Technical Memorandum

To:	Rebecca Jackson	**From:**	Matthew Adamson
Company:	Otago Regional Council	**SLR Consulting NZ**	
cc:	Sam Isles (SLR)	**Date:**	8 November 2023
		Project No.	13556

RE: RM23.185 - Green Island Landfill Geotechnical Technical Review

1.0 Introduction

SLR Consulting NZ (SLR) has been engaged by Otago Regional Council (ORC) to conduct a technical review of the resource consent application (including subsequent attachments and request for information (RFI) responses submitted by Dunedin City Council (the applicant) for the operation, expansion and closure of the Green Island Landfill.

Dunedin City Council is proposing to continue to extend the life of the Green Island Landfill to allow acceptance of waste until between December 2029 and March 2031, following which closure operations and landfill aftercare will commence.

2.0 Response

ORC posed the following questions relating to general technical matters and specific to geotechnical matters. The subsequent SLR responses to both sets of questions are presented in **Table 1** and **Table 2**.

The objectives of the geotechnical assessment, scope of works, reviewed material and assessment are provided in **Section 3.0** to **Section 8.0** of this technical memorandum.

Table 1 ORC General Technical Matter Questions

All technical disciplines	
Q:	***Is the technical information provided in support of the application robust, including being clear about uncertainties and any assumptions? Yes, or no. If not, what are the flaws?***
R:	The methodology used to perform the slope stability assessment is considered reasonable. The interpretations of the geotechnical parameters based on in-situ field testing and lab results are considered reasonable, with any review comments on the parameters deemed not critical to the overall findings of the slope stability and liquefaction assessment.
Q:	***Are there any other matters that appear relevant to you that have not been included? Or is additional information needed? Please specify what additional info you require and why [please explain]***
R:	The interpretations of the key geotechnical parameters (undrained shear strength, effective friction angle, effective cohesion) were provided upon request. The raw CPT data and laboratory results were also provided to assist with the geotechnical review.

All technical disciplines	
Q:	***If granted, are there any specific conditions that you recommend should be included in the consent?***
R:	No further conditions.

Table 2 ORC Specific Geotechnical Questions

Geotechnical	
Q:	***Is the geological and geotechnical information provided sufficient to understand the site and the land stability effects from the continued operation, closure, and aftercare of the landfill?***
R:	The desktop study and intrusive geotechnical investigations (boreholes and CPTs) performed provided sufficient detail to inform the subsoil layering and geotechnical characteristics. In addition to investigation data and laboratory test results, literature reviews and past local experience was used to determine the design parameters. The closure landfill design geometry was used to model the long-term static, seismic and post-seismic load cases.
Q:	***Does the Liquefaction and Stability Report (appendix 11) adequately address potential effects on landfill stability?***
R:	The natural soils were assessed for their liquefaction potential and their behaviour post-earthquake considered in the slope stability assessments. Where the required slope stability factors of safety were not achieved, seismic slope displacement and lateral spreading analysis was performed. Based on the assessment and findings, proposed remedial measures were discussed. Expected differential settlements due to liquefaction were calculated to be reasonably small and anticipated impact on infrastructure was considered to be minimal.
Q:	***Do you agree with the conclusions reached as to slope stability assessments?***
R:	The slope stability assessment methodology and the cross-sections selected to represent the full range of conditions across the site are considered acceptable. The required factors of safety were met for the static, long-term load cases for all cross-sections. Under SLS seismic, non-liquefaction conditions cross-sections 1, 2 and 6 did not meet the target FoS however the anticipated slope displacements were below the allowable limits. Under ULS seismic, non-liquefaction conditions, all cross-sections did not meet the target FoS however the anticipated slope displacements were below the allowable limits. Lateral spreading was calculated for the ULS seismic, liquified load case with the slope displacements below the allowable limits.
Q:	***Are the measures proposed in the Design Report (appendix 3) appropriate to minimise the release of contaminants to the environment during and following an ultimate limit state seismic event?***
R:	Localised damage to infrastructure (e.g., pipe work, capping) was identified during and post a ULS seismic event. For the section of the landfill that will experience the largest lateral deformation (but within the tolerable limits), the leachate trench has not been installed. It is recommended that the proposed trench be designed with resilience to these deformations. For the remaining sections of the landfill where leachate trenches already exist, differential settlements were expected to be minimal with redundancy measures put in place should a seismic event occur. Where the leachate pipes discharge into a buried header pipe and sewer system, remedial actions are proposed in which existing buried sewer systems be replaced with surface pipes which can accommodate ground displacement and movement. These measures are considered reasonable to mitigate the effects of a ULS seismic event.

