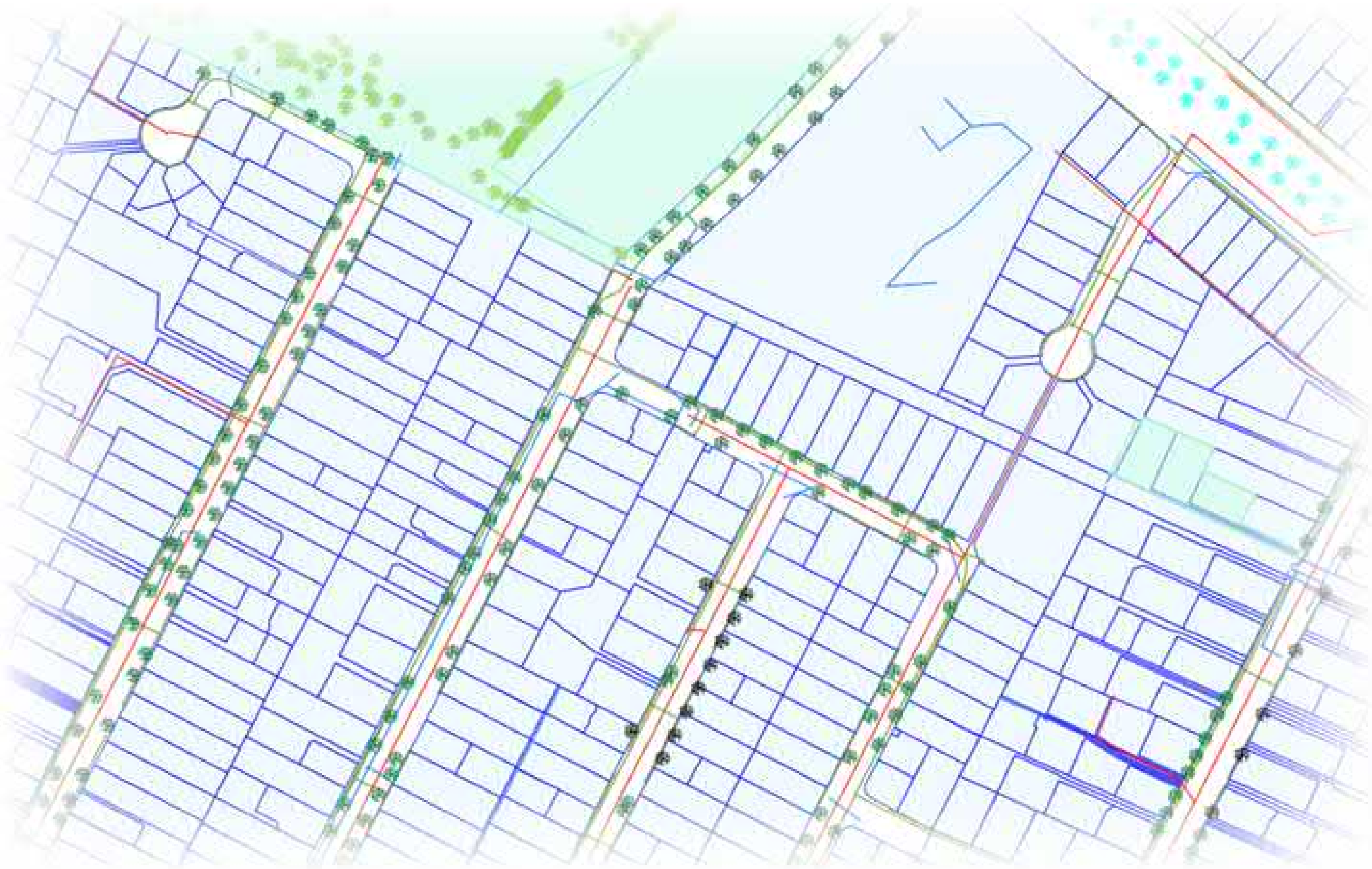
Land Information Memorandum



Property address:
133 Prestons Park Drive

Christchurch City Council
53 Hereford Street, PO Box 73015
Christchurch 8154, New Zealand
Tel 64 3 941 8999
Fax 64 3 941 8984
www.ccc.govt.nz

LIM number: 70222155

Land Information Memorandum

Christchurch City Council

Application details

Please supply to	PROFESSIONALS CHRISTCHURCH 33 HALSWELL ROAD HORNBY CHRISTCHURCH 8025
Client reference	CDL
Phone number	338 5924
Fax number	
Date issued	28 June 2019
Date received	27 June 2019

Property details

Property address	133 Prestons Park Drive
Valuation roll number	21823 73900
Valuation information	Capital Value: $ Land Value: $ Improvements Value: $ *Please note: these values are intended for Rating purposes*
Legal description	Lot 3030 DP 531672
Existing owner	CDL Land New Zealand Limited PO Box 3248 Auckland 1140

Council references

Debtor number	4124228
Rate account ID	73190994
LIM number	70222155
Property ID	1185315

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Document information

This Land Information Memorandum (LIM) has been prepared for the purpose of section 44A of the Local Government Official Information and Meetings Act 1987 (LGOIMA). It is a summary of the information that we hold on the property. Each heading or "clause" in this LIM corresponds to a part of section 44A.

Sections 1 to 10 contain all of the information known to the Christchurch City Council that must be included under section 44A(2) LGOIMA. Any other information concerning the land as the Council considers, at its discretion, to be relevant is included at section 11 of this LIM (section 44A(3) LGOIMA).

The information included in this LIM is based on a search of Council records only and there may be other information relating to the land which is unknown to the Council. Council records may not show illegal or unauthorised building or works on the property. The applicant is solely responsible for ensuring that the land is suitable for a particular purpose.

If there are no comments or information provided in any section of this LIM this means that the Council does not hold information on the property that corresponds to that part of section 44A.

A LIM is only valid at the date of issue as information is based only upon information the Council held at the time of that LIM request being made.

Property file service

This Land Information Memorandum does not contain all information held on a property file. Customers may request property files by phoning the Council's Customer Call Centre on (03) 941 8999, or visiting any of the Council Service Centres. For further information please visit www.ccc.govt.nz .

To enable the Council to measure the accuracy of this LIM document based on our current records, we would appreciate your response should you find any information contained therein which may be considered to be incorrect or omitted. Please telephone the Customer Call Centre on (03) 941 8999.

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A search of records held by the Council has revealed the following information:

1. Special features and characteristics of the land

Section 44A(2)(a) LGOIMA. This is information known to the Council but not apparent from the district scheme under the Town and Country Planning Act 1977 or a district plan under the Resource Management Act 1991. It identifies each (if any) special feature or characteristic of the land concerned, including but not limited to potential erosion, avulsion, falling debris, subsidence, slippage, alluvion, or inundation, or likely presence of hazardous contaminants.

(For enquiries, please phone (03) 941 8999 or visit www.ccc.govt.nz.

- **ECan Liquefaction Assessment**

  ECan holds indicative information on liquefaction hazard in the Christchurch area.Information on liquefaction can be found on the ECan website at www.ecan.govt.nz/liq or by calling ECan customer services on Ph 03 353 9007. The Christchurch City Council may require site-specific investigations before granting future subdivision or building consent for the property, depending on the liquefaction potential of the area that the property is in.

- **Consultant Report Available**

  Land Information New Zealand (LINZ) engaged Tonkin and Taylor to provide a Geotechnical Report on Ground Movements that occurred as a result of the Canterbury Earthquake Sequence. The report indicates this property may have been effected by a degree of earthquake induced subsidence. The report obtained by LINZ can be accessed on their website at https://www.linz.govt.nz/land/surveying/earthquakes/canterbu ry-earthquakes/ information-for-canterbury-surveyors

- **Coastal Hazard Inundation**

  The Council has a report, Coastal Hazard Assessment for Christchurch and Banks Peninsula (2017), that indicates this property or part of this property may be susceptible to coastal inundation (flooding by the sea). The 2017 report considers four sea level rise scenarios through to the year 2120. A copy of the 2017 report and other coastal hazard information can be found at www.ccc.govt.nz/coastalhazards.

- **Fill**

  This property is located in an area known to have been filled. The year the fill occurred is 2016. The filling was, according to the Councils records carried out in a controlled manner and comprises Sand.

- **Contains or contained a Tank**

  Council Records indicate that this site contains or contained a Tank Details of Tank are as follows:Date Installed: NA Tank Function: Diesel Volume(l): 4500 Underground or Above Ground: Underground Tank Status: Tank Does Not Exist Date Removed: 13-04-1994 Condition when Removed: Good

- **Contains or contained a Tank**

  Council Records indicate that this site contains or contained a Tank Details of Tank are as follows:Date Installed: NA Tank Function: Septic Tank Volume(l): NA Underground or Above Ground: Underground Tank Status: Tank Exists Date Removed: NA Condition when Removed: NA

Related information

- There is attached a sub division soil investigation report covering this property.
- Please find attached an aerial shot showing the approximate location of the current/ or removed tanks at this site

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2. Private and public stormwater and sewerage drains

Section 44A(2)(b) LGOIMA. This is information about private and public stormwater and sewerage drains as shown in the Council's records.

(For stormwater and sewerage enquiries, please phone (03) 941 8999 or visit www.ccc.govt.nz.

Related information

- No up-to-date drainage plan is available for the development of this site. However, the installation of a water connection along with sewer and stormwater drains is checked by the Council prior to the issue of a Code Compliance Certificate.

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3. Drinking Water Supply

Section 44A(2)(ba) and (bb) LGOIMA. This is information notified to the Council about whether the land is supplied with drinking water, whether the supplier is the owner of the land or a networked supplier, any conditions that are applicable, and any information the Council has about the supply.

Please note the council does not guarantee a particular water quality to its customers. If you require information on current water quality at this property please contact the Three Waters & Waste Unit.

(For water supply queries, please phone (03) 941 8999 or visit www.ccc.govt.nz.

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4. Rates

Section 44A(2)(c) LGOIMA. This is information on any rates owing in relation to the land.

(For rates enquiries, please phone (03) 941 8999 or visit www.ccc.govt.nz.

(a) Annual rates

Annual rates to 30/06/2019: $ 0.00

	Instalment Amount	Date Due
Instalment 1	$	
Instalment 2	$	
Instalment 3	$	
Instalment 4	$	

Rates owing as at 28/06/2019: $ 0.00

(b) Excess water charges

$ 0.00

(For water charge enquiries, please phone (03) 941 8999 or visit www.ccc.govt.nz.

(c) Final water meter reading required?

No Reading Required

(To arrange a final water meter reading, please phone (03) 941 8999 or visit www.ccc.govt.nz.

Related information

- The rates are not showing for this site. For this information please contact the rates team on 941-8999.

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5. Consents, certificates, notices, orders, or requisitions affecting the land and buildings

Section 44A(2)(d) LGOIMA. This is information concerning any consent, certificate, notice, order, or requisition, affecting the land or any building on the land, previously issued by the Council.

Section 44A(2)(da) LGOIMA. The information required to be provided to a territorial authority under section 362T(2) of the Building Act 2004. There is currently no information required to be provided by a building contractor to a territorial authority under section 362T(2) of the Building Act 2004. The Building (Residential Consumer Rights and Remedies) Regulations 2014 only prescribed the information that must be given to the clients of a building contractor.

(For building enquiries, please phone (03) 941 8999, email EPADutyBCO@ccc.govt.nz or visit www.ccc.govt.nz.

(a) Consents

- BCN/2019/772 Applied: 11/02/2019 Status: Completed
  133 Prestons Park Drive Burwood
  Exemption from building consent approved 18/02/2019
  A two meter high (max), 189m long retaining wall situated on the western boundary of Prestons Park Subdivision.
- BCN/2019/3699 Applied: 12/06/2019 Status: On Hold - Building Consent Officer Processing
  133 Prestons Park Drive Burwood
  Accepted for processing 13/06/2019
  Construction of dwelling with attached garage - Lot 217

(b) Certificates

Note: Code Compliance Certificates were only issued by the Christchurch City Council since January 1993.

(c) Notices

- **Ministry of Business, Innovation & Employment Foundation Design**

  Some properties have experienced land damage and considerable settlement during the sequence of Canterbury earthquakes. While land in the green zone is still generally considered suitable for residential construction, houses in some areas will need more robust foundations or site foundation design where foundation repairs or rebuilding are required. Most properties have been assigned a technical category. Details of the MBIE guidance can be found at www.building.govt.nz/

(d) Orders

(e) Requisitions

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6. Certificates issued by a building certifier

Section 44A(2)(e) LGOIMA. This is information notified to the Council concerning any certificate issued by a building certifier pursuant to the Building Act 1991 or the Building Act 2004.

(For building enquiries, please phone (03) 941 8999, email EPADutyBCO@ccc.govt.nz or visit www.ccc.govt.nz.

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7. Weathertightness

Section 44A(2)(ea) LGOIMA. This is information notified to the Council under section 124 of the Weathertight Homes Resolution Services Act 2006.

(For weathertight homes enquiries, please phone (03) 941 8999 or visit www.ccc.govt.nz.

If there is no information below this means Council is unaware of any formal Weathertight Homes Resolution Services claim lodged against this property.

