**BEFORE A HEARINGS PANEL APPOINTED BY THE OTAGO REGIONAL COUNCIL**

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| **IN THE MATTER OF** | the Resource Management Act 1991 ("the Act" or "the RMA") |
| **AND** | |
| **IN THE MATTER OF** | An application RM23.185 by Dunedin City Council for the continued operation and closure of the Green Island Landfill, Dunedin |

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**STATEMENT OF EVIDENCE OF DR LARICAR DOMINIC ORTEGA TRANI ON BEHALF OF OTAGO REGIONAL COUNCIL**

**21 FEBRUARY 2025**

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## 1. INTRODUCTION

1.1 Qualification and Experience

(a) My full name is Dr Laricar Dominic Ortega Trani.

(b) I have a PhD (Geotechnical Engineering) from the University of Wollongong in 2010, a Master of Engineering (Geotechnical Engineering) from the Asian Institute of Technology in 2006, and a Bachelor of Science (Civil Engineering) from the University of the Philippines in 2001. I am a long-standing professional member of the Institute of Engineers Australia, as well as the Australian Geomechanics Society. I am a Chartered Engineer (Australia), I maintain registration under the National Engineering Register NSW and the Board of Professional Engineers QLD and currently applying for the Victorian Engineering Registration. Under the Mutual Recognition Agreement with New Zealand, I am also currently applying for registration under the Engineering New Zealand.

(c) I am Technical Director in Geotechnical Engineering and NSW Team Lead – Geotechnics at SLR Consulting, where I worked since February 2020. Since completing my PhD degree, I hold the position of Honorary Principal Fellow at the University of Wollongong where I deliver lectures in advanced geotechnical engineering subjects once or twice a year.

(d) I have more than 20 years extensive geotechnical design and construction experience both in major infrastructure and relatively smaller projects including foundation for bridges, major construction platforms, embankments, retaining structures, landfill closure and post closure developments, slope stability assessments & deformation analyses of seismic sensitive structures such dams and ports, and ground improvement designs. I have a comprehensive understanding of the various aspects of geotechnical engineering as they relate to major projects, having been involved in projects from the tendering and project inception stages, feasibility assessment, concept and detailed design, through to construction management and handover.

## 2. ENGAGEMENT AND OBJECTIVE

2.1 In December 2022, I was engaged by Otago Regional Council (ORC) to conduct a geotechnical 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 (GIL).

2.2 Dunedin City Council (DCC) is proposing to continue to extend the life of the GIL to allow acceptance of waste until between December 2029 and March 2031, following which closure operations and landfill aftercare will commence.

2.3 The objective for this geotechnical review scope is to perform a technical review on the previous work undertaken associated with the planned extension of the landfill site's design life. As the landfill height increases, the overburden stresses on the underlying ground also increases. Subject to the proposed change in conditions, the stability of the landfill embankments must continue to satisfy the factor of safety requirements during landfill operation and into closure/ aftercare stage.

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

## 3. SCOPE OF EVIDENCE

3.1 My evidence addresses the above objective which considers the following key aspects:

(a) Review of relevant documents made available by ORC and associated consultants;

(b) Review of the interpreted geotechnical parameters used to characterise the existing geotechnical conditions;

(c) Review of the slope stability assessment, including the seismic and liquefaction analysis; and

(d) 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.

3.2 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.

3.3 I have visited the site on 4 February 2025. I am familiar with the site layout, and surrounding environment.

3.4 My evidence builds on my review of the following documents:

(a) Green Island Landfill Closure: Assessment of Environmental Effects (Boffa Miskell Limited), version 0, dated 16 March 2023;

(b) Appendix 02, General Arrangement Plan at Closure (Boffa Miskell Limited), revision D, dated 16 March 2023;

(c) Appendix 03, Design Report: Waste Futures – Green Island Landfill Closure (GHD), revision 1, dated 16 February 2023;

(d) Appendix 10, 2022 Geotechnical Investigation Factual Report: Waste Futures – Green Island Landfill Closure (GHD), revision 3, dated 5 March 2023; and

(e) Appendix 11, Liquefaction and Stability Assessment: Waste Futures – Green Island Landfill Closure (GHD), revision 3, dated 20 February 2023.