Geotechnical	
Q:	***In your opinion, are the proposed conditions of consent appropriate to mitigate adverse effects on persons and the environment?***
R:	Yes, the slope stability and liquefaction assessment have provided an understanding of the associated risks and anticipated ground displacements and movements. All cross-sections satisfy the target slope factors of safety together with the displacement tolerance limits for all SLS and ULS load cases considered. Remedial measures have been recommended which minimise the level of adverse effects on people and the environment.
Q:	***Do you agree with the Applicant's conclusion as the level of adverse effects (associated with land stability risks) on persons and the environment?***
R:	Yes, no adverse effects are expected due to non-seismic stability conditions. Any differential settlements experienced by subsurface drainage due to liquefaction are expected to be minimal. Lateral spreading and ground movement due to a ULS seismic event can be designed for (for new sections of subsurface drainage) or mitigation and monitoring procedures can be put in place for existing subsurface drainage infrastructure to limit adverse effects on persons and the environment to within acceptable tolerance levels.

3.0 Objective

The objective for this geotechnical scope is to perform a technical review on the previous work undertaken associated with the planned extension of the landfill sites design life. As the landfill height increases, the overburden stresses on the underlying ground so to increase. As a result, the stability of the landfill embankments must continue to satisfy the factor of safety requirements for both operation and closure conditions.

This review includes the intrusive geotechnical investigations performed, ground condition classifications, geotechnical design parameter interpretations and slope stability analysis and assessments.

4.0 Scope of Work

To address the above objective, the following geotechnical scope of works as part of this review include:

- Review of relevant documents made available by ORC and associated consultants;
- Review of the interpreted geotechnical parameters used to classify the existing geotechnical conditions;
- Review of the slope stability assessment, including the seismic and liquefaction analysis; and
- Review the assessment of lateral stresses and displacements to be induced on the subsurface drainage and infrastructure due to the proposed increase in landfill height.

Following a review of the Application, a Section 92 Request for Further Information was submitted to the Applicant. This review considers the information presented in the RFI response.

5.0 Available Documentation

SLR has reviewed the following background documentation to inform the geotechnical assessment:

- Green Island Landfill Closure: Assessment of Environmental Effects (Boffa Miskell Limited), version 0, dated 16 March 2023;
- Appendix 02, General Arrangement Plan at Closure (Boffa Miskell Limited), revision D, dated 16 March 2023;
- Appendix 03, Design Report: Waste Futures – Green Island Landfill Closure (GHD), revision 1, dated 16 February 2023;
- Appendix 10, 2022 Geotechnical Investigation Factual Report: Waste Futures – Green Island Landfill Closure (GHD), revision 3, dated 5 March 2023; and
- Appendix 11, Liquefaction and Stability Assessment: Waste Futures – Green Island Landfill Closure (GHD), revision 3, dated 20 February 2023.

The following material was requested and provided to SLR to supplement the design documentation listed above:

- Ground Design Parameter Derivation (GHD), dated 17 November 2022;
- Laboratory test data – Particle Size Distributions, Water Content and Plasticity Index Results (provided by GHD); and
- Cone Penetration Testing raw data files (provided by GHD).

6.0 Assessment of Geotechnical Conditions

6.1 2022 GHD Geotechnical Investigations

Geotechnical investigations were undertaken by GHD between 17 October 2022 and 11 November 2022 to assess the ground conditions of the site. The intrusive ground investigations consisted of seven cone penetration tests (CPTs) and twelve boreholes. The location of the CPTs were performed around the toe of the landfill to classify the geotechnical conditions outside the extent of the landfill embankment. In addition, laboratory testing (Atterberg limits and particle size distribution (PSD)) was performed on samples extracted from the boreholes from varying depths and geological units.

6.2 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 based on previous studies. The KEF was divided into upper and lower layers (members), that being the Upper Kaikorai Estuary Member (UKEM), approximately 4.5 m thick, and the Lower Kaikorai Estuary Member (LKEM), approximately 6.5 m thick.

The engineering geological units encountered around the toe of the landfill are presented in **Table 3**. Note, not all boreholes were conducted around the landfill toe. The boreholes and CPTs used were: BH100 to BH104, BH108, BH111, CPT100 to CPT105, and CPT108.

