (Phi=0)	17					75	1
🟨	Fill	Mohr-Coulomb	14.5			3	25		1
🟪	Final capping (undrained)	Undrained (Phi=0)	17					100	1
🟦	LKEM (undrained)	SHANSEP	15.5	15	0.23				3
▨	Sludge	Mohr-Coulomb	13			0	24		1
🟩	UKEM	Mohr-Coulomb	16			0	26		2
⬜	Weathered Mudstone (undrained)	Undrained (Phi=0)	18					200	

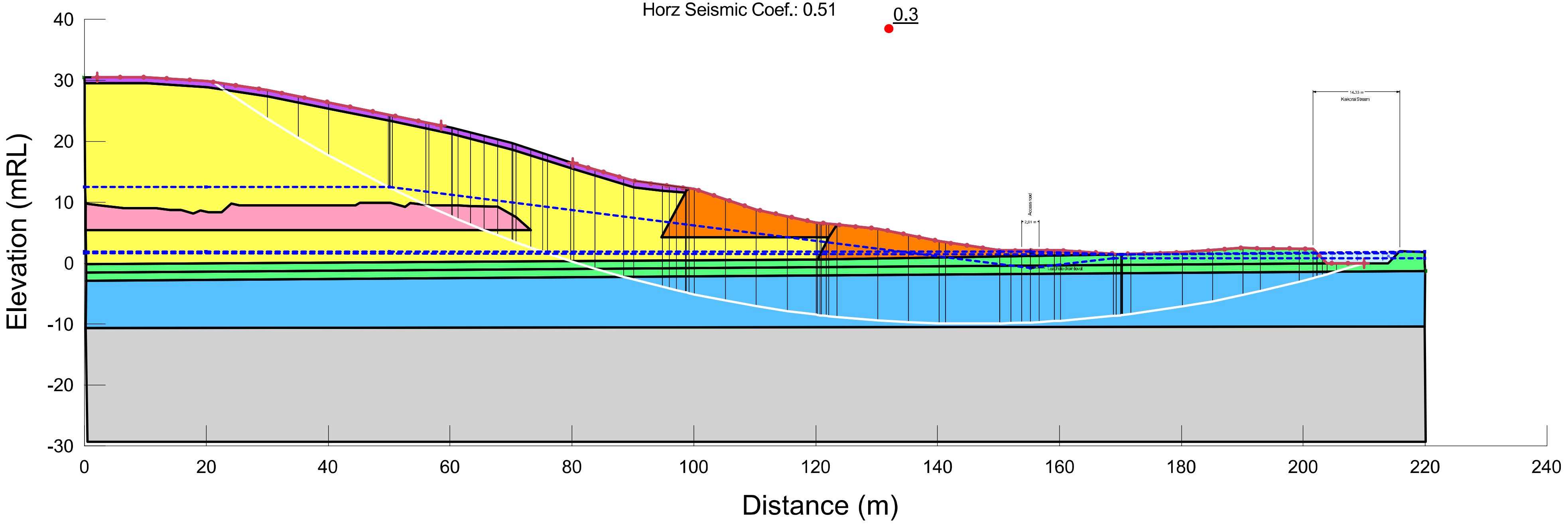


06. Seismic SLS (undrained)
GILF Section 2 revC.gsz
07/12/2022 1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Minimum Strength	Tau/Sigma Ratio	Effective Cohesion	Effective Friction Angle	Total Cohesion	Piezomet Line
🟧	Bund (undrained)	Undrained (Phi=0)	17					75	1
🟨	Fill	Mohr-Coulomb	14.5			3	25		1
🟪	Final capping (undrained)	Undrained (Phi=0)	17					100	1
🟦	LKEM (undrained)	SHANSEP	15.5	15	0.23				3
🟥	Sludge	Mohr-Coulomb	13			0	24		1
🟩	UKEM	Mohr-Coulomb	16			0	26		2
⬜	Weathered Mudstone (undrained)	Undrained (Phi=0)	18					200	

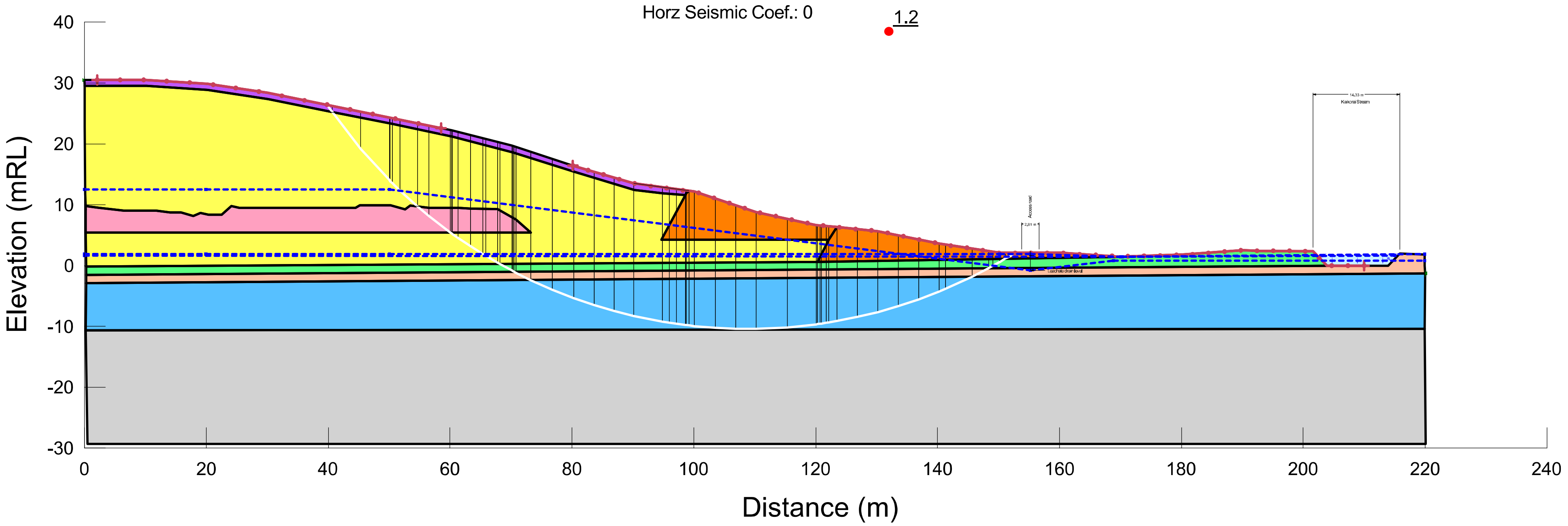
Horz Seismic Coef.: 0.51



08. Seismic ULS (undrained)
GILF Section 2 revC.gsz
07/12/2022 1:700

Analysis Type: Morgenstern-Price

		Material Model	Weight	Strength	Ratio	Cohesion	Friction Angle	Cohesion	Line
🟧	Bund (undrained)	Undrained (Phi=0)	17					75	1
🟨	Fill	Mohr-Coulomb	14.5			3	25		1
🟪	Final capping (undrained)	Undrained (Phi=0)	17					100	1
🟦	LKEM (undrained)	SHANSEP	15.5	15	0.23				3
▨	Sludge	Mohr-Coulomb	13			0	24		1
🟩	UKEM	Mohr-Coulomb	16			0	26		2
▨	UKEM (liq)	SHANSEP	16	0	0.08				2
⬜	Weathered Mudstone (undrained)	Undrained (Phi=0)	18					200	2



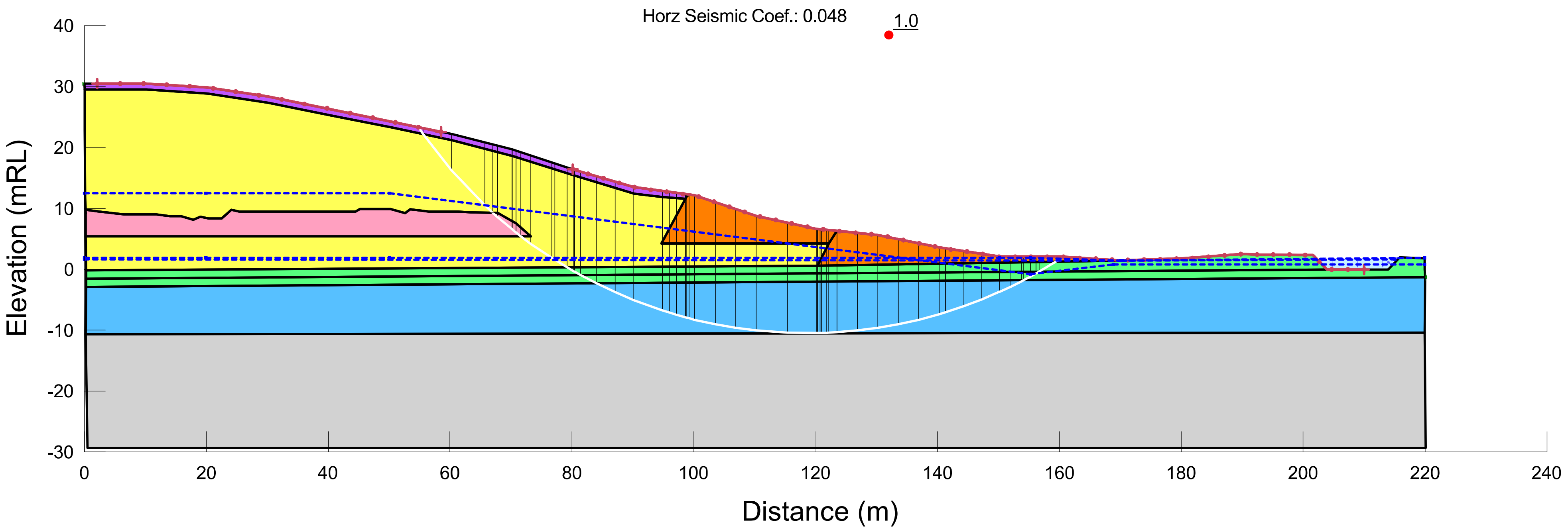
10. Post seismic

GILF Section 2 revC.gsz

07/12/2022 1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Minimum Strength	Tau/Sigma Ratio	Effective Cohesion	Effective Friction Angle	Total Cohesion	Piezomet Line
🟧	Bund (undrained)	Undrained (Phi=0)	17					75	1
🟨	Fill	Mohr-Coulomb	14.5			3	25		1
🟪	Final capping (undrained)	Undrained (Phi=0)	17					100	1
🟦	LKEM (undrained)	SHANSEP	15.5	15	0.23				3
🟥	Sludge	Mohr-Coulomb	13			0	24		1
🟩	UKEM	Mohr-Coulomb	16			0	26		2
⬜	Weathered Mudstone (undrained)	Undrained (Phi=0)	18					200	



11. Seismic yield (undrained)

GILF Section 2 revC.gsz

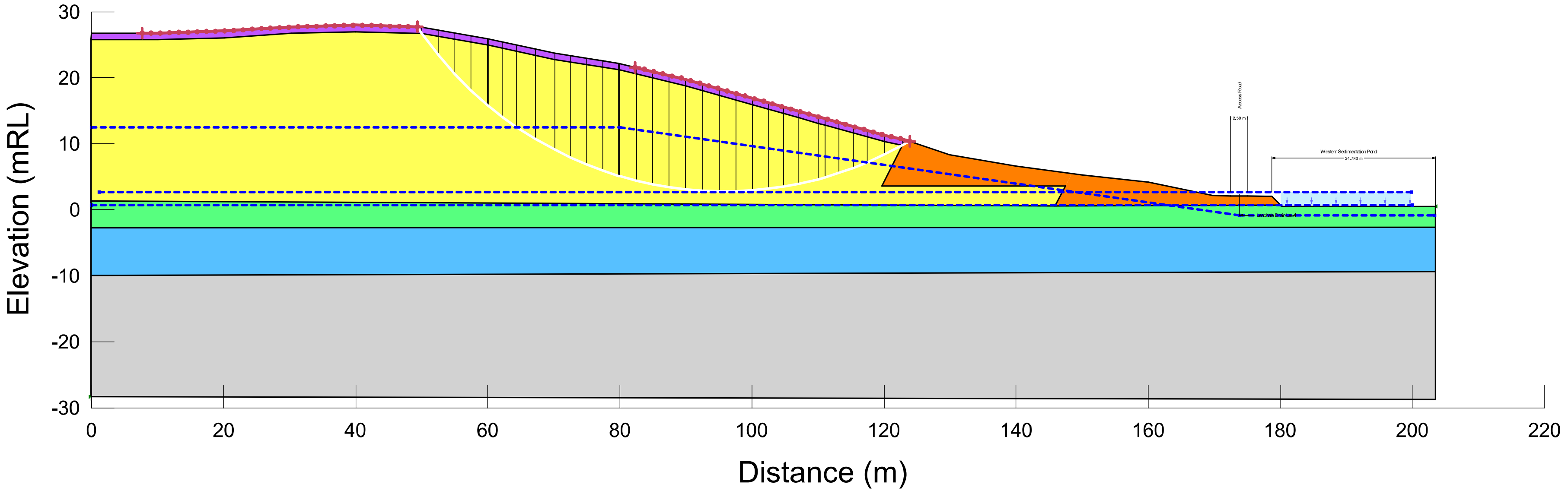
07/12/2022 1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟧	Bund	Mohr-Coulomb	17	1	27	1
🟨	Fill	Mohr-Coulomb	14.5	3	25	1
🟪	Final capping	Mohr-Coulomb	17	2	29	1
🟦	LKEM	Mohr-Coulomb	15.5	0	24	3
🟩	UKEM	Mohr-Coulomb	16	0	26	2
⬜	Weathered Mudstone	Mohr-Coulomb	18	10	32	

● 2.1

Horz Seismic Coef.:



01. Static long term groundwater level - local

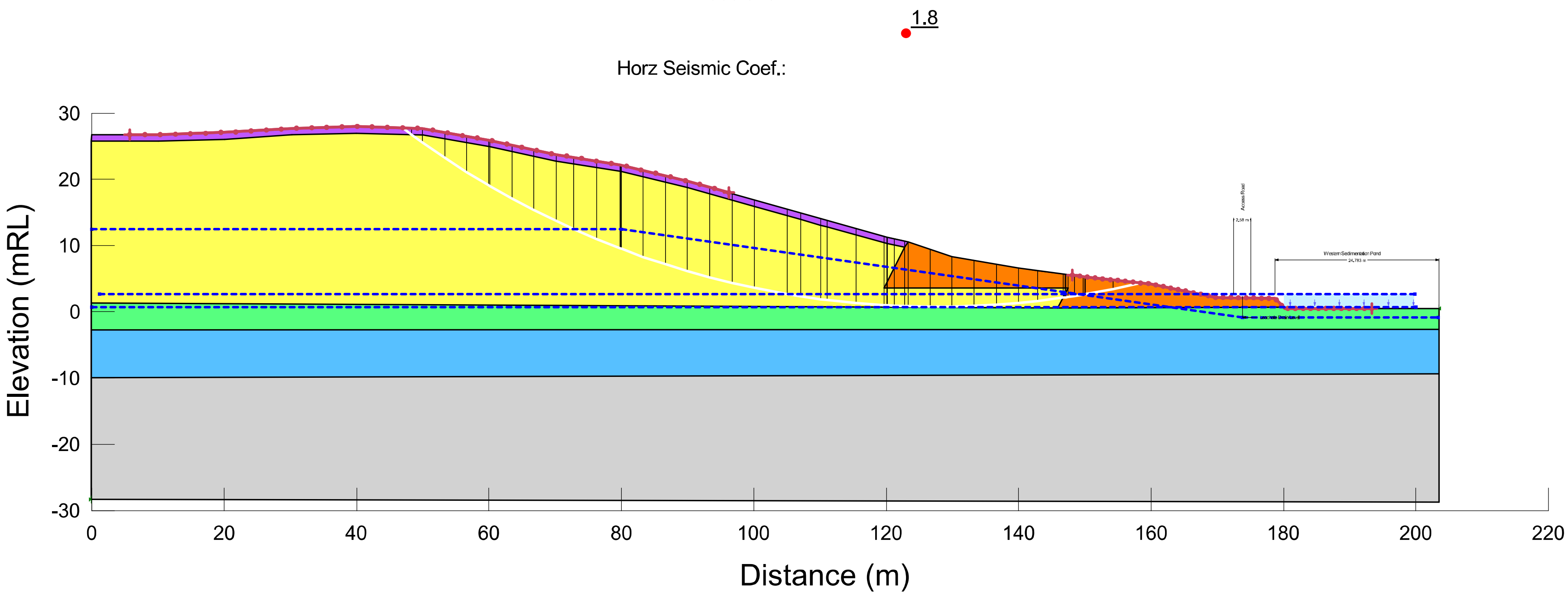
GILF Section 3 revD.gsz

07/12/2022 1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟧	Bund	Mohr-Coulomb	17	1	27	1
🟨	Fill	Mohr-Coulomb	14.5	3	25	1
🟪	Final capping	Mohr-Coulomb	17	2	29	1
🟦	LKEM	Mohr-Coulomb	15.5	0	24	3
🟩	UKEM	Mohr-Coulomb	16	0	26	2
⬜	Weathered Mudstone	Mohr-Coulomb	18	10	32	

Horz Seismic Coef.:



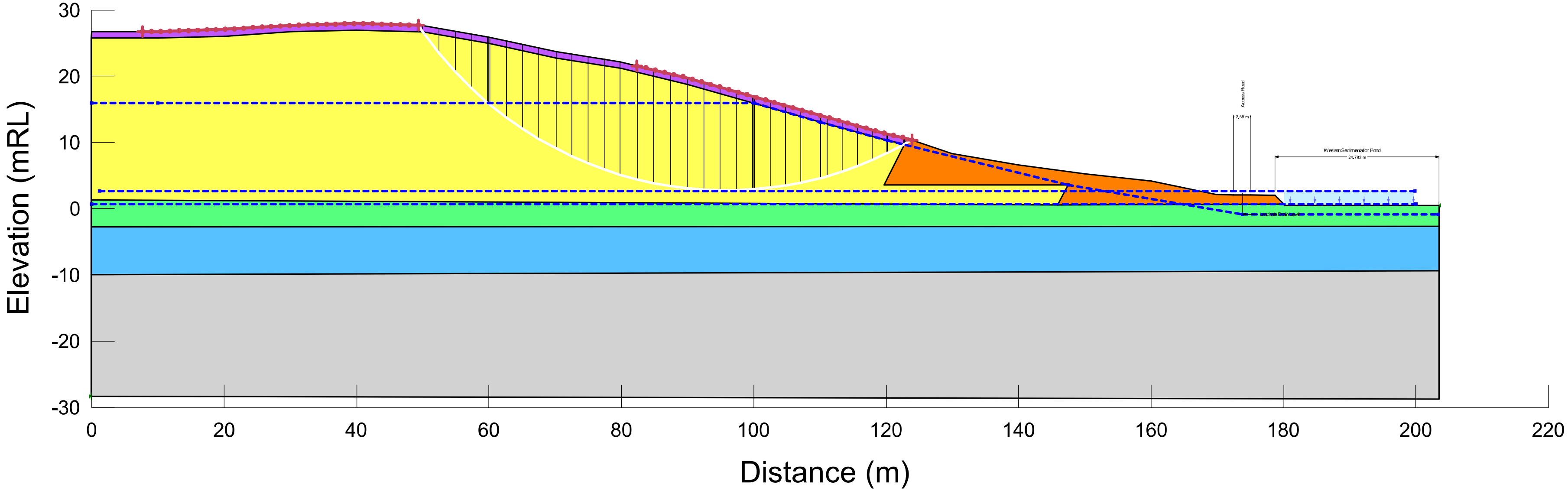
02. Static long term groundwater level - global	
GILF Section 3 revD.gsz	
07/12/2022	1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟧	Bund	Mohr-Coulomb	17	1	27	1
🟨	Fill	Mohr-Coulomb	14.5	3	25	1
🟪	Final capping	Mohr-Coulomb	17	2	29	1
🟦	LKEM	Mohr-Coulomb	15.5	0	24	3
🟩	UKEM	Mohr-Coulomb	16	0	26	2
⬜	Weathered Mudstone	Mohr-Coulomb	18	10	32	

1.6

Horz Seismic Coef.:



03. Static elevated groundwater level - local

GILF Section 3 revD.gsz

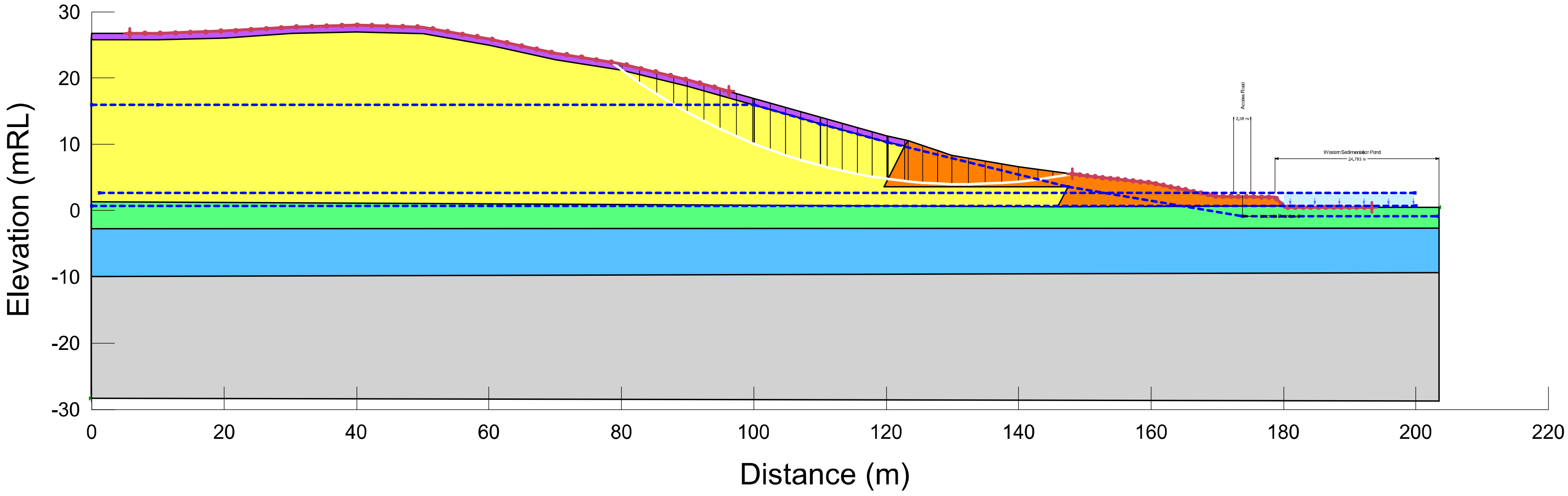
07/12/2022 1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟧	Bund	Mohr-Coulomb	17	1	27	1
🟨	Fill	Mohr-Coulomb	14.5	3	25	1
🟪	Final capping	Mohr-Coulomb	17	2	29	1
🟦	LKEM	Mohr-Coulomb	15.5	0	24	3
🟩	UKEM	Mohr-Coulomb	16	0	26	2
⬜	Weathered Mudstone	Mohr-Coulomb	18	10	32	

● 1.3

Horz Seismic Coef.:

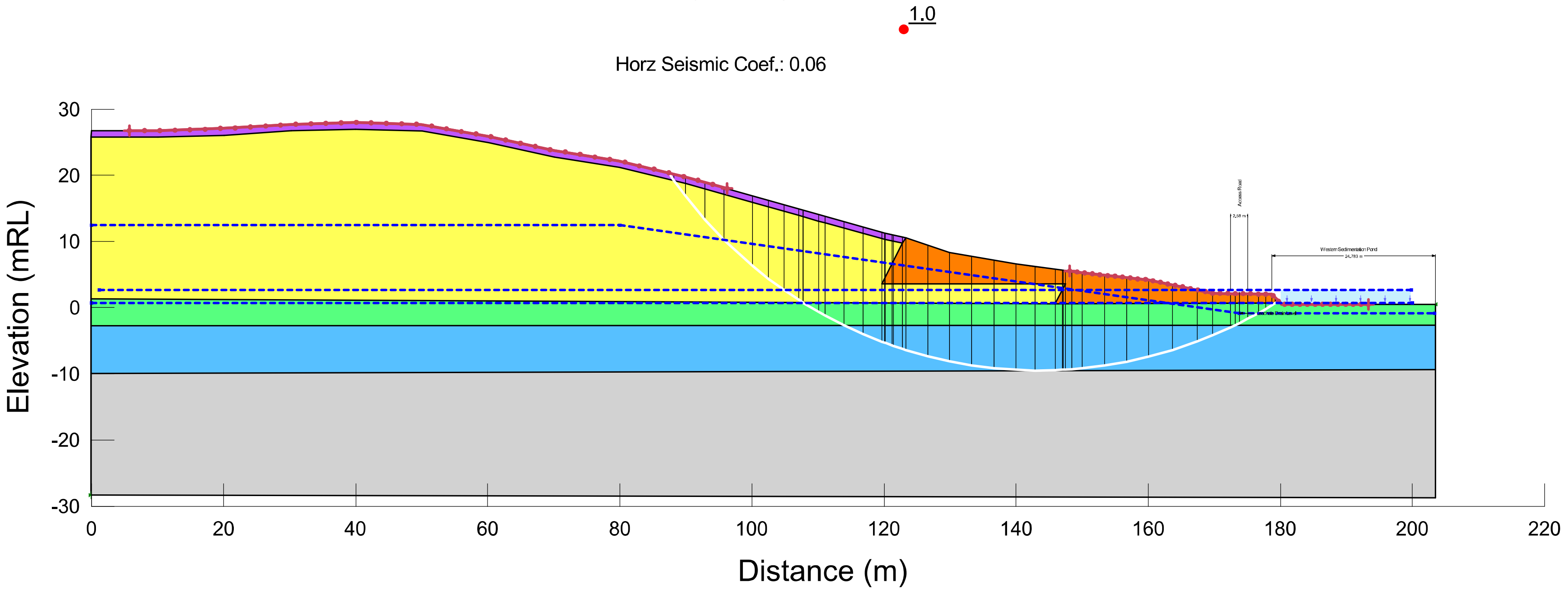


04. Static elevated groundwater level - global	
GILF Section 3 revD.gsz	
07/12/2022	1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Minimum Strength	Tau/Sigma Ratio	Effective Cohesion	Effective Friction Angle	Total Cohesion	Piezometric Line
🟧	Bund (undrained)	Undrained (Phi=0)	17					75	1
🟨	Fill	Mohr-Coulomb	14.5			3	25		1
🟪	Final capping (undrained)	Undrained (Phi=0)	17					100	1
🟦	LKEM (undrained)	SHANSEP	15.5	15	0.23				3
🟩	UKEM	Mohr-Coulomb	16			0	26		2
⬜	Weathered Mudstone (undrained)	Undrained (Phi=0)	18					200	

Horz Seismic Coef.: 0.06



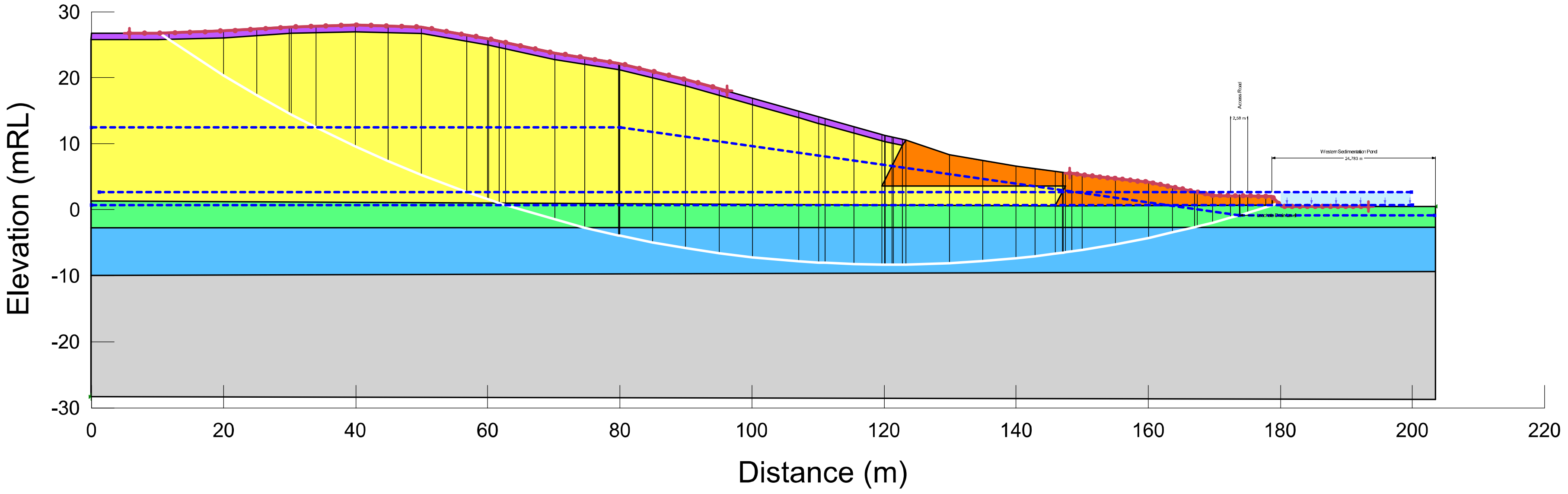
06. Seismic SLS (undrained)
GILF Section 3 revD.gsz
07/12/2022 1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Minimum Strength	Tau/Sigma Ratio	Effective Cohesion	Effective Friction Angle	Total Cohesion	Piezometric Line
🟧	Bund (undrained)	Undrained (Phi=0)	17					75	1
🟨	Fill	Mohr-Coulomb	14.5			3	25		1
🟪	Final capping (undrained)	Undrained (Phi=0)	17					100	1
🟦	LKEM (undrained)	SHANSEP	15.5	15	0.23				3
🟩	UKEM	Mohr-Coulomb	16			0	26		2
⬜	Weathered Mudstone (undrained)	Undrained (Phi=0)	18					200	