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8. Land use and conditions

Section 44A(2)(f) LGOIMA. This is information relating to the use to which the land may be put and conditions attached to that use. The planning information provided below is not exhaustive and reference to the Christchurch District Plan and any notified proposed changes to that plan is recommended: https://ccc.govt.nz/the-council/plans-strategies-policies-and-bylaws/plans/christchurch-district-plan/.

There maybe some provisions of the Christchurch City Plan or Banks Peninsula District Plan that affect this property that are still operative.

(For planning queries, please phone (03) 941 8999, email DutyPlanner@ccc.govt.nz or visit www.ccc.govt.nz.

- **Regional plan or bylaw**

There may be objectives, policies or rules in a regional plan or a regional bylaw that regulate land use and activities on this site. Please direct enquiries to Canterbury Regional Council (Environment Canterbury).

(a)(i)Christchurch City Plan & Banks Peninsula District Plan

(ii)Christchurch District Plan

- **Liquefaction Management Area (LMA)**

  Property or part of property within the Liquefaction Management Area (LMA) Overlay which is operative.

- **Outline Development Plan**

  Property or part of property is within an Outline Development Plan area which is affected by specific provisions that are operative.

- **Flood Management Area**

  Property or part of property within the Flood Management Area (FMA) Overlay which is operative.

- **Fixed Minimum Floor Overlay**

  This property or parts of the property are located within the Fixed Minimum Floor Overlay level in the Christchurch District Plan. Under this plan pre-set minimum floor level requirements apply to new buildings and additions to existing buildings. The fixed minimum floor level can be searched at http://ccc.govt.nz/floorlevelmap. For more information please contact a CCC duty planner on 941 8999.

- **District Plan Zone**

  Property or part of property within the Residential New Neighbourhood Zone which is operative.

(b) Resource consents

If there are any land use resource consents issued for this property the Council recommends that you check those resource consents on the property file. There may be conditions attached to those resource consents for the property that are still required to be complied with.

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- RMA/2012/462 - Subdivision Consent
  153 Mairehau Road Burwood
  400 LOT SUBDIVISION s223 issued 13/8/13 LT 466017 - Historical Reference RMA92019798
  Status: Consent issued
  Applied 30/03/2012
  Decision issued 19/09/2012
  Granted 19/09/2012

- RMA/2015/1309 - Subdivision Consent
  155 Mairehau Road Burwood
  Fee Simple - Sixty Five Lots - Residential Layout changes - applicant to respond - Historical Reference RMA92029567
  Status: On hold - processing suspended by applicant
  Applied 15/05/2015

- RMA/2015/2996 - Subdivision Consent
  155 Mairehau Road Burwood
  Fee Simple Subdivision - Three Lots - Historical Reference RMA92031377
  Status: Processing complete
  Applied 28/10/2015
  s223 Certificate issued 08/08/2017
  s224 Certificate issued 08/08/2017
  Decision issued 15/01/2016
  Granted 14/01/2016

- RMA/2013/1085 - Subdivision Consent
  3 Roys Street Burwood
  434 LOT FEE SIMPLE SUBDIVISION - STAGE 2 - Historical Reference RMA92022731
  Status: Consent issued
  Applied 18/06/2013
  Decision issued 26/06/2014
  Granted 26/06/2014

- RMA/2018/1970 - Combined subdivision / land use consent
  74 Prestons Park Drive Burwood
  Eighteen lot fee simple subdivision (with two road allotments to vest) and associated land use application
  Status: Consent issued
  Applied 15/08/2018
  Decision issued 17/12/2018
  Granted 17/12/2018

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9. Other land and building classifications

Section 44A(2)(g) LGOIMA. This is information notified to the Council by any statutory organisation having the power to classify land or buildings for any purpose.

(For land and building enquiries, please phone (03) 941 8999 or visit www.ccc.govt.nz.

Please refer to Section 1 for details

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Christchurch City Council

10. Network utility information

Section 44A(2)(h) LGOIMA. This is information notified to the Council by any network utility operator pursuant to the Building Act 1991 or the Building Act 2004.

(For network enquiries, please phone (03) 941 8999 or visit www.ccc.govt.nz.

l **None recorded for this property**

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11. Other information

Section 44A(3) LGOIMA. This is information concerning the land that the Council has the discretion to include if it considers it to be relevant.

(For any enquiries, please phone (03) 941 8999 or visit www.ccc.govt.nz.

(a) Kerbside waste collection

- Your recycling is collected Fortnightly on the Week 2 collection cycle on a Wednesday. Please leave your recycling at the Kerbside by 6:00 a.m. Your nearest recycling depot is the Styx Mill EcoDrop.

- Your refuse is collected Fortnightly on the Week 2 collection cycle on a Wednesday. Please leave your rubbish at the Kerbside by 6:00 a.m. Your nearest rubbish depot is the Styx Mill EcoDrop.

- Your organics are collected Weekly on Wednesday. Please leave your organics at the Kerbside by 6:00 a.m.

(b) Other

- **Community Board**

  Property located in Coastal-Burwood Community Board
- **Guest Accommodation**

  Guest accommodation (including whole unit listings on Airbnb; BookaBach; etc.) generally requires a resource consent in this zone when the owner is not residing on the site. For more information, please refer to: https://ccc.govt.nz/providing-guest-accommodation/.
- **Electoral Ward**

  Property located in Burwood Electoral Ward
- **Listed Land Use Register**

  Hazardous activities and industries involve the use, storage or disposal of hazardous substances. These substances can sometimes contaminate the soil. Environment Canterbury identifies land that is used or has been used for hazardous activities and industries. This information is held on a publically available database called the Listed Land Use Register (LLUR). The Christchurch City Council may not hold information that is held on the LLUR Therefore, it is recommended that you check Environment Canterbury's online database at www.llur.ecan.govt.nz
- **Spatial Query Report**

  A copy of the spatial query report is attached at the end of this LIM. The spatial query report lists land use resource consents that have been granted within 100 metres of this property.

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18 May 2018

Sean Ward
Senior Planner
Christchurch City Council
Christchurch, PO Box 7301332
Christchurch 8154

Dear Sean

Prestons South Stage 3 Subdivision Resource Consent Geotechnical Report – Review of 99 Mairehau Road, Marshland, Christchurch
Our Ref: 235361

CDL Land NZ Ltd wish to subdivide the sawmill site for residential development and the site will form part of the Prestons South Subdivision Stage 3 works. This letter provides a review of geotechnical information to support subdivision consent of the Royal Timber Supplied property located at 99 Mairehau Road, Mashland, Christchurch.

Previously 99 Mairehau Road was included in the wider Prestons South subdivision area however the sawmill site was later excluded from the Subdivision Consent application (referred to as Stage 2) due to potential land contamination. Details of the geotechnical investigations are provided in the *Prestons South Subdivision Consent Geotechnical Report, Aurecon NZ Ltd., Ref. 235361, Rev 1* dated 6 June 2013.

Since lodging subdivision consent for Prestons South Subdivision (Stage 2) and subsequent development of the wider area CDL Land Ltd wish to develop the land at 99 Mairehau Road. Aurecon NZ were requested to undertake a review the Prestons South Subdivision Consent Geotechnical Report (2013) in alignment with the proposed subdivision works. This letter will accompany the resource consenting process for subdivision for Prestons South Subdivision Stage 3.

Details of the review are summarised below:

- Aurecon NZ has undertaken detailed geotechnical investigations and assessment for the wider Prestons North (Stage 1) and South (Stage 2) Subdivisions for the purposes of plan change, subdivision resource consent application, liquefaction assessment, technical classification of the entire subdivision, observation of bulk earthworks construction, and site specific geotechnical investigations and assessment of individual lots for building consent applications purposes.
- The sawmill site (at 99 Mairehau Road) was included in the Prestons South Subdivision area, see *Appendix A Figures, Prestons South Subdivision Consent Geotechnical Report, Aurecon NZ Ltd., Ref. 235361, Rev 1* dated 6 June 2013, however the sawmill site was later excluded from the initial Subdivision Consent application due to continued use as a sawmill and the site's potential for land contamination. Geotechnical Report (2013) attached for reference.
- However, the sawmill site area located at 99 Mairehau Road was included as part of the geotechnical investigation and reporting, see attached Figure 235361-GIS-23-C for area extent.
- Engineering considerations and recommendations provided in *Section 6* of the Prestons South Subdivision Consent Geotechnical Report (Aurecon, 2013) were reviewed and considered relevant to the saw mill site.

- We note that given the sites historic use as a sawmill there is potential for fill and environmentally impacted soil that may impact on the land being able to be used for subdivision development without being remediated. As part of our scope of work we aim to complete at subdivision development stage an extensive geotechnical testing regime to identify any unsuitable materials and propose remediation measures should those be required. Remediation measures are likely to comprise removal of unsuitable materials and replacement with suitable fills to allow the development of residential infrastructure.
- Changes are noted to *Section 7* and *Table 13* in the Prestons South Subdivision Consent Geotechnical Report (Aurecon, 2013) due to recent amendments to the Resource Management Act (RMA) in 2017. In alignment with these recent amendments, a risk assessment approach has been undertaken on the significant geotechnical hazards that may affect the site. The revised risk assessment has been attached for reference. Based on this assessment we consider that there may be a moderate risk for liquification induced ground damage due to the nature of the soils. Other issues include ground surface settlement due to the potential for compression of shallow, soft organic layers at Prestons Park. However, provided that the geotechnical recommendations, provided within the Prestons South Subdivision Consent Geotechnical Report (Aurecon, 2013) including amendments outlined in this letter are followed, and the appropriate engineering measures are implemented, then we consider that the development is unlikely to be affected by significant geotechnical hazards nor will the development worsen, accelerate or result in material damage. Therefore, from a geotechnical perspective we consider that the Prestons South Subdivision Stage 3 development will comply with RMA Section 106 (1).

Based on our review of the Prestons South Subdivision Consent Geotechnical Report (Aurecon, 2013), considering our past work on the wider subdivision development and subsequent detailed geotechnical investigations and assessment for the wider Prestons North and South Subdivisions, we consider our past geotechnical recommendations to be applicable for subdivision consent for Prestons Stage 3. We note changes to the RMA in 2017 and provide additional information on geohazards using a risk based assessment.

Yours faithfully,

Kieran Foote
Geotechnical Engineer

Approved by,

Dr Jan Kupec
Technical Director – Ground Engineering

Enc: Prestons South Subdivision Consent Geotechnical Report, Aurecon NZ Ltd., Ref. 235361, Rev 1 dated 6.6.2013
Figures 235361-GIS-23-C and 235361-DW-SU-PS-S3-SP-01[A] showing Sawmill site extent
Amendments to Table 13, Section 7 Geohazard Risk Assessment (RMA)

Project: Prestons South Subdivision

Resource Consent Geotechnical Report

Reference: 235361

Prepared for: CDL Land Ltd

Revision: 1

6 June 2013

Document Control Record

Document prepared by:

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1	6 June 2013	Issue to Client	J Muirson	J Muirson	J Kupec	T Browne
Current Revision		**1**				

Approval			
Author Signature		Approver Signature	
Name	James Muirson	Name	Tim Browne
Title	Engineering Geologist	Title	Technical Director

Contents

Appendices

1. Executive Summary

CDL Land Ltd. is proposing to develop a large residential subdivision with associated commercial lots. The site is located on an area of land between Prestons Road and Mairehau Road on the north east side of Christchurch. The subdivision is currently referred to as Prestons South. The Prestons South subdivision is part of the larger Prestons subdivision, which extends north to Lower Styx Road.

Aurecon NZ Ltd has previously carried out extensive geotechnical investigations and assessment for the purpose of the plan change, earthworks and subdivision resource consent for the entire Prestons Subdivision. Since the release of these reports the liquefaction methodology assessment has been defined by the Ministry of Business, Innovation and Employment (MBIE) in their guidelines dated December 2012. In addition, the subdivision layout has been adjusted. Therefore we have reviewed the existing geotechnical information against the latest MBIE liquefaction methodology to confirm the suitability of the land for residential development as part of the resource consent.

Our ground investigations indicated relatively consistent and predictable ground conditions in line with our past experience in the Prestons Road area. Based on the investigation results the site geomorphology has been defined into three distinct zones, which are detailed in the report and presented in the attached figures. The geological model has been defined predominantly by the upper surficial soils that will be affected by earthworks to form the residential subdivision. The soils at depth are relatively consistent across the entire site.

The geological model delineated by these areas is in general agreement with the published geological maps of the area and is reflected to a certain extent in the site geomorphology. We note that the site is very large and changes in geomorphology and geology were expected. The groundwater monitoring does identify that the groundwater levels vary across the site depending on the time of year and local elevation.

In addition to the geotechnical investigation a trial with the Landpac impact compactor was carried out to confirm the suitability of the method to improve the underlying ground conditions from a liquefaction potential aspect. The trials indicate that improvement in the q_c was achieved in the sandy layers but no improvement in the peat layers, which was expected. Similar results were gained on Prestons North.

A liquefaction assessment has been carried out using the CPT information. For the assessment we have reviewed three levels of seismic shaking.

1. Serviceability Limit State (SLS) design level earthquake, as defined by MBIE.
2. Intermediate design level earthquake of 1 in 150 year event.
3. Ultimate Limit State (ULS) design level earthquake, as defined by MBIE.