Table 3 Encountered engineering geological units (Appendix 11, GHD)

Geological Unit	Description	Depth to top of unit [mbgl]	Thickness [m]
Bund	Silty, sand, and clay with MSW and wood fragments	0.0	1.3 - 13.5
UKEM	Sandy silt with minor to some clay and trace to some organics	1.3 - 5.5	0 - 3.2
LKEM	Silt, sand, and clay with pockets of organic, trace seashell	3.95 - 13.5	0 - 8.55
Abbotsford mudstone	Weathered mudstone or siltstone extremely to very weak	4.5 - 16.2	unproven

6.3 Geotechnical Design Parameters

The geotechnical design parameters adopted by GHD for the slope stability assessment are presented in **Table 4**. These parameters were derived based on the available geotechnical investigation data, laboratory test results, literature review and their past local experiences.

Table 4 Geotechnical design parameters adopted by GHD (Appendix 11, GHD)

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

6.4 Assessment

6.4.1 Geological Unit Stratification

Based on the interpretation of the CPT data, the geological unit stratification determined by GHD (Liquefaction and Stability Assessment, Appendix D) are considered accurate. There are distinct changes in cone resistance (q_t) with depth when the UKEM, LKEM and Mudstone units are encountered below the bund fill. The depth of the units below ground level are summarised in **Table 5**.

Table 5 Geological units encountered in CPT boreholes

Geological unit	Depth to top of unit [mbgl]						
	CPT100	CPT101	CPT102	CPT103	CPT104	CPT105	CPT108
Fill	0.0	0.0	0.0	0.0	0.0	0.0	0.0
UKEM	1.9	1.1	1.4	1.5	1.0	0.6	2.6
LKEM	3.3	4.2	3.6	4.4	2.6	4.0	3.8

Geological unit	Depth to top of unit [mbgl]						
	CPT100	CPT101	CPT102	CPT103	CPT104	CPT105	CPT108
Mudstone	11.7	10.5	11.1	12.6	6.2	12.0	10.8

6.4.2 Geotechnical Parameters

The geotechnical design parameters adopted for the slope stability assessment were "*derived based on the available geotechnical investigation data, laboratory test results, literature review and/or our past local experiences*." (Appendix 11, GHD). GHD have provided their geotechnical parameter derivations based on the borehole data and their CPeT-IT and CLiq results output. The GHD calculation sheet is presented in **Appendix A**. The assessment of the GHD geotechnical design parameters is provided in **Table 6**.

Commentary

Generally, the geotechnical design parameters used in the slope stability analysis for the UKEM, LKEM and Abbotsford mudstone units are considered reasonable. It should be noted that similar to LKEM, the UKEM unit would have an undrained behaviour under certain conditions given the soils high silt and clay content and low cone resistance values as observed from the CPT results. No undrained shear strength was provided.

It should also be noted that without the presence of advanced geotechnical laboratory tests (such as undrained triaxial tests) on samples from the UKEM and LKEM units, the selection of effective parameters (friction angle and cohesion) would be based on literature review and past local experience. Similarly, no geotechnical information was provided on the waste material, sludge/biosolids or final capping material. Therefore, no review can be undertaken on these design values however they appear reasonable.

Table 6 SLR assessment of the GHD geotechnical design parameters

Geotechnical Parameter	Geological Unit			
	Bund	UKEM	LKEM	Abbotsford mudstone
Unit weight [kN/m^3]	The output ranged from 16.4 to 18.5. A design value of 17 is considered reasonable for a silty sandy clayey material.	The output ranged from 15.7 to 16.4. A design value of 16 is considered reasonable for a sandy silt material.	The output ranged from 14.8 to 15.9. A design value of 15.5 is considered reasonable for a sandy silt material.	The output ranged from 19.3 to 20.3. A design value of 18 is considered reasonable for weathered mudstone.
Effective friction angle [^{o}]	The output ranged from 38.3^o to 43.5^o. A design value of 27^o is considered conservative and reasonable.	No friction angle output were provided. A design value of 26^o is considered is reasonable for a sandy silt material.	No friction angle output were provided. A design value of 24^o is considered reasonable for a clay material.	The output ranged from 39.4^o to 41.3^o. A design value of 32^o is considered conservative and reasonable.
Effective cohesion [kPa]	No cohesion output were provided. A design value of 1 is considered reasonable for a silty sandy clayey material.	No cohesion output were provided. A design value of 0 kPa is considered conservative but reasonable for a sandy silt material.	No cohesion output were provided. A design value of 0 is considered conservative but reasonable for a clay material.	No cohesion output were provided. A design value of 10 is considered reasonable for weathered mudstone.
Undrained shear strength [kPa]	The output of 131.3 was provided. A design value of 75 is considered reasonable.	The output ranged from 30 to 60.9. No design value was provided however the PSD results (FC > 52 % minimum), and q_t data (0.1-3.2 MPa) suggest a clay soil. It is therefore reasonable to assume undrained behaviour and undrained strength could be provided.	The output ranged from 25.7 to 44.5. A design SHANSEP relationship of 0.23 x overburden stress, with a minimum strength of 15 kPa was used. Given the overburden (UKEM) layer is up 3.2 m thick and groundwater close to ground level, the relationship is considered reasonable.	The output of 444.8 was provided. A design value of 200 is considered reasonable.
Liquified shear strength ratio	The output of 0.08 was provided. The material was considered to behave like a clay and classified as non-liquefiable.	The output of 0.08 was provided. A design value of 0.08 is considered reasonable.	The output of 0.08 was provided. The material was considered to behave like a clay and classified as non-liquefiable.	The output of 0.08 was provided. The material was considered to behave like a clay and classified as non-liquefiable.

