K210215-1

8 April 2021

Geotechnical Engineering Investigation
Proposed Subdivision
Sabys Estate
Halswell
Christchurch

Prepared For:

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Geotechnical Engineering Investigation

Proposed Subdivision

Sabys Estate

Halswell

Christchurch

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K210215-1

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EXECUTIVE SUMMARY

Geotechnical Engineering Investigation
Proposed Subdivision
Sabys Estate
Halswell
Christchurch

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<tr><td rowspan="3">SUB-SOIL CONDITIONS</td><td>Lithology</td><td colspan="3">The sub-surface conditions comprise topsoil over interbedded silts, peat and sands, with gravels below.</td></tr>
<tr><td>Soil Classification as per NZS 1170.5:2004</td><td colspan="3">Class ‘D’ (Deep and soft soil sites)</td></tr>
<tr><td>Groundwater Depth</td><td colspan="3">Assumed to be at 1.5m depth for geotechnical design</td></tr>
<tr><td rowspan="8">SEISMIC ASSESSMENT</td><td rowspan="2">Vertical Displacement under Earthquake Ground Motions (Index Value)</td><td colspan="2">SLS</td><td>ULS</td></tr>
<tr><td colspan="2">Up to 50mm</td><td>Up to 100mm</td></tr>
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<tr><td colspan="2">Minor (up to 100mm)</td><td>Minor (up to 50mm)</td></tr>
<tr><td rowspan="2">Seismic Technical Categorisation</td><td>MBIE</td><td colspan="2">N/A – Rural and Unmapped</td></tr>
<tr><td>Site Specific</td><td colspan="2">TC2</td></tr>
<tr><td>SUBDIVISION ASSESSMENT</td><td colspan="4">According to our assessment, TC2 style foundations (as per the MBIE Guidance document) would be suitable for dwellings within the proposed new Lots.

Static settlements are anticipated following earthworks across the property. Please refer to Sections 10 and 11.3 of this report.

This report is a geotechnical assessment in terms of suitability of the land for subdivision and general recommendations regarding proposed development. This report is not suitable for building consent application.</td></tr>
<tr><td>REPORT DISTRIBUTION</td><td colspan="4">A full copy of this report must be provided to all relevant parties involved in the project. This should include, but not be limited to, owner, architectural designers, engineers (civil and structural) and the earthworks/building contractor.</td></tr>
<tr><td>CONSENT</td><td colspan="4">The site has been assessed as suitable for subdivision from a geotechnical point of view, and a Statement of Professional Opinion is included within Appendix E.

This report has been prepared for Subdivision Consent purposes and is insufficient for Building Consent.</td></tr>
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KGA GEOTECHNICAL

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1. INTRODUCTION

At the instruction of Davie Lovell-Smith (DLS) on behalf of Danne Mora Holdings Ltd, we have undertaken a geotechnical engineering investigation on the property at Sabys Estate in Halswell, Christchurch. The work was carried out in accordance with our Agreement dated 11 March 2021.

As part of the ground investigation, we have undertaken a detailed site and environs walkover inspection, and referenced available information from the New Zealand Geotechnical Database (NZGD), Environment Canterbury (ECan) and Christchurch City Council (CCC) websites. We have also referenced previous onsite investigations by KGA including reports K190726-1, K200095-1, K200151-1 and K200277-1. This report is intended to supersede the previously issued documentation. We have conducted a subsurface investigations comprising hand auger boreholes with associated Scala penetrometer tests and piezometers, as well as cone penetrometer testing in order to assess ground conditions and likely behaviour of the ground, as well as to provide guidance on the proposed subdivision development.

The investigation was carried out with reference to the Ministry of Business, Innovation & Employment (MBIE) Guidance - Version 3, dated December 2012 - “Repairing and rebuilding houses affected by the Canterbury earthquakes” and subsequent updates.

This report presents our findings and conclusions. It has been prepared in the context of providing geotechnical information for use in assessing the suitability of the land for subdivision. This report is not sufficient for site specific building foundation design as part of a Building Consent application.

2. SITE DESCRIPTION

The property comprises 23 and 43 Quaifes Road as well as 84 Sabys Road; it is legally described as Pt Lot 2, DP 8256, Lot 1, DP 79515 and Lot 4, DP 82666, and has an overall area of approximately 12.86 hectares. The property is irregular in shape and is located on the southwestern corner of the Quaifes Road and Sabys Road intersection in Halswell. It is otherwise bounded by rural residential properties to the north and Sabys Road Stormwater Ponds to the west, as shown on our Site Plan, in Attachment A. The site and nearby surroundings are near level, though there is a water channel between 23 and 43 Quaifes Road (through the centre of the site) and a water channel from the Stormwater Ponds extends along the western site border.

