A1 Introduction

The Canterbury region of New Zealand has been affected by a series of earthquakes and aftershocks with the four most significant earthquakes occurring on 4 September 2010 (Mw 7.1), 22 February 2011 (Mw 6.2), 13 June (Mw 5.6 and Mw 6.0 separated by 80 minutes), and 23 December 2011 (Mw 5.8 and Mw 5.9 separated by 80 minutes). The earthquake shaking from these events triggered localised-to-widespread liquefaction resulting in mapped land damage ranging from none-to-minor is some suburbs and moderate-to-severe in other suburbs.

Ground surface subsidence resulting from liquefaction related volumetric densification, surface ejecta of liquefied soil material, topographic re-levelling and lateral spreading were the principal ground deformation modes that caused differential settlement which damaged the residential dwellings in the Canterbury region.

While extensive triggering of liquefaction was observed in the September 2010, February 2011, and June 2011 earthquakes, this triggering had little-to-no consequence on the built environment in some areas where state-of-the-practice liquefaction procedures suggested severe ground failure and damaging effects should have been anticipated. Initially, the results of existing liquefaction vulnerability assessment procedures were compared with the observed land damage datasets and to understand the reasons why some parts of Christchurch were affected more seriously by liquefaction. Existing liquefaction vulnerability procedures were not able to capture the observed damage data well.

There is extensive literature on liquefaction phenomenon and liquefaction triggering evaluation procedures, but there is substantially less literature on the quantification of liquefaction land damage and associated vulnerability assessment methods that address the consequences of liquefaction for residential land.

This appendix provides a review of existing liquefaction vulnerability assessment methods, and compares the results obtained for these methods with the liquefaction related land damage observations obtained after major events in the Canterbury Earthquake Sequence (CES). A new liquefaction vulnerability parameter, Liquefaction Severity Number (LSN), has been developed and compared to the observed liquefaction related land damage datasets. The advantages and disadvantages of this parameter are discussed relative to data and observations associated with liquefaction and the existing methods for evaluating the severity of liquefaction. The focus of this appendix is the assessment of vulnerability to free field liquefaction related land damage. Vulnerability to lateral spreading is discussed separately in Appendix F.

A1.1 The Liquefaction Vulnerability Assessment Process

The assessment of liquefaction vulnerability in the Christchurch area involves the following steps:

1. Estimating the liquefaction susceptibility of the soils being assessed;
2. Estimating whether or not liquefaction will be triggered in a susceptible soil layer for a given depth to groundwater and level of ground shaking;
3. Estimating the vulnerability to liquefaction damage at the ground surface for a given soil profile; and
4. Verification of the liquefaction vulnerability assessments with the observations from the CES.

For each of these steps there are a number of assessment methods which have been developed and evolved over time. With the use of the geotechnical investigation data in the Canterbury Geotechnical Database (CGD) and through extensive studies, the results from these assessment methods have been correlated to the land damage observation data.
A1.2 Purpose and Outline

The purpose of this appendix is to document the current scientific understanding of the assessment of liquefaction vulnerability in the Christchurch area, as relevant to the ILV Assessment Methodology. The results of these studies are summarised as they relate to each of the steps in Section A1.1.

This appendix is laid out in the following sections:

- Section A2 liquefaction susceptibility;
- Section A3 liquefaction triggering;
- Section A4 liquefaction vulnerability and verification; and
- Section A5 discusses the limitations of CPT-based liquefaction vulnerability assessment methods.

The main conclusion of this appendix is that of the various liquefaction assessment tools that were evaluated, the LSN parameter is the most appropriate index for the assessment of liquefaction vulnerability for the Christchurch ground conditions and the assessment of the increase in liquefaction vulnerability as a result of the ground surface subsidence caused by the CES.

However all liquefaction assessment methodologies, including LSN, have inherent limitations. When undertaking any liquefaction vulnerability assessment, these limitations should be considered in conjunction with any available land damage observations.

A2 Liquefaction Susceptibility

Liquefaction susceptibility is a physical characteristic of a soil that determines whether or not it is able to liquefy. Soils that are susceptible to liquefaction typically have no to low plasticity, and low to moderate permeability. Liquefaction susceptibility is independent of the level of shaking required to trigger liquefaction; this is part of the assessment of the liquefaction resistance of the soil (refer to Section A3 for discussion about liquefaction triggering).

As described in Section A1, the first step in the liquefaction assessment process is to determine whether or not a particular soil layer is susceptible to liquefaction (Kramer, 1996). If a soil layer is not susceptible to liquefaction, by definition, liquefaction cannot be triggered and that layer will not contribute to liquefaction vulnerability. Therefore, liquefaction triggering assessments should only be undertaken on soil layers that have been assessed as being susceptible to liquefaction.

To assess liquefaction susceptibility, Robertson and Wride (1998) suggested adopting a default soil behaviour type index ($I_c$) ’cutoff’ value of 2.6 beyond which soil material can be assumed to be non-liquefiable (i.e. not susceptible to liquefaction). One caveat with adopting this default value is that soils with $I_c > 2.6$ and $FC \leq 1.0\%$ can be very sensitive. The application of an $I_c$ cut-off value has proven to be contentious although it is generally accepted that for many soils where $I_c > 2.6$, published liquefaction triggering methodologies are generally approaching their limitations.

Regional liquefaction analyses carried out in Canterbury to date (Tonkin & Taylor, 2013; van Ballegooy, et al., 2014b; van Ballegooy, et al., 2015c) have been performed assuming a default $I_c$ cutoff of 2.6. The laboratory data available in the CGD has been examined to evaluate the validity of this assumption and to understand its influence.