Based on the liquefaction assessment the liquefiable layers appear to be predominantly within the upper 3m to 4m of the soil profile, although some tests indicate thin liquefiable layers at depths of 7m and additional layers between 10m and 15m depth.

The liquefaction assessment identified that due to the potential for liquefiable soil layers in the upper 3m to 4m, there is a potential for lateral spreading of the soil adjacent to any new stormwater basins/channels. The lateral spreading values determined as part of the assessment are considered to be typical lateral displacements that may occur in a seismic event if no ground improvement is carried out.

Based on the liquefaction results only, parts of the site can currently be classified as Technical Category 1 (TC1) while other parts can be classified as Technical Category 2 (TC2). Although parts of the site can be classified as TC2, the extent of the liquefaction induced ground damage and settlements are at a level where we consider that suitable engineering options are available to potentially improve the ground to a TC1 equivalent performance level.

In terms of lateral spreading, the DBH guidelines indicate that no lateral spreading should occur for a site to be classified as TC1. The liquefiable layers appear to be predominantly within the upper 3m to 4m of the soil profile and therefore there is the potential for lateral spreading adjacent to the stormwater basins and channels if ground treatment is not undertaken. Hence, from a lateral spreading assessment perspective the site cannot be classified as TC1. However, as the depth of the liquefiable soil layers are limited to the upper soil profile, it is considered that there are suitable engineering options available that will minimise the potential for liquefaction induced lateral spreading.

Although the site can be classified from a liquefaction assessment as TC1 and TC2 the client's preference is to develop the Prestons South land into TC1 equivalent performance level.

The ground conditions and liquefaction potential at Prestons South is similar to that of Prestons North. The Prestons North earthworks have included the use of an impact compactor to densify the upper soil profile and hence minimise liquefaction. In addition, gravel embankments have been constructed to mitigate the lateral spreading potential susceptibility adjacent to stormwater basins/channels. We recommend that these mitigation measures are used on Prestons South.

A trial of the impact compactor was carried out on Prestons South and the results indicate that the liquefaction induced settlements can be reduced significantly with the use of the impact compactor. As part of the detailed design of the Prestons South subdivision, further geotechnical analysis will be required to confirm the extent of the required impact compaction. In addition, during the impact compaction extensive quality assurance testing with CPTs will be required to confirm that the required level of ground densification is being achieved. The results of the trial and extent of the quality assurance testing are detailed in this report.

The impact compactor does improve ground conditions and hence reduces the potential lateral spreading. The resulting lateral spreading displacements are still high to be TC1 but are within the deformation limits for TC2. Although the potential for lateral spreading is reduced there still remains a lateral spreading potential adjacent to the stormwater basins that cannot be addressed with the impact compactor alone, particularly if TC1 land is preferred. Therefore the construction of a wide gravel embankment founded below the liquefiable layer is to be considered. An assessment of gravel embankment option has been carried out and is detailed in this report. However, if the gravel embankment method is found to be not feasible then alternative options such as stone columns or vibrofloatation can be considered. As part of the detailed design of the Prestons South subdivision, geotechnical design will be required to confirm the gravel embankment design for each of the stormwater features or if required detailed design for an alternative mitigation options.

As part of the ground improvement and earthworks construction monitoring, quality assurance testing using CPTs will be carried out. The purpose of the testing is to determine if the ground improvement has achieved the required results and define the final technical category. The testing will include 90 pre-impact compactor CPTs, 90 post impact compaction CPTs, 75 post earthworks CPTs and 38 verification CPTs carried out one month after earthworks are completed.

Suitable foundation types for the various technical categories have been defined in the MBIE Guidelines (2012). For TC1 areas the MBIE Guidelines has recommended Standard NZS3604:2011 type foundations with tied slabs. In the unlikely case where residential sites cannot be improved to TC1 classification then TC2 type enhanced foundations will be required.

Recommendations are also provided for the infrastructure. Although we note that liquefaction induced ground damage under a SLS event is very unlikely. The benefits in building additional seismic resilience into the residential development infrastructure for large earthquake events are discussed in this report.

General comments regarding site earthworks are also provided. We note that areas of near surface peat are present across the site as well thin layers at depth. Where there will be residential buildings or subdivision infrastructure we recommend that the peat at ground level is removed as part of the subdivision development.

We have carried out an assessment of the thin layers of peat across the site. The levels of settlement calculated are unlikely to cause any significant issues to the earthworks and subdivision infrastructure. Primary consolidation of the peat is likely to occur within 4 to 6 weeks of the bulk earthfill placement, especially as the peat is interbedded with more permeable, free draining sandy soil. Secondary compression is likely to occur over a much longer time period, and is unlikely to be observable over the life time of the subdivision. Recommendations with regard to design for the presence of peat are provided.

In our opinion and based on our assessment, we consider that under Section 106 of the RMA there are no geotechnical reasons preventing the development, provided the appropriate engineering measures as recommended in this report are carried out.

Our limitations are at the end of this report and this report shall be read as a whole.

2. Introduction

CDL Land Ltd. is proposing to develop a large residential subdivision with associated commercial lots. The site is located on an area of land between Prestons Road and Mairehau Road on the north east side of Christchurch. The subdivision is current referred to as Prestons South. The Prestons South subdivision is part of the larger Prestons subdivision, which extends north to Lower Styx Road. The greater Prestons Subdivision is approximately 150ha, whilst Prestons South part is approximately 75ha.

Aurecon NZ Ltd has previously carried out a geotechnical investigation and assessment for the purpose of the earthworks and subdivision resource consent for the entire Prestons Subdivision. The resource consent geotechnical report, dated 5 March 2012, identified the liquefaction and lateral spreading risk associated with the site and defined the technical classification of the entire subdivision area. Further to this reporting, a detailed design geotechnical report, date 12 July 2012, was prepared for the entire Prestons Subdivision area. This report provided additional geotechnical information and results, defined the technical classification further and provided indicative ground improvement options. These ground improvement options have since successfully been used in the Prestons North bulk earthworks.

Since the release of these reports the liquefaction methodology assessment has been refined by the Ministry of Business, Innovation and Employment (MBIE) in their guidelines dated December 2012. In addition, the subdivision layout has been adjusted. Therefore we have reviewed the existing geotechnical information against the latest MBIE liquefaction methodology to confirm the suitability of the land for residential development.

The scope of work included the following geotechnical investigations:

- Review of existing geotechnical and geological information on the site,
- Cone Penetrometer Tests (CPT) across the site to provide information on the soil at depth and data to allow a liquefaction assessment to be undertaken.
- Window sampling of the soils at depth to confirm the soil type and calibrate the CPT logs.
- Machine excavated test pits to determine that nature of the shallow soil profile and obtain soil samples for laboratory testing.
- A review of ongoing groundwater measurements from the installed standpipe piezometers.
- Laboratory testing of soil samples.
- Carry out and monitor impact compactor trials.

The assessment of the geotechnical investigation results included:

- Liquefaction analysis using latest MBIE Guidelines to identify the liquefaction potential of the underlying natural soils and confirm the technical categories across the site based on the liquefaction assessment.
- Assessment of the impact compactor results.
- Provide indicative engineering measures required on the site to address liquefaction and lateral spreading potential.
- Preparation of a geotechnical report to present the above information.

This geotechnical report presents the results of our geotechnical investigations and assessment, confirms the suitability of the land for residential development as well providing recommendations for development of the site.

Our limitations are attached as Section 8 of this report. This report shall be read as a whole.

3. Site Conditions

3.1 Site Description

The Prestons Road subdivision is located on the north eastern fringes of Christchurch City. The site is made up of a series of adjacent properties forming an irregular and elongated rectangle shape, orientated approximately north to south. The total area of the overall Prestons Subdivision site is approximately 150ha of which Prestons South is approximately 75ha. Prestons South extends from Prestons Road, through to Mairehau Road to the south, as shown in Figure 1 in Appendix A.

The main features of the overall subdivision site are as follows:

- Topography of the site ranges from flat through to gently undulating, with localised high points generally consisting of paleo sand dunes.
- The bulk of the southern block is divided up into a series of fields with windbreaks in the form of mature trees around the edges. The north eastern part of this block is currently used as a market garden, growing commercial quantities of vegetables. A sawmill is located on the southern boundary and accessed off Mairehau Road.

3.2 Access

The Prestons South site can be accessed from Prestons Road and Mairehau Road. Internal access through the site is via sealed and unsealed farm tracks.

3.3 Surface Water

No natural surface drainage channels remain on the site and the existing drainage is highly modified on the overall site. There are localised drainage ditches throughout the site. The majority of these ditches appear to be shallow. During the initial investigations a number of the drainage ditches were filled with water and did not appear to be draining as these appeared to be in a state of disrepair. A water table is present in these drains and may reflect the underlying groundwater level. In the low lying areas, surface ponding of water was noted at the time of the investigations, particularly during the wetter months of the year.

3.4 Regional Geology

The geology of the site is described in the 1:100,000 scale geological map – 'Geology of the Christchurch Urban Area" published in 1992 by the Institute of Geological and Nuclear Sciences. Note this map has been referenced as it is at an appropriate scale and covers the entire site. The geological map indicates several different material types. The map indicates the following underlying geology:

- *"Dominantly sand of fixed and semi-fixed dunes and beach deposits"* are present over the majority of the Prestons South site.
- *"Peat swamps now drained"* are present in the north east and south east corners as well as in the eastern block of Prestons South.

The GNS Active Fault System database (GNS, 2012a) indicates that the site is located approximately:

- 27km north east of the eastern end of the Greendale Fault System. Movement on the Greendale Fault System was responsible for the Magnitude 7.1 Darfield (Canterbury) Earthquake on 4 September 2010.
- 14km north of the epicentre of the Magnitude 6.2 Christchurch Earthquake on 22 February 2011.

- 12km north west of the Magnitude 6.0 earthquake on 13 June 2011.
- 8km north west of the Magnitude 5.9 earthquake on 23 December 2012.

3.5 Previous Work

Aurecon has been involved in the geotechnical assessment for the Prestons Subdivision since 2005. Aurecon Ltd has been engaged on this project from 2005 onwards in a continuous manner. Previous documentation which has been reviewed as part of this geotechnical assessment includes the following:

- *"Prestons Road Subdivision, Detailed Geotechnical Design Report"*, dated 12 July 2012
- "Geotechnical Assessment Report for Resource Consent", dated 5 March 2012, which included delineation of the technical categories across the site.
- *"Geotechnical Assessment Report for Earthworks Consent"*, dated 28 November 2011.
- *"Supplementary Evidence (Post 22 February 2011)"*, dated March 2011, provided as part of Plan Change 30.
- *"Prestons Road Rezoning Liquefaction Reassessment"*, dated October 2010, which reviewed the liquefaction risk to the site following the Darfield Earthquake.
- *"Prestons Road Rezoning Geotechnical Investigation Report"*, dated August 2008, which included intrusive investigations and a geotechnical assessment on the suitability of the area for development.
- *"Stage 1 Environmental Assessment Report, Prestons Road Development Area, Christchurch"*, dated August 2008.
- Aerial photographs dating back to 1955, used as part of the environmental study.

4. Geotechnical Investigation

4.1 Introduction

The objective of the geotechnical investigation was to determine the nature and composition of the underlying ground conditions and identify the relevant geotechnical issues. The investigation for Prestons South was carried out as part of the investigation for the larger Prestons subdivision.

The geotechnical investigation for Prestons South comprised the following:

- Undertake a site walkover to identify the geomorphological features of the site.
- Review previous investigation results.
- Carry out 75 CPT's across the site to confirm ground condition at depth and provide information for liquefaction assessment.
- Machine excavated 38 test pits to determine that nature of the shallow soil profile and obtain soil samples for laboratory testing.
- Window sampling of the soils at depth to confirm the soil type and calibrate the CPT logs.
- Monitoring of six standpipe piezometers to determine changes in groundwater levels across the site over time.
- Laboratory testing including compaction tests and particle size distributions.
- Review Environment Canterbury (ECan) borehole logs on or near the site.

Our ground investigation indicated a relatively consistent and predictable geology in line with our past geotechnical investigation experience in the Prestons Road area.

A detailed description of the geotechnical investigations and the results are provided in the following sections.

4.2 Site Walk Over

A site walk over was carried out by an experienced Engineering Geologist on 27 to 28 July 2011 and again on the 29 August 2011. The purpose of the site walk over was to identify geomorphological features of the site and to identify any ground damage resulting from the recent seismic activity.

As noted above, Aurecon Ltd. has been involved in the geotechnical assessment for the Prestons Subdivision since 2005. Numerous site inspections have been carried out by Senior Geotechnical Engineers prior to the recent investigation. These include a site walk over by a Senior Geotechnical Engineer following the 4 September 2010 Darfield Earthquake, the 22 February 2011 Christchurch Earthquake, the 15 June 2011 Earthquakes and 23 December 2011 Earthquake.