3. EXISTING STRUCTURE AND PROPOSED DEVELOPMENT

3.1 Existing Structure

The property is currently occupied by residential dwellings and various outbuildings at 43 and 23 Quaifes Road, as well as an outbuilding at 84 Sabys Road near the southern corner of the site. The remainder of the property comprises grassed paddocks, with mature trees and hedges between paddocks.

3.2 Proposed Development

We have been provided with a conceptual plan titled 'Proposed Subdivision of Lot 1 DP 79515, Lot 4 DP 82666, Sec 6 SO……. & Pt RS 1202 Public Drain' by DLS (Ref: E20106, dated: March 2021). These detail that the subject site will be subdivided into a 169 Lot development, to be completed in seven stages. We have also been provided with an 'Earthworks Concept Plan' by DLS that indicates minor areas of cut and fill, with fill depths of up to 0.9m . We have included these plans within Appendix A, and expect that subdivision design will be determined based on the findings of this report.

4. BACKGROUND INFORMATION

4.1 Sources Consulted

The following third party information sources were consulted and referred to in this report:

- KGA Sabys Road Stormwater Pond Geotechnical Report (Ref. 190726-1, dated 5 December 2019)
- KGA Geotechnical Engineering Investigation Reports for the Proposed Subdivision at Quaifes Road and Sabys Road (Ref. 200095-1, dated 14 February 2020 and Ref. K200151-1, dated 7 April 2020)
- KGA Geotechnical Engineering Investigation Report for the Proposed Site Subdivision at 43 Quaifes Road (Ref. K200277-1, dated 20 May 2020)
- New Zealand Geotechnical Database (NZGD)
- Environment Canterbury (ECan)
- Christchurch City Council (CCC)

A summary of our understanding of the information contained in the reports is presented below. We note that the below summary of information is intended to present facts contained in third party sources and does not constitute interpretation or endorsement by KGA (excepting the reports completed by KGA).

4.1 KGA (2019) Sabys Road Stormwater Pond Investigations (Ref. 190726-1)

As part of Sabys Road Stormwater Pond extension, KGA was engaged to assess potential effects of proposed pond excavations to the land at 23 Quaifes Road. The investigations included two CPTs adjacent to 23 Quaifes Road property boundary; based on this data, lateral spreading analyses were undertaken in order to assess potential impact to the adjacent property at 23 Quaifes Road. The results of the investigations are presented in the KGA geotechnical report, “Geotechnical Engineering Investigation, Sabys Road Stormwater Pond, 86 Sabys Road, Halswell, Christchurch” (Ref. 190726-1, dated 5 December 2019). The results of the investigations indicate that the potential for lateral movement of the land at 23 Quaifes Road is low for peak ground accelerations of up to 0.35g (i.e. ULS level of shaking for IL2 residential structures) due to the proposed ponds.

4.2 KGA Geotechnical Engineering Investigation Reports, Proposed Subdivision at Quaifes Road and Sabys Road (Ref. K200095-1 and K200151-1)

KGA completed a geotechnical investigation for the proposed subdivision at Quaifes and Sabys Road (Ref. K200151-1, dated 7 April 2020) inclusive of the southern portion of the subject site. The report referenced a previous KGA investigation at 23 Quaifes Road (Ref. K200095-1, dated 14 February 2020), conducted to preliminarily assess the potential of the site for subdivision purposes. The investigations included a total of 23 CPTs and ten hand auger boreholes with Scala penetrometer tests within the subject site. According to the liquefaction analyses included within the report, the majority of the property is likely to behave as a TC2 site. CPT and hand auger/Scala penetrometer data from the K200151-1 investigation is referenced further in Sections 7.1 and 7.3, and the CPT data has been used for the liquefaction and settlement analyses conducted within this report. Test locations and logs are included within Attachments B and C.

Piezometers were installed across the property at four of the hand auger locations in order to conduct ongoing measurements of groundwater depth. Preliminary measurements undertaken in March 2020 indicate that groundwater was present at 1.7m to 2.9m depth within the area investigated.

4.3 KGA Geotechnical Engineering Investigation Report, Proposed Subdivision at 43 Quaifes Road (Ref. K200277-1)

KGA completed a geotechnical investigation for the proposed subdivision at 43 Quaifes Road (Ref. K200277-1, dated 20 May 2020) inclusive of the northern portion of the subject site. The investigation included six CPTs and four hand auger boreholes with Scala penetrometer tests within the area of interest. The liquefaction analyses included within the report indicated that the property is likely to behave as a TC2 site. CPT and hand auger/Scala penetrometer data from the K200277-1 investigation is referenced further in Sections 7.1 and 7.3, and the CPT data has been used for the liquefaction and settlement analyses conducted within this report. Test locations and logs are included within Attachments B and C.