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

(a) Ground Design Parameter Derivation (GHD), dated 17 November 2022;

(b) Laboratory test data – Particle Size Distributions, Water Content and Plasticity Index Results (provided by GHD); and

(c) Cone Penetration Testing raw data files (provided by GHD).

## 4. CODE OF CONDUCT STATEMENT

4.1 While this is not an Environment Court hearing, I nonetheless confirm that I have read and agree to comply with the Code of Conduct for Expert Witnesses in the Environment Court Practice Note 2023.

4.2 I am satisfied that the matters which I address in my evidence are within my field of expertise. I am not aware of any material facts that I have omitted which might alter or detract from the opinions I express in my evidence.

## 5. ASSESSMENT OF GEOTECHNICAL CONDITIONS

### 2022 GHD Geotechnical Investigations

5.1 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 characterise the geotechnical conditions outside the extent of the landfill embankment. In addition, laboratory testing (Atterberg limits and particle size distribution (PSD)) was performed on soil samples extracted from the boreholes from varying depths and geological units.

### Geology

5.2 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 footprint 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.

5.3 The engineering geological units encountered around the toe of the landfill are presented in Table 4 of the GHD Liquefaction and Stability Report (Appendix 11). 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.

### Geological Unit Stratification

5.4 Based on the interpretation of the CPT data, the geological unit stratification presented in Appendix D of the GHD Liquefaction and Stability Assessment Report (Appendix 11) are considered acceptable. There are distinct changes in cone resistance (qt) 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 1.

Table 1. 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* |
| Mudstone | 11.7 | 10.5 | 11.1 | 12.6 | 6.2 | 12.0 | 10.8 |

5.5 With the exception of the depths presented in underlined italics, the geological unit stratification as inferred from the CPT data is generally in agreement with the GHD Liquefaction and Stability Report (Appendix 11). The discrepancies could be explained by a delay of CPT signals registering the change in cone resistance which is not uncommon. Overall, the discrepancies are within acceptable margin of error.

**Geotechnical Parameters**

5.6 The geotechnical design parameters adopted by GHD for the slope stability assessment are presented in Table 5 of the GHD Liquefaction and Stability Report (Appendix 11). It is understood that these parameters 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 derived geotechnical parameter based on the borehole data and their CPeT-IT and CLiq results output. The GHD summary calculation sheet is presented in Appendix A of this evidence. It was therefore difficult to review the interpretations and derivations of the geotechnical design parameters. In saying that, it appears some level of conservatism was taken to parameter selection.

5.7 Generally, the geotechnical design parameters used in the slope stability analysis for the UKEM, LKEM and Abbotsford mudstone units are considered reasonable. The SLR assessment of the GHD geotechnical design parameters is provided in Table 2.

Table 2. 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 in shear 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. |

5.8 It should be noted that similar to LKEM, the UKEM unit would have an undrained behaviour under certain conditions given the encountered presence of high silt and clay content and low cone resistance values as observed from the CPT results. No undrained shear strength was provided or modelled by GHD.

5.9 The geotechnical parameters output does not include the formulas used to derive and interpret the raw data. For example, the interpretation of the effective friction angle and effective cohesion for the UKEM and LKEM units is not explained. Similarly, the Nkt factor used to derive the undrained shear strength from the CPT data is not presented.

5.10 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 or interpretations were provided on the waste material, sludge/biosolids or final capping material. Therefore, no review can be undertaken on these design values however they are considered reasonable based on previous experience of similar materials. In the absence laboratory testing, appropriate characterisation of strength of soil materials would be challenging.