4.2.1 Site Geomorphology

Based on our site walk over and review of the general landscape, the site can be defined into three distinct geomorphological zones. The zones are shown in Figure 2 in Appendix A and are as follows:

Zone 1 – High Dune Areas

The high dune areas comprise the sand ridges and upper terrace. The sand ridges are the distinct, topographically high sand dunes which are elevated above the surrounding topography. The crests of the ridges are about 3m to 4m above the surrounding topography and have moderately steep sides. These are located in the south west corner of the site, adjacent to Mairehau Road.

Zone 2 – Low Lying Areas

The low lying areas comprise the parts of the site which are relatively flat and do not have a distinctive undulating ground surface. Within the Prestons South these areas are located on the eastern side and a small area in the south west corner. These areas were typically wet and soft underfoot with ponded water present in localised depressions, the latter following rainfall.

Zone 3 – Low Dune Areas

The low dune areas encompass the remainder of the site not covered by the previously described zones. The low dunes typically comprise a gently undulating ground surface of lower topographical height, relative to Zone 1. For the most part these landscape features appear to be undisturbed, however there are parts, in particular the northern side of Prestons South around the market garden, where it appears the dunes have been earthworked to form flat farm pastures. In addition, it appears that some historic land disturbance, such as excavation of the sands dunes has occurred in the south east corner of the site, adjacent to Mairehau Road.

4.2.2 Ground Damage

Based on a number of site walkovers we note the following:

- Based on our site walkover carried out following the earthquake events as well as sizeable aftershocks, no liquefaction or lateral spreading was observed on the site.
- Evidence of liquefaction surface ejecta (i.e. sand boils) was not apparent during the site walkover carried out as part of this investigation. Even though the site walkover was carried out shortly after the seismic events, there was no apparent residual evidence, such as degraded sand boils or distinct mounds that would indicate sand boils had been present, nor was there any evidence of accumulation of sand and silt brought up by liquefaction within the drainage ditches.
- Other evidence of ground damage such as ground cracking or lateral spreading adjacent to the drainage ditches was not apparent on the site.
- A review of high resolution aerial photographs from the Canterbury Geotechnical Database did not identify any apparent surface manifestation of liquefaction on the site.

4.3 Cone Penetrometer Tests

4.3.1 CPT Investigations

Cone penetrometer tests (CPT) across Prestons South have been carried out in a number of stages as part of the larger Prestons Subdivision. The details of the CPT within Prestons South are as follows:

- Three CPTs were carried out by McMillan Drilling Services using a truck mounted CPT rig and the investigations were carried out in April 2007. The test locations (labelled CPT6 (2007)) are shown on Figure 3 in Appendix A and the CPT logs are attached in Appendix B.
- 30 CPTs were carried out by McMillan Drilling Services, using a track mounted CPT rig. The CPT testing was carried out from the 10 to 18 August 2011. The test locations (labelled CPT1 to CPT30) are shown on Figure 3 in Appendix A and the CPT logs are attached in Appendix C.
- 42 were carried out by McMillan Drilling Services, using both a truck and track mounted CPT rig. The truck mounted CPT testing was carried out from the 26 January to 9 February 2012, with the track mounted CPT testing carried out on the 1 March 2012. The

test locations (labelled CPT101 to CPT141 and CPT179) are shown on Figure 3 in Appendix A and the CPT logs are attached in Appendix D.

Extensive CPT testing has been carried out as part of the earthworks for Preston North, to the north of the site. In addition Aurecon has been involved in the Alpine View Retirement Village development to the east, which included CPT testing. At this stage this information has not been included with this report due to client confidentiality. However it has been used in our assessment to provide an understanding of the ground conditions immediately outside of Prestons South and if required, this additional testing information will be provided via the Canterbury Geotechnical Database (CGD).

The majority of the tests had a target depth of 15m and were extended until refusal was met (defined as sustained tip resistance of over 40MPa) or the target depth was reached. Of the CPT tests carried out, seven were extended past 15m in order to confirm the ground conditions at depth.

4.4 Test Pit Excavations

Test pits were excavated across the site from 27 July 2011 to 5 August 2011, with additional test pitting on the 25 to 26 August 2011. For the majority of the test pits, a 20t excavator supplied by Texco was used along with a 5t excavator supplied by Reardon Contracting Ltd. The purpose of the test pits was to identify the upper soil profile, groundwater levels, allow the installation of standpipe piezometers and calibrate the CPT information against hand samples. In total 38 test pits were carried out across Prestons South.

The test pits were excavated to depths ranging from 2.8m to 4.7m below ground level. Test pit collapses occurred at shallow depths due to the presence of sandy soils and infiltrating groundwater.

The test pits were logged by an Engineering Geologist in accordance with the New Zealand Geotechnical Society's "Guide for the Field Classification and Description of Soil and Rock for Engineering Purposes". The test pits were backfilled with the excavated soil.

The locations of the test pits are shown on Figure 3 in Appendix A and the logs are attached in Appendix E.

4.5 Window Sampler

The McMillans Drilling CPT rig used has the capability to carry out window sampling. Window sampling was taken alongside the CPT probe hole to allow calibration of the CPT log with soil type, particularly at depth. Window samples were taken at various depths between 3.6m to 10m, from three CPT locations. Soils were logged by an Engineering Geologist from Aurecon. Window sampler logs are presented in Appendix F.

The soil obtained from the window samplers predominantly consists of fine to medium grained sand, with minor gravels. The soils logged are in reasonable agreement with the adjacent CPT logs.

4.6 Prestons South Geological Model

The following geological model has been derived based on the desk study and intrusive investigation described in this report. The site geological model is defined predominantly by the upper surficial soils, as the soils at depth are relatively consistent across the site. Based on the surficial soil profile, the geological model for each of the blocks is discussed below. The extent of surficial geology for the overall subdivision described below is presented in Figure 4 in Appendix A.

4.6.1 Geological Areas

Area 1 - Main Area

Depth to Top of Unit (m)	Depth to Base of Unit (m)	Soil Unit
0	0.2 to 0.75	TOPSOIL.
0.2 to 0.75	3	SAND, loose to medium dense, with silty PEAT layers up to 0.3m thick within the upper 3m.
3	15+	SAND, medium dense to dense, becoming very dense with depth. Trace PEAT and SILT layers at depths of 10m+.

Within the market garden area on the north side of Prestons South, localised areas of peat were noted, in particular CPT24. These areas appeared to be limited in extent and where encountered, extended to a depth no greater than 0.7m below existing ground level.

In addition, thicker layers of peat in the order of 0.5m to 0.6m were identified in TP23, CPT119 and CPT120 in the order of 1m to 1.5m depth. These areas appeared to be limited in extent, where encountered.

Area 2 - East Side

Depth to Top of Unit (m)	Depth to Base of Unit (m)	Soil Unit
0	0.2 to 0.5	TOPSOIL
0.2 to 0.5	0.5 to 1.3	SILT non plastic and hard.
0.5 to 1.3	3	SAND loose to medium dense, with silty PEAT layers up to 0.3m thick within the upper 3m.
3	15	SAND medium dense to dense, becoming very dense with depth. Trace PEAT and SILT layers at depths of 10m+.

Area 3 - South West Corner

Depth to Top of Unit (m)	Depth to Base of Unit (m)	Soil Unit
0	0.5	TOPSOIL
0.5	0.8	SILT low plasticity and firm.
0.8	3	SAND medium dense to dense.

The geological model delineated by these areas is in general agreement with the geological maps of the area and is reflected to a certain extent in the site geomorphology.

4.6.2 Peat

Using the original and recent geotechnical testing, the extent of peat (i.e. depth and thickness) has been defined across the site and presented in Figures 5 and 6. We note that with the exception of localised thicker pockets (i.e. greater than 0.5m thick) across the site, all other surficial peat is not represented in these figures as it will most likely be removed as part of the topsoil stripping prior to earthworks. The peat layers typically are less than 2m deep.

Peat thickness across the site was typically less than 0.3m but thicker layers were identified in localised pockets across the site, ranging from near surface to 2m depth. These localised pockets are presented on Figure 6.

4.7 Standpipe Piezometers

During the initial geotechnical investigation, standpipe piezometers were installed in Test Pits TP2, TP9, TP11, TP19, TP22 and TP32. Since installation of the piezometers TP2 and TP32 has been destroyed.

The piezometers consisted of a 65mm diameter PVC pipe with perforated slots in the bottom metre. The perforated pipe was wrapped in geosynthetic cloth and placed within selected test pits to depths ranging from 2m to 3m below ground level. The location of the standpipe piezometers is shown on Figure 3 in Appendix A.

Piezometers have been monitored at regular intervals since early August 2011. The results of the groundwater monitoring are attached in Appendix G.

The groundwater monitoring graphs indicate that there is a general increase in groundwater levels within the winter months, which then drop off during the summer months. It is possible that the initial peak measured in August 2011 reflects stabilisation of the groundwater level in the standpipe or is possibly due to the snow melt from the heavy snowfall that Christchurch experienced on 15 to 16 August 2011. Such groundwater levels will need to be borne in mind for the design of the site stormwater system and critical infrastructure.

We have reviewed these groundwater levels alongside the event specific groundwater information provided in the Canterbury Geotechnical Database (CGD). The median groundwater levels present in the CGD are reasonably consistent with the groundwater levels measured on site.

4.8 Laboratory Testing

Laboratory testing was carried out as part of the geotechnical investigation and the purpose of the testing was as follows:

- Particle size distribution on sand samples from the window samplers to confirm the particle size distribution and fines content.
- Standard compaction tests and minimum dry density tests on the bulk samples from the test pits to provide indicative compaction curves for the earthwork.

Central Testing Services Ltd. based in Alexandra, carried out particle size distribution and standard compaction tests.

The results of the laboratory testing relating to Prestons South are attached in Appendix H and summarised in the following tables.

Table 1 - Standard compaction test results

Test Pit	Depth	Natural Water Content	Optimum Moisture Content	Maximum Dry Density	Minimum Dry Density	Soil Type
TP6	1m	4.9%	17%	1.59t/m^3	N/A	SAND
TP12	1.5m	21.4%	19%	1.58t/m^3	N/A	SAND
TP22	1m	24.5%	19%	1.58t/m^3	N/A	SAND
	Average		18.5%	1.58t/m^3		

The standard compaction curves show a distinct peak at the maximum dry density with the curve falling off relatively sharply with increased moisture content but gently sloping with a decrease in moisture content. The compaction curves indicate that the sand will be sensitive to moisture contents in excess of the optimum moisture content.

In Test Pits TP6 natural water contents were significantly different to the other tests. These samples were taken from test pits on the high sand ridges and hence the samples are likely to have been taken from the soil profile above the groundwater level and the capillary rise zone.

Table 2 - Particle size analysis test results

Test Pit	Depth	Percentage Less than 0.06mm	Percentage Less than 0.2mm	Percentage Less than 0.6mm	Percentage Less than 2mm	Soil Type Based on Grading
TP10	1m	3%	91%	100%	100%	SAND
TP19	2m	4%	91%	100%	100%	SAND
TP35	1.5m	2%	88%	100%	100%	SAND
CPT127	8.5 to 9.3m	8%	88%	92%	97%	SAND with trace gravel
CPT140	4.7 to 5.9m	4%	99%	100%	100%	SAND

Particle size analysis results indicate that the sand at near surface and at depth is predominantly medium to fine grained.

4.9 Environment Canterbury Borehole Logs

A review of the Environment Canterbury (ECan) GIS website indicates a number of boreholes are present within the area. The ECan boreholes are shown in Figure 7 in Appendix A and the logs are in Appendix I. The borehole information is summarised in the Table 3.

Table 3 - Summary of ECan Borehole Logs

Hole Reference	Distance from Site	Depth of Borehole (m bgl)	Summary
M35/1575	50m to the south	81.3	Blue and brown sand to 29.6m underlain by clay to 31.4m. Further underlain by gravel with sand and minor peat/clay layers to 81.3m. Groundwater was artesian due to depth of borehole.
M35/1608	On site along southern boundary	64.3	Sand and clay overlying underlain by gravel with sand and minor peat/clay layers to 64.3m. Groundwater was artesian due to depth of borehole.
M35/4577	On site along northern boundary	40.4	Soil and peat to 2.09m, overlying fine sand to 26.2m. Further underlain by pug to 27.6m and peat to 27.9m, then underlain by gravel with minor clay layers to 40.4m. Groundwater at 0.9m bgl.
M35/5690	50m to the south	36	Sand to 31.5m with a layer of wood at 26m. gravel from 31.5m to 36m. Groundwater at 0.6m bgl.
M35/6362	50 to north west	30.5	Clay to 2m, underlain by gravel and sand to 5m and sand to 26m. Further underlain by peat to 27m and gravel to 30.5m. Groundwater not recorded.
M35/8069	100m west	35	Sand to 4m and sand with gravel to 31m. Underlain by gravel to 35m. Groundwater at 0.6m bgl.
M35/10124	30m west	35.9	Sandy topsoil to 1.8m underlain by sand to 27.5m. Further underlain by sand with peat to 27.8m and gravel with minor clay layers to 35.9m. Groundwater at 0.7m bgl.
M35/11720	On site within northern part	10	Peaty topsoil to 0.5m overlying sand to 10m.
M35/11720	On site at north east corner	10	Peaty topsoil to 0.5m overlying sand with trace gravel to 8m and gravel to 10m.

The ECan borehole information is consistent with the soil profile defined by our geotechnical investigation information.