Piezometers were installed across the property at the four hand auger locations in order to conduct ongoing measurements of groundwater depth. Preliminary measurements undertaken on 13 May 2020 indicate that groundwater was present at 2.4m to 2.7m depth within the area investigated. Water levels from the Piezometers installed for the K200151-1 investigation indicated groundwater depths between 1.1m to 2.6m bgl.

4.4 New Zealand Geotechnical Database (NZGD) & Environment Canterbury (ECan)

The NZGD website was searched for geotechnical information in the form of relevant well/borehole logs and CPT data. One machine borehole and four Cone Penetrometer Tests (CPTs) were found in the vicinity of the property; These logs are reproduced in Attachment C, and are referenced further in Section 7.2 of this report.

The ECan website indicates that the site is overlaying a "Coastal Confined Gravel Aquifer."

Aerial photographs taken following the 22 February 2011 earthquake (included within Appendix B) as presented on the NZGD, did not show signs of liquefaction ejecta across the grassed areas. Google imagery taken immediately following the 4 September 2010 event did not indicate areas of potential ejecta, except locally to the north of the horse arena. There were no aerial photos taken during the other major events of the Canterbury Earthquake Sequence (CES).

A summary of the Peak Ground Acceleration (PGA) experienced on the site during the major earthquake events of the Canterbury Earthquake Sequence (CES) is given in Table 1. We have also scaled the PGA to a magnitude 7.5 event to be able to compare the recorded PGA to the ULS and SLS values recommended in the MBIE Guidance ($PGA_{7.5}$(ULS)=0.35g, $PGA_{7.5}$(SLS)=0.13g).

For the magnitude scaling factor assessment, it is important to note that the 13 June and 23 December 2011 aftershocks each comprised two separate earthquakes. In both cases, the second of the two earthquakes was the more damaging, but the first earthquake is inferred to have caused elevated pore water pressure and increase the susceptibility for liquefaction to occur. To take the two earthquakes into consideration for modelling purposes, the Tonkin & Taylor Liquefaction Vulnerability Study considers the second earthquake with an increased magnitude to include the effects of the first one. The design earthquake magnitude for each event is presented in Table 1, and copies of the Property Summary Reports are included in Appendix B.

Table 1: Summary Desk Study Information

	September 2010 (M 7.1)	February 2011 (M 6.2)	June 2011 (M 6.0)	December 2011 (M 5.9)
PGA (g)	0.30	0.34	0.14	0.13
Scaled $PGA_{7.5}$ (g)*	0.27	0.24	0.10	0.09

** Scaled to M7.5 using Idriss and Boulanger recommendations (2008)*

4.5 Christchurch City Council (CCC) – Flooding Risk

According to the flood data provided by the CCC, the property is within a CCC District Plan Flood Management Area. There are currently no recommended Finished Floor Levels for this property. It is recommended that new dwellings for the proposed subdivision are elevated well above the surrounding ground level to provide a default level of protection from surface flooding. According to information provided by DLS, we understand that the site is subject to flooding and the ground levels for the subdivision are to be raised from current levels as required.

The DLS 'Earthworks Concept Plan' plan indicates fill areas of up to 0.9m; we assume that their design accounted for potential flooding at the site.

4.6 Environment Canterbury (ECan) – Listed Land Use Register

Environment Canterbury has identified sites where hazardous activities and industries have been located throughout Canterbury, and maintains the Listed Land Use Register (LLUR) database which alerts about potential contamination before site works commence. After consultation of the database regarding the site, the following information was obtained: "The Listed Land Use Register has information relating to this land parcel." The HAIL activity identified by the LLUR is, "A10 – Persistent pesticide bulk storage or use". The area along the northeastern site boundary is also identified to have contained multiple glass houses and garden plots in aerial photographs pre-1994 to 2011. The relevant extracts from the LLUR are presented in Appendix B.

In light of the existing information, we recommend that a specialist geo-environmental consultant is engaged to review potential for contamination at the site.

5. SITE OBSERVATIONS

During our site walkovers between March 2020 and March 2021, the following was noted:

- The site is predominantly grassed paddocks.
- Medium height trees and hedges are scattered along fence lines separating different paddocks.
- Grown trees are present along the northern boundary of the overall site.
- There is a water channel through the centre of the site is approximately 2.2m wide at the top and at least 1m deep.
- A water channel is present along the western site boundary, with larger pond areas close to the site boundary in places.
- Desiccation cracks were observed in the non-grassed areas of the northern portion of the overall site, indicative of surficial groundwater.
- No visible damage to the dwellings and surrounds were apparent at 23 and 43 Quaifes Road.
- During the March 2021 site walkover, a bulk load of soils (approximately 2m high with a footprint of approximately 20m by 30m) noted near the southern corner of the site; we understand that this was placed in order to monitor the potential risk of static settlements at the site.