5.11 Due to the absence of the values of material stiffnesses, we could not further substantiate our opinion on the deformations calculated.

**Groundwater Levels**

5.12 Two groundwater/ leachate level design scenarios were considered by GHD in the slope stability analysis based on available data from monitoring wells. These were long-term groundwater/ leachate level and elevated groundwater/ leachate level.

5.13 Within the landfill embankment including final capping and bund, the long-term groundwater level was modelled assuming the current and future subsurface drainage is functioning as designed and that the capping of the landfill reduces surface water infiltration. The drain levels constructed across the site range from 11 m above mean seal level (amsl) to 14 m amsl, with the average long-term groundwater level of 12 m amsl adopted in the modelling.

5.14 For the elevated groundwater level scenario, it was assumed that the subsurface drainage is temporarily not functioning and hence the groundwater level of 16 m amsl was adopted. This elevated level was considered conservative by GHD relative to the groundwater models documented in their Groundwater Technical Assessment Report.

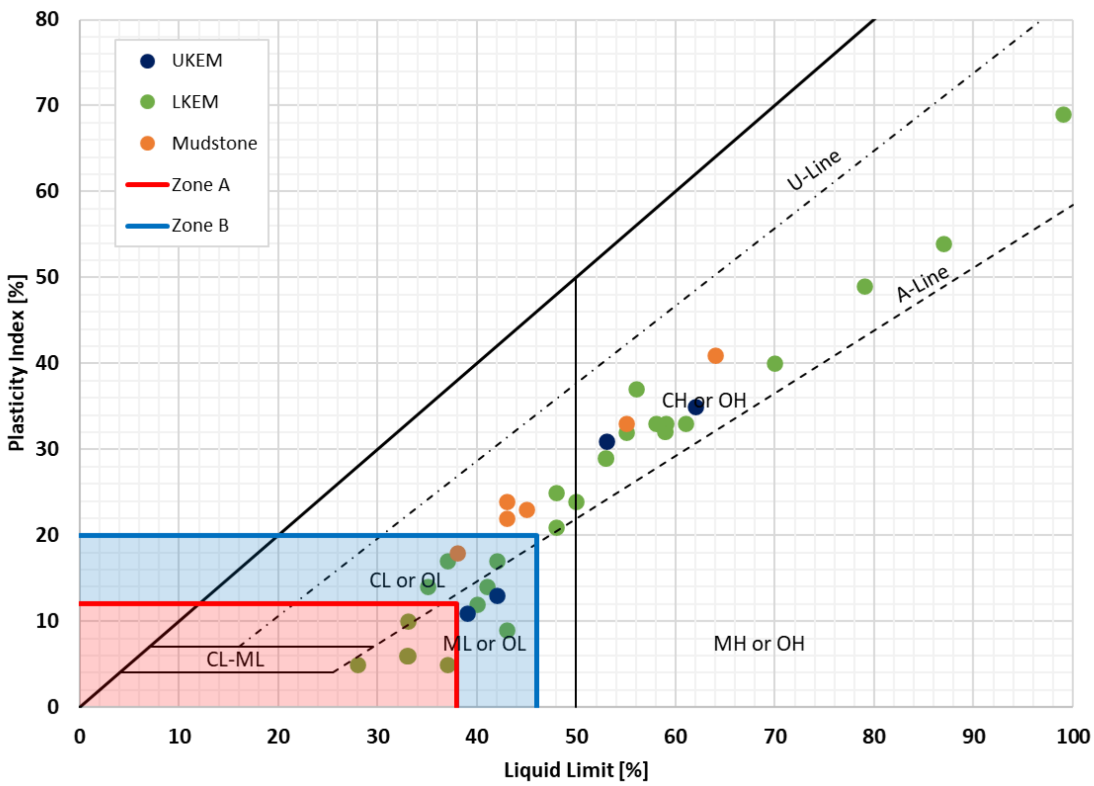
5.15 The underlying foundation layers, UKEM and LKEM were treated as fully saturated in the slope stability assessment for both long-term and elevated design cases. The piezometric lines in the model for these layers were set based on monitoring wells around the perimeter of the landfill, which closely reflected the river level around the perimeter of the site.