The liquefaction susceptibility of fine grained soils is generally governed by the soils plasticity. The Atterberg limits is a commonly used laboratory test which calculates the water contents ($w_c$) of fine grained soils according to standard performance criteria such as its Plastic Limit ($PL$) and Liquid Limit ($LL$). PL is the $w_c$ at which the behaviour of a soil changes from plastic to non-plastic.
LL is the $w_c$ at which the behaviour of a soil changes from plastic to liquid. The plasticity index (PI) is a measure of the plasticity of a soil. It is defined as the difference between the LL and the PL (i.e., $PI = LL - PL$). Soils with a high PI tend to be clays, soils with a low PI tend to be silts and soils with a PI of 0 tend to be neither silt nor clay.

Recent research into the liquefaction susceptibility of fine grained soils by Bray and Sancio (2006) has shown that PI is a good indicator of the liquefaction susceptibility of fine grained soils. Based on their research they have proposed the following liquefaction susceptibility criteria:

- Soils with $PI < 12$ and $w_c/LL > 0.85$ are susceptible to liquefaction;
- Soils with $12 < PI < 18$ and $w_c/LL > 0.8$ are systematically more resistant to liquefaction; and;
- Soils with $PI > 18$ are not susceptible to liquefaction.

Lees et al. (2015) used CPT and laboratory test data from the CGD to investigate the liquefaction susceptibility of soils within the Christchurch area. Figure A2.1 (a), reproduced from Lees et al. (2015), shows the laboratory test data plotted on the Bray and Sancio (2006) liquefaction susceptibility criteria. The symbol shapes denote the corresponding $I_c$ value at the same vertical location from which the soil samples were obtained for laboratory testing purposes.

Figure A2.1: (a) Associated Atterberg Limit, $w_c$ and $I_c$ data obtained from geotechnical investigation data in the CGD from the Christchurch area plotted onto the Bray and Sancio (2006) liquefaction susceptibility criteria. (b & c) percentage of dataset identified as not susceptible with values below the $I_c$ cutoff threshold (defined by point x) and susceptible above the $I_c$ cutoff threshold (defined by point y). Figure reproduced from Lees et al. (2015).

Inspection of Figure A2.1 (a) shows that there are some data points which may be susceptible to liquefaction where the $I_c$ value is greater than 2.6 and similarly there are some data points which are potentially not susceptible to liquefaction that have an $I_c$ value less than 2.6. This variability is partially attributed to the application of the Bray and Sancio (2006) criteria on a regional scale (without the ability to apply site-specific engineering judgement).

Figure A2.1 (b) shows two pie graphs which separate the influence of an $I_c$ cut-off of 2.6 on the 2,400 point dataset. The top pie graph shows the distribution of $I_c$ for the not susceptible soils (according to the Bray and Sancio (2006) criteria) and the bottom pie graph shows the distribution of $I_c$ for the susceptible soils (according to the Bray and Sancio (2006) criteria). These two pie graphs indicate that the $I_c$ cut-off is generally in the right place. For 25% of the not susceptible grouping, the $I_c$ cutoff should be lower (the blue section of the upper pie graph). For 18% of the susceptible grouping, the data indicates that the $I_c$ cut-off should be higher (the red section of the lower pie graph). Figure A2.1 (c) indicates that an $I_c$ cutoff between 2.5 and 2.6 is the optimal
value in the Christchurch area between false positive and false negative identification of whether or not soil layers are susceptible to liquefaction.

Lees et al., (2015) demonstrated that there was no clear spatial correlation between $I_c$ cutoff and the geologic units of the Christchurch soils. It also demonstrated that regionally adopting the default $I_c$ cutoff of 2.6, as suggested by Robertson and Wride (1998), is justified. However, there are soil layers in localised areas where adopting a slightly lower or slightly higher $I_c$ cutoff could be justified when supported by appropriate laboratory test data.

A3 Liquefaction Triggering

Liquefaction triggering is the initiation of liquefaction from ground shaking, commonly caused by earthquakes. This shaking must be sufficiently intense to trigger liquefaction for a particular soil. Smaller earthquakes do not tend to trigger liquefaction as readily as larger earthquakes. The shaking level that causes liquefaction is the trigger depends on the resistance of the soil layer being assessed.

As described in Section A1, the second step in the liquefaction assessment process is to estimate whether or not liquefaction is likely to be triggered in a given soil layer for a given level of ground shaking. Liquefaction triggering assessments should only be undertaken on soil layers which have been assessed as susceptible to liquefaction and should not be applied to soil layers which have been assessed as not susceptible to liquefaction (refer to Section A2).

A3.1 Liquefaction Triggering Assessment

In an earthquake, liquefaction is triggered in a soil when the seismic demand exceeds the ability of the soils to resist the demand. The seismic demand imposed by an earthquake to trigger liquefaction is represented by the Cyclic Stress Ratio (CSR). The Cyclic Resistance Ratio (CRR) is a representation of the ability of the ground to resist liquefaction demand imposed on the ground by seismic shaking, and is related to the relative density and Fines Content (FC) of the soil. When the resistance of the soil is less than the seismic demand (i.e. CRR < CSR), liquefaction triggering occurs.

The extent of liquefaction within a soil profile is typically assessed by analysing CPT results using a recognised simplified liquefaction triggering method to obtain a continuous evaluation over the full depth profile of which layers is likely to liquefy, and which is unlikely to liquefy, for a given level of shaking and groundwater level. The extent of liquefaction that is likely to be triggered for a specific soil profile may vary considerably depending on the level of shaking.

The four most commonly used CPT-based methods for assessing liquefaction triggering are Robertson and Wride (1998), Seed et al. (2003) as set out in Moss et al. (2006), Idriss and Boulanger (2008) and Boulanger and Idriss (2014). Each of these methods are empirical relationships developed from liquefaction case histories. The steps used to develop each of these methods are, in general terms, as follows:

1. Identify case history sites with CPT investigations that have clear critical liquefaction susceptible soil layers that are more likely to liquefy relative to the other soil layers in the profile at that site;
2. Estimate the CSR for that critical soil layer. CSR is a function of the levels of ground shaking (Peak Ground Accelerations (PGA) and Magnitude (Mw)) and the Groundwater Depth (GWD) below the ground surface;
3. Calculate the normalised clean sand equivalent CPT tip resistance ($q_{C1NCS}$). $q_{C1NCS}$ is a function of the CPT tip resistance ($q_c$), the GWD, the FC as determined from laboratory testing and soil density;
4 Plot CSR vs. $q_{CINCS}$ for the critical soil layer at each of the case history sites; and

5 Draw an envelope curve that best separates the case histories sites where liquefaction manifestation has occurred from those sites where liquefaction manifestation has not occurred. This envelope curve is generally called the CRR empirical equation.