No obvious signs of earthquake related ground damage or settlement were noted during our site visits; however, considering time since the CES and persistent use as a horse/livestock grazing over the years, it is likely that any evidence has been obscured.

Relevant site photographs are provided in Appendix B.

6. GEOLOGY

The GNS Science published geology of the Christchurch Area shows the site is underlain by grey river alluvium comprising gravel, sand, and silt, beneath plains or low-level terraces. These deposits can range from veneers of sediment up to many tens of meters thick.

7. GROUND INVESTIGATION

7.1 Scala Penetrometer Tests and Hand Auger Boreholes

Previous KGA shallow soils investigations comprise fourteen hand auger boreholes and associated Scala penetrometer tests across the subject site (K200151-HASP01 to K200151-HASP10 and K200277-HASP01 to K200277-HASP04). Tests were conducted between 16 to 18 March and 7 May 2020; all tests achieved the target depth of 3.0m. The test locations and borehole logs are included within Attachment B.

The Scala penetrometer probe consists of a hand operated dynamic cone penetrometer and is used to evaluate the penetration resistance of a soil. A 9kg hammer weight is dropped over a distance of 510mm driving a 20mm diameter cone into the ground. The number of hammer blows is recorded for each 50mm of penetration. Testing is in accordance with Test 6.5.2:1988 of NZS 4402.

Hand auger boreholes were drilled in conjunction with the Scala penetrometer testing using a 50mm diameter hand auger and a soil description log in accordance with the NZGS Guidelines "Field description of soil and rock" is presented for each borehole. Scala Penetrometer testing was carried out at nominal 1.0m depth intervals with hand auger boreholes drilled over the Scala test points to the same depth to reduce the friction on the rod for subsequent testing.

7.2 Piezometers

In order to conduct ongoing measurements of groundwater depth, piezometers were installed at four of the hand auger locations from the K200151-1 investigation (K200151-HASP03, K200151-HASP05, K200151-HASP07 and K200151-HASP10), and at each of the hand auger borehole locations from the K200227-1 investigation. The Piezometers were installed to a depth of 3.0m below ground level, with drainage slots spaced at alternating 0.1m intervals from 1.5m to 3.0m depth.

7.3 NZGD Subsurface Information

A search of the NZGD website identified one machine borehole and four CPTs within the vicinity of the site. We considered these tests relevant, and the CPT data has been used in our liquefaction analyses. The investigation locations and logs are reproduced in Attachment C, and the data is referenced in more detail within Table 2.

Table 2: NZGD Data

NZGD Test Reference	Source	Source Reference	Probe Depth (m)
BH_110260	Beca (test located southeast of subject site)	BH02	15.5
CPT_137088	Ground Investigation / KGA Report 190726	CPT01	10.2
CPT_138931	Ground Investigation / KGA Report 190726	CPT02	10.2
CPT_128369	Pro-Drill	CPT14	14.3
CPT_128301	Pro-Drill	CPT13	15.0

7.4 KGA Cone Penetrometer Testing

Previous KGA deep soils investigations comprise 29 CPTs across the subject site, conducted between 7 February and 7 May 2020 (K200095-01 to K200095-04, K200151-01 to K200151-19, and K200227-01 to K200227-06). The test target depth was to a minimum depth of 15m or refusal, whichever came first; tests achieved depths of 7.8m to 15.0m. The test locations and CPT logs are included within Attachment C.

The CPT testing was undertaken by Ground Investigation Ltd with a 1.1 tonne Pagani TG63-150, using a cone of $10cm^2$ cross-sectional area, and a $150cm^2$ friction sleeve area. Continuous measurement of pore water pressure was undertaken during testing (u2). Tests were undertaken in accordance with A.S.T.M. Standard D 5778-12 procedure.

In order to supplement the existing CPT information, further project specific deep ground investigation was undertaken. The subsurface conditions on site were explored by drilling 14 cone penetrometer tests (K210215-01 to K210215-14) as shown in Table 3. The CPT investigation was undertaken by Ground Investigation Ltd on 16 and 22 March 2021. The test locations and CPT logs are included within Attachment C.

Table 3: Schedule of Exploratory Holes

Test Ref.	Depth (m)	Reason for Termination	Date of Test
K210215-01	15.2	Target depth	16 March 2021
K210215-02	15.1	Target depth	16 March 2021
K210215-03	15.1	Target depth	16 March 2021
K210215-04	8.1	Refusal in dense gravel	16 March 2021
K210215-05	15.1	Target depth	16 March 2021
K210215-06	15.0	Target depth	16 March 2021
K210215-07	14.0	Refusal in dense gravel	22 March 2021
K210215-08	15.0	Target depth	16 March 2021
K210215-09	14.0	Refusal in dense gravel	22 March 2021
K210215-10	15.1	Target depth	22 March 2021
K210215-11	14.2	Refusal in dense gravel	22 March 2021
K210215-12	14.5	Refusal in dense gravel	22 March 2021
K210215-13	13.4	Refusal in dense gravel	22 March 2021
K210215-14	12.9	Refusal in dense gravel	22 March 2021

The exploratory borehole locations were selected to provide representative indications of the subsurface ground conditions within the area being assessed.