5. Impact Compactor Trial

5.1 Introduction

A trial on the Landpac impact compactor was carried out at the greater Preston Subdivision to confirm the suitability of the method to improve the underlying ground conditions from a liquefaction potential aspect. A number of trial areas were carried out over the greater Prestons Subdivision area, of which one was located in Prestons South.

The main issues identified in the previous geotechnical reporting were as follows:

- The liquefaction assessment identified the site as Technical Category TC1/TC2 equivalent. Although foundation options are available for both technical categories there is benefit in improving the ground to TC1.
- The site will require stormwater basins/channels that will range in depth from 2m to 3m. Our liquefaction assessment work identified that liquefiable soils are present in the upper 4m of the site and hence coupled with the adjacent basins/channels, lateral spreading of the ground adjacent to the basin/channel could occur.

As the liquefaction potential is typically governed by the liquefiable soils in the upper 3m to 4m, the impact compactor was trialled to identify it as a potential option to use for ground improvement.

5.2 Trial Compaction

The trial compaction was carried out using a three sided dual drum impact compactor from Landpac. Of the number of trial areas carried out over the greater Prestons Subdivision area, one was located in Prestons South (Site 4). The test area is present in Figure 8 in Appendix A. The trial area was chosen as it was representative of the greater site ground conditions.

As part of the assessment of the trial, CPTs were carried out within the trial area. The CPTs were limited to 6m depth, as it was understood from Landpac that the depth of influence of the impact compactor was unlikely to be greater than 3m. This was confirmed in the compliance testing regime at Prestosn North. Three CPTs were carried out in the trial area, with one located at the centre of the trial and two either side at approximately quarter points. The CPT testing carried out pre and post trials were taken at approximately the same location to allow the pre and post-trial results to be compared.

The sequence of impact compactor passes and testing carried out as part of the trial are presented in Table 4.

Table 4 - Sequence of trial compaction and CPT testing

Stages	Site 4
1) Pre-Trial CPT	CPT214 to CPT216
2) Compaction	40 passes
3) CPT Testing	CPT414 to CPT416

5.3 Trial Compaction Test Results

Pre and post-trial CPT logs are presented in Appendix J and a comparison of the CPT results, based on the cone resistance (q_c) is present in Appendix K.

Based on these results we can make the following comments:

Site 4

- Strength and density improvement in the upper sand layer.
- No improvement in the thin peat layer.
- Strength and density improvement in sand layer below the peat.

Overall the compaction trail concluded that granular soil layers can be effectively improved. This was observed on the Prestons North site. Key findings were that there is no significant improvement of the layer of sandy soil either side of the silt/peat layers, where it is present in the soil profile. Thicker layer of topsoil or compressible soil may limit the degree of improvement within the upper soil horizon. A detailed assessment of the results is provided in Section 6.5.

6. Engineering Considerations

6.1 Introduction

CDL Land Ltd is proposing to develop the Prestons South subdivision, located on Prestons Road, Christchurch. The subdivision will be predominantly residential but there will be commercial area allotted for shops. The site earthworks will involve cutting of the existing high dunes across the site and the placement of engineered fill over the majority of the site. In addition, the site development will incorporate a stormwater retention basin and channels running through the center of the site.

Based on the ground conditions encountered during the geotechnical investigation we consider that the following geotechnical aspects need to be considered as part of the subdivision:

- Potential for seismically induced liquefaction.
- Recommendations for liquefaction mitigation measures, including assessment of the impact compactor.
- Implications for building foundations.
- Recommendations for infrastructure construction.
- Identify the presence of the peat and the affect it may have on the residential buildings or infrastructure.
- Provide recommendations with regard to site earthworks.
- Assessment against Resource Management Act (RMA) Section 106 a) to c).

Each of these is discussed in detail in the following sections along with recommendations for engineering mitigating measures.

6.2 Liquefaction Assessment

6.2.1 Liquefaction Potential Assessment

The three primary factors that contribute to liquefaction potential are:

- Loose, uniformly graded soils.
- High groundwater table.
- Sufficiently high, earthquake induced ground acceleration and sustained shaking.

Each of these is considered below together with conclusions on the site liquefaction potential.

Soil Grading and Density

Liquefiable soils generally have a Coefficient of Uniformity of less than 5 and a low proportion of soil finer than 75 microns in size (typically less than 5% to 10%, but up to 30%). Based on the laboratory testing that was carried out on the typical soil encountered at the site the soil is predominantly sand with typically less than 5% fines content, as seen in Table 2. Therefore from a grading and density perspective some soil layers may be potentially liquefiable.

Groundwater

Groundwater levels within Prestons South range from 0.4m to 1.8m below existing ground level. Soils are therefore potentially liquefiable from depths of 0.4m to 1.8m below ground level, depending on the location within the southern block.

Groundwater levels used in our analysis are based on standpipe piezometer measurements from late winter to spring and are likely to be relatively high and hence are considered to be conservative. Groundwater levels will vary depending on the time of year.

Earthquake Intensity and Soil Resistance to Liquefaction

Earthquake induced ground acceleration and sustained shaking, leading to sufficient load cycles, is a requirement and a potential trigger of liquefaction. For the assessment we have reviewed three levels of seismic shaking.

1. Serviceability Limit State (SLS) design level earthquake, as defined by MBIE.
2. Intermediate design level earthquake.
3. Ultimate Limit State (ULS) design level earthquake, as defined by MBIE.

Each of these earthquake cases is discussed in detail below:

Serviceability Limit State (SLS) Earthquake

From the MBIE Guidelines, we have used a Peak Ground Acceleration (PGA) of 0.13g for a SLS event with a Magnitude 7.5 earthquake.

Intermediate Level (Int) Earthquake

Subdivision Consent Condition RMA92019798, which covers Stage 1 of Prestons South, indicates that liquefaction mitigation measures for the subdivision infrastructure shall be designed for a 1 in 150 year return period, as defined by NZS1170.5:2004.

Based on NZS1170.5:2004 for an Importance Level 2 (IL2) structure, with an increased Z hazard factor of 0.3, we have derived a PGA of 0.2g for a 1 in 150 year return period. A Magnitude 7.5 has been assumed.

Ultimate Limit State (ULS) Earthquake

The MBIE guidelines (2012) recommend a PGA of 0.35g for residential buildings in Christchurch. We have adopted this PGA value with a Magnitude 7.5 earthquake for our ULS assessment.

6.2.2 Liquefaction Methodology

In assessing the liquefaction potential, two methods have been used to assess the potential settlement for each of the three design level events. Previous experience indicates that some methods can over predict liquefaction induced settlements and the use of two settlement prediction methods will give us a range of results. The two settlement prediction methods are discussed below.

Idriss and Boulanger Method

The liquefaction assessment was carried out using the method developed by Idriss and Boulanger (2008), in accordance with the MBIE Guidelines (2012) for residential properties. The assessment was carried out using an excel spread sheet developed by Aurecon.

The method of Robertson and Wride (1998) with the modified fine content was used to assess the liquefaction potential from the CPT results. The method of Zhang et al (2002) was used for estimating the liquefaction induced settlements from CPT results. The method of Ishihara and Yoshimine (1992) was used for estimating the liquefaction induced settlements from CPT results.

NCEER Method

The liquefaction assessment was carried out using the National Centre for Earthquake Engineering Research (NCEER) method as outlined by Youd et al. (2001), and recommended in the NZGS (2010) guidelines. The assessment was carried out using Version 5 of the CivilTech Corporation LiquefyPro computer programme.

The method of Robertson and Wride (1998) with the modified fine content was used to assess the liquefaction potential from the CPT results. The method of Ishihara and Yoshimine (1992) has been used for calculating potential liquefaction induced settlements for CPT results.

Based on recent experience in the Canterbury Region this method appears to give consistent results with liquefaction damage observed elsewhere in the region and based on the back analysis carried out as part of our geotechnical investigation report, dated March 2012, appears to be relatively consistent with the observed (or lack of observed) damage at the site.

The liquefaction assessment has been carried out on the full CPT profile as well as the upper 10m of the soil profile, as the MBIE Guidelines indicates that for technical classification of sites the upper 10m needs to be assessed.

6.2.3 Liquefaction Analysis Results

A summary of the calculated potential liquefaction induced settlements for the overall subdivision area is presented in Table 5 and 6. Detailed results are presented in Appendix L.

Table 5 – Summary of calculated settlements for upper 10m of the soil profile

	Liquefaction Induced Settlements (in mm)					
	I&B Method (10m)			NCEER Method (10m)		
	SLS	INT	ULS	SLS	INT	ULS
Maximum	15	45	65	20	30	40
Minimum	0	0	0	0	0	0
Average	<5	10	25	<5	<5	10

Note: The settlements presented above are to the nearest 5mm. There are inherent assumptions in the analysis methods used that may cause the actual site settlements to vary from those calculated.

Table 6 – Summary of calculated settlements for full depth of CPT the CPT test

	Liquefaction Induced Settlements (in mm)					
	I&B Method (Full Depth)			NCEER Method (Full Depth)		
	SLS	INT	ULS	SLS	INT	ULS
Maximum	30	65	165	20	35	70
Minimum	0	0	0	0	0	0
Average	5	25	55	<5	10	25

Note: The settlements presented above are to the nearest 5mm. There are inherent assumptions in the analysis methods used that may cause the actual site settlements to vary from those calculated.

The liquefiable layers appear to be predominantly within the upper 3m to 4m of the soil profile, although some tests indicate thin liquefiable layers at depths of 7m and additional layers between 10m and 15m depth. For the ULS earthquake events (i.e. worst case scenarios), the thickness of the liquefiable layers in the upper 3m to 4m range up to 1m but are typically 0.6m thick or less.

6.2.4 Ground Damage

Published information (after Ishihara, 1985) can be used to assess the potential for surface expression of liquefaction and hence the likelihood of ground induced damage. Our assessment of liquefaction induced ground damage is based on the liquefaction assessment plots for the Idriss and Boulanger Method. The results are present in Appendix L and summarised below in Table 7.

Table 7 – Number and percentage of tests indicating potential for liquefaction induced ground damage at the site

Induced Ground Damage	SLS & INT EQ	ULS EQ
Yes	11 (15%)	38 (51%)
No	64 (85%)	37 (49%)

6.2.5 Lateral Spreading

Flow failures caused by seismically induced liquefaction can occur when the shear stress required for static equilibrium of a soil mass is greater than the shear strength of the soil in its liquefied state (Kramer, 1996). Lateral spreading can occur where there is a continuous liquefiable layer through to the free face, such as a stream or river back. Lateral spreading can also occur where the ground slopes at greater than 1:100.

Lateral spreading damage was not observed at the site during the Canterbury Earthquake sequence. We do note that the ground shaking the site has experienced from the recent earthquakes is likely to be less than a ULS design earthquake event. As the liquefiable layers appear to be predominantly within the upper 3m to 4m of the soil profile, there is the potential for lateral spreading adjacent to the stormwater retention basins and channels if land is left untreated. However, engineering works such as filling and compaction will alter the susceptibility.

To assess the lateral spreading risk we have carried out an assessment based on the estimated shear strain potential as detailed in Idriss and Boulanger (2008). From previous experience around Christchurch we consider that this method does provide reasonably accurate results and gives an estimate of the lateral displacement towards a free-face edge. We have carried out our assessment based on the following parameters:

- An indicative stormwater basin profile with a free face slope of 4H:1V and height of 3m.
- The analysis is based on the three design level event detailed in Section 6.2.1.
- The lateral displacements have been calculated based on a number of the CPT tests located adjacent to the channels and basins. This assessment infers that the soil profile in the CPT is continuous to the free face.

The potential lateral displacement has been calculated for various setbacks from the crest of the basins/channels and the results of the analysis are present in Table 8. We note that the potential displacements are based on the existing ground conditions and are likely to improve once engineering works are complete.

Table 8 – Potential lateral spreading displacements

	Lateral Displacement (mm)								
	5m Setback			10m Setback			20m Setback		
CPT No.	**SLS**	**INT**	**ULS**	**SLS**	**INT**	**ULS**	**SLS**	**INT**	**ULS**
CPT15	75	310	550	60	250	455	45	185	330
CPT19	5	30	150	5	25	125	5	15	90
CPT106	25	235	290	20	190	240	15	140	175
CPT117	0	5	25	0	5	20	0	0	15
CPT121	80	155	180	65	125	145	45	95	110
CPT122	0	10	40	0	5	30	0	5	25
CPT123	0	5	30	0	5	25	0	0	20
CPT124	0	10	45	0	5	35	0	5	25
CPT129	5	55	80	5	45	65	5	30	50
CPT134	5	25	125	5	20	100	0	15	75
CPT137	5	40	190	5	35	155	5	25	115
CPT139	0	10	45	0	10	35	0	5	30

Note: The displacements presented above are to the nearest 5mm.

6.2.6 Land Classification Technical Categories

For the Christchurch Region, MBIE has released a classification system for residential ‘Green Zone’ land on the flat in regard to the liquefaction susceptibility. The classification system is divided into three technical categories that reflect both the liquefaction experienced to date and future land performance expectations. The categories and corresponding criteria are summarised as follows:

- **Technical Category 1 (TC1)** – future land damage from liquefaction is unlikely, and ground settlements are expected to be within normally accepted tolerances.
- **Technical Category 2 (TC2)** – Minor to moderate land damage from liquefaction is possible in future large earthquakes.
- **Technical Category 3 (TC3)** – Moderate to significant land damage from liquefaction is possible in future large earthquakes.