7.0 Assessment of Liquefaction Risk

7.1 Liquefaction Assessment

A total of 38 Atterberg Limits were carried out for UKEM, LKEM and Weathered Abbotsford Mudstone samples ranging in depth from 1.95 m to 17.5 m below ground level. The samples tested are predominately low-plasticity clays (CL), low-plasticity silty (ML) or high-plasticity clays (CH). The results of the index testing are shown on the plasticity chart in **Figure 1**.

Liquefaction potential screening criteria has been included based on Seed et al., where Zone A soils (highlighted in red) are considered potentially susceptible to liquefaction. Zone B (highlighted in blue) may be susceptible to liquefaction. Soils plotting outside zone A and B are generally not considered to be susceptible to liquefaction triggering but may be sensitive.

The criteria is applicable for soils with fines content ≥ 20 % and plasticity index > 12 % or fines content ≥ 35% and plasticity < 12 %. Based on the particle size distribution tests, all geological units have fines contents > 35 %.

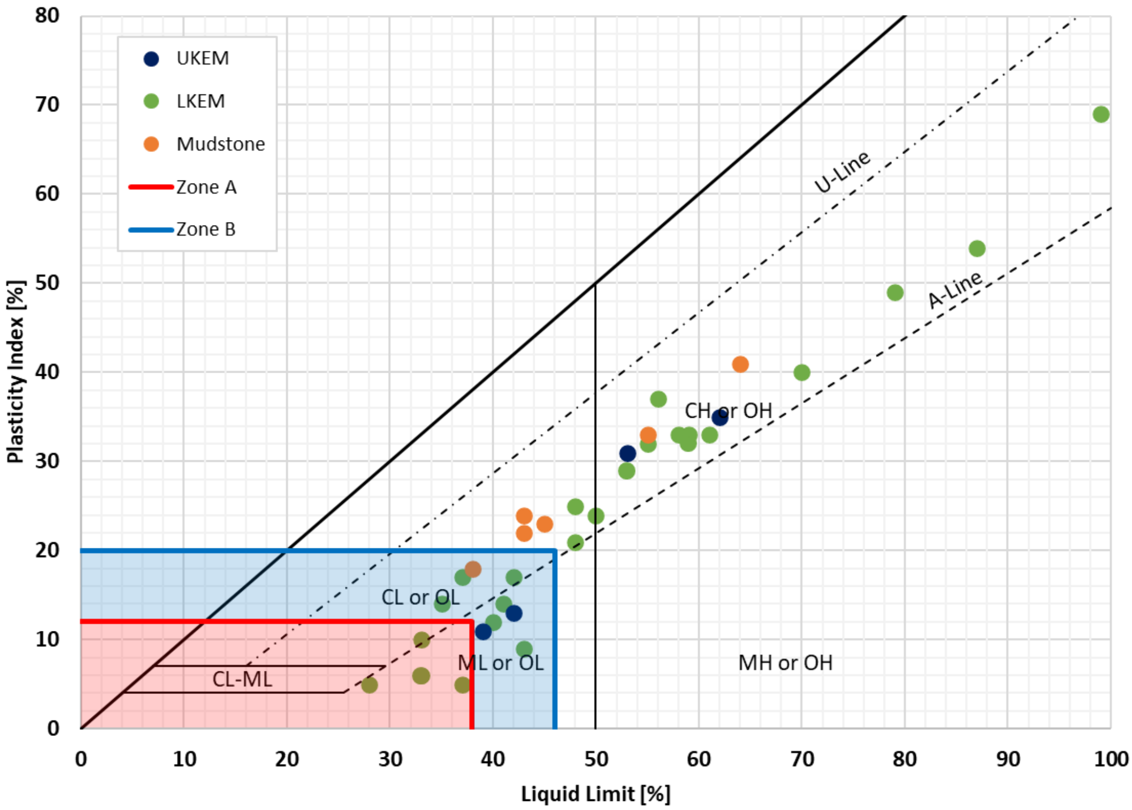

Figure 1 Plasticity Chart

<u>Commentary</u>

Based on the Seed classification, a number of samples from both the UKEM and LKEM units reflect liquefaction potential (Zone A and B). These samples have a higher level of sand content (sandy clay, sandy silty clay, sandy silt, silty sand).