7.5 Subsurface Conditions

The results of our investigation indicate that the subsurface ground conditions are variable across the site, but generally comprise the following soil profile:

- Topsoil is present to depths of 0.4m below ground level.
- Silts and sands to 1.5m to 3.5m depth.
- Silts and peat to depths of up to 5.0m to 6.9m depth. The shallow investigation encountered peat soils from 2.2m to 2.9m depth, and the onsite CPTs indicate lenses of organic silt are present from 1.5m to 5.9m depth.
- Silts and sands to depths between 6.8m and 10.9m.
- Interbedded silts, sands, and "clays" to depths between 7.2m and 15.0m; we note that this unit is absent at locations K200151-14 and K210215-04 near the southern corner of the site. However, this unit was encountered at K200151-15 located in the same area.
- Inferred dense sand and gravels below 12.8m to 14.5m depth (confirmed by nearby borehole BH_110260).

For a full detailed description, reference should be made to the investigation logs included within Attachments B and C.

7.6 Groundwater

Initial groundwater measurements conducted at the piezometer locations on 13 May 2020 indicated water levels between 1.1m and 2.8m bgl; measurements from our return visit on 16 March 2021 indicated water levels between 1.2m to 2.9m bgl. Groundwater was measured between ground level to 3.6m depth within the onsite CPTs.

Based on the above, we consider that there is significant variation in the measured groundwater table, which may be a result of sub-artesian groundwater conditions measured within the CPTs relative to the shallower hand auger boreholes. Groundwater levels can and will vary with changes in weather patterns, particularly following periods of prolonged wet or dry weather.

8. LIQUEFACTION ASSESSMENT

8.1 General

Liquefaction analyses were completed for both SLS and ULS design criteria, using the CPT data referenced in Sections 7.2 and 7.3. Calculations were performed over the full investigation depth and the MBIE ‘Index’ depth of 10.0m. We note that K200151-14 and K210215-04 were terminated at 7.8 and 8.1m depth respectively due to refusal within dense gravels. We consider that liquefaction settlements within the gravel layer are unlikely, and that analysis results from the full depth of these tests is equivalent to MBIE ‘Index’ results.

This section presents two parameters commonly used to assess the liquefaction vulnerability; the liquefaction induced settlements and the Liquefaction Severity Number (LSN).

The seismic design requirements adopted for use in the analyses are defined in Part C of the MBIE Guidelines "Repairing and rebuilding houses affected by the Canterbury earthquakes" with the recent update regarding Boulanger & Idriss (2014) liquefaction analysis methodology. These are:

- Buildings of normal use (Importance Level 2).
- Deep or soft soil sites (Class D).
- Magnitude M7.5 EQ event and peak Ground Acceleration (PGA) of 0.13g and magnitude M6.0 EQ event and peak Ground Acceleration (PGA) of 0.19g for annual exceedance probabilities of 1/25 (SLS1 and SLS2) – considering the highest calculated total volumetric strain from either scenario adopted.
- Magnitude M7.5 EQ event and 0.35g for annual exceedance probabilities of 1/500 (ULS).
- Boulanger and Idriss (2014) methodology for liquefaction triggering.
- Zhang et al. (2002) volumetric densification calculation.
- Fines content coefficient (C_{FC}) of 0.2.
- Transitional layer correction applied due to the interbedded nature of the subsoils on site.

Based on the results of our investigations, the groundwater table was assumed at 1.5m bgl both for long term and during earthquake shaking.

8.2 Calculated Settlements

We have analysed the CPT data using the current software 'CLiq' and a copy of the output from the analyses has been included within Appendix C. The software includes for normalisation of the data for overburden pressure and is considered to provide improved indications of liquefaction potential. The numerical results for SLS1, SLS2 and ULS levels of seismicity are presented within Attachment D and noted upon the site plan at the relevant CPT locations.

The majority of the calculated settlements are indicated to be within TC2 categorisation, as per the Liquefaction Analyses Plan included within Attachment D. At SLS1 and ULS levels of seismicity, the calculated settlements from all onsite CPTs are within TC2 categorisation parameters. For SLS2 levels of seismicity, calculated settlements from CPTs K200095-04, K200151-05, K200151-08, K200151-10 and CPT_128301 slightly exceed TC2 liquefaction induced settlement parameters. We have reviewed these individual CPT profiles, and have concluded that liquefiable layers are not present above 4.5m depth; we therefore consider that the entire site is likely to behave as a TC2 site as per the MBIE Guidance, based on the thickness of the upper non-liquefiable 'crust'. We further note that the DLS 'Earthworks Concept Plan' indicates that fill materials will be placed over the majority of the property, providing additional separation between the liquefiable layers and the overlying structures.