The MBIE has indicated the following liquefaction deformation limits for house foundations as summarised in Table 9.

Table 9 – Liquefaction deformation limits and house foundation requirements

Technical Category	Liquefaction Deformation Limits				Likely Implication for House Foundations (subject to individual assessment)
	Vertical		Lateral Spread		
	SLS	ULS	SLS	ULS	
TC1	15mm	25mm	Nil	Nil	Standard NZS3604 type foundations with tied slabs
TC2	50mm	100mm	50mm	100mm	MBIE enhanced foundation solutions
TC3	>50mm	>100mm	>50mm	>100mm	MBIE TC3 specific foundations

Based upon the results of the liquefaction assessment only, parts of the site can currently be classified as Technical Category 1 (TC1) while other parts can be classified as Technical Category 2 (TC2). These areas are shown on Figure 9 in Appendix A.

Although parts of the site can be classified as TC2, the extent of the liquefaction induced ground damage and settlements are at a level where we consider that suitable engineering options are available to potentially improve the ground to a TC1 level. These potential mitigation measures are identified in the following sections, and have been successfully used on the Prestons North subdivision.

In terms of lateral spreading, the MBIE Guidelines indicate that no lateral spreading should occur for a site to be classified as TC1. The liquefiable layers appear to be predominantly within the upper 3m to 4m of the soil profile and therefore there is the potential for lateral spreading adjacent to the stormwater basins and channels if ground improvement is not undertaken. Hence, from a lateral spreading assessment perspective the site cannot be classified as TC1. However, as the depth of the liquefiable soil layers are limited to the upper soil profile, it is considered that there are suitable engineering options available that will minimise the potential for liquefaction induced lateral spreading. These potential mitigation measures are identified in the following sections and these have been employed at the Prestons North Subdivision.

6.3 Compliance with the Definition of 'Good Ground'

Based on the review of the results of the geotechnical site investigations to date, it is inferred based on considerations of soil strength and compressibility, that the site is non-compliant with the intent of the definition of 'Good Ground' in terms of the New Zealand Standards Timber Framed Buildings (NZS3604:2011) and Concrete Masonry Buildings Not Requiring Specific Engineering Design (NZS4229:1999).

Therefore, irrespective of any potential liquefaction risk at the site, typical light weight timber framed or masonry houses (which would generally be designed within the guidelines of NZS3604:2011 or NZS4229:1999) would either require specific foundation design or the land improved. We note that civil engineering works are commonly undertaken to improve the shallow ground bearing capacity and should be considered as part of the detailed civil engineering design and earthworks construction.

6.4 Engineering Mitigation Measures

The liquefaction assessment indicates that based on the potential liquefaction induced vertical deformations, Prestons South can be classified as TC1 and TC2, with a potential lateral spreading adjacent to the basins/channels.

6.4.1 Liquefaction

Current MBIE Guidelines indicate that there are various foundation solutions available for constructing on TC1 and TC2 land. For TC1 land NZS3604:2011 type foundations are suitable provide the required bearing capacity can be achieved. For TC2 land enhance raft foundations (gravel raft, thickened slab or generic grid and beam slab or waffle slab) or piles could be used. So at present no specific ground improvement is required as there are foundation solutions available for construction on TC1 and TC2 land. However, at this stage the client's preference is to develop the Prestons South land into TC1 equivalent performance.

The liquefaction assessment indicates that the majority of the liquefaction is occurring in the upper 3m to 4m of the soil profile. Liquefaction could potentially occur at depths in the order of 10m but at this depth this is likely to be beyond the zone of influence for a residential building on shallow foundations. Similar ground conditions have been encountered in Prestons North and as part of the earthwork for Prestons North, ground improvement with an impact compactor supplied by Landpac has been carried out in areas of TC2. Based on our extensive construction monitoring, which has included numerous CPTs, the use of the Landpac compactor to improve ground from TC2 to TC1 has been successful.

As the ground conditions are similar to Prestons North and the preference is to form the land as TC1, we propose to use the Landpac compactor on the areas of TC2 within Prestons South. A trial of the Landpac compactor was carried out within the Prestons South site and our assessment of the compactor is provided in Section 6.5. In addition we have provided an indicative construction monitoring regime and earthwork requirements for the Landpac compactor.

6.4.2 Lateral Spreading

The lateral spreading assessment indicates that there is a potential for minor lateral spreading adjacent to the stormwater basins and channels. The extent of the lateral displacement varies and would have an impact on the site technical classification and any building foundation. We therefore recommend that as the lateral spreading risk is likely to govern the land classification and infrastructure resilience, mitigation measures are utilised to eliminate or limit the potential lateral spreading.

Similar lateral spreading potential and ground conditions have been encountered in Prestons North. As part of the earthwork for Prestons North, a geotechnical assessment of lateral spreading was carried out and gravel embankments were installed along the sides of the basins/channels, and embedded at a depth below any liquefiable layers. Lateral spreading occurs where there is a continuous liquefiable layer through to the free face, so by installing wide gravel embankments along any free edges the lateral spreading was eliminated. From our experience around Christchurch, observational and quantitative evidence indicates the presence of shallow gravel layers within the river banks has resulted in negligible lateral spreading adjacent to rivers.

As the lateral spreading potential and ground conditions are similar to that of Prestons North, we propose to use gravel embankment adjacent to the stormwater basins and channels. However, if the gravel embankment method is not feasible then alternative options such as stone columns or vibrofloatation may need to be considered. The gravel embankment assessment is provided in Section 6.6 as well a discussion of alternative methods.

6.5 Impact Compactor Trial Assessment

6.5.1 Liquefaction Assessment

It is intended to carry out ground improvement on TC2 areas within Prestons South using the Landpac impact compactor. It is noted that liquefaction is occurring at shallow depths across the site it is proposed to use the Landpac compactor to densify the shallow soil profile and reduce the liquefaction potential. By reducing the liquefaction potential the ground can possibly be improved from TC2 to TC1.

To confirm the extent that the impact compactor has had on the liquefaction potential on the underlying soil in Prestons South, a liquefaction analysis on the pre-trial CPTs and the post-trial CPTs (i.e. after 40 passes with the impact compactor) has been carried out. The liquefaction analysis is based on the three design level event detailed in Section 6.2.1.

A summary of the pre and post liquefaction induced settlement is presented in Table 10. Although the initial liquefaction induced settlements were already relatively low, compaction further reduces settlements in the upper soil profile.

Table 10 - Liquefaction induced vertical settlements for pre and post compaction CPT to 6m depth

Earthquake Magnitude 7.5, Water Depth 0.7m						
CPTs	**SLS Design Event (0.13g)**		**Intermediate Design Event (0.20g)**		**ULS Design Event (0.35g)**	
	Settlement (mm)		**Settlement (mm)**		**Settlement (mm)**	
	Pre	***Post***	*Pre*	***Post***	*Pre*	***Post***
CPT214/414	10	**<5**	30	**5**	35	**10**
CPT215/415	5	**0**	25	**<5**	35	**5**
CPT216/416	<5	**<5**	15	**10**	30	**15**

Note: The displacements presented above are to the nearest 5mm.

The results indicate that the liquefaction induced settlements can be reduced by a significant amount with the use of the Landpac compactor.

In addition to these results we have used the Landpac compactor on Prestons North and where it has been applied within the TC2 areas in Stage 1 Prestons North, the ground has been improved so it is equivalent to TC1.

We consider that this ground improvement method is likely to be suitable for the Prestons South site. As part of the detailed design of the Prestons South subdivision, further geotechnical assessment will be required to confirm the extent of the required impact compaction. During the impact compaction extensive quality assurance testing with CPTs will be required to confirm that the level of ground densification is being consistently achieved.

6.5.2 Lateral Spreading

Lateral spreading or horizontal ground movement during an earthquake event towards an open channel or stormwater basin has been identified as a geotechnical issue during the investigations. As with the liquefaction assessment, we have carried out a lateral spreading analysis on the CPTs carried out pre-trial and those carried out after 40 passes with the impact compactor. The lateral spreading analysis is based on the three design level event detailed in Section 6.2.1.

Note these values are based on assessments at the CPT locations, which may either be located a reasonable distance away from a basin/channel locations or alternatively at the crest. To rationalise the results so these are comparable to each other we have carried a lateral spreading assessment based on the assumption the soil profile in the CPT extrapolates to a setback distance of 10m from the crest of the basin/channel. The results based on a 10m setback are presented in Table 10.

Table 11 – Liquefaction induced lateral movements for pre and post compaction CPT to 6m depth

Earthquake Magnitude 7.5, Water Depth 0.7m						
CPTs	**SLS Design Event (0.13g)**		**Intermediate Design Event (0.20g)**		**ULS Design Event (0.35g)**	
	Settlement (mm)		**Settlement (mm)**		**Settlement (mm)**	
	Pre	***Post***	*Pre*	***Post***	*Pre*	***Post***
CPT214/414	40	**<5**	180	**10**	320	**35**
CPT215/415	25	**<5**	130	**<5**	300	**20**
CPT216/416	10	**5**	55	**35**	170	**90**

Note: The displacements presented above are to the nearest 5mm.

The impact compactor does improve ground conditions and hence significantly reduces the potential lateral spreading. The resulting lateral spreading displacements are still high to be TC1 but are within the deformation limits for TC2.

Although the potential for lateral spreading is reduced there still remains a lateral spreading potential adjacent to the stormwater basins/channels that cannot be addressed with the impact compactor alone, particularly if TC1 land is preferred. Other mitigation methods, such as the gravel embankment, need to be considered.

6.5.3 Quality Assurance Testing

Any area treated with the impact compactor will need to undergo pre and post CPT testing to confirm that the required ground improvement has been achieved. The following quality assurance testing is proposed for areas where ground improvement is to be carried out.

- Carry out pre-compaction CPTs to a depth of 10m. The CPTs will be undertaken at a frequency of 2 CPT tests per hectare of treated area. As the total treatment area is approximately 45 hectares this will amount to 90 pre-compaction CPTs.
- Carry out post compaction CPTs to a depth of 10m adjacent to the pre-compaction CPT locations. These can be compared to the pre-CPTs but also be used to assess technical category assessment in accordance with MBIE Guidelines. The post compaction CPTs testing frequency will be the same as above. As the total treatment area is approximately 45 hectares this will amount to 90 post compaction CPTs.

In addition to the above testing we propose the following quality assurance testing that will be carried out over the entire Prestons South site:

- One CPT test per hectare to a depth of 10m once the earth filling has been completed. This is to quantify the extent the earthfill has had on the liquefaction susceptibility. The CPTs will be taken to 10m depth so a technical category assessment in accordance with MBIE Guidelines can be undertaken. As the total Prestons South area is approximately 75 hectares this will amount to 75 CPTs.

- Additionally we recommended that CPTs are carried out 1 month after completion of the earthworks to further confirm the technical category. At this stage we recommend 0.5 test per hectare to a depth 10m. As the total Prestons South area is approximately 75 hectares this will amount to 38 CPTs.

6.5.4 Impact Compactor Site Works

Based on our experience of the impact compactor on Prestons North, pre-treatment of the site will be required prior to the ground improvement. This will need to include stripping of the topsoil, peat and any other unsuitable soil and placement of a working layer of 300mm gravel fill across the proposed treatment area. As observed in Prestons North a thick layer of topsoil can reduce the effectiveness of the impact compactor on the upper soil profile and hence there will be a requirement for all topsoil to be stripped from the treatment area prior to compaction.

6.5.5 Impact Compactor Limitations

We do note that the ground improvement with impact compactor may be variable depending on such aspects as presence of groundwater level and soil profile. As a result there will be risks associated with the use of ground improvement method. Given the nature of the ground conditions and liquefaction potential to the site, the only alternative to utilising the impact compactor is to accept that the land is TC2 and all new foundations are designed in accordance with MBIE Guidelines.

We also note that the impact compactor will cause vibrations that may be felt by neighbouring property owners, especially where compaction is being carried out close to the neighbouring properties. We recommend that neighbours are consulted on the short term effects during the impact compaction and that pre-condition surveys are undertaken on properties within close proximity to areas of potential ground improvement areas.

6.5.6 Conventional Compaction

Site earthworks are likely to include the use of conventional compaction plant. Such plant has been used successfully in the past to improve the strength of soils to shallow depths. However the depth of penetration for conventional compactors is limited. It is unlikely that conventional compaction plant could carry out ground improvement to a depth of 3m like the impact compactor.

Conventional compaction plant may be of use in improving the ground for residential lots, especially if dewatering is carried out prior to compaction. However, the effectiveness of this will depend on the depth of the dewatering and may still be limited to 1m depth or less using a smooth wheeled compactor.

Even if conventional compaction plant cannot densify fully to the required depth, the method still has potential merit as it tends to increase the density of the upper, non-saturated soil layers and hence improve the soil bearing capacity.

Based on our experience on Prestons North, conventional compaction tends to compliments the impact compactor, as it densifies the upper 0.5m and forms a firm work surface for the impact compactor. Conventional compaction also can improve the bearing capacity of the upper soil layers.