0.3

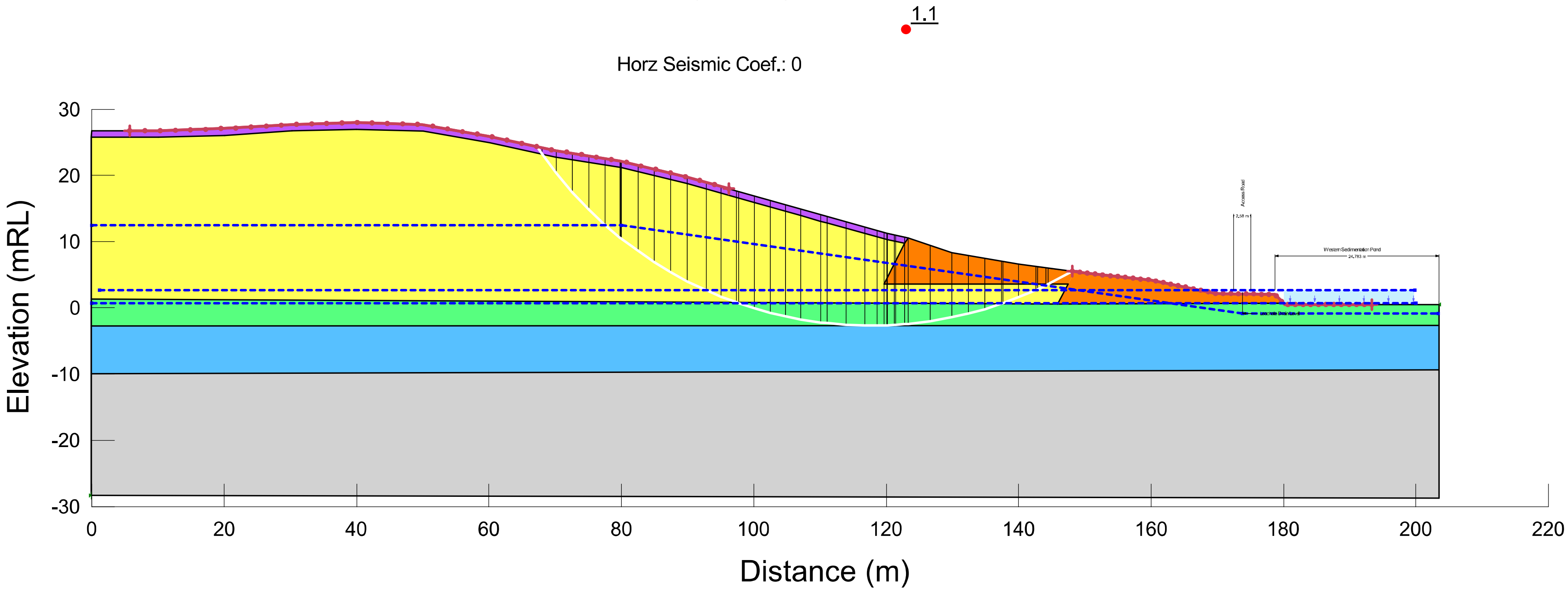
Horz Seismic Coef.: 0.51



08. Seismic ULS (undrained)
GILF Section 3 revD.gsz
07/12/2022 1:700

Analysis Type: Morgenstern-Price

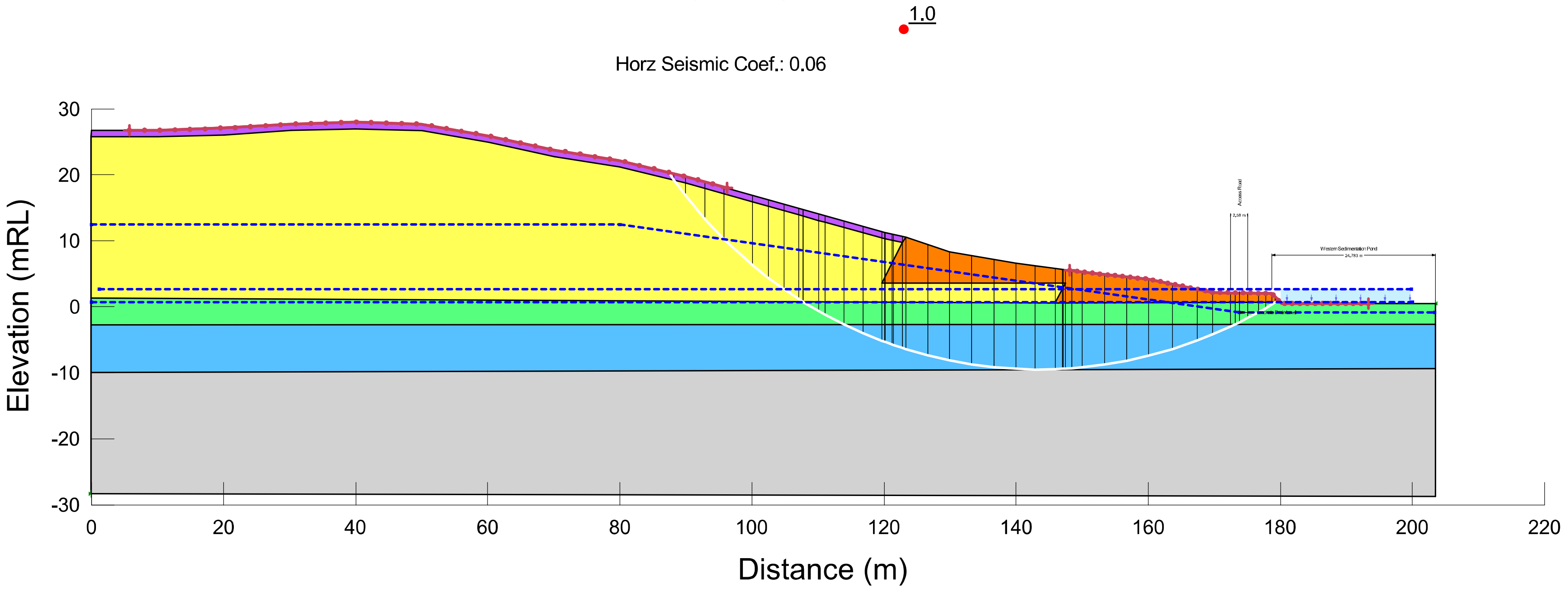
Color	Name	Slope Stability Material Model	Unit Weight	Minimum Strength	Tau/Sigma Ratio	Effective Cohesion	Effective Friction Angle	Total Cohesion	Piezometric Line
🟧	Bund (undrained)	Undrained (Phi=0)	17					75	1
🟨	Fill	Mohr-Coulomb	14.5			3	25		1
🟪	Final capping (undrained)	Undrained (Phi=0)	17					100	1
🟦	LKEM (undrained)	SHANSEP	15.5	15	0.23				3
🟩	UKEM (liq)	SHANSEP	16	0	0.08				2
⬜	Weathered Mudstone (undrained)	Undrained (Phi=0)	18					200	



10. Post seismic
GILF Section 3 revD.gsz
07/12/2022 1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Minimum Strength	Tau/Sigma Ratio	Effective Cohesion	Effective Friction Angle	Total Cohesion	Piezometric Line
🟧	Bund (undrained)	Undrained (Phi=0)	17					75	1
🟨	Fill	Mohr-Coulomb	14.5			3	25		1
🟪	Final capping (undrained)	Undrained (Phi=0)	17					100	1
🟦	LKEM (undrained)	SHANSEP	15.5	15	0.23				3
🟩	UKEM	Mohr-Coulomb	16			0	26		2
⬜	Weathered Mudstone (undrained)	Undrained (Phi=0)	18					200	

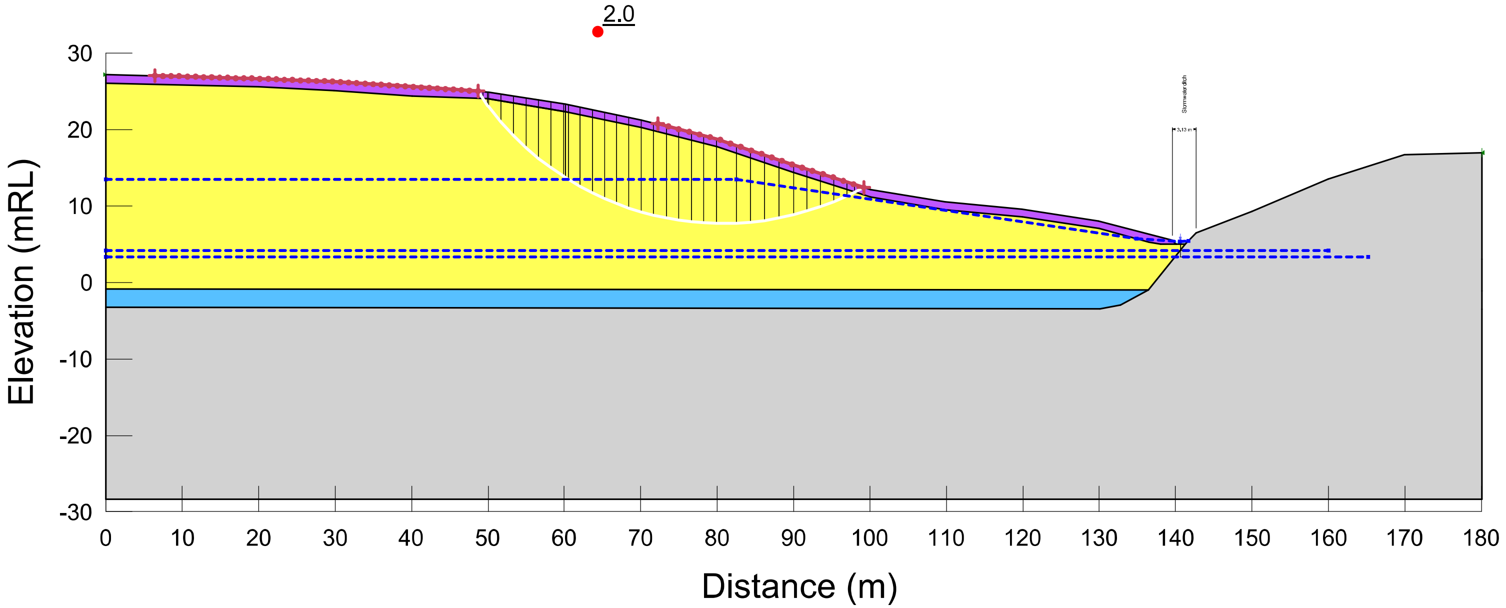


11. Seismic yield - undrained	
GILF Section 3 revD.gsz	
07/12/2022	1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟨	Fill	Mohr-Coulomb	14.5	3	25	1
🟪	Final capping	Mohr-Coulomb	17	2	29	1
🟦	LKEM	Mohr-Coulomb	15.5	0	24	3
⬜	Weathered Mudstone	Mohr-Coulomb	18	10	32	

Horz Seismic Coef.:

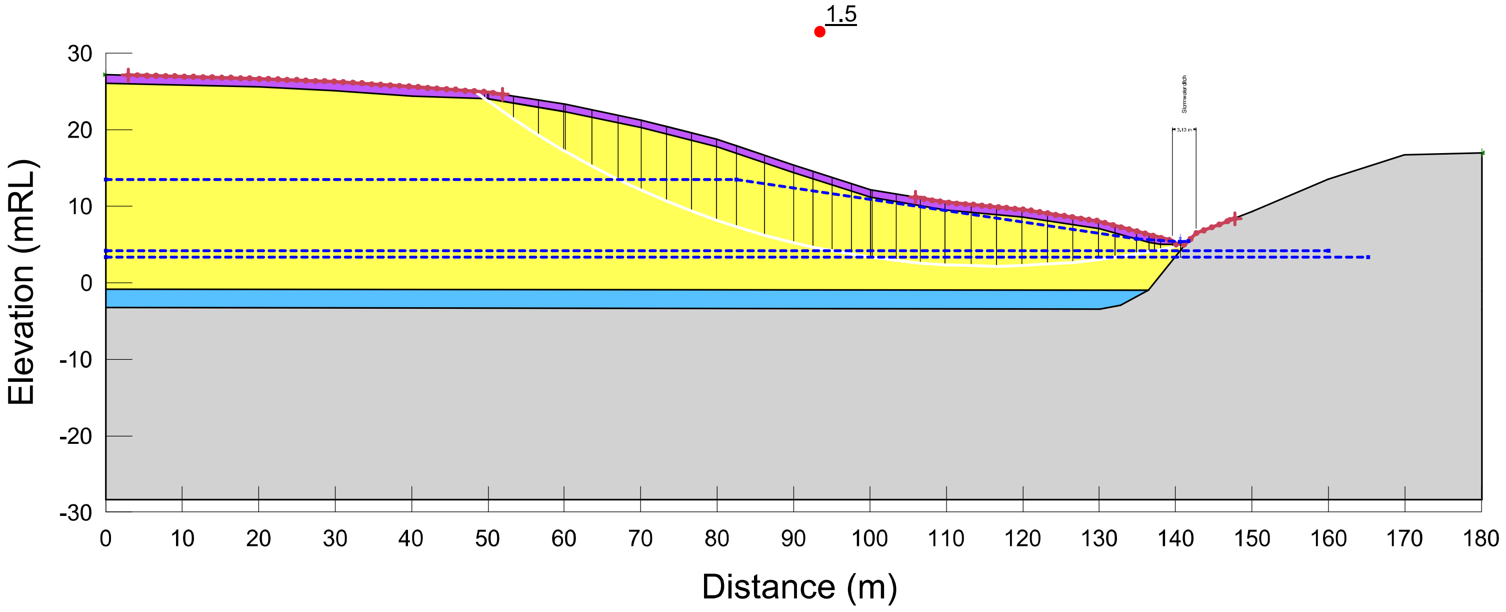


01. Static Long Term Groundwater Level - Local
GILF Section 4 revD.gsz
07/12/2022
1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟨	Fill	Mohr-Coulomb	14.5	3	25	1
🟪	Final capping	Mohr-Coulomb	17	2	29	1
🟦	LKEM	Mohr-Coulomb	15.5	0	24	3
⬜	Weathered Mudstone	Mohr-Coulomb	18	10	32	