The UKEM unit is predominantly characterised as generally 'sand like with occasional thin lenses of clay' while LKEM is generally 'clay like with occasional thin lenses of sand'. As a result, it is reasonable to assign the UKEM with a 'High' liquefaction risk, and the LKEM unit with a 'Low' risk.

7.2 Settlements and Risk to Underground Infrastructure

A maximum ULS free field settlement of 35 mm (CPT103) as a result of liquefaction for a return period of 2500 years was documented. SLS free field settlement was considered negligible.

As reported by GHD (Appendix 3, Design Report), "*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 liquefaction impact on the landfill and other infrastructure at the site is likely to be minimal*."

Commentary

Given the leachate interception trench (gravel-infilled trench with a slotted PVC drainage pipe) is constructed around the toe/perimeter of the landfill embankment, it is considered reasonable to assume there will be little to no impact on the leachate drainage systems from increasing the landfill embankment height.

8.0 Assessment of Slope Stability Analysis

8.1 Slope Stability Methodology

Six geological cross-sections were generated around the perimeter of the landfill and used for the slope stability assessment on the landfill closure landform. The cross-sections were selected to represent 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.

As the landfill is unlined but capped, the acceptable displacements for SLS and ULS events were considered to be < 0.3 m and < 1.0 m, respectively.

The slope stability load cases and design criteria used by GHD to perform their assessment have been summarised and presented in **Table 7**.

Table 7 Summarised slope stability load cases used by GHD (Appendix 11, GHD)

Load case	Design Criteria	Geotechnical behaviour modelled	Groundwater conditions modelled	Target FoS
Static	Local and global slip planes	Drained soil parameters to be adopted.	long term groundwater and leachate levels	≥ 1.5
			elevated groundwater and leachate levels	≥ 1.2
Seismic - SLS - non liquefied	-	Bund, final capping, LKEM and weathered mudstone units to adopt the undrained parameters. Drained parameters for the remaining units (UKEM, waste/fill and sludge/biosolids).	long term groundwater and leachate levels	≥ 1.0 (or displacement < 0.3 m)
Seismic - ULS - non liquefied	This load case is only valid when liquefaction is not anticipated.			≥ 1.0 (or displacement < 1.0 m)
Seismic - ULS - liquefied	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.			
Post-earthquake - flow failure	This load case is only valid when liquefaction is anticipated.	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 (waste/fill and sludge/biosolids).	long term groundwater and leachate levels	≥ 1.05

Commentary

Given the significantly high fines contents in the UKEM unit and cone penetration testing profiles, it is not reasonable to assume that the soil would behave drained after a seismic, non-liquefaction event. It can not be concluded if adopting the drained parameters for the UKEM layer for seismic analysis is conservative or not without modelling both scenarios. That being said, the slope stability analysis for both the SLS and ULS seismic, non-liquified load cases yielded unsatisfactory factor of safety (FoS) for the majority of cross-sections. Therefore, the behaviour of the UKEM layer during seismic, non-liquefaction can be considered non-critical and hence the analysis methodology can be considered acceptable.

8.2 Slope Stability Results

Commentary

The slope stability assessment was performed using the limit equilibrium analysis based on the Morgenstern-Price method. The static condition load cases resulted in satisfactory FoS for all cross-sections for long-term and elevated groundwater levels. Under SLS seismic, non-liquefaction conditions cross-sections 1, 2 and 6 did not meet the target FoS however the anticipated slope displacements were below the allowable limits. Under ULS seismic, non-liquefaction conditions, all cross-sections did not meet the target FoS however the anticipated slope displacements were below the allowable limits. Lateral spreading was calculated for the ULS seismic, liquified load case with the slope displacements below the allowable limits.

The parameter interpretations (yield accelerations, shear wave velocity) and the calculations for the displacements and lateral spreading were not included in the reporting so comment can not be made on the accuracy of the assessment however the design criteria and methodology outlined are considered reasonable.

9.0 Closure

SLR trusts that this technical memorandum is adequate for its purpose. We are happy to discuss any aspects of our assessment and work collaboratively with you to undertake additional revisions if required. We also draw your attention to our standard limitations (**Section 10**), which provides additional detail about the utilisation of this memo.

Regards,

SLR Consulting NZ

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Appendix A – GHD Geotechnical Design Parameters Derivation Documentation