For each of the steps in the development of these liquefaction triggering methodologies there are uncertainties associated with the collection of the data. These uncertainties include:

- Selection of the critical soil layer;
- Estimation of the PGA and $M_w$ that caused the liquefaction at the case history site;
- GWD at the time of the earthquake shaking that cause liquefaction;
- Measurement error of $q_c$; and
- Estimation of FC.

Therefore, these uncertainties are inherent in each of the liquefaction triggering methods. As these liquefaction triggering methodologies form the basis for the liquefaction vulnerability methodologies (discussed in Section A4) these uncertainties are also inherent in the liquefaction vulnerability methodologies.

Each of the four most commonly used liquefaction triggering methodologies mentioned above are discussed below.

**A3.1.1 Robertson and Wride (1998) – referred to as RW**

The use of the CPTs as a tool for the assessment of liquefaction triggering began in the 1990s. A series of workshops on liquefaction held by the National Centre for earthquake Engineering Research culminated in the Robertson and Wride (1998) paper being adopted in Youd et al. (2001) as the preferred liquefaction triggering analysis method. This method estimates FC using empirical relationships with $I_c$ as derived by the CPT. Normalisation of the CPT data was deterministic, as iterative normalisation was not in widespread use.

**A3.1.2 Moss and Seed (2006) – referred to as MS**

In 2003 Seed et al. adopted developed revised triggering relationships from an extended body of case history data. Critical layers from that database were used by Moss et al. (2006) to develop a CPT-based relationship. This relationship included the probabilistic assessment of critical layers. CPT data was normalised using an iterative procedure.

**A3.1.3 Idriss and Boulanger (2008) – referred to as IB-2008**

In 2008 Idriss & Boulanger presented their liquefaction triggering method in their Earthquake Engineering Research Institute monograph. The primary advantage of this method over the previous two methods is that it allowed for the correction of FC from site specific laboratory test results. This means that these FC-$I_c$ profiles can be tailored to suit specific site conditions rather than being solely dependent on an empirical relationship derived from a limited case history database.

**A3.1.4 Boulanger and Idriss (2014) – referred to as BI-2014**

In 2014 Boulanger & Idriss updated the IB-2008 liquefaction triggering methodology. This update included the following main changes to the assessment methodology:

- **Clean Sand Equivalent Correction** – The FC is a means of characterising how much silt is in a soil. Soils with higher FC are generally siltier and have a higher resistance to liquefaction than soils with lower FC with and equivalent value of $q_c$ and hence require a greater seismic
demand before they will liquefy. For IB-2008 the relationship between FC and CRR was derived empirically based on case histories of liquefying and non-liquefying soils. Recent work in this area, including 50 case histories based on the CES has been incorporated into this empirical relationship for BI-2014. Therefore, the revised clean sand equivalent correction as a function of FC is believed to represent an improved correlation to Canterbury soils. These changes have a minor impact on the prediction of liquefaction triggering in sandy soils but a more significant impact in silty soils with higher FC compared to the IB-2008 method.

- **Cyclic Resistance Ratio (CRR)** – CRR is a measure of the soils resistivity to seismic demand. Soils with a higher CRR indicate a greater resistance to liquefaction and hence require stronger levels of shaking before they will liquefy. Minor revisions to the way CRR is calculated have resulted in BI-2014 predicting slightly higher values of CRR for loose soils and slightly lower values of CRR for dense soils. Note, these changes only have a minor impact on the prediction of liquefaction triggering compared to the IB-2008 method.

- **Magnitude Scaling Factor (MFS)** – The MFS is used to account for earthquake duration effects on the triggering of liquefaction. In the IB-2008 MFS relationship, a single formula was developed for non-cohesive soil material. The revised MFS relationship now has a different relationship for loose soils and dense soils. For loose soils the IB-2008 MFS relationship was under estimating the effect of shorter duration smaller magnitude earthquakes (and hence under estimating the triggering of liquefaction at smaller magnitude earthquakes) whereas for dense soils the IB-2008 MFS relationship was over estimating the effect of shorter duration smaller magnitude earthquakes (and hence over estimating the triggering of liquefaction at smaller magnitude earthquakes).

In addition to these main changes to the methodology, Boulanger and Idriss (2014) provided a recommended FC-Ic correlation that can be calibrated for areas based on laboratory test data.

**A3.1.5 Comparison of Liquefaction Triggering Methods**

Van Ballegooy et al. (2015b) compared the four most commonly used liquefaction triggering methodologies by analysing the CPTs available in the CGD with the LSN liquefaction vulnerability parameter. The LSN parameter was adopted because van Ballegooy et al. (2014b) demonstrated that it provides the best fit of the available liquefaction vulnerability parameters for the land damage observations during the CES (discussed further in Section A4).

In general for each of the liquefaction triggering methods areas with high LSN values spatially correlate well with areas of more severe land damage observations and areas of low LSN values spatially correlate well with areas of less severe land damage observations.