## 6. REVIEW OF THE LIQUEFACTION RISK ASSESSMENT

### Liquefaction Potential Screening

6.1 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.

Figure 1. Plasticity chart



6.2 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.

6.3 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 %.

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

6.5 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 susceptibility, and the LKEM unit with a 'Low' liquefaction susceptibility.

**Settlements and Risk to Underground Infrastructure**

6.6 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 by GHD.

6.7 As reported in the GHD Design Report (Appendix 3), "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."

6.8 The anticipated free field settlement calculated using CPeT-IT is a one-dimensional (1-D) vertical assessment. A 1-D analysis could be considered a simplified approach, and a two-dimensional approach could be used to refine the expected lateral and vertical displacements of the embankment and the subsoil infrastructure

6.9 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.

## 7. REVIEW OF THE SLOPE STABILITY ASSESSMENT

**Representative Geometries for Slope Stability Assessment**

7.1 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.

7.2 The cross-sections analysed include the worst-case geometry and are considered reasonable. Both the long-term and elevated groundwater conditions were considered which were determined from piezometric monitoring.

**Representative Load Cases for Slope Stability Assessment**

7.3 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.

7.4 The slope stability load cases and design criteria used by GHD to perform their assessment have been summarised and presented in Table 10 of the GHD Liquefaction and Stability Report (Appendix 11).

7.5 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 cannot 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 adopted model of the UKEM layer during seismic, non-liquefaction can be considered non-critical and hence the analysis methodology can be considered acceptable.

**Results of the Slope Stability Analyses**

7.6 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.

## 8. CONCLUSIONS AND RECOMMENDATIONS

8.1 This geotechnical review is based on the documents and information provided, and not an independent verification of the interpretation or assessment of the geotechnical parameters used to perform the slope stability and liquefaction assessment.

8.2 It is noted that no geotechnical interpretation report was done, only a factual report and a design/ analysis report. It was therefore difficult to review the interpretations and derivations of the geotechnical parameters used in the slope stability analysis. In saying that, it appears some level of conservatism was taken to parameter selection.

8.3 The desktop study and intrusive geotechnical investigations (boreholes and CPTs) performed provided sufficient detail to inform the subsoil layering and geotechnical characterisation. It should be noted that there has been no geotechnical assessment performed on the landfill materials (waste material, sludge/biosolids etc.) which introduces some uncertainties to the analysis. In addition to the investigation data and laboratory test results, GHD stated that literature reviews and past local experience was used to derive the design parameters.

8.4 The closure landfill design geometry was used to model the long-term static, seismic and post-seismic load cases. A seismic hazard study was performed to characterise the sites subsoil class, design earthquake accelerations and return periods for both SLS and ULS conditions (1/25 and 1/2500 years, respectively) for landfill closure, with a design life of 100 years.

8.5 The methodology used by GHD to perform the slope stability assessment is considered reasonable. The interpretations of the geotechnical parameters of the natural soils by GHD based on in-situ field testing and lab results are considered reasonable. It should be noted that there has been no geotechnical assessment performed on the landfill materials (waste material, sludge/ biosolids etc.) which introduces some uncertainties to the analysis. The provided review comments by myself on the natural soil parameters are deemed non-critical to the overall findings of the slope stability and liquefaction assessment and should be received as recommendations and/ or notes. In lieu of information regarding interpretation of factual data, I cannot comment on the uncertainties or assumptions in the derivation of analysis inputs.

8.6 The natural soils underlying the site were assessed for their liquefaction susceptibility and their behaviour post-earthquake considered in the slope stability assessments. Where the required slope stability factors of safety were not achieved (FoS < 1.0), seismic slope displacement and lateral spreading analysis were performed. Based on the assessment and findings, the proposed remedial measures were presented and are considered acceptable from geotechnical perspective.