Horz Seismic Coef.:

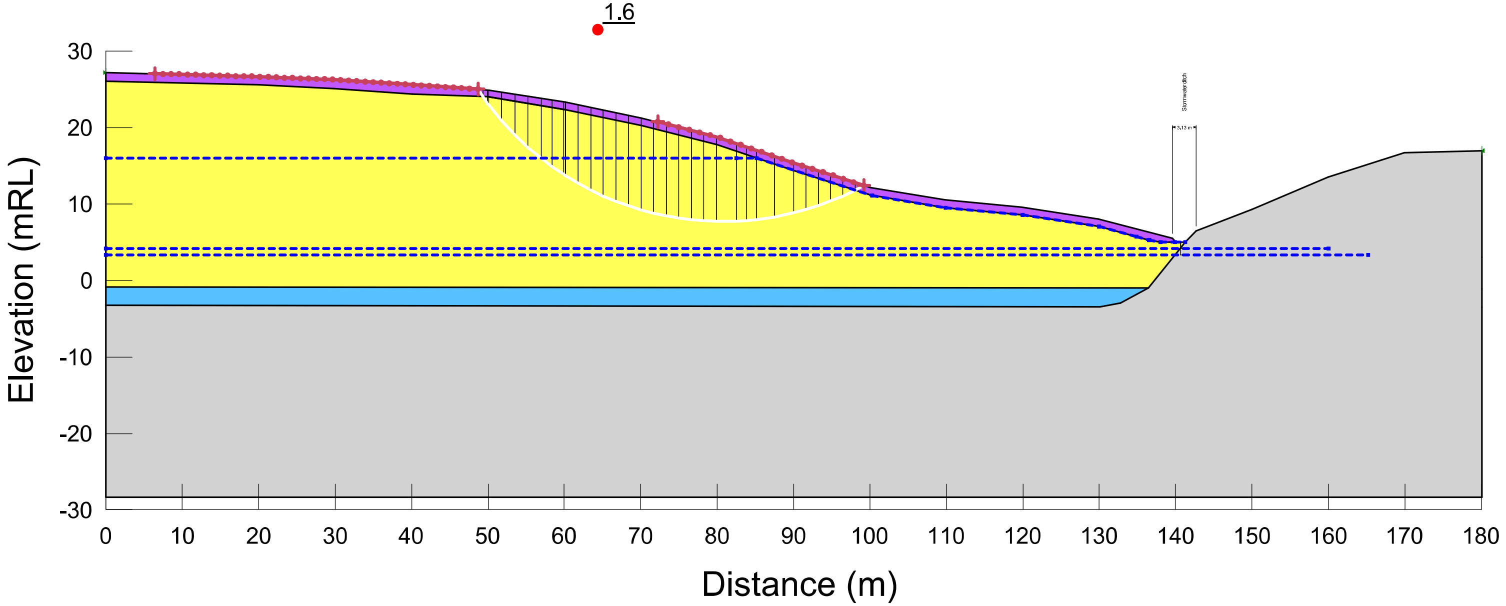


02. Static Long Term Groundwater Level - Global	
GILF Section 4 revD.gsz	
07/12/2022	1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟨	Fill	Mohr-Coulomb	14.5	3	25	1
🟪	Final capping	Mohr-Coulomb	17	2	29	1
🟦	LKEM	Mohr-Coulomb	15.5	0	24	3
⬜	Weathered Mudstone	Mohr-Coulomb	18	10	32	

Horz Seismic Coef.:

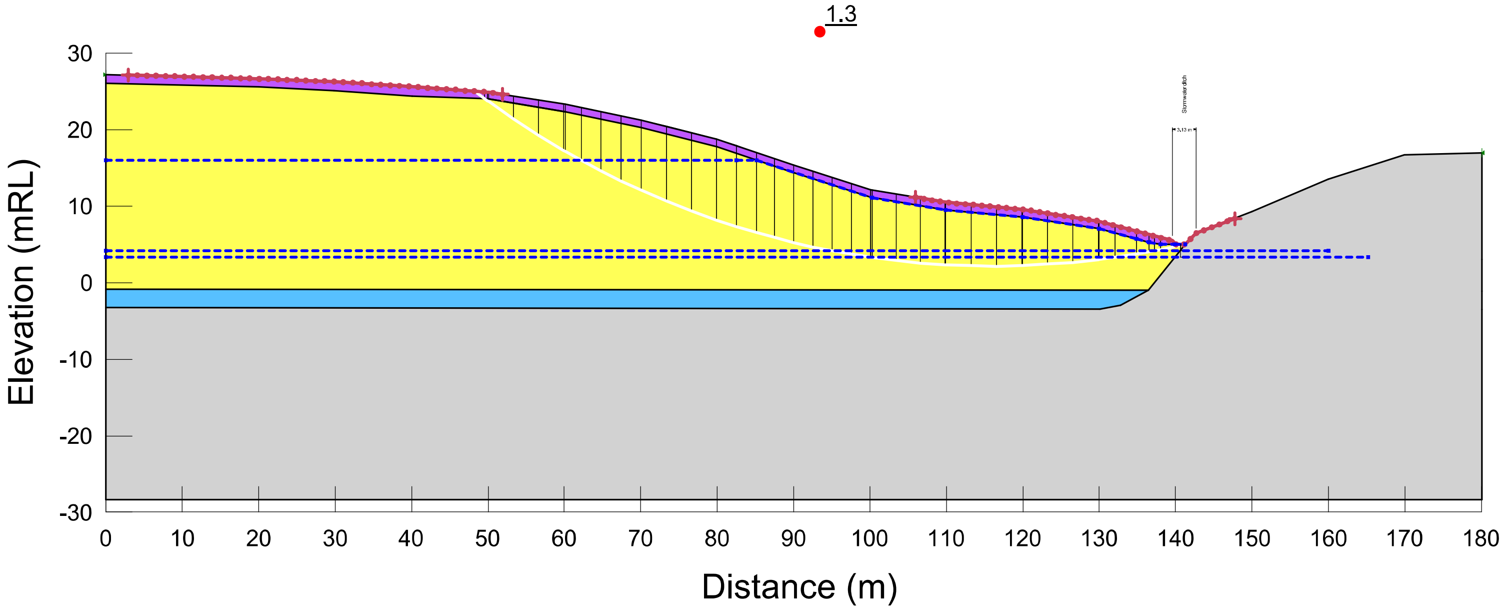


03. Static Elevated Groundwater Level - Local
GILF Section 4 revD.gsz
07/12/2022 1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟨	Fill	Mohr-Coulomb	14.5	3	25	1
🟪	Final capping	Mohr-Coulomb	17	2	29	1
🟦	LKEM	Mohr-Coulomb	15.5	0	24	3
⬜	Weathered Mudstone	Mohr-Coulomb	18	10	32	

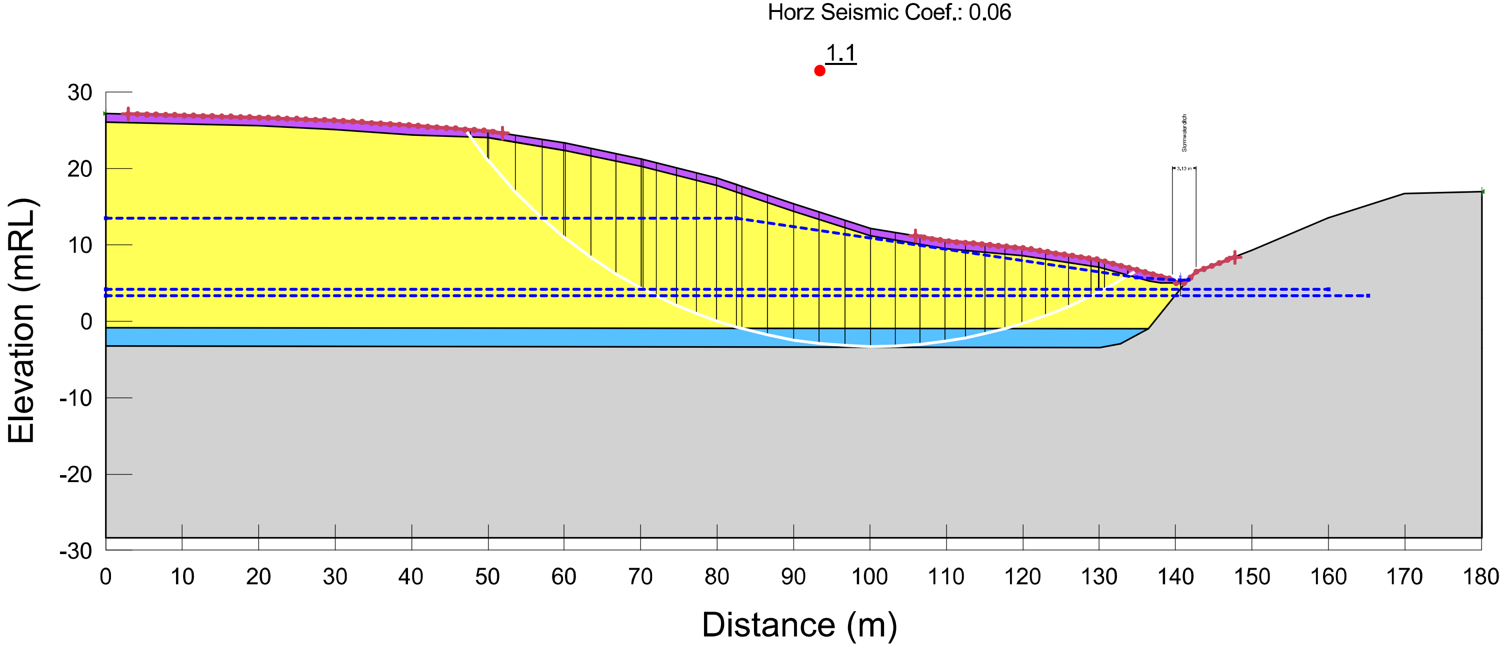
Horz Seismic Coef.:



04. Static Elevated Groundwater Level - Global
GILF Section 4 revD.gsz
07/12/2022 1:700

Analysis Type: Morgenstern-Price

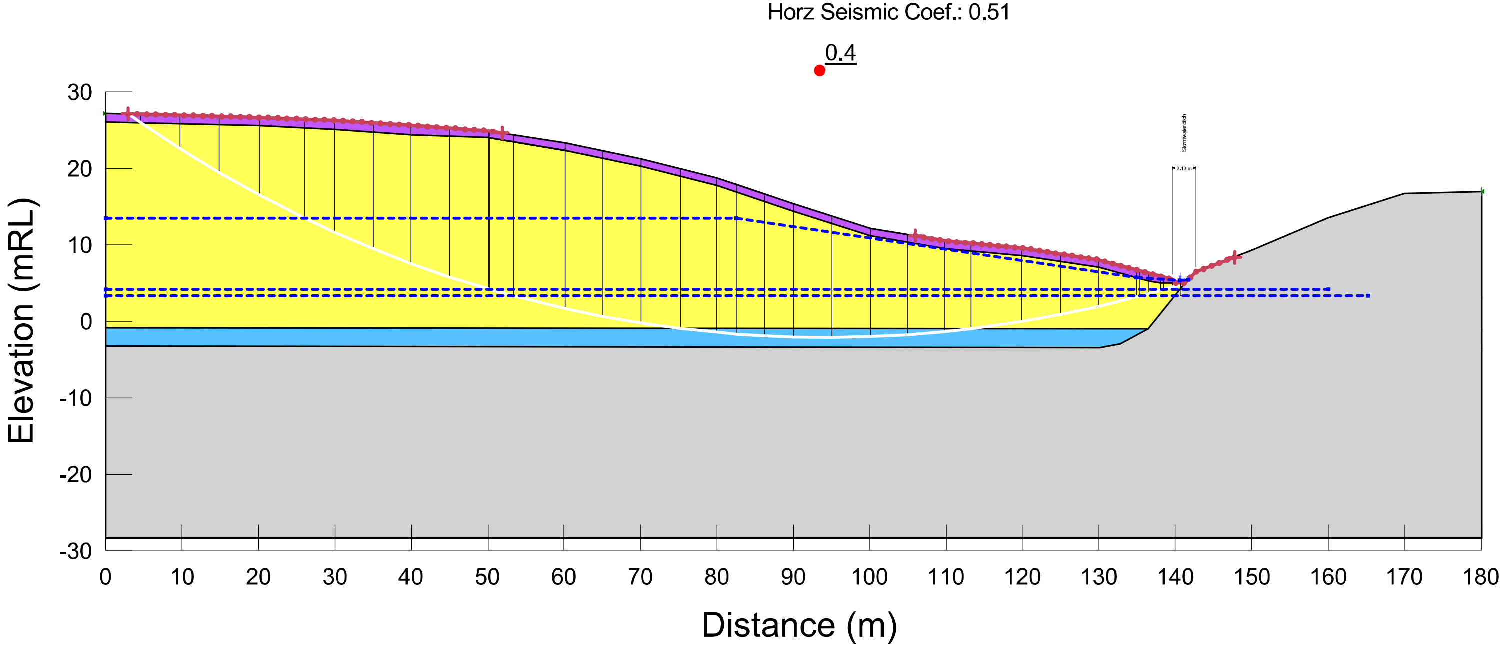
Color	Name	Slope Stability Material Model	Unit Weight	Minimum Strength	Tau/Sigma Ratio	Total Cohesion	Effective Cohesion	Effective Friction Angle	Piezomet Line
🟨	Fill	Mohr-Coulomb	14.5				3	25	1
🟪	Final capping (undrained)	Undrained (Phi=0)	17			100			1
🟦	LKEM (undrained)	SHANSEP	15.5	15	0.23				3
⬜	Weathered Mudstone (undrained)	Undrained (Phi=0)	18			200			