Figure A3.1 compares the mapped liquefaction observations for the September 2010, February 2011 and June 2011 earthquake events with the estimated LSN values for each of the four main liquefaction triggering methodologies. The estimated LSN values for IB-2008, MS and RW have been presented as LSN difference maps from the BI-2014 LSN map. This has been done to accentuate the differences between each of the liquefaction triggering methodologies.
Figure A3.1: Map series of liquefaction severity observations and estimated LSN for all CPTs available in the CGD using the BI-2014, IB-2008, MS and RW simplified liquefaction evaluation methods for the September 2010, February 2011 and June 2011 earthquake events. PGA contours (Bradley and Hughes 2012) are overlaid on the liquefaction severity observation maps. Difference maps are shown between BI-2014 and the other three methods (IB-2008, MS and RW) to accentuate the differences between the LSN maps. Positive values on the difference maps indicate areas where BI-2014 predicts higher values and negative values indicate areas where BI-2014 predicts lower values. Figure reproduced from van Ballegoooy et al. (2015b).
The key differences from Figure A3.1 as discussed in van Ballegooy et al (2015b) are as follows:

- Of the assessed methods BI-2014 provides the best spatial correlation with the land damage observations for the September 2010, February 2011 and June 2011 earthquake events;
- The IB-2008 method tends to produce slightly higher LSN values in western parts of Christchurch and slightly lower LSN values in eastern parts of Christchurch but overall it is generally consistent with BI-2014;
- Of the assessed methods, MS is least aligned to the spatial distribution of the land damage observations; and
- The RW method is not as well aligned as the BI-2014 method to the land damage observations for each event. In particular this applies to the February event where it significantly under predicts liquefaction in the eastern parts of Christchurch.

Further analysis of this data with frequency histograms and summary statistics of estimated LSN values for each of these events is available in van Ballegooy et al. (2015b). These analyses were categorised into the three liquefaction land damage observation categories shown on Figure A3.1.

The key differences between each of the four liquefaction triggering methodologies were as follows:

- The MS method provided the best calibration for the September 2010 event however it was the least consistent for the other events;
- The IB-2008 method and RW method provided comparable results; and
- The BI-2014 method provided the best fit to the mapped liquefaction induced land damage for the regional prediction of liquefaction triggering for the Christchurch soils.

In summary from this comparison of the liquefaction triggering methods it has been concluded that while each of the methods provides reasonable correlations with the land damage observations, the BI-2014 method is best suited for the Christchurch soils.

### A3.2 Input Parameters for Boulanger & Idriss (2014)

In order to undertake an assessment of liquefaction triggering using the BI-2014 methodology it is necessary to adopt default input parameters. The parameters and the associated values that have been adopted for the assessment of liquefaction triggering in the Christchurch are listed in Table A3.1.
### Table A3.1: Input Parameters for Boulanger & Idriss (2014)

<table>
<thead>
<tr>
<th>Input parameter</th>
<th>Default value adopted</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil Density</td>
<td>18 kN/m³</td>
<td>Not sensitive to the typical variability in soil density in Christchurch (Tonkin &amp; Taylor, 2013)</td>
</tr>
<tr>
<td>FC - $I_c$ correlation</td>
<td>$C_{FC} = 0.0$</td>
<td>Appropriate upper bound value for Christchurch soils (Lees, van Ballegoooy, &amp; Wentz, 2015)</td>
</tr>
<tr>
<td>$I_c$ - cutoff</td>
<td>$I_c$ cutoff = 2.6</td>
<td>Appropriate value for Christchurch soils (Lees, van Ballegoooy, &amp; Wentz, 2015)</td>
</tr>
<tr>
<td>Level of earthquake shaking</td>
<td>$M_w = 6.0$, $PGA = 0.3g$</td>
<td>Critical case for 100 year return period levels of earthquake shaking using the BI 2014 methodology</td>
</tr>
<tr>
<td>Probability of Liquefaction ($P_l$)</td>
<td>$P_l = 15%$</td>
<td>Based on standard engineering design practice</td>
</tr>
</tbody>
</table>
| Depth to Groundwater (GWD)       | Surrogate median groundwater surface | Based on the GNS groundwater model (van Ballegoooy, et al., 2014a) Two key assumptions associated with GWD are:  
  - The groundwater profile is hydrostatic below the groundwater surface; and  
  - The soils are fully saturated below the groundwater surface. |

Each of the input parameters listed in Table A3.1 and the reasoning for why the associated values have been adopted are discussed in further detail below. Unless otherwise stated, these values have been adopted as default parameters for the liquefaction triggering analyses in this document.

#### A3.2.1 Soil Density

Robertson and Cabal (2010) presented a correlation between the normalised CPT tip resistance, friction ratio ($f_r$) and soil density. To consider whether or not implementing the correlation in Christchurch soils would be useful, Tonkin & Taylor (2013) assessed the sensitivity of LSN, using the IB-2008 liquefaction triggering methodology, to variations in soil density.

This work demonstrated that the LSN using IB-2008 was not sensitive to variations in soil density that can be reasonably expected within the soils in the Christchurch area. The BI-2014 liquefaction triggering methodology accounts for soil density in the same manner as the IB-2008 methodology. Therefore, these results are also applicable to the BI-2014 methodology and as such it is appropriate to adopt a default soil density value of 18 kN/m³ for all soils in Christchurch.

#### A3.2.2 Fines Content Correlations with $I_c$

As discussed in Section A3.1.4, for BI-2014 the relationship between FC and $I_c$ was derived empirically based on case histories (including 50 case histories in Christchurch) of liquefying and non-liquefying soils. The general relationship derived is expressed with the following equations:

$$I_c = \frac{(FC + 137)}{80} + \varepsilon$$

Where $\varepsilon$ is an error term which has a mean of 0 and a standard deviation of 0.29 based on the case history database used. This equation is then modified and rearranged for the estimation of FC from CPT data.

$$FC = 80(I_c + C_{FC}) - 137$$

where $0\% \leq FC \leq 100\%$

A new fitting parameter, $C_{FC}$ is introduced which is used to calibrate the basic equation to site specific conditions. On an individual site scale, the $C_{FC}$ should be determined for individual
geological units. A targeted laboratory testing regime can facilitate the development of a specific FC-Ic relationship for a given unit which still maintains the general shape of the mean curve (i.e. $C_{FC} = 0$).