Post-earthquake general observations show that the empirical calculation of the settlement is uncertain with perhaps a ± 50% margin to the calculated results. We therefore recommend that engineering judgment is applied when interpreting the computed settlements.

We point out that the calculated settlements describe the settlements of the ground not occupied by a building, occurring due to dissipation of excess pore water pressure generated during earthquake shaking. Additional settlements may also occur due to yield of the liquefied soils under foundation loading, soil loss of volume (liquefaction ejecta) and lateral spreading, but these components are very difficult to predict. Subsurface conditions may vary across the site, making accurate prediction of future settlements even more difficult.

8.3 Liquefaction Severity Number (LSN)

The Liquefaction Severity Number (LSN) is a calculated parameter developed by Tonkin & Taylor (on behalf of the Earthquake Commission) in 2013 for use in Christchurch and presented in more detail in the Liquefaction Vulnerability Study report (T&T, 2013).Unlike the settlement parameter, the LSN assessment includes for the thickness of the non-liquefiable crust (largely controlled by depth to groundwater level) and depth at which the liquefaction will occur.

We have analysed the CPT data using the current software ‘CLiq’; at SLS2 levels of seismicity, the values range from 1 to 10 indicating a land performance with ‘little to none’ expression of liquefaction. At ULS levels of seismicity, the LSN values range from 6 to 25 (‘little to none’ to ‘moderate’ expression of liquefaction), with most results falling within the ‘minor’ expression of liquefaction parameters. Such performance is typical for TC2 sites.

8.4 Cyclic Softening Potential

There is a tendency for very soft to soft or sensitive saturated clays and silts to undergo cyclic softening (loss of shear strength and stiffness) as a result of strong shaking. According to the onsite CPT profiles, the soft organic soil layers between 1.5m and 15m depth could be prone to cyclic softening during a strong seismic event. We have therefore undertaken a cyclic softening potential assessment, based on Idriss and Boulanger (2008) methodology as implemented in Cliq software, to gain more insight into the cyclic softening potential of the soils at the site under larger levels of shaking.

The results of cyclic softening potential assessment indicate that some of the layers are predicted to undergo cyclic softening under ULS level seismic shaking. The cyclic softening of those soils is likely to contribute to additional seismic induced settlements i.e. site settlements are expected to be greater than indicated from the liquefaction analyses. Cyclic softening is not predicted at SLS levels of seismicity.

Robertson et al (2015) indicates that volumetric strains are typically 0.5% in clay-like soils, indicating potential for settlements of up to 30mm assuming weak silt layers up to 6m thick within the upper 10m depth. This indicates that there is potential for areas within the southern portion of the site to experience levels of settlements just beyond the TC2 threshold when liquefaction related settlements are combined with cyclic softening settlements of the clays and silts under ULS levels of shaking. We therefore recommend that earthworks in the area south of the ‘Local Purpose Utility Reserve to vest in CCC’ (as shown on the DLS Proposed Subdivision Plan; i.e. Lots 1 to 10, 21 to 28, 35 to 116, and 131 to 147) include at least 400mm of engineered fill over a geotextile; we recommend that areas not scheduled for 400mm of fill are over-excavated to accommodate this recommendation, which is described in greater detail within Section 11.2. This area is indicated within the KGA Earthworks Plan included as Attachment E.

9. SITE DESIGNATION AND POTENTIAL FOR LATERAL SPREADING

The site is classified as Class “D” – (Deep or soft soil) for structural purposes in the New Zealand design standards (NZS 1170.5:2004). Our aerial photograph interpretation indicates that no liquefaction ejecta was observed following the February 2011 Canterbury Earthquake.

The site experienced M_w 7.5 scaled peak ground accelerations (PGAs) of 0.1g to 0.24g, as per Table 1. Therefore, it may be classified as ‘sufficiently tested’ for an SLS event according to Section 13.5 of the MBIE Guidance document (a level of shaking more than 170% of design SLS).

According to the lateral spreading assessment for the adjacent Sabys Road Stormwater Ponds (as presented in the 2019 KGA report), the lateral spreading potential is considered to be within TC2 limits (50mm at SLS seismicity and 100mm at ULS seismicity) across the central portion of the proposed subdivision (23 Quaifes Road); we note that this analysis was based on a 4m setback of the ponds from the subdivision site. Based on the similarity of the soil profiles along the western boundary of the site as well as our understanding that the proposed development is to be based at least 7m from the ponds (as per the ‘Earthworks Concept Plan’ by DLS) and that the ponds will be limited to approximately 1.5m depth in the close proximity to the proposed subdivision, we consider that the lateral spreading potential across the entirety of the western site boundary is within TC2 limits. The presence of a minimum 4.5m non-liquefiable crust indicates that the lateral spreading potential for the remainder of the site is low at SLS levels of shaking. Furthermore, we note that the ‘Earthworks Concept Plan’ by DLS indicates that the water channel through the central portion of the site will be filled, and therefore no free-faces will be present within the development area.