8.7 Without a proper definition of the landfill embankment materials and their geotechnical design parameter derivation, the proposed values used in the analysis introduce uncertainties into the stability model.

8.8 The slope stability assessment methodology and the final landform cross-sections selected to represent the range of conditions across the site, including the worst-case geometry, subsurface profile and groundwater conditions are considered acceptable. The derived strength parameters used in the design are generally within the acceptable range of values representative of the materials considered, with some conservatism noted. There is an absence of material stiffness values presented in the report which are required when performing deformations analysis.

8.9 Under working stress design method, the required factors of safety (FoS) were met for the static, long-term load cases and groundwater conditions for the cross-sections considered. Under limit state design method, and under SLS seismic, non-liquefaction conditions cross-sections 1, 2 and 6 did not meet the target FoS of 1.0 (i.e., failed) with the anticipated slope displacements below threshold values. Under limit state design method, and under ULS seismic, non-liquefaction conditions, all six cross-sections did not meet the target FoS of 1.0 (i.e., failed) with the anticipated slope displacements below threshold values. Lateral spreading was calculated for the ULS seismic, liquified load case with the slope displacements below threshold values. It is understood that the calculation of displacements and lateral spreading were based on empirically derived one-dimensional relationships incorporating design earthquake accelerations.

8.10 It must be emphasised that in our review of available geotechnical reports, derivation and development of representative stiffnesses of the various materials was not encountered. Considering that under limit state design method the values of FoS of the slope/s were considered unacceptable, by correlation there is a high likelihood that the deformations could exceed threshold values. Due to the absence of the values of material stiffnesses, we could not further substantiate our opinion on the deformations potentially exceeding threshold values. Once values of stiffnesses are known, it is recommended that some closed-form 2D analytical solutions be undertaken as first-pass estimates of deformations (both displacements and lateral spreading).

8.11 Assessment of deformations resulting from seismic events using the presented empirical techniques, particularly in seismically active settings, may be an oversimplification of a complex behaviour. More sophisticated analysis commensurate with the associated levels of risk such as at this site may be more appropriate means of assessing deformation. It should be noted, however, to further assess with improved level of accuracy, it is expected additional advanced field/ laboratory testing and instrumentation monitoring would be required to further understand the behaviour of the soils and define inputs to any advanced computer modelling. Free field deformations are dependent on the modulus of the materials, applied vertical stresses and layer thicknesses which have not been presented in the reports for review. Investment in a digital twin which comprised of the following activities may be warranted:

(a) More advanced modelling and analysis (e.g. finite difference methods, finite element methods, etc); and

(b) Modelling to be adopted through a phased approach to allow improvements based on changes to the geometry and some advanced laboratory and investigation testing and/or installation of monitoring equipment).

8.12 Localised damage to infrastructure (e.g., pipe work, capping) was identified by GHD 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 identified), the leachate trench has not been installed. It is recommended that the proposed new section of leachate collection trench be designed with resilience to these deformations. For the remaining sections of the landfill where leachate trenches already exist, differential settlements calculated by GHD were expected to be minimal with redundancy measures put in place should a seismic event occur.

8.13 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 the ULS seismic event considered in the report.

8.14 Overall, the approach taken by GHD for slope stability and liquefaction assessment have provided an understanding of the associated risks, anticipated ground displacements and movements. The cross-sections are understood to represent critical conditions, satisfy the target slope factors of safety together with the displacement tolerance limits outlined by GHD for all SLS and ULS load cases with appropriate groundwater conditions considered. Remedial measures have been recommended by GHD which minimise the level of adverse effects on people and the environment and are considered reasonably acceptable.