Lees et al. (2015) used laboratory test data and CPTs from the CGD to investigate the correlation between FC and Ic and how this data conforms to the empirical relationship with the $C_{FC}$ fitting parameter. Figure A3.2(a) and Figure A3.2(b) compare the international case history database with the Christchurch data. Figure A3.2 (b) indicates that the majority of FC and Ic points for the Christchurch data plot below the $C_{FC} = 0$ line. This observation is supported by the cumulative frequency graphs in Figure A3.2 (c) which show that 49% of the back-calculated $C_{FC}$ values are below a value of 0.2.

Lees et al. (2015) concluded that for the regional Christchurch dataset, the best fit $C_{FC}$ is approximately 0.2. This means that adopting a $C_{FC}$ of 0 provides an appropriate upper bound for the assessment of liquefaction triggering in Christchurch.

Figure A3.2: (a) Plots of RW and BI-2014 FC-Ic correlations overlaid on the international liquefaction case history data. (b) FC vs. median Ic for data in the Christchurch area with the BI-2014 FC-Ic correlations with a $C_{FC}$ of -0.29, 0 and 0.29 overlaid. (c) Percentage of the FC-Ic dataset below the BI-2014 FC-Ic correlation for a varying site specific fitting parameter, $C_{FC}$. Figure reproduced from Lees et al. (2015).

Leeves et al. (2015) investigated the sensitivity of LSN, using the BI-2014 liquefaction triggering methodology and the CPT data available in the CGD, to changes in $C_{FC}$. Figure A3.3 is reproduced from that paper and provides a comparison of variation in $C_{FC}$ and Ic cutoff from the default parameters of $C_{FC} = 0$ and Ic cutoff $= 2.6$ at 100 year return period levels of ground shaking assuming a $P_l = 15\%$ with the surrogate median groundwater surface. Inspection of the middle column of Figure A3.3 shows that LSN is very sensitive to variation in $C_{FC}$. Using a less conservative value of $C_{FC} = 0.2$ typically results in a decrease in LSN of 5 to 10 points.

The sensitivity of LSN to variations in Ic cutoff is discussed in Section A3.2.4.
A3.2.3 Soil Behaviour Type Index (Ic) Cutoff

As discussed in Section A2, the Ic cutoff is used for the assessment of liquefaction susceptibility. Accordingly it is one of the key input parameters for the assessment of liquefaction triggering. As also discussed in Section A2, adopting an Ic cutoff = 2.6 is appropriate for a regional assessment of liquefaction susceptibility and therefore it is an appropriate value to adopt for the CPT-based assessment of liquefaction triggering.

Inspection of the top row of Figure A3.3 shows that in general in eastern parts of Christchurch the LSN parameter is relatively insensitive to changes in Ic cutoff value. This is because most of the soil layers in that area have Ic values less than 2.4. In western parts of Christchurch (in particular to the north and south of the CBD) the LSN parameter is relatively sensitive to changes in the Ic cutoff value. This is because the soil layers in that area typically have Ic values that vary between 2 and 3.

A3.2.4 Level of Earthquake Shaking

Adopting an appropriate level of earthquake shaking is a critical assumption in the assessment of liquefaction triggering because, as discussed in Section A3.1, the extent of liquefaction that is likely to be triggered for a specific soil profile may vary considerably depending on the level of shaking.

The MBIE Guidelines for rebuilding in Canterbury (MBIE, 2012) recommend the use of PGA values of 0.13g and 0.35g for Serviceability Limit State (SLS) and Ultimate Limit State (ULS) design cases respectively with a Mw of 7.5. This applies when undertaking liquefaction triggering analysis on deep or soft soil site (Class D).

Work undertaken by van Ballegoooy at al. (2015c) has demonstrated that in Christchurch soils the critical magnitude for the assessment of liquefaction triggering using BI-2014 is Mw = 6.0 for PGA with equivalent return period levels of shaking compared to the Mw = 7.5 case. As a result of this
work MBIE (2014) recommended that when using the BI-2014 liquefaction triggering method liquefaction assessments at SLS should also be undertaken at 0.19g $M_w$ 6.0.

In addition to this an update to the MBIE guidelines (MBIE, 2015) recommended undertaking a sensitivity check at an Intermediate Level of Shaking (hereinafter referred to as ILS) – nominally at 100 year return period levels of ground shaking. In Christchurch, this level of ground shaking is 0.3g for $M_w$ = 6.0 relative to MBIE (2014) specified SLS and ULS levels of earthquake shaking.

Table A3.2 summarises the different PGA for SLS, ILS and ULS design cases.

**Table A3.2: Summary of seismic demands for liquefaction triggering analysis in the Canterbury earthquake region**

<table>
<thead>
<tr>
<th>Design Case</th>
<th>Annual Probability of Exceedance</th>
<th>PGA at $M_w$ = 7.5</th>
<th>PGA at $M_w$ = 6.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>SLS</td>
<td>1 in 25 years</td>
<td>0.13g</td>
<td>0.19g</td>
</tr>
<tr>
<td>ILS</td>
<td>1 in 100 years</td>
<td>0.20g</td>
<td>0.30g</td>
</tr>
<tr>
<td>ULS</td>
<td>1 in 500 years</td>
<td>0.35g</td>
<td>0.52g</td>
</tr>
</tbody>
</table>

The spatial distribution of where liquefaction triggering is predicted anywhere in the top 10m of the soil profile for SLS, ILS and ULS levels of earthquake shaking is shown in Figure A3.4. It is noted that prediction of liquefaction triggering could only be made in the areas where a sufficient density of CPT data is available (i.e. the red and blue areas).