Based on these considerations, we believe the site presents potential for minor global lateral movement (up to 100mm) and lateral stretch (up to 50mm) of the ground across the majority of the site.

The site is situated in a “N/A – Rural and Unmapped” area as per the MBIE mapping available on the NZGD. Based on the above and according to our liquefaction analyses, the site may be classified as TC2 as per the MBIE Guidance document and provided that our recommendations contained within Section 11.2 are followed.

10. LONG TERM SETTLEMENT POTENTIAL

Soft to firm silts, organic soils, and peat were encountered within the shallow testing and inferred within the CPT profiles at relatively shallow depths. These soils could undergo long term static settlement when loaded either from buildings or engineered fill where required to raise site levels. According to the provided ‘Earthworks Concept Plan’ by DLS, fill depths of up to 0.9m are proposed for the new subdivision.

KGA has been provided with monitoring data from onsite settlement plates on top of an approximate 2m soil mound at the southern end of the site, conducted by DLS between 3 June and 14 September 2020. In order to simulate the behaviour of the natural soil under ‘fill’ loads, we understand that a bulk load of soils (approximately 2m high with a footprint of approximately 20m by 30m) was placed near the southern corner of the site, near CPT locations K200151-06, K200151-10, and K210215-06 to K210215-08). The settlements were monitored in three locations, and the measurements indicate static settlements of up to approximately 75mm for the initial two months of loading (3 June to 4 August 2020), and additional secondary static settlements of up to approximately 10mm for the third month

(4 August to 14 September 2020). The provided location plan, data, and graphical outputs are included within Appendix A of this report.

Using CPeT-IT software based on CPTs in the vicinity of the DLS settlement test area (K200151-06, K200151-10, and K210215-06 to K210215-08) and assuming site fill of up to 2m in thickness, KGA was able to calibrate the CPeT-IT static settlement calculations with the measurements from DLS. Our analyses were found to under or overestimate the settlements by 20% to 30%.

The onsite CPT data was then analysed through the CPeT-IT software assuming site fill of up to 1m in thickness and then applied a correction factor of 30% based on the aforementioned back-analysis. The majority of primary settlements are indicated to be below 25mm; at locations where calculated settlements are greater than 25mm, we reanalysed the CPT data based on the provided fill depths from the DLS ‘Earthworks Concept Plan’.

Our analyses indicate that the soils across the site are variable, and we therefore recommend a three month delay between placement of any fill across the site, with ongoing settlement monitoring to confirm the primary settlement has substantially ceased prior to construction of dwellings. Site specific geotechnical investigation will also be required for the development of each Lot, and the foundation specifically designed for the site ground conditions.

We note that our assessment is specific to the provided DLS ‘Earthworks Concept Plan’; should fill levels deviate from those indicated within this document, KGA should be engaged to revisit the static settlement assessment.

11. GEOTECHNICAL CONSIDERATIONS FOR PROPOSED DEVELOPMENT

11.1 Geotechnical Considerations

Based on our investigation results and observations, the following recommendations should be considered for new dwellings:

- Topsoil was encountered to depths of up to 0.4m below ground level at the hand auger boreholes.
- There is potential for ongoing static settlements due to the proposed increase in site levels by up to 0.9m.
- The local groundwater level is considered to exist below 1.5m depth based upon long-term groundwater monitoring.
- According to our assessment, the property is likely to behave as equivalent to a TC2 site as defined in the MBIE Guidance document for residential sites.
- Geotextile and at least 400mm of fill material are required for Lots south of the indicated flow path (denoted by a green line on the DLS 'Earthworks Concept Plan'; i.e. Lots \1 to 10, 21 to 28, 35 to 116, and 131 to 147). Refer to KGA earthworks plan in Attachment E.
- Site-specific shallow investigations will be required in order to confirm foundation design for the development of each new Lot.

Taking into account the above constraints, we recommend that the following are considered for the proposed development:

11.2 Earthworks Fill

According to the DLS 'Earthworks Concept Plan', up to 0.9m of fill material will be placed over the subject site. This material must comprise well compacted silts placed and suitably compacted in accordance with the specifications outlined in NZS 4431:1989 (Code of practice for earth fill for residential development).