6.6 Lateral Spreading Mitigation Measures

The impact compactor assessment indicates that the lateral spreading displacement can be significantly reduced but not entirely eliminated. Therefore to mitigate against lateral spreading other options need to be considered and based on the liquefaction risk as well as our experience on

Prestons North, the most appropriate is likely to be gravel embankments. Although depending on the depth of the liquefiable layers stone columns or vibrofloatation may need to be considered.

6.6.1 Gravel Embankment Assessment

The gravel embankment would comprise of a block of well compacted sandy gravel that is founded below the liquefiable layers and is wide enough to prevent lateral displacement towards the stormwater basin/channel. Lateral spreading occurs where there is a continuous liquefiable layer through to the free face, so by installing wide gravel embankments along any free edges the lateral spreading can be eliminated.

In determining the extent of the mitigation measures to minimise the lateral spreading we need to consider where the liquefiable layers are present in the soil profile and how this relates to the proposed basin/channel configuration.

At this stage the stormwater basins will comprise of two typical sections:

- 3m deep basin with bank slopes of 4H:1V and a safety bench 3m wide 2m below the crest of the bank.
- 3m deep channel located in the southern part of Prestons South with bank slopes of 4H:1V.
- 2m deep channels with bank slopes of 4H:1V.

For the proposed stormwater basin and channels we have reviewed CPT logs and the liquefaction analysis to confirm where liquefaction is occurring. The results of this assessment are presented in Appendix M. The depth of the liquefiable layers has been determined for the most part to be less than 2.5m below existing ground level. However there are a few CPTs adjacent to the stormwater channel where liquefiable layers are between 3m and 4m.

The extent and depth of liquefiable layers are similar to those identified in Prestons North and gravel embankment have been designed and constructed to eliminate the lateral spreading potential. As the depth of the liquefiable layers is predominantly in the upper 3m to 4m with further liquefiable layers at depths of greater than 7m, we consider the use of gravel embankment adjacent to the stormwater basin/channel is a feasible mitigation option.

Where liquefiable layers are present at depths of 3m to 4m, options are either to excavate out and replace with compacted gravel. This would require relatively deep excavations but such excavations have been completed successfully in Prestons North.

An alternative is to excavate to 3m depth place a layer of compacted gravel than use the impact compactor to densify the soil below the gravel, before building up the remainder of the gravel embankment. As this option involves ground densification quality assurance testing with CPT will be required to confirm that the required ground improvement has been achieved.

As part of the detailed design of the Prestons South subdivision, further geotechnical assessment will be required to confirm the gravel embankment design for each of the stormwater features.

Indicative recommendations for construction of the gravel embankment are as follows:

- Embankment fill material will consist of free draining, well graded sandy gravel fill.
- Fill is placed in a dry toe to allow maximum compaction.
- Base of excavation is clear of organics and should be inspected by a geotechnical engineer prior to fill placement.
- If any inspection identifies loose soil in the base of the excavation, the soil will need to be compacted or removed and replaced with gravel fill.

- Construction of the embankment will require gravel to be compacted to 98% of standard compaction in accordance with NZS4402:1986 – *Methods of Testing Soils for Civil Engineering Purposes*, Test 4.1.1.

6.6.2 Stone Columns

If the detailed design of the embankment indicates that the depth of liquefaction is significant enough that excavation of the embankment is not feasible, then stone columns many need to be considered. Stone columns would effectively densify the ground and reduce the potential for the ground to liquefy and hence reduce the potential for lateral spreading.

The installation of stone columns only become cost effective if the depth of densification exceeds 3m. Typically stone columns would be installed on a grid over a zone ranging from 10m to 15m in width, depending on the basin configuration, and to a depth of 6m. As part of the detailed subdivision design the stone columns will need to be designed by a geotechnical engineer.

As part of the detailed subdivision design the stone columns will need to be designed by a geotechnical engineer. A field trial is recommended, in order to determine the degree of ground improvement, optimise the column spacing, determine the viability of the proposed works and if viable, optimise the column spacing. Stone columns are typically carried out by a specialist contractor, whose input would be required in carrying the detailed design.

6.6.3 Vibrofloatation

An alternative densification method to stone columns is to undertake vibrofloatation compaction adjacent to the basins/channels. This essentially involves using a crane mounted vibrating probe that is inserted into the ground. As the probe penetrates the soil skeleton around the probe collapses and densifies due to the high frequency vibration. The probe would be inserted on a triangular grid, at approximately 2.5m to 3.5m centres, across the site to the required depth of penetration.

The vibrofloatation method leaves hollows in the ground following treatment (due to the volumetric reduction in the soil volume caused by the soil densification) that will then need filling. If this method is utilised it is recommend that the surface is rolled with an impact compactor post filling as the vibrofloatation method can potentially loosen the upper 600mm of soils.

This method of compaction has the advantage that it can be undertaken below the water table in saturated soils. Therefore, no dewatering is required for the compaction process to be undertaken.

As part of the detailed subdivision design the vibrofloatation will need to be designed by a geotechnical engineer. A field trial is recommended, in order to determine the degree of ground improvement, optimise the probe spacing, determine the viability of the proposed works and if viable, optimise the probe spacing. Vibrofloatation is typically carried out by a specialist contractor, whose input would be required in carrying the detailed design.

6.7 Foundation Implications

6.7.1 TC1 Compliance

Suitable foundation types for the various technical categories have been defined in the MBIE Guidelines (2012). For Technical Category 1 areas the MBIE Guidelines has recommended Standard NZS3604:2011 type foundations with tied slabs provided there is suitable bearing. As required under the MBIE Guidelines for detailed house design, a site specific geotechnical assessment shall be carried out by suitability qualified chartered engineer with experience in residential house development.

6.7.2 TC2 Compliance

Where residential sites cannot be improved to TC1 classification then TC2 type foundations will be required. For Technical Category 2 areas the MBIE Guidelines (2012) has recommended types of enhanced foundation systems. The appropriate foundation system will depend on the ultimate bearing capacity of the foundation soil. Schematics and typical cross sections of these foundation systems are presented in the guidelines. For detailed house design, a site specific geotechnical assessment shall be carried out by suitability qualified chartered engineer with experience in residential house development.

As part of the detailed foundation design, particular attention should be paid to detailing the connection joints of buried services (water and sewer pipes, power conduits, etc.) between the house foundation and the in situ ground. The design should allow sufficient movement and ductility to account for seismic shaking and liquefaction induced movement, and to allow for easy reinstatement if they were to be damaged during a future seismic event.

6.8 Infrastructure

Despite the minor potential liquefaction risk on the site, buried services at the site are still potentially vulnerable to seismically induced liquefaction if inserted into potentially liquefiable soils. The liquefaction analysis indicates the potential liquefaction induced ground damage is unlikely in a SLS event, with potential ground damage in a ULS event if no ground improvement is carried out.

It is recommended to design the mitigation measures against the effects of a 1 in 150 year earthquake. For seismic events with a return period greater than 1 in 150 years the system may become progressively less serviceable. The proposed liquefaction mitigation measures are in line with the Christchurch City Council Capital Programme Group Technical Memorandum *'Earthquake Learnings – Amendments to the IDS and the CSS for Pipes Infrastructure in Christchurch City, to Mitigate Against Future Earthquake Damage.'* This section outlines the possible liquefaction mitigation measures for the infrastructure at the site. Although liquefaction induced ground damage under a SLS or 1 in 150 year events is unlikely there would be benefits in building additional seismic resilience into the residential development infrastructure for large earthquake events.

6.8.1 Buried Structures

All buried services such as manhole risers, pump station chambers, etc. founded below the groundwater level, should be designed to have neutral buoyancy and accommodate uplift forces associated with liquefied soil, not just hydrostatic groundwater buoyancy forces. This is in order to minimise lifting / floatation of these buried services. Spaces around buried structures should be backfilled with free draining granular non-liquefying fill in order to alleviate pore water pressure build up during a large seismic event thereby reducing the potential for liquefaction in the soils immediately surrounding the buried structure.

As it is unlikely that buried services are able to be founded directly into the underlying dense non-liquefiable sand due to the depth, the manhole inverts and pipe entry and exit levels should be designed to accommodate differential settlement post liquefaction event. Based upon differential settlements calculated as part of the geotechnical assessment, the differential settlements are expected to be in the order of 25mm. Essentially the hydraulic design of the pipes coming into and out of the manhole risers should be designed to accommodate both positive (i.e. pipe gradient getting steeper) and negative (i.e. pipe gradient getting shallower) differential settlements of 25mm. The civil engineer will need to assess if this level of settlement will affect the hydraulic design of the pipe and detailed the required engineering measures.

Manhole risers should have strap rings to hold the manhole riser sections together in order to reduce lateral displacement of the manhole risers. Additionally, manhole connectors with greater than 90mm sealing lengths should be used to minimise the potential for joint pull-out.

It is recommended that the finalised design of each buried service (manhole riser, pump station, etc.) is confirmed on a case by case basis during construction, as each development stage will require site specific design. This specific design is needed to define the mass concrete for dead weight, tie down anchors, etc. for each buried structure, if required.

6.8.2 Pipes and Service Conduits

Pipes and service conduits should be made from flexible material (i.e. plastic) where practicable. For gravity reticulated sewer lines all pipe joints and intersections with manhole risers should be installed with short slip collars to allow greater capacity of joint movement and increase joint resilience. For pressurised sewer lines, all PE pipes should have end restraints at pump stations. Combined with the PE pipe material, well designed end restraints will improve the resilience of the pressure line and help prevent damage.

Hydraulic pipes (sewer, and stormwater and possibly reticulated water), the pipe sizes and gradients should be designed in such a way that it can accommodate post liquefaction differential settlement, both positive (i.e. pipe gradient getting steeper) and negative (i.e. pipe gradient getting shallower). For design, differential settlements of 25mm between manhole risers should be used.

All pipes and conduits should be founded into the non-liquefiable crust material where possible. If the founding depth of the pipes and conduits is in the liquefiable silty sandy material the service trenches should be backfilled with non-liquefiable geotechnically competent fill.

All service trenches located below the water table should be lined with a geosynthetic filter fabric material (i.e. Bidim A19 or similar) to separate potentially liquefiable soils from non-liquefiable granular bedding and backfill material. For shallow service trenches founded above the water table then filter fabric is not required but generally recommended.

By providing a filter fabric and filling the service trenches with non-liquefiable geotechnically competent fill, the trench becomes non-liquefiable and will therefore limit liquefaction induced settlement. Additionally, if a pipe was to rupture, by having a filter fabric encasing the bedding material there is less likelihood of sand material infiltrating into and blocking the pipeline.

6.8.3 Pavement

At this stage based upon our liquefaction assessment it is inferred that the pavement is unlikely to be significantly affected by seismically induced liquefaction in a SLS or 1 in 150 year events. However, to ensure robustness of the pavement following a liquefaction inducing major earthquake it is recommended that the pavement be designed to accommodate the potentially adverse effect of seismically induced liquefaction. The pavement should be designed in such a way that it can bridge any localised voids / settlements that may be caused by seismically induced liquefaction, and prevent liquefiable soil from penetrating into the pavement structure.

If subsoil drains are to be installed as part of the subdivision development for stormwater control, then it is recommended extending the subsoil drainage to below the footprint of the roading network. This will extend the thick non-liquefied crust below the pavement areas as well as the residential sections, thereby minimising the likelihood of liquefaction induced damage.

A geosynthetic filter fabric (i.e. Bidim A19 or similar) should be placed directly onto the in situ sub-grade material prior to the placement of the granular sub-base fill. This filter fabric will act as a barrier to any fines migration from the sub-grade to the sub-base during a liquefaction inducing seismic event. Therefore, the pavements sub-base will not lose strength post the seismic event through fines infiltration and associated loss of effective thickness.

6.9 Organic Soil Layers

6.9.1 General

As the site has isolated areas of peat, as well as peat layers within the soil profile, a specific geotechnical review of the presence of the peat has been carried out. In this section we review the presence the organic soil layers (peat) and its effect on the subdivision development.

The geotechnical investigations identified localised pockets of peat in the Prestons South. Peat was also located in thin bands across other parts of the site. The extent of the peat and depth is presented in Figures 5 and 6. We note that with the exception of localised thicker pockets (i.e. greater than 0.5m thick) across the site, all other surficial peat is not represented on these figures, as it will most likely be removed as part of the topsoil strip prior to earthworks. The peat layers typically are less than 2m in depth.

Peat thickness across the site was typically less than 0.3m but thicker layers were identified in localised pockets across the site, ranging from near surface to 2m depth. These localised pockets are presented on Figure 6.

6.9.2 Peat at Ground Level (Surficial Peat)

Based on our geotechnical test information, the peat at the ground surface has been identified in localised pockets and is typically less than 0.5m in depth. Where there will be residential buildings or subdivision infrastructure we recommend that the peat at ground level is removed as part of the subdivision development. However, for any proposed green spaces removal of the peat is not essential.

6.9.3 Buried Peat Layers

Thin peaty layers typically less than 0.3m thick were identified across the site, at depths ranging from 1m to 2m below existing ground level. As the peat is relatively thin and discontinuous in parts, removal is not considered viable. Therefore, we need to consider the effect of the proposed earthworks (both cutting and filling) on the peat layers.