The maps indicate that liquefaction is predicted to be triggered (somewhere in the soil profile) over a large proportion of Christchurch even at a SLS levels of shaking. For ULS levels of shaking (similar to the level of shaking which was experienced in much of the centre and south of the city in the February 2011 event), some portion of the soil profile is predicted to liquefy virtually over the whole of the Christchurch area where CPT investigations have been undertaken.

**Figure A3.4: Maps showing areas where liquefaction triggering is predicted for SLS, ILS and ULS levels of earthquake shaking assuming an Ic cutoff = 2.6, $C_{FC} = 0$ and $P_L = 15%$.**

### A3.2.5 Probability of Liquefaction ($P_L$)

The BI-2014 CPT liquefaction triggering methodology also incorporates a probability of liquefaction ($P_L$) parameter which allows the estimation of the likelihood of liquefaction occurring across a range of probabilities as a result of the uncertainty in the estimation of the CRR (as discussed in Section A3.1).

The likelihood of liquefaction triggering for a given seismic demand (PGA and $M_w$) and GWD can now be calculated for 15%, 50% and 85% probability of liquefaction triggering whereas previously only the 15% probability of liquefaction triggering was calculated. Adopting a $P_L$ factor of 15%
indicates the soil has a 85% likelihood of liquefaction being triggered for a given seismic demand (PGA and $M_w$) and GWD ($1 - P_L$). As such it represents a conservative assessment.

When the liquefaction triggering is incorporated into the LSN liquefaction vulnerability parameter, the estimated LSN value at $P_L = 15\%$ can be interpreted as the estimated index value with a 15\% probability of exceedance, $P_L = 50\%$ can be interpreted as a 50\% probability of exceedance and $P_L = 85\%$ can be interpreted as 85\% probability of exceedance for a given seismic demand (PGA and $M_w$) and GWD.

Lacrosse et al. (2015) compared the sensitivity of the LSN parameter to $P_L$ by analysing the CPT database available in the CGD with the LSN liquefaction vulnerability parameter and adopting $P_L = 15\%, 50\%$ and $85\%$. The LSN parameter was adopted because Tonkin & Taylor (2013) and van Ballegooij et al. (2014b) demonstrated that it provides an appropriate was of comparing the $P_L$ with the land damage observations from the CES (LSN is discussed further in Section A4).

Figure A3.5 compares the mapped liquefaction observations for the September 2010, February 2011 and June 2011 earthquake events with the estimated LSN values for $P_L = 15\%, 50\%$ and $85\%$. This figure shows that the observed spatial extent and severity of the land damage from the main CES events is reasonably well captured by the range of estimated LSN values. That is, the map of LSN values at $P_L = 15\%$ provides an upper bound and at $P_L = 85\%$ provides a lower bound of predicted land damage.

Figure A3.5 also shows that the land damage in eastern parts of Christchurch is more aligned with adopting a $P_L = 15\%$ whereas land damage in western parts of Christchurch is more aligned with adopting a $P_L = 85\%$. In general it can be observed that areas indicating a high LSN at $P_L = 15\%$ are areas where there is a higher level of certainty of liquefaction damage occurring.
Lacrosse et al. (2015) concluded that for the areas where the CPT-based liquefaction assessments are over predicting the liquefaction vulnerability relative to the land damage observations, the different $P_L$ curves could help engineers understand the degree of potential over estimation of assessed liquefaction.

Lacrosse et al. (2015) investigated the sensitivity of LSN to varying $P_L$ at SLS, ILS and ULS levels of ground shaking. Figure A3.6 is reproduced from that paper and provides a comparison of LSN values for $P_L = 15\%$, $50\%$ and $85\%$ at SLS, ILS and ULS levels of earthquake shaking.
As anticipated, the difference maps in Figure A3.6 show that the estimated LSN at $P_L = 50\%$ is smaller throughout the whole area for all three ground motions and smaller again for the $P_L = 85\%$ case when compared to the $P_L = 15\%$ case, which is typically adopted in deterministic design-based calculations. The difference is much more significant at the SLS and ILS ground motions compared to the ULS ground motions.

In large parts of the city, the LSN difference for the $P_L = 50\%$ case at SLS is in the order of 10 LSN points. At ILS, the difference is in the order of 2 to 5 points. At ULS, it is between 0 to 2 points in the central and western parts of Christchurch and 2 to 5 points in the eastern Christchurch suburbs. Given that the absolute LSN values at SLS are lower than at ULS, the percentage
difference in LSN between SLS and ULS is even more significant. This is mainly because at larger ground motions the CSR increases and hence the likelihood of liquefaction increases.

A3.2.6 Depth to Groundwater

Adopting an appropriate depth to groundwater is a critical assumption in the assessment of liquefaction triggering because liquefaction can only occur if the soil is saturated. It is standard engineering practice to assume that the soil layers above the groundwater surface are not fully saturated and hence are not evaluated for liquefaction triggering.

Van Ballegooy et al. (2014a) developed groundwater surfaces for the 15th, 50th and 85th percentile conditions in recognition of the fact that the groundwater surface fluctuates both seasonally and inter-annually. Two surfaces were developed for the 50th percentile conditions known as the median and the median surrogate groundwater surfaces. The median groundwater surface only considered groundwater records with 12 months or more of data at the time the surfaces were developed. The median surrogate groundwater surface considered all groundwater records (i.e. included those with less than 12 months of data). Comparison of the two groundwater surfaces with the groundwater records undertaken since the groundwater study was published indicates that the surrogate median groundwater surface is the more appropriate representation of the ongoing post-CES groundwater conditions.

For engineering design purposes it is appropriate to assume critical (or upper bound) cases for groundwater conditions such as the maximum and minimum or 15th and 85th percentile groundwater surfaces (i.e. for design of uplift pressures of basements for large buildings). This is because it is highly likely that these conditions will occur during the design life of the structure that is being designed.