06. Seismic SLS (undrained)
GILF Section 4 revD.gsz
07/12/2022 1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Minimum Strength	Tau/Sigma Ratio	Total Cohesion	Effective Cohesion	Effective Friction Angle	Piezomet Line
🟨	Fill	Mohr-Coulomb	14.5				3	25	1
🟪	Final capping (undrained)	Undrained (Phi=0)	17			100			1
🟦	LKEM (undrained)	SHANSEP	15.5	15	0.23				3
⬜	Weathered Mudstone (undrained)	Undrained (Phi=0)	18			200			

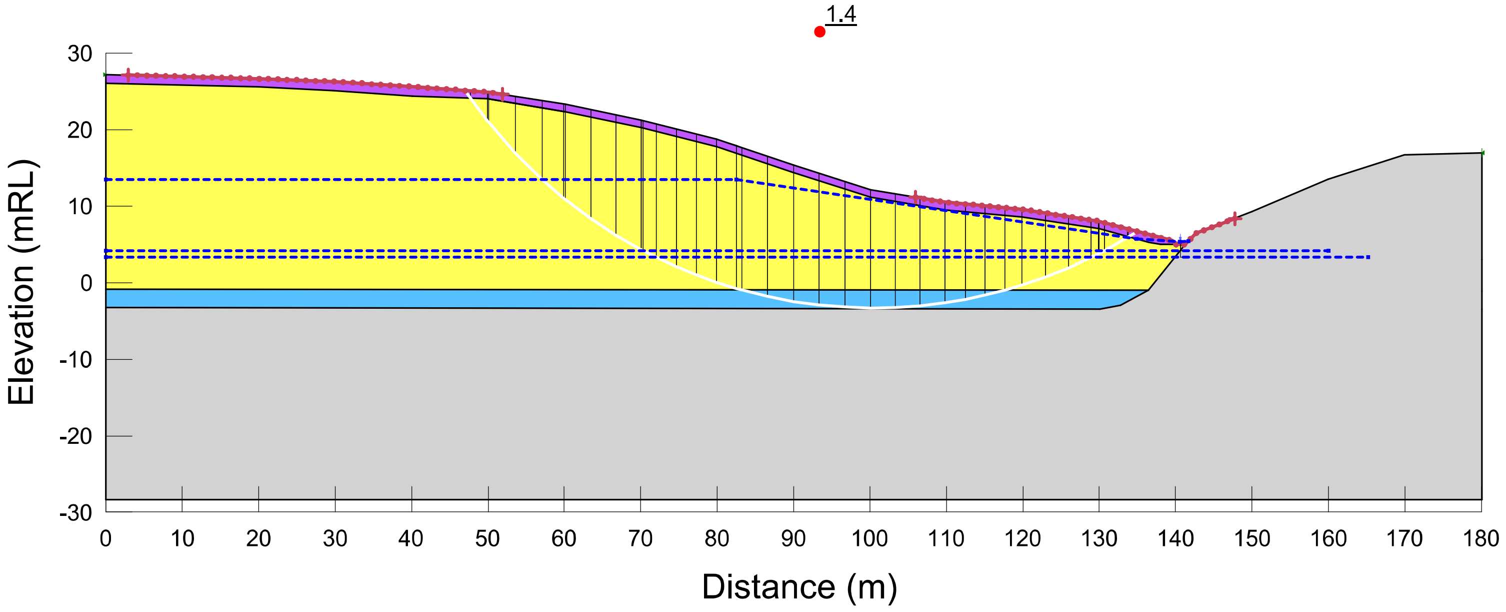


08. Seismic ULS (undrained)
GILF Section 4 revD.gsz
07/12/2022 1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Minimum Strength	Tau/Sigma Ratio	Total Cohesion	Effective Cohesion	Effective Friction Angle	Piezomet Line
🟨	Fill	Mohr-Coulomb	14.5				3	25	1
🟪	Final capping (undrained)	Undrained (Phi=0)	17			100			1
🟦	LKEM (undrained)	SHANSEP	15.5	15	0.23				3
⬜	Weathered Mudstone (undrained)	Undrained (Phi=0)	18			200			

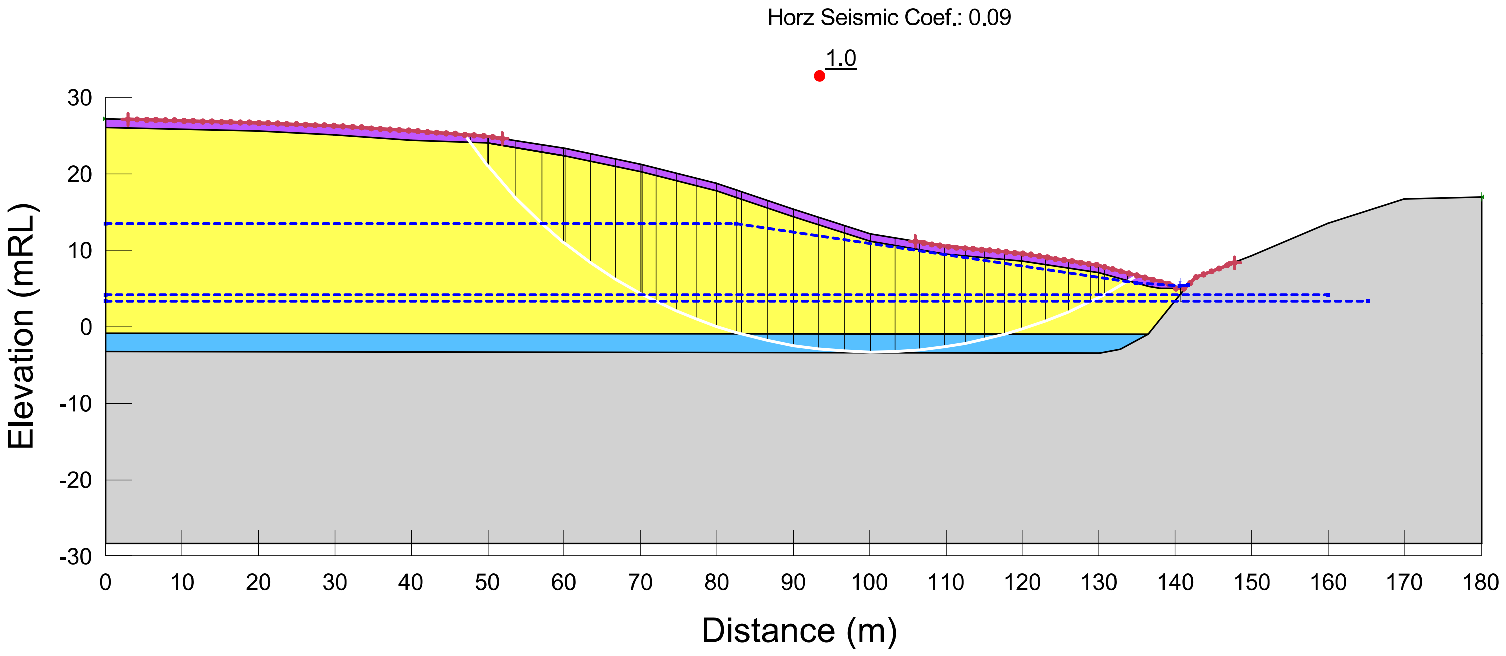
Horz Seismic Coef.: 0



10. Post seismic	
GILF Section 4 revD.gsz	
07/12/2022	1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Minimum Strength	Tau/Sigma Ratio	Total Cohesion	Effective Cohesion	Effective Friction Angle	Piezomet Line
🟨	Fill	Mohr-Coulomb	14.5				3	25	1
🟪	Final capping (undrained)	Undrained (Phi=0)	17			100			1
🟦	LKEM (undrained)	SHANSEP	15.5	15	0.23				3
⬜	Weathered Mudstone (undrained)	Undrained (Phi=0)	18			200			

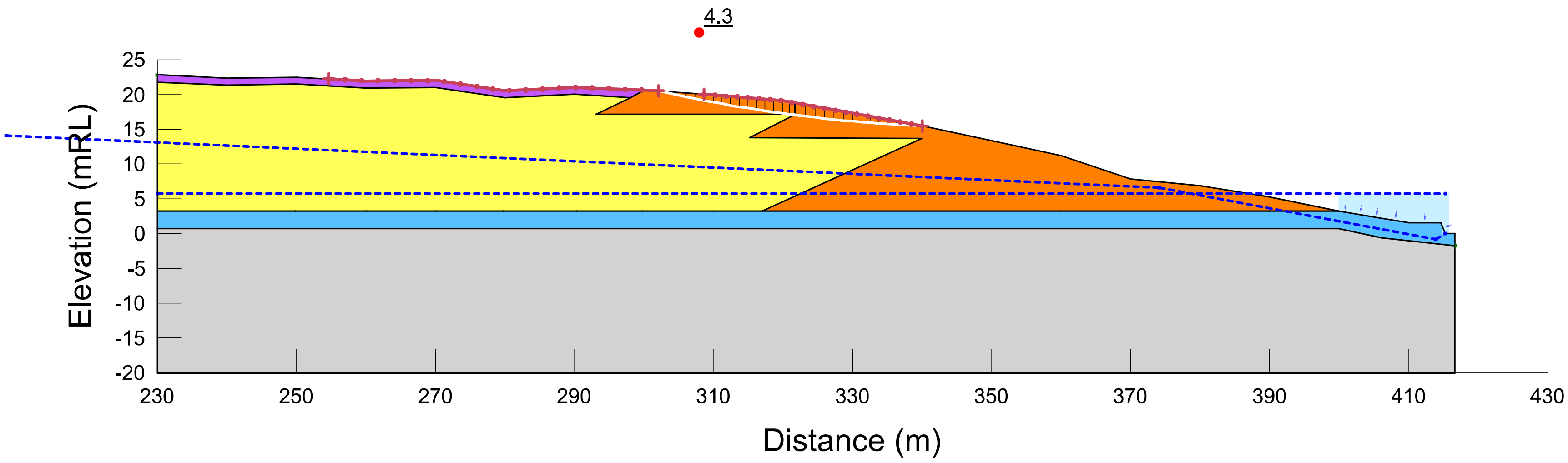


11. Seismic yield - undrained	
GILF Section 4 revD.gsz	
07/12/2022	1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟧	Bund	Mohr-Coulomb	17	1	27	1
🟨	Fill	Mohr-Coulomb	14.5	3	25	1
🟪	Final capping	Mohr-Coulomb	17	2	29	1
🟦	LKEM	Mohr-Coulomb	15.5	0	24	2
⬜	Weathered Mudstone	Mohr-Coulomb	18	10	32	

Horz Seismic Coef.:

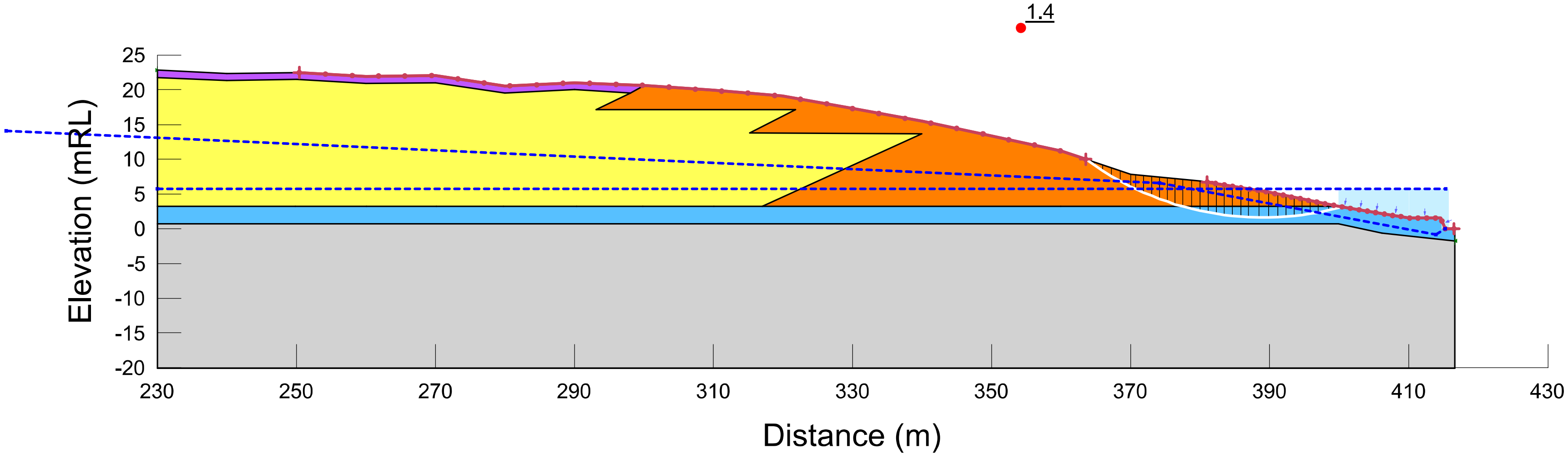


01. Static long term groundwater level - local
GILF Section 5 revC.gsz
07/12/2022
1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟧	Bund	Mohr-Coulomb	17	1	27	1
🟨	Fill	Mohr-Coulomb	14.5	3	25	1
🟪	Final capping	Mohr-Coulomb	17	2	29	1
🟦	LKEM	Mohr-Coulomb	15.5	0	24	2
⬜	Weathered Mudstone	Mohr-Coulomb	18	10	32	

Horz Seismic Coef.:



02. Static long term groundwater level - global

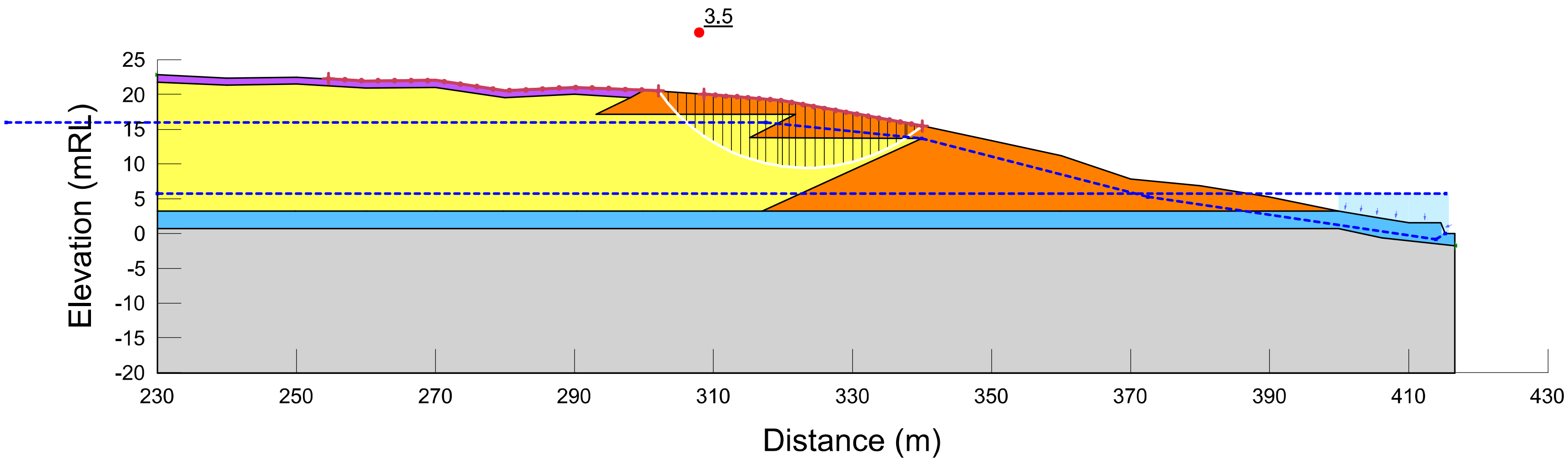
GILF Section 5 revC.gsz

07/12/2022 1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟧	Bund	Mohr-Coulomb	17	1	27	1
🟨	Fill	Mohr-Coulomb	14.5	3	25	1
🟪	Final capping	Mohr-Coulomb	17	2	29	1
🟦	LKEM	Mohr-Coulomb	15.5	0	24	2
⬜	Weathered Mudstone	Mohr-Coulomb	18	10	32	

Horz Seismic Coef.:

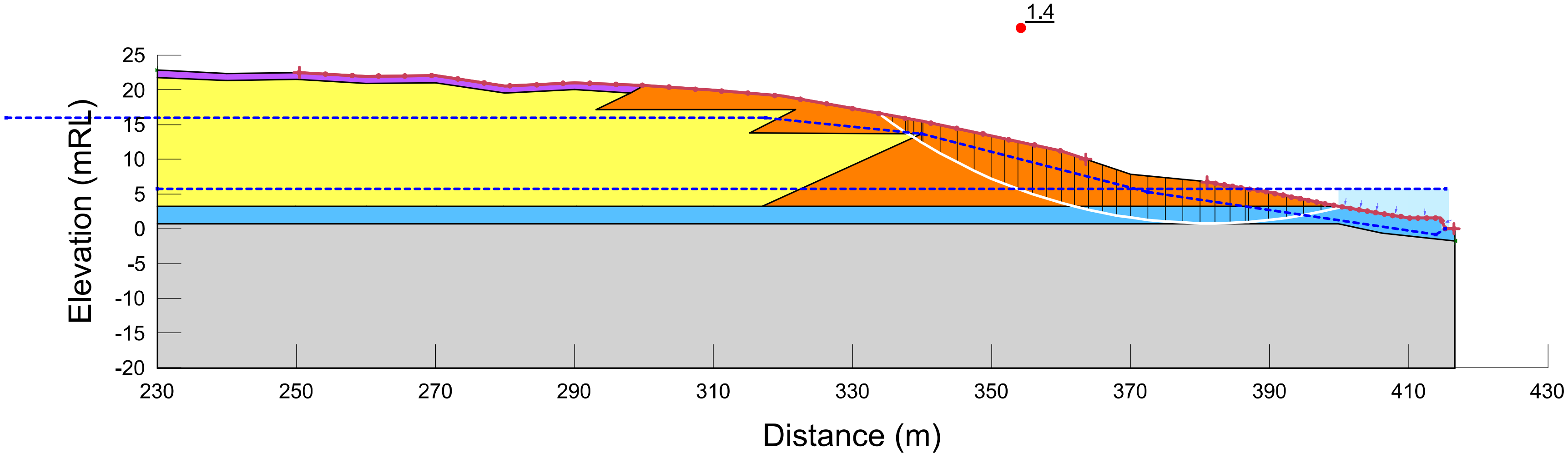


03. Static elevated groundwater level - local
GILF Section 5 revC.gsz
07/12/2022 1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟧	Bund	Mohr-Coulomb	17	1	27	1
🟨	Fill	Mohr-Coulomb	14.5	3	25	1
🟪	Final capping	Mohr-Coulomb	17	2	29	1
🟦	LKEM	Mohr-Coulomb	15.5	0	24	2
⬜	Weathered Mudstone	Mohr-Coulomb	18	10	32	

Horz Seismic Coef.:



04. Static elevated groundwater level - global

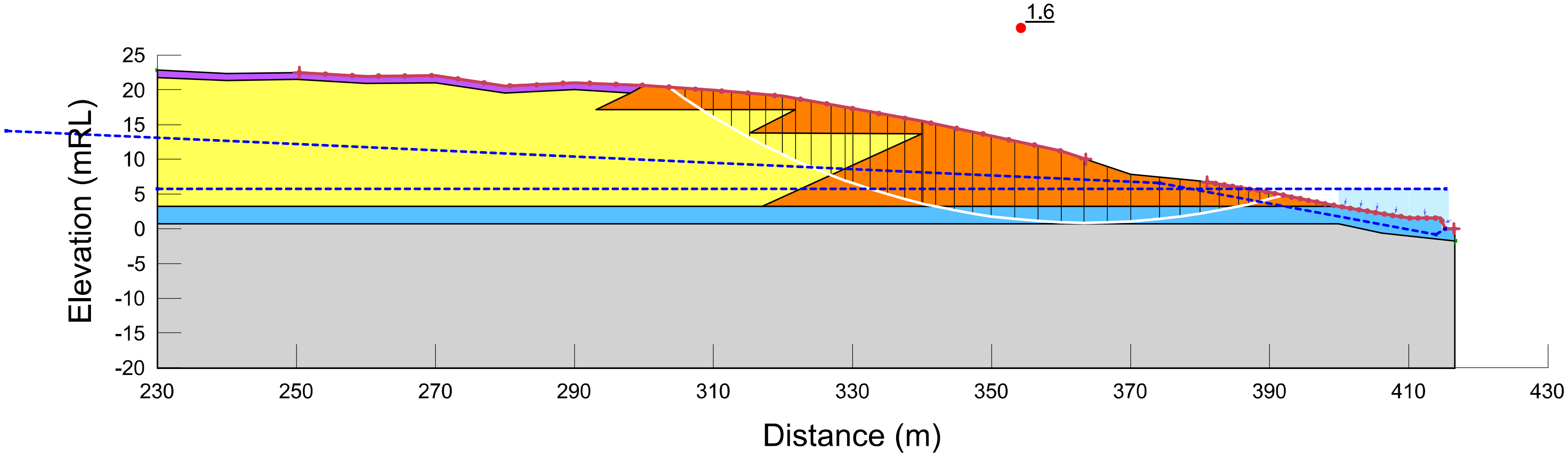
GILF Section 5 revC.gsz

07/12/2022 1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Minimum Strength	Tau/Sigma Ratio	Effective Cohesion	Effective Friction Angle	Total Cohesion	Piezometric Line
🟧	Bund (undrained)	Undrained (Phi=0)	17					75	1
🟨	Fill	Mohr-Coulomb	14.5			3	25		1
🟪	Final capping (undrained)	Undrained (Phi=0)	17					100	1
🟦	LKEM (undrained)	SHANSEP	15.5	15	0.23				2
⬜	Weathered Mudstone (undrained)	Undrained (Phi=0)	18					200	

Horz Seismic Coef.: 0.06

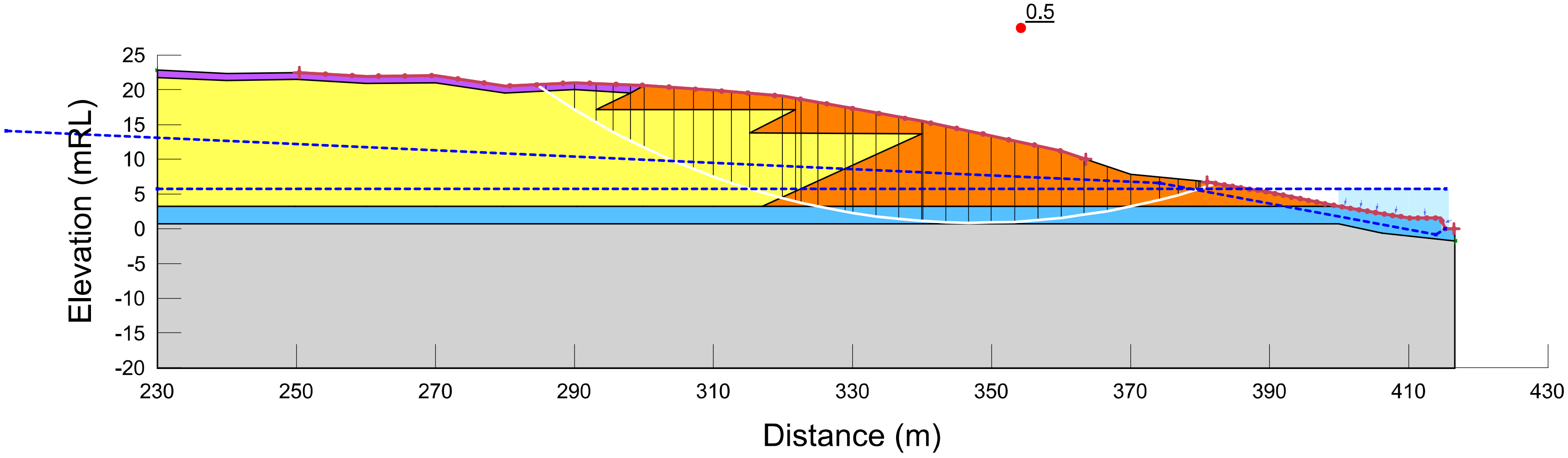


06. Seismic SLS (undrained)
GILF Section 5 revC.gsz
07/12/2022
1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Minimum Strength	Tau/Sigma Ratio	Effective Cohesion	Effective Friction Angle	Total Cohesion	Piezometric Line
🟧	Bund (undrained)	Undrained (Phi=0)	17					75	1
🟨	Fill	Mohr-Coulomb	14.5			3	25		1
🟪	Final capping (undrained)	Undrained (Phi=0)	17					100	1
🟦	LKEM (undrained)	SHANSEP	15.5	15	0.23				2
⬜	Weathered Mudstone (undrained)	Undrained (Phi=0)	18					200	

Horz Seismic Coef.: 0.51



08. Seismic ULS (undrained)

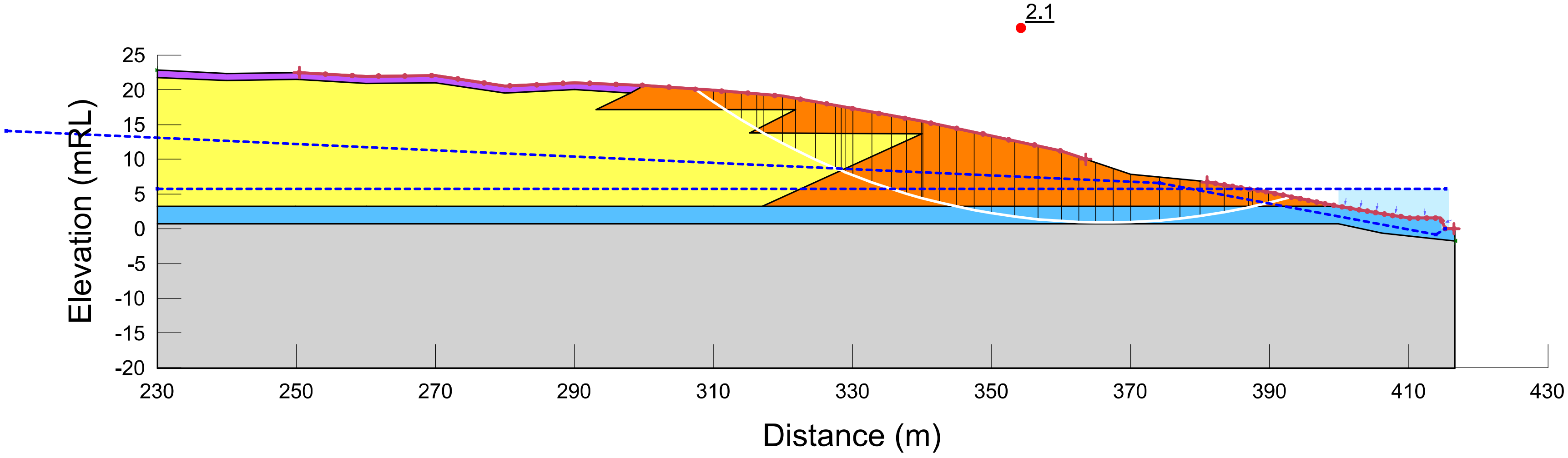
GILF Section 5 revC.gsz

07/12/2022 1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Minimum Strength	Tau/Sigma Ratio	Effective Cohesion	Effective Friction Angle	Total Cohesion	Piezometric Line
🟧	Bund (undrained)	Undrained (Phi=0)	17					75	1
🟨	Fill	Mohr-Coulomb	14.5			3	25		1
🟪	Final capping (undrained)	Undrained (Phi=0)	17					100	1
🟦	LKEM (undrained)	SHANSEP	15.5	15	0.23				2
⬜	Weathered Mudstone (undrained)	Undrained (Phi=0)	18					200	

Horz Seismic Coef.:

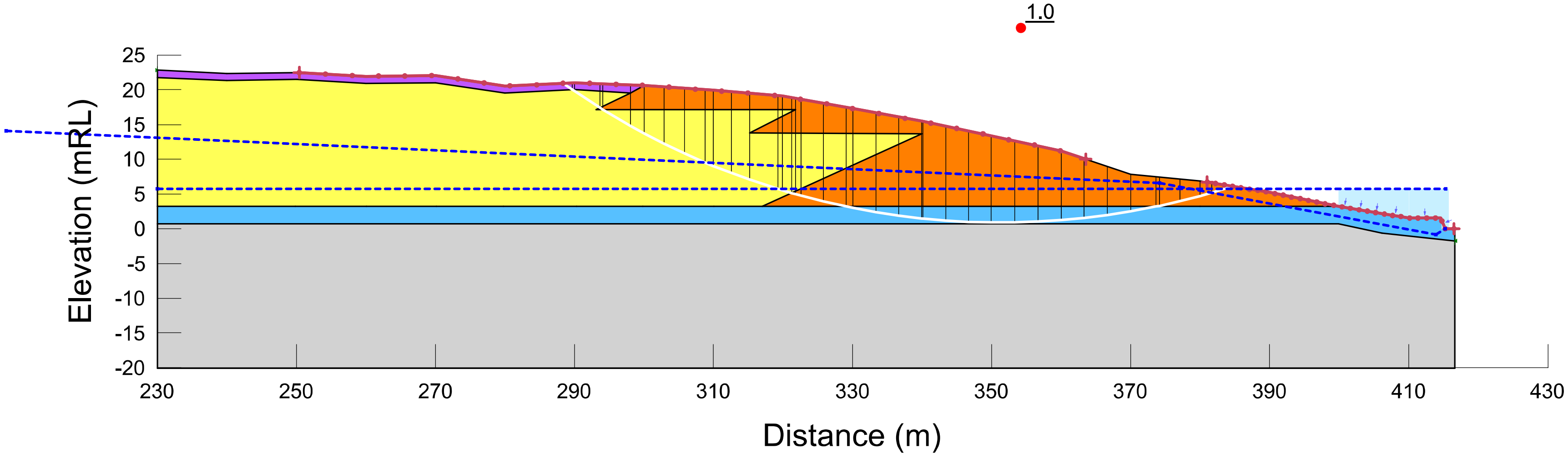


09. Post seismic
GILF Section 5 revC.gsz
07/12/2022 1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Minimum Strength	Tau/Sigma Ratio	Effective Cohesion	Effective Friction Angle	Total Cohesion	Piezometric Line
🟧	Bund (undrained)	Undrained (Phi=0)	17					75	1
🟨	Fill	Mohr-Coulomb	14.5			3	25		1
🟪	Final capping (undrained)	Undrained (Phi=0)	17					100	1
🟦	LKEM (undrained)	SHANSEP	15.5	15	0.23				2
⬜	Weathered Mudstone (undrained)	Undrained (Phi=0)	18					200	

Horz Seismic Coef.: 0.19



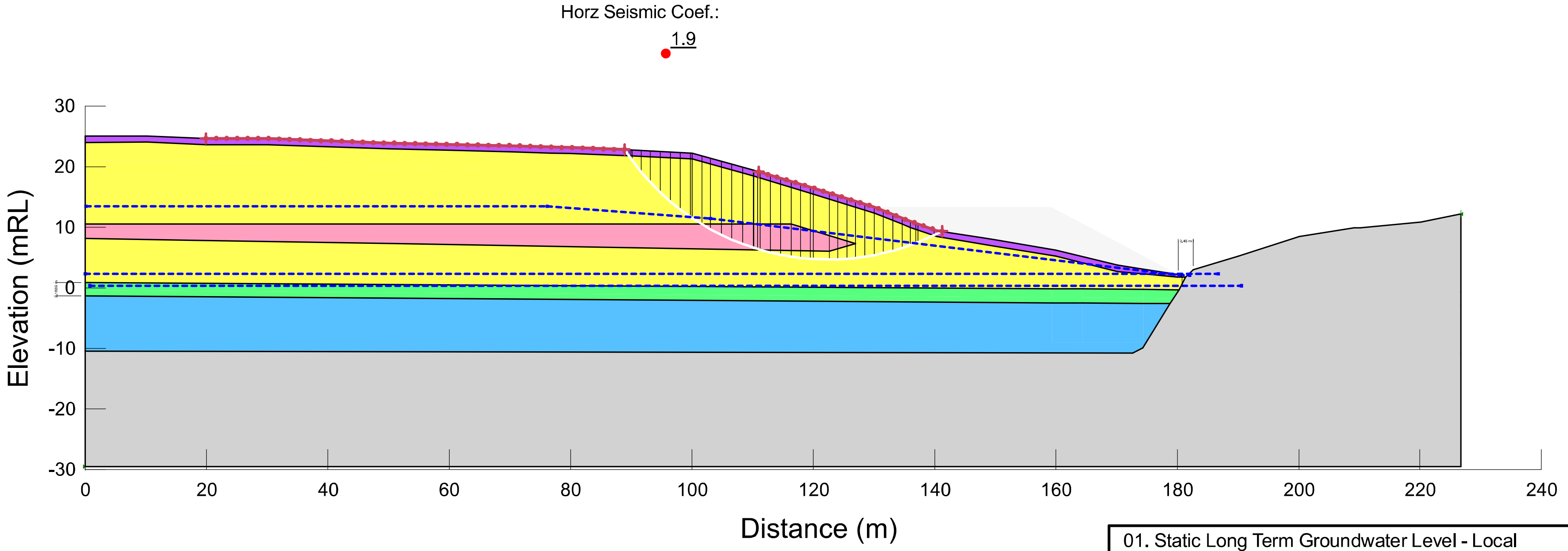
10. Seismic yield

GILF Section 5 revC.gsz

07/12/2022 1:700

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟨	Fill	Mohr-Coulomb	14.5	3	25	1
🟪	Final capping	Mohr-Coulomb	17	2	29	1
🟦	LKEM	Mohr-Coulomb	15.5	0	24	3
🟥	Sludge	Mohr-Coulomb	13	0	24	1
🟩	UKEM	Mohr-Coulomb	16	0	26	2
⬜	Weathered Mudstone	Mohr-Coulomb	18	10	32	



01. Static Long Term Groundwater Level - Local

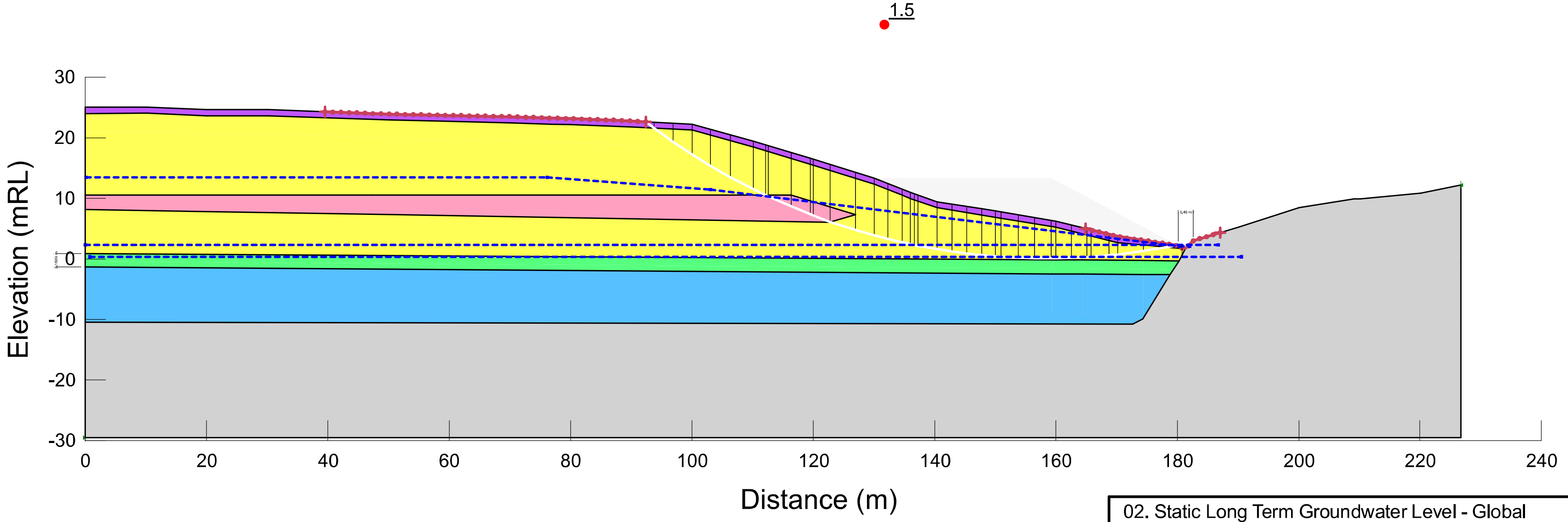
GILF Section 6 revG.gsz

07/12/2022 1:750

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟨	Fill	Mohr-Coulomb	14.5	3	25	1
🟪	Final capping	Mohr-Coulomb	17	2	29	1
🟦	LKEM	Mohr-Coulomb	15.5	0	24	3
🟥	Sludge	Mohr-Coulomb	13	0	24	1
🟩	UKEM	Mohr-Coulomb	16	0	26	2
⬜	Weathered Mudstone	Mohr-Coulomb	18	10	32	

Horz Seismic Coef.:



02. Static Long Term Groundwater Level - Global

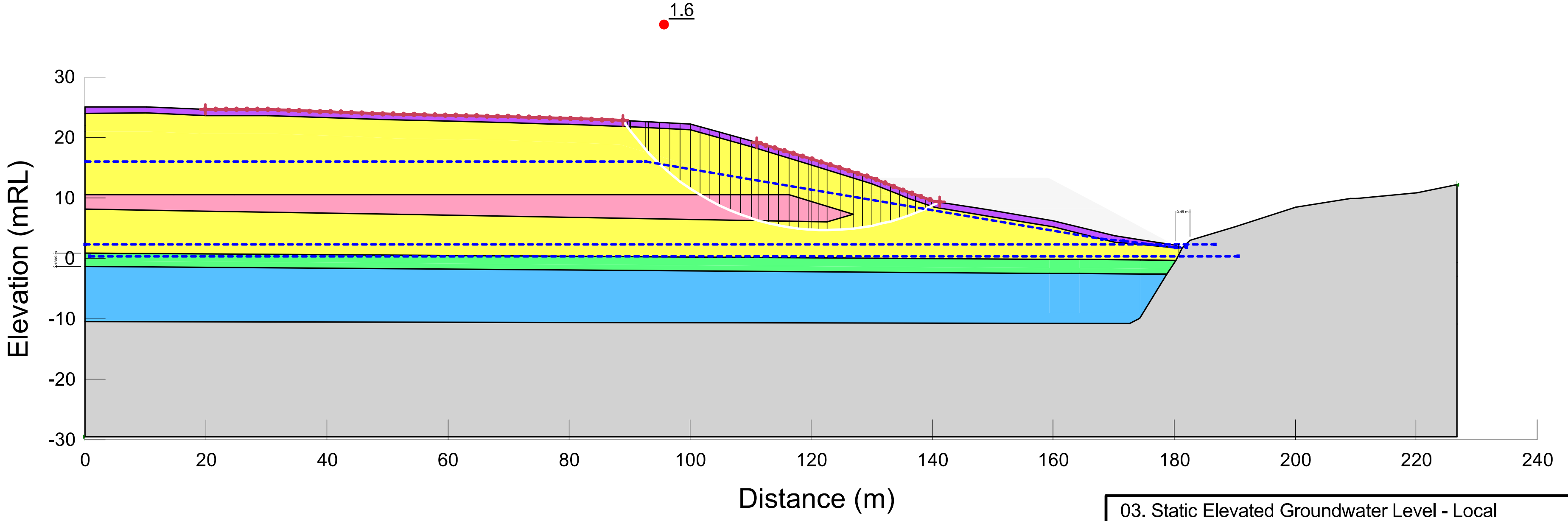
GILF Section 6 revG.gsz

07/12/2022 1:750

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟨	Fill	Mohr-Coulomb	14.5	3	25	1
🟪	Final capping	Mohr-Coulomb	17	2	29	1
🟦	LKEM	Mohr-Coulomb	15.5	0	24	3
🟥	Sludge	Mohr-Coulomb	13	0	24	1
🟩	UKEM	Mohr-Coulomb	16	0	26	2
⬜	Weathered Mudstone	Mohr-Coulomb	18	10	32	

Horz Seismic Coef.:

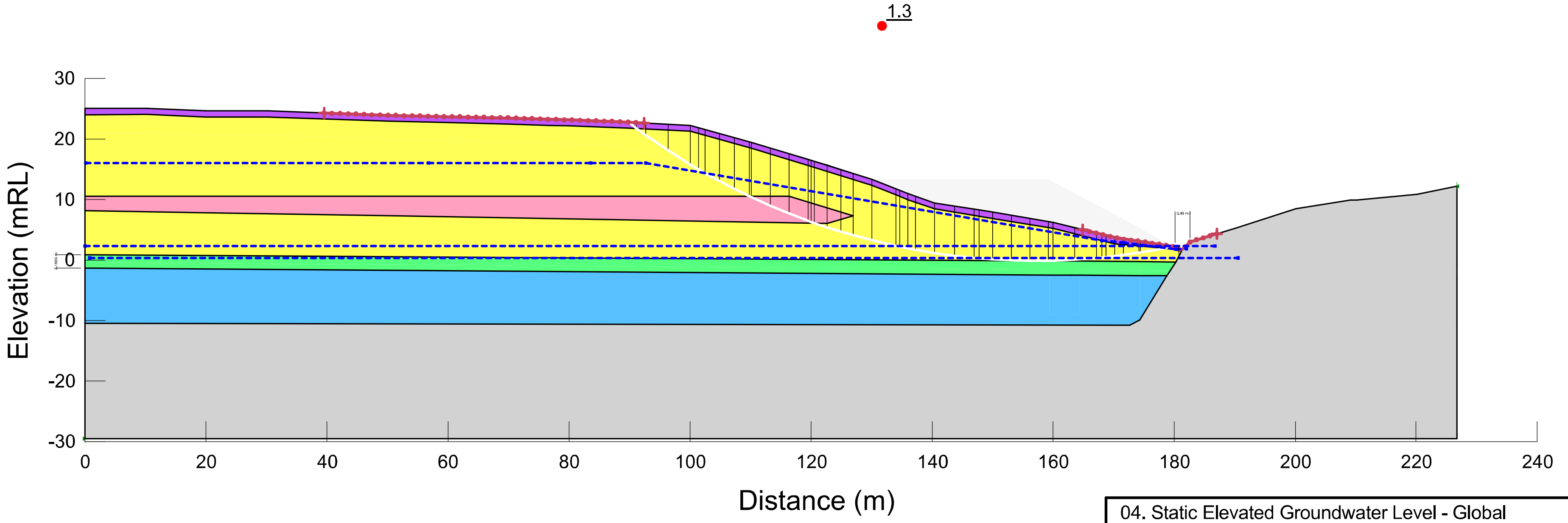


03. Static Elevated Groundwater Level - Local
GILF Section 6 revG.gsz
07/12/2022 1:750

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟨	Fill	Mohr-Coulomb	14.5	3	25	1
🟪	Final capping	Mohr-Coulomb	17	2	29	1
🟦	LKEM	Mohr-Coulomb	15.5	0	24	3
🟥	Sludge	Mohr-Coulomb	13	0	24	1
🟩	UKEM	Mohr-Coulomb	16	0	26	2
⬜	Weathered Mudstone	Mohr-Coulomb	18	10	32	