Based on our understanding of the earthworks levels, the majority of the site is likely to require earthfill in the order of 0.5m to 1.5m above existing ground levels. We have carried out an assessment of the potential primary consolidation settlement and secondary compression (long term settlement) that may occur due to the raising of the ground levels. The assessment is based on compression indices defined from empirical correlations with the water content and plasticity index values for peat sampled from Prestons North. These tests are summarised in the Detailed Design Report, dated July 2012. The potential consolidation settlements are presented in Table 12.

Table 12 – Potential settlements levels of peat layers

Thickness of Peat	Primary Consolidation Settlement Magnitude	Secondary Compression Settlement
100mm	<5mm to 20mm	<5mm to 10mm
200mm	5mm to 35mm	5mm to 20mm
300mm	10mm to 50mm	10mm to 30mm

Primary consolidation of the peat is likely to occur within 4 to 6 weeks of the earthfill placement, especially as the peat is interbedded with more permeable, free draining sandy soil. Secondary compression is likely to occur over the long term.

6.9.4 Effect of Groundwater Lowering on Peat

The proposed stormwater basins/channels may lower the groundwater which could lead to additional settlement in the peat. However, we understand that the channels are likely to lower the groundwater level to depths of about 1m below existing ground level and hence the majority of the peat layers are likely to remain below the water table and therefore fully saturated. If groundwater levels are lowered to below the peat layers settlements may occur but we consider that the likely settlement of the peat could occur relatively fast (i.e. within weeks of the groundwater table lowering) and will not be a long term issue to the subdivision.

6.9.5 Earthworks

Earthworks in the order of 3m to 4m deep will be required to form the stormwater retention basins and channels. At these depths the majority of the peat layers will be removed and therefore are unlikely to affect the basins and channels.

For excavations to form the residential lots, we recommend that where the excavations are deeper than 1m, the upper peat layers (within the upper 2m of the existing soil profile) are excavated and replaced with earthfill. Where localised thick layers are present at shallow depths, as defined in Figures 5 and 6 these areas should be over excavated and the peat removed.

The above recommendations will need to be confirmed on site during earthworks by a geotechnical engineer, to verify that all the peat and other organic soils have been successfully removed.

Provided the above recommendations are followed and accounted for in the detailed design, we consider the presence of the peat should not prevent the subdivision development.

6.9.6 Residential Lots

Given the thickness of the peat layers we consider that the primary consolidation settlement should be within tolerable limits for a light weight residential building. The placement of the fill is likely to remove a significant amount of the potential settlement.

Once the earthfill is in place the upper peat layers will be in the order of 2m below finished ground level. At these depths the residential building loads are unlikely to cause significant additional loading on the peat layers and hence it is unlikely that typically shallow type foundations will experience additional settlements, above those already calculated.

Cutting will be required in parts of the side, which will reduce the overlying soil thickness over the peat layers. The effect of this has been discussed in the following section and specific geotechnical investigation and design maybe required for the affected residential lots.

Once the final earthfill levels have been confirmed the depth of the filling and cutting in relation to the identified peat layers will be reviewed by a geotechnical engineer. Any site specific geotechnical investigations for residential lots will need to take into account the presence of peat, if present, and provisions included in the foundation design to mitigate any potential adverse settlements.

6.9.7 Infrastructure

These levels of settlement are unlikely to cause any significant issues to the earthworks and subdivision infrastructure. However, as part of any detailed design for the subdivision infrastructure, such as pump stations and road pavement, the peat layers will need to be identified and appropriate engineering measures implemented. For instance, depending on where the peat layer is within the soil profile, it may be excavated and replaced with hardfill or the infrastructure placed on foundations that extend below the peat layer. The appropriate engineering measures will need to be assessed on a case by case basis during the subdivision detailed design.

6.10 Earthworks

6.10.1 Cut Excavations

It is proposed to form a series of stormwater basins and channels as part of the development. The formation of these will required excavations into the existing ground surface with cut slopes in the order of 4H:1V. Based on the investigation results we make the following comments:

- Cuts are likely to encounter predominantly loose to medium dense sandy soil with interbedded peat and silt layers. We anticipate that the soils will be easy to excavate with conventional earth moving equipment.
- Cut slopes of 4H:1V are likely to maintain global stability for static and seismic cases. However, there is a potential for the cuts to be affected by lateral spreading. The lateral spreading risk has been discussed in Section 6.6 and mitigation measures will be required.
- Groundwater is present at relatively shallow depths across the site and is likely to be encountered during cut slope construction. Earthworks will need to be carried out so that the presence of the groundwater does not adversely affect the stability of the cuts. It is anticipated that groundwater seeps are initially likely to be present in the cut faces however the levels are likely to equalise in the long term.
- If groundwater takes a relatively long time to equalise, de-watering of the ground within the basins and along the channels may be required.
- If significant groundwater inflows are encountered and left untreated, slumping of cuts could occur. Hence, site specific treatment should be adopted on an as required basis.
- Cut slopes will be vulnerable to erosion and therefore should be treated or otherwise protected as soon as practicable after excavation.

6.10.2 Earthfill

It is proposed to reuse the soils from the stormwater basin/channel excavations as earthfill across the site as well as imported fill to meet the balance of the site. The majority of the insitu soils consist of fine to medium grained sand with peat and silt in the upper layers, and thin peat layers at depth. Based on the anticipated soil types we make the following comments:

- **Peat:** The peat is unsuitable for fill and would need to be cut to waste or retained as landscape fill.
- **Silt:** The silt is marginally suitable for fill, as it is moisture sensitive and can be difficult to compact. It would be preferable that the silt is used as landscaping fill, where achieving high levels of compaction are not essential. However, following a field trial this material may be considered as fill in appropriate locations.
- **Sand:** The sand is considered to be suitable as an earthfill material. However, the compaction tests indicate that the maximum dry density will be sensitive to moisture contents in excess of the optimum moisture content. The moisture content of the sand will need to be controlled during fill placement to ensure appropriate compaction is achieved. Alternatively, where the sand is too dry, wetting may be required.
- **Imported Fill:** In case of a shortfall of site won fill, imported fill will be required. We understand that a suitable source of fill is to be confirmed. When the fill source site has been identified, an inspection of the material by a geotechnical engineer and review of laboratory testing results by a geotechnical engineer should be carried out to confirm the fill suitability.

Aurecon NZ Ltd. as part of the detail earthworks design will be providing a detailed earthworks specification. However, we make the following preliminary recommendations with regard to the fill placement:

- Filling shall generally be carried out in accordance with NZS4431:1989 – *Code of Practice for Earth Fill for Residential Development*, with appropriate on site quality control;
- All fill material should be compacted to at least 95% of standard compaction, or to a higher degree where required, in accordance with NZS4402:1986 – *Methods of Testing Soils for Civil Engineering Purposes*, Test 4.1.1.
- All areas where earthfill is to be placed should be stripped of topsoil and other organic material and stockpiled. Peaty layers, where encountered, will need to be removed especially at the base of earthfill embankments.
- Benching to key the fills into the ground is required wherever the existing ground slope is steeper than 4H:1V.
- Fill slopes are likely to be stable at a slope of 3H:1V. If fill slopes are required to be steeper, then the use of reinforcement may be required to retain the fill edge.
- The fill slope could be vulnerable to erosion if concentrated stormwater flows develop. To control runoff from the fill batters and scouring of the fill, the front face should be protected and stormwater runoff directed away from the fill face.
- The design of any fill slope will need to take into account the potential for liquefiable soils and the presence of peat at the base of the slope.

6.10.3 Earthworks Volumes

In considering earthworks volumes the following aspects need to be considered.

Impact Compactor Induced Settlements

The impact compactor trials identified that the compaction method will result in settlement of the ground. We recommend that where the impact compactor is to be used it should be assumed that at least 100mm of settlement of the natural ground will occur and hence an additional 100mm thickness of filling will be required.

Impact Compactor Working Surface

The impact compactor will need a working surface to allow ease of movement as well as improve its effectiveness. To form the working surface the topsoil and any other unsuitable soil will need to be removed and a working layer of 300mm gravel fill placed across the proposed ground improvement area. It may be possible to re-use parts or all of the gravel layer.

Peat Settlement

As previously identified in the Section 6.9, primary settlements induced by the placement of the earthfill may occur relatively quickly following placement of the fill. The extent of the settlement will be in the order of 10mm to 50mm. It may be prudent to allow for additional filling following the primary settlement of the peat.

7. Assessment Against RMA

Section 106 of the Resource Management Act (RMA) states *inter alia*

> *… "a consent authority may refuse to grant a subdivision consent, or may grant a subdivision consent subject to conditions, if it considers that:*

a) *the land in respect of which a consent is sought, or any structure on the land, is or is likely to be subject to material damage by erosion, falling debris, subsidence, slippage, or inundation from any source; or*

b) *any subsequent use that is likely to be made of the land is likely to accelerate, worsen, or result in material damage to the land, other land, or structure by erosion, falling debris, subsidence, slippage, or inundation from any source; or*

c) *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 below. A summary is provided in Table 6 with further details following.

Table 13 - Assessment against RMA

Hazard	Potential Susceptibility	
	Current (Section 106 1a)	**Post Development (Section 106 1b)**
Falling Debris	The site is essentially flat to gently sloping so there is no risk of falling debris.	
Slippage	The site is essentially flat to gently sloping so there is no risk of slippage.	
Erosion	No significant erosion observed on site apart from nominal erosion along the drainage channels and the ephemeral stream.	Sandy soils on site have the potential to erode if topsoil and vegetation layer is removed and underlying soil is left unprotected.
Subsidence	Due to the potential for seismically induced liquefaction especially in the areas that fall within Technical Category TC1 and TC2, we infer that parts of the site are susceptible to subsidence (settlement) from liquefaction.	Provided the appropriate engineering mitigation measures are implemented the subsidence potential is unlikely to be worsen.
Inundation	Due to the potential for seismically induced liquefaction especially in the areas that fall within Technical Category TC1 and TC2, we infer that parts of the site are susceptible to inundation (sand boils) from liquefaction. A full flood risk assessment is outside the scope of this report.	Provided the appropriate engineering mitigation measures are implemented the inundation potential is unlikely to be worsen. Provided the storm water discharge is appropriately managed, inundation potential is unlikely to be worsen.

No significant erosion was observed on the site. However the silty soils that directly underlie parts of the site are inferred to be potentially susceptible to erosion when left unvegetated or exposed. We infer that the site is not susceptible to falling debris or slippage due to the topographical location. It is noted that issues surround stormwater discharge are being dealt with in the detailed civil engineering design by Aurecon. Therefore any potential “inundation” susceptibility due to stormwater has already been addressed.

We infer that the site is susceptible to subsidence (settlement) and inundation (sand boils) from liquefaction. However, if the appropriate liquefaction mitigation measures, as outlined in this report, are undertaken then the risk of subsidence and inundation from liquefaction is significantly reduced. Therefore, with appropriate liquefaction mitigation measures where required, the site in our opinion will generally be free of “erosion”, “falling debris”, “subsidence”, “slippage”, or “inundation”. **The proposed subdivision development therefore generally complies with the intent of Section 106 1 (a).**

Due to the site being partially underlain by fine grained soils, there exists the potential for erosion and rilling of the sandy and silty soils if vegetation cover is removed for prolonged periods of time from both stormwater runoff if it is not discharged in a controlled manner, and from the wind. This susceptibility to erosion of the sandy and silty soils can be minimised with appropriate industry standard design measures undertaken during construction.

The site has been identified as being susceptible to seismically induced liquefaction and hence has the potential for “subsidence”, “and “inundation.” Provided that appropriate liquefaction mitigation measures are implemented, as recommended in this report, subsequent use of the land following development is unlikely to accelerate, worsen, or result in material damage to the land, other land, or structures. **In our opinion the development will comply with the intent of Section 106 1 (b).**

Although not a geotechnical matter we note that the site has multiple access points from Prestons Road and Mairehau Road, and hence Section 106 1(c) is fulfilled.

Therefore in our opinion, under Section 106 of the RMA, there are no geotechnical reasons preventing the development, provided the appropriate engineering measures as recommended in this report are carried out.

8. Limitations

We have prepared this report in accordance with the brief as provided. The contents of the report are for the sole use of the Client and no responsibility or liability will be accepted to any third party. Data or opinions contained within the report may not be used in other contexts or for any other purposes without our prior review and agreement.

The recommendations in this report are based on data collected at specific locations and by using appropriate investigation methods with limited site coverage. Only a finite amount of information has been collected to meet the specific financial and technical requirements of the Client's brief and this report does not purport to completely describe all the site characteristics and properties. The nature and continuity of the ground between test locations has been inferred using experience and judgment and it must be appreciated that actual conditions could vary from the assumed model.

Subsurface conditions relevant to construction works should be assessed by contractors who can make their own interpretation of the factual data provided. They should perform any additional tests as necessary for their own purposes.

Subsurface conditions, such as groundwater levels, can change over time. This should be borne in mind, particularly if the report is used after a protracted delay.

This report is not to be reproduced either wholly or in part without our prior written permission.

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