However, for the purposes of liquefaction assessment, the median groundwater surface is adopted because it ensures that the 25, 100 and 500 year return period (SLS, ILS and ULS) assessments for the liquefaction hazard are maintained. For example, if a higher than average groundwater surface (e.g. 85th percentile) were to be adopted in conjunction with a 100 year intensity of earthquake shaking, then the return period for this combination of events would be much greater than the 100 year return period.

The groundwater level in Canterbury varies naturally from season to season, and from year to year. This means that the liquefaction vulnerability (which can be predicted using the LSN parameter) also fluctuates above and below the median value. However, liquefaction vulnerability responds more to a rise in the groundwater level than it does to a lowering in the level. This means that the variability in the liquefaction vulnerability (modelled using the LSN parameter) over time is not equally distributed about the median.

Also, the presence of layers of non-liquefiable soils within the range of groundwater fluctuation will affect the distribution of LSN over time. Liquefaction assessments can incorporate this non-uniform variation in vulnerability to calculate a mean value of LSN over time (Lacrosse, van Ballegooy, & Bradley, 2015) for a given return period level of earthquake shaking. However, this is not a straightforward exercise and requires the LSN to be estimated at a range of groundwater levels in order to ultimately calculate the mean LSN value. Further discussion about the use of the median and mean LSN for liquefaction vulnerability assessment purposes is provided in Appendix H.

Other key assumptions associated with adopting a groundwater surface for liquefaction triggering analysis are as follows:

- The soils below the groundwater surface are fully saturated – Chaney (1978), Yoshimi et al. (1989), Grozic et al. (1999), Tsukamoto et al. (2002) amongst others have studied the
effects of partial saturation on liquefaction triggering and found that a reduction in the saturation ratio ($S_r$) resulted in an increase in the CRR of soils susceptible to liquefaction (i.e. partially saturated soils require an increased level of shaking to trigger liquefaction). Results from geophysical testing presented in Stokoe et al. (2014) and Wotherspoon et al. (2015) indicate the natural soils in Christchurch are only partially saturated below the groundwater surface. Soils become fully saturated at depths ranging between 0.5 to 2m below the ground water surface in eastern Christchurch and at even greater depths in some areas in western Christchurch.

However, because the degree of saturation below the groundwater surface level is not well understood and is potentially seasonably variable it is most appropriate to conservatively assume full saturation below the groundwater surface level.

- **The pressure profile below the groundwater surface is hydrostatic** – In liquefaction triggering assessments it is typical to assume a hydrostatic groundwater profile however in Christchurch soils this is not always the case. In some areas the groundwater is partially perched resulting in a groundwater pressure profile which is less than hydrostatic. In other areas there are upward pressure gradients (from the underlying artesian aquifer) resulting in a groundwater pressure profile which is greater than hydrostatic.

In either case it is not practical to evaluate these highly localised groundwater conditions at a regional level. Therefore it is reasonable to assume a simple hydrostatic groundwater pressure profile.

### A3.3 Liquefaction Triggering Envelopes

Curves that represent liquefaction triggering envelopes for sandy soils and silty soils for SLS, ILS ULS levels of shaking and GWD at 0.5 and 1.0m using the BI-2014 method are presented in Figure A3.7. Soils with normalised CPT tip resistance ($q_{c1N}$) which are on the left of the liquefaction triggering envelopes shown in Figure A3.7 are likely to liquefy the assessed level of earthquake shaking and GWD. Soils with $q_{c1N}$ values to the right of the liquefaction triggering curves are unlikely to liquefy.

Figure A3.7 demonstrates the significantly higher liquefaction resistance of the silty soils (with the dashed lines) as compared to the clean sands (with solid lines). That is, a soil layer with a $q_{c1N}$ value of 100 atm at 3m depth is unlikely to liquefy if it is silty and has an $I_c = 2.4$ but is likely to liquefy if it is sandy and has an $I_c < 1.8$. 
Liquefaction triggering curves for low various levels of shaking, with sandy and silty soils and GWD of 0.5m and 1m assuming the BI-2014 liquefaction triggering method and with $C_{uc} = 0$ and $P_L = 15\%$.

Trends of increasing resistance to liquefaction in soils with higher strength, silty soils and increasing depth to groundwater are noted.

### A3.4 Liquefaction Triggering Analysis of CPT Profiles

Liquefaction triggering assessments can be visualised through the use of CPT profiles showing which layers are likely to liquefy at a given level of shaking and GWD. Figure A3.8 shows the estimated liquefaction triggering for two CPT profiles for SLS, ILS and ULS levels of ground shaking at the median GWD at each CPT location. Green shading indicates that liquefaction is unlikely, with yellow, orange and red shading indicating liquefaction triggering of $P_L = 15\%$, $P_L = 50\%$ and $P_L = 85\%$ respectively.

Comparison of these two CPTs demonstrates the different response of soil profiles to liquefaction triggering to varying levels of ground shaking. At SLS levels of ground shaking similar levels of liquefaction triggering are predicted for these CPT profiles. For both CPTs relatively thin bands of soil are predicted to liquefy at $P_L$ of more than 50%. At ILS levels of ground shaking CPT_2892 is estimating significantly more of the soil profile resulting in liquefaction triggering than CPT_2360. The majority of the estimated liquefaction triggering in CPT_2892 is for $P_L$ of more than 50%. At ULS levels of ground shaking CPT_2892 is indicating a thick band of liqueifying material at $P_L = 15\%$ whereas CPT_2360 is indicating only isolated lenses of liquefying material.
A4 Liquefaction Vulnerability and Verification

A4.1 Introduction

As demonstrated in Figure A3.4, liquefaction is predicted to be triggered (somewhere in the soil profile) over a large proportion of the areas with CPT investigations within Christchurch at SLS levels of shaking (M6 0.19g). However, when compared with the land damage observations (refer to Figure A3.1) where higher levels of shaking occurred, it can be seen that not all liquefaction triggering results in land damage. This is because there are other factors, such as the thickness of the liquefying soil layers and the thickness of the non-liquefying crust, which influence the vulnerability of land to liquefaction related damage. Therefore, it is necessary to use liquefaction vulnerability parameters which capture these main factors, to estimate vulnerability to liquefaction induced damage in future earthquake events.