For the area indicated within the KGA Earthworks Plan included within Attachment E, we further recommend that a geotextile (Bidim A29 or similar) is placed between the in-situ soils and the engineered fill and that the minimum thickness of the fill is 400mm, placed in well-compacted layers no greater than 200mm thick. Areas not scheduled for 400mm of fill should be over-excavated to accommodate this recommendation.

11.3 Static Settlement Potential Assessment

Static settlements are considered likely following earthworks across the site. KGA recommends either applying a three month delay between fill placement and Lot development, with ongoing settlement monitoring during the three month period.

11.4 Lateral Spreading Assessment

Due to presence of a minimum 4.5m non-liquefiable crust under SLS levels of shaking, we consider that the lateral spreading potential at the site is low at SLS levels of shaking. However, shallow liquefiable layers were calculated at ULS levels of shaking based on nearby CPTs. Based on the similarity of the soil profiles along the western boundary of the site as well as our understanding that the proposed development is to be based at least 7m from the ponds (as per the 'Earthworks Concept Plan' by DLS) and that the ponds will be limited to approximately 1.5m depth in the close proximity to the proposed subdivision, we consider that the lateral spreading potential across the entirety of the western site boundary is within TC2 limits.

12. SUITABILITY OF SITE FOR SUBDIVISION

Section 106 of the Resource Management Act (RMA) states that a consent authority may refuse to grant subdivision consent, or may grant conditional subdivision consent, if it considers that:

- There is a significant risk from natural hazards
- Sufficient provision has not been made for legal and physical access to each allotment to be created by the subdivision.

Our assessment of the site against the requirements of Section 106 is presented in Table 4.

Table 4: Assessment of the site against the RMA requirements

Hazard	Potential Susceptibility	
	Current (Section 106, 1A)	Post Development (Section 106, 1Ac)
Erosion	No evidence of erosion or gullying across site.	Provided stormwater is discharged in a controlled manner and appropriate engineering design is implemented, erosion is unlikely to worsen due to development or have significant adverse effects on the development.
Falling Debris	The site is flat and located away from any hills, with no risk of falling debris.	--
Slippage / lateral movement	The site presents a minor to nil potential for global horizontal movement and lateral stretch.	The development will not have any effect that could worsen the existing potential for horizontal movement on site.
Subsidence	Potential liquefaction hazard, cyclic softening, and static settlement is present on site.	Significant damage is unlikely to occur to the proposed development provided building foundations are appropriately designed as per recommendations within Section 11 of this geotechnical report and Lot-specific investigations are undertaken.
Inundation	The site is within a flood management area as per the CCC District Plan. The CCC should be contacted to comment on floor level requirements.	Provided stormwater discharge is appropriately managed, any CCC finished floor level requirements are respected, and/or advise is sought from an experienced civil engineer, we consider that the risk of inundation will not exacerbated by developing the land and may be appropriately managed.

Based on these considerations, we believe on reasonable grounds that the site would likely be suitable for the proposed subdivision in terms of geotechnical constraints. A Statement of Professional Opinion on the suitability of land for subdivision is presented in Appendix E. It should be noted that other natural hazards (such as tsunamis) not specifically included in Table 4 are outside of the scope of works for our geotechnical investigation.

13. FURTHER WORKS

This report is a geotechnical assessment in terms of suitability of the land for subdivision and general recommendations regarding proposed development. Details of proposed building development were not known at the time of writing this report and further coordination will be required to conduct site-specific investigations and confirm foundation design for each new Lot. KGA should be engaged to review the conclusions of this report should the earthworks diverge from those indicated on the provided plan.

14. LIMITATIONS

Our report was prepared in line with the current MBIE Guidance. To satisfy the requirements of the Building Code and the New Zealand Standard “NZS 1170 - Structural Design Actions”, foundations must be designed so that the building must remain functional under SLS level loads; minor damage is acceptable provided the damage is readily repairable, and the building does not collapse under ULS level loads, but could suffer moderate to significant structural damage.

Desk study data was obtained from several investigation and modelling study sources made available to the public and engineering industry post the Canterbury earthquake sequence. Acknowledgment is given regarding the free use of the New Zealand Geotechnical Database, Christchurch City Council and Environment Canterbury websites. The data is used in good faith and no responsibility can be taken for the accuracy or completeness of the data.

This report was prepared in the context defined in Section 1 above and must not be relied upon by any other party other than our Client, for whom it was prepared, and the relevant Territorial Authority. It has been compiled with respect to the brief given to us, and must not be relied upon in any other context, recreated for any other purpose, or used by any person who is not our Client without first obtaining our written permission.

We point out that our conclusions are based on desk study material, a visual surface inspection of the site, third party investigation data from nearby sites and discrete exploratory hole positions. Ground conditions may vary between investigation points. The recommendations given in this report are provided as an overall strategy to minimise risks from geotechnical hazards. It should be noted that they are unlikely to remain effective if they are adopted in a piecemeal manner.

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