Horz Seismic Coef.:



04. Static Elevated Groundwater Level - Global

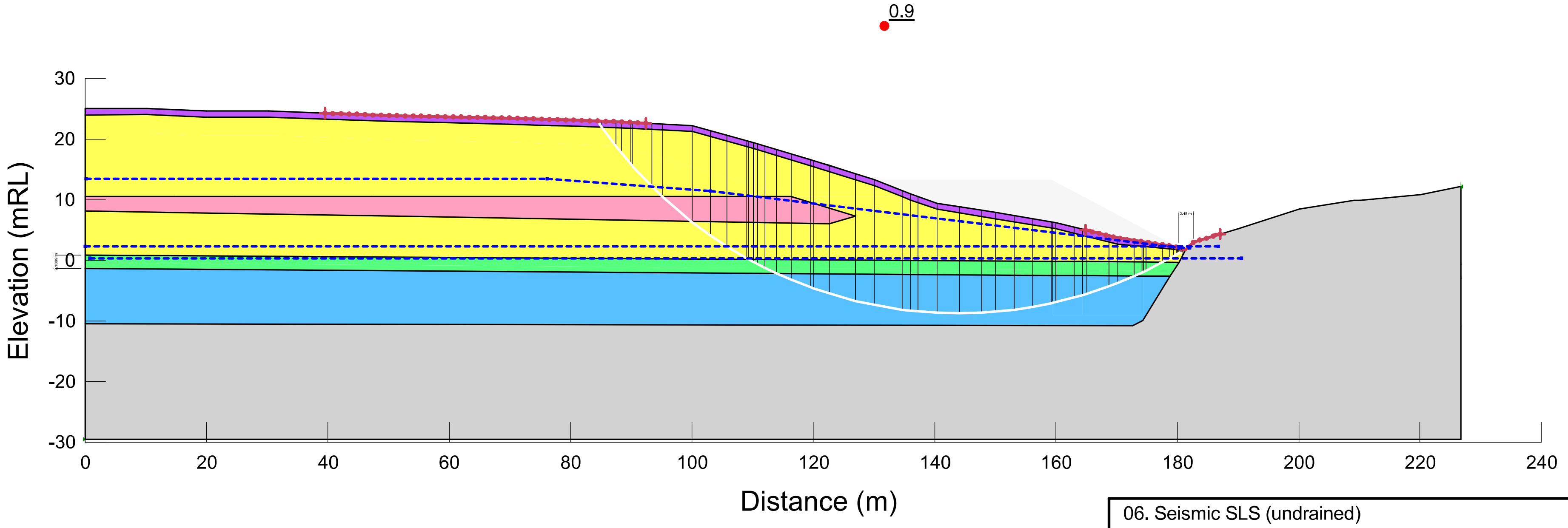
GILF Section 6 revG.gsz

07/12/2022 1:750

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Minimum Strength	Tau/Sigma Ratio	Total Cohesion	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟨	Fill	Mohr-Coulomb	14.5				3	25	1
🟪	Final capping (undrained)	Undrained (Phi=0)	17			100			1
🟦	LKEM (undrained)	SHANSEP	15.5	15	0.23				3
🟥	Sludge	Mohr-Coulomb	13				0	24	1
🟩	UKEM	Mohr-Coulomb	16				0	26	2
⬜	Weathered Mudstone (undrained)	Undrained (Phi=0)	18			200			

Horz Seismic Coef.: 0.06

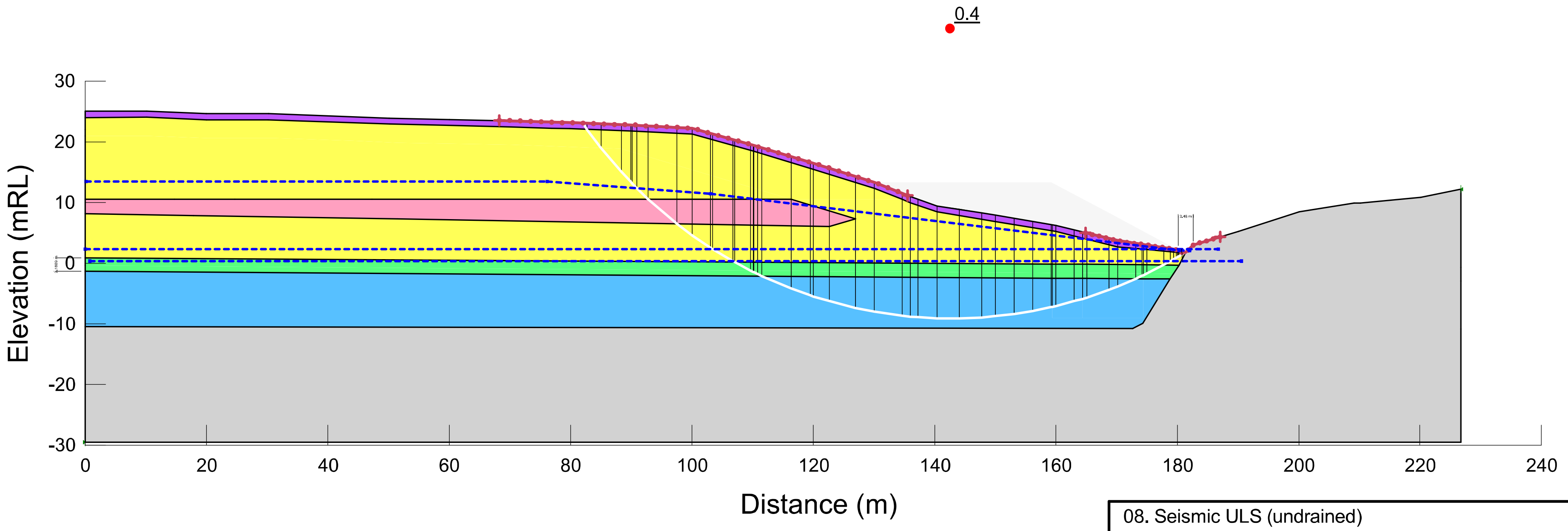


06. Seismic SLS (undrained)
GILF Section 6 revG.gsz
07/12/2022 1:750

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Minimum Strength	Tau/Sigma Ratio	Total Cohesion	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟨	Fill	Mohr-Coulomb	14.5				3	25	1
🟪	Final capping (undrained)	Undrained (Phi=0)	17			100			1
🟦	LKEM (undrained)	SHANSEP	15.5	15	0.23				3
🟥	Sludge	Mohr-Coulomb	13				0	24	1
🟩	UKEM	Mohr-Coulomb	16				0	26	2
⬜	Weathered Mudstone (undrained)	Undrained (Phi=0)	18			200			

Horz Seismic Coef.: 0.51

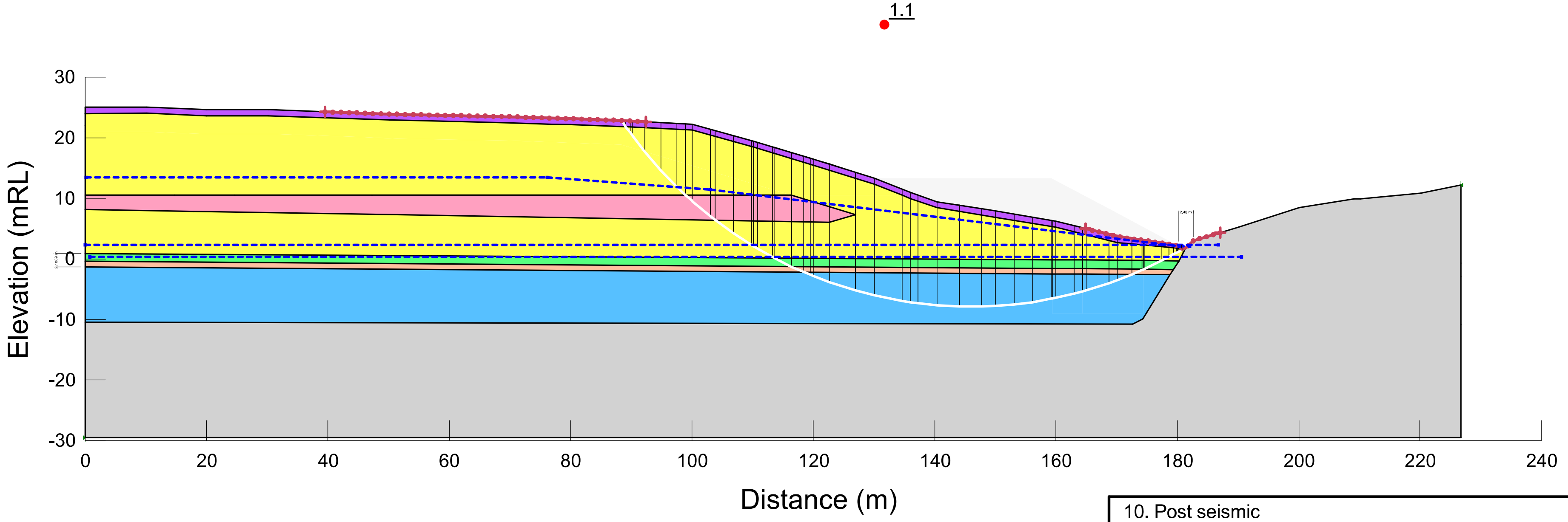


08. Seismic ULS (undrained)
GILF Section 6 revG.gsz
07/12/2022 1:750

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Minimum Strength	Tau/Sigma Ratio	Total Cohesion	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟨	Fill	Mohr-Coulomb	14.5				3	25	1
🟪	Final capping (undrained)	Undrained (Phi=0)	17			100			1
🟦	LKEM (undrained)	SHANSEP	15.5	15	0.23				3
🟥	Sludge	Mohr-Coulomb	13				0	24	1
🟩	UKEM	Mohr-Coulomb	16				0	26	2
🟧	UKEM (liq)	SHANSEP	16	0	0.08				2
⬜	Weathered Mudstone (undrained)	Undrained (Phi=0)	18			200			

Horz Seismic Coef.: 0

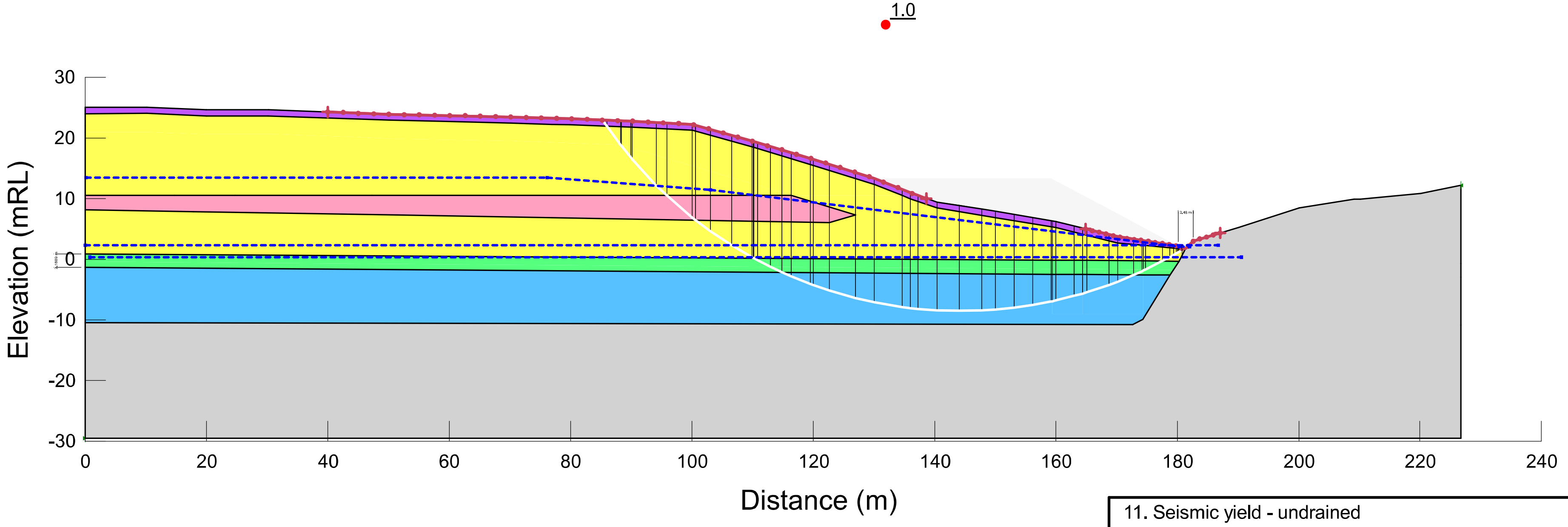


10. Post seismic
GILF Section 6 revG.gsz
07/12/2022 1:750

Analysis Type: Morgenstern-Price

Color	Name	Slope Stability Material Model	Unit Weight	Minimum Strength	Tau/Sigma Ratio	Total Cohesion	Effective Cohesion	Effective Friction Angle	Piezometric Line
🟨	Fill	Mohr-Coulomb	14.5				3	25	1
🟪	Final capping (undrained)	Undrained (Phi=0)	17			100			1
🟦	LKEM (undrained)	SHANSEP	15.5	15	0.23				3
🟥	Sludge	Mohr-Coulomb	13				0	24	1
🟩	UKEM	Mohr-Coulomb	16				0	26	2
⬜	Weathered Mudstone (undrained)	Undrained (Phi=0)	18			200			

Horz Seismic Coef.: 0.043



11. Seismic yield - undrained
GILF Section 6 revG.gsz
07/12/2022
1:750

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