**Dr Laricar Dominic Ortega Trani**

21 February 2025

**Appendices** Appendix A: GHD Geotechnical Design Parameters Derivation Documentation

# Appendix A

GHD Geotechnical Design Parameters Derivation Documentation

# CALCULATIONS

| | | | | | |
|---|---|---|---|---|---|
| Client | DCC | Project No. | 12547621 | Sheet | 1 of |
| Project | GILF Closure Consent | Calculations by | KT | Date | 17/11/22 |
| Subject | Ground Design Parameter Derivation | Checked by | | Date | |

Aim: GHD has been engaged by DCC to undertake various assessments to support the renewal of the resource consent for the Green Island landfill site located in Dunedin. The purpose of this calculation package is to document the design parameter derivation process.

Inputs:
- CPT data
- BH data

Method:
- CPeT-IT
- CLiq (for liq shear strength only)

Outputs from CPeT-IT and CLiq:

| Unit | Unit weight (kN/m3) | Friction Angle (°) | Cohesion (kPa) | Undrained shear strength (kPa) | Liquefied strength ratio |
|---|---|---|---|---|---|
| Fill | 16.4 - 18.5 (17.2) | 38.3 - 43.5 (40.4) | - | 131.3 | 0.08 |
| UKEM | 15.7 - 16.4 (15.9) | - | - | 30 - 60.9 (36.4) | 0.08 |
| LKEM | 14.8 - 15.9 (15.6) | - | - | 25.7 - 44.5 (36.5) | - |
| Mudstone | 19.3 - 20.3 (19.8) | 39.4 - 41.3 (40.2) | - | 444.8 | 0.15 |

Equivalent SPT N60
4.5, 4.2, 10.2, 5.5,4.6, 4.4,9 (6.1)
2.7, 2.3, 2.3, 2.5, 2.1, 2.9, 4.8 (2.8)
2.8, 2.8, 2.8, 3.7, 2.4, 3.4, 3.7 (3.1)
22.8, 34.9, 25.8, 22.3, 25.6, 22.3,29.6 (26.2)

BH summary

| Unit | SPT raw |
|---|---|
| Fill | 6,0,2,50+,10,50+,4,5,3,2,12,21,9,11,8,3,7,4,15,9,6,6 (7 excludes 50+ ) |
| UKEM | 6,2,1,2,1,2,2,1,1,1,1,1,0,2,1,3 (1.7) |
| LKEM | 1,1,1,6,2,1,1,1,1,1,6,2,1,1,1,1,5,8,5,5,1,1,1,1,1,1,1,1,9,11,2,9,0,1,1,4,4 (2.7) |
| Mudstone | 25,34,26,30,34,8,28,28,33,37,50+,46,50+,50+,50+,21,34,50+,43,50+,50+,50+,50+,50+,18,50+46, 50+,50+,50+,50+,50+,50+,50+,50+,50+,38,40,50+,50+ (41.7) |

| Unit | Shear vane |
|---|---|
| Fill | - |
| UKEM | 8/3,2/1,5/2,3/1,9/3,5/1 (5) |
| LKEM | 15/3,5/3,3/1,5/2,8/2,11/4,5/2,5/2,5/2,5/3,5/2,4/2,2/1,5/3,5/1,22/7 (6.9) |
| Mudstone | 7/1 |

Derived based on Skempton
Su/overburden = 0.11+0.0037PI

Recommended design values

| Unit | Unit weight (kN/m3) | Friction Angle (°) | Cohesion (kPa) | Undrained shear strength (kPa) | Liquefied strength ratio |
|---|---|---|---|---|---|
| Waste | 14.5 | 25 | 3 | - | - |
| UKEM | 16 | 26 | - | - | 0.08 |
| LKEM | 15.5 | 24 | 0 | 15+0.23*overburden | - |
| Weathered Mudstone | 18 | 32 | 10 | 200 | - |
| Final capping | 17 | 29 | 2 | 100 | - |
| Bund | 17 | 27 | 1 | 75 | - |
| Sludge | 13 | 24 | 0 | - | - |