Figure A3.8: Liquefaction triggering analysis for two example CPTs, for various strengths of ground shaking. Green shading indicates that liquefaction is unlikely, with yellow, orange and red shading indicating $P_L = 15\%$, $P_L = 50\%$ and $P_L = 85\%$ respectively.
Following the widespread liquefaction damage in Christchurch as a result of the CES, a major focus has been on assessing and developing approaches for evaluating liquefaction vulnerability. A range of liquefaction vulnerability indicators are available for predicting land damage including:

- The Ishihara (1985) Criteria;
- Cumulative Thickness of Liquefaction (CTL);
- One-dimensional volumetric reconsolidation settlement ($S_{1vo}$) using the method of Ishihara and Yoshimine (1992), as incorporated in Zhang et al. (2002);
- Liquefaction Potential Index (LPI) developed by Iwasaki (1978, 1982);
- Ishihara inspired LPI ($LPI_{ish}$) developed by Maurer et al. (2014a); and
- Liquefaction Severity Number (LSN) developed by Tonkin & Taylor (2013).

Sections A4.2 to A4.5 discuss the advantages and disadvantages of each of these liquefaction vulnerability parameters with respect to their application to the soils in the Christchurch area.

Section 0 discusses the assessment of liquefaction vulnerability using the LSN parameter at SLS, ILS and ULS levels of ground shaking.

It is important to note that the uncertainties inherent in the assessment of triggering liquefaction as described in Section A3 above will also apply to the calculation of CPT-based liquefaction vulnerability parameters.

**A4.2 Ishihara (1985) Boundary Curves**

Ishihara (1985) recognised that liquefied soils needed to be of sufficient thickness and close enough to the surface for damaging effects of liquefaction to be expressed on the ground surface. It was identified that surface liquefaction manifestation was prevented with increasing non-liquefying crust thickness, depending on the ratio of non-liquefying surface layer (crust) to the thickness of underlying liquefying material.

Case study observations were divided into two main categories; sites that showed surface expression of liquefaction at the ground surface and sites that did not. Boundary curves were drawn to separate the respective sites and to provide a ratio at which surface manifestation of liquefaction was unlikely to occur. Figure A4.1 is a reproduction of these boundary curves from Ishihara (1985).
Figure A4.1: Relationship between thickness of a liquefying layer, $H_2$ (m) and thickness of a non-liquefiable overlying layer, $H_1$ (m) at sites for which surface manifestation of level-ground liquefaction has been observed. Figure reproduced from Ishihara (1985).

The Ishihara (1985) method provides a useful and simple method for assessing liquefaction induced damage, however it was based on observations for only two earthquakes representing a limited range of ground accelerations. Youd and Garris (1995) extended this concept by considering additional case studies and presented boundary curves for different PGAs. Both papers showed that there was a critical thickness of the upper non-liquefied material surface layer beyond which the ground surface manifestation of liquefaction was unlikely to occur regardless of the thickness of the underlying liquefied soil. Both papers considered manifestation of ejected material as an indicator of ground damage, however did not consider building damage which may still occur where there is no manifestation. Further case histories supporting the Ishihara curves are presented in Juang (2005).

The available CPTs from the CGD have been applied to the Ishihara (1985) Criteria. However, unlike the simple soil profiles from which the Ishihara (1985) Criteria have been developed, Christchurch soil materials do not typically divide into two discrete units of a non-liquefying layer over a liquefying layer. Accordingly, the non-liquefying crust thickness ($H_2$) was plotted against the cumulative thickness of liquefying materials in the soil profile as a proxy for the liquefying layer ($H_2$). The results of these analyses showed that there is no clear division between those sites which were or were not affected by liquefaction based on a visual inspection of liquefaction and foundation damage (Tonkin & Taylor, 2013; van Ballegooy, et al., 2015a).

It is concluded that the Ishihara criteria is not the most suitable indicator of liquefaction damage observed in Canterbury, due to the difficulty of representing the interbedded soil profile as two simple layers (van Ballegooy et al., 2015a).

Due to the difficulty of implementing the Ishihara (1985) procedure for the soil profiles in Christchurch, studies have been undertaken by Tonkin & Taylor (2013), Maurer et al. (2014; 2014;
2015) and van Ballegooy et al. (2014b; 2015b; 2015c) to assess and/or develop CPT-based liquefaction vulnerability parameters for use in evaluating the severity of surficial liquefaction manifestations in Christchurch. The CPT-based liquefaction vulnerability parameters are discussed in Section A4.3.

### A4.3 CPT-based Liquefaction Vulnerability Parameters

Extensive studies have been undertaken on assessing the vulnerability of land to liquefaction damage. Tonkin & Taylor (2013) and van Ballegooy et al. (2014b) show that liquefaction triggering of soil layers more than 10m below the ground surface provides a negligible contribution to liquefaction damage at the ground surface. Hence the liquefaction vulnerability studies by Lacrosse et al. (2015), Lees et al. (2015), Leeves et al. (2015) and van Ballegooy et al. (2014b, 2015a; 2015b; 2015c) all assess the liquefaction triggering in the upper 10m of the soil profile only.

#### A4.3.1 Cumulative Thickness of Liquefaction (CTL)

CTL is the total thickness of soil layers that are predicted to liquefy during a seismic event.

\[
CTL = \int_0^z (FS < 1) \, dz
\]

It is a useful gauge for broadly assessing vulnerability to liquefaction and can be used as a sensibility check for the other CPT-based liquefaction vulnerability parameters.

For example, if a high value is estimated for a vulnerability parameter (e.g. \( S_{VID} \), LPI, LPI\(_{ISH} \) or LSN) then the CTL value can be used as a sense check. If the CTL value is also high then this indicates a reasonable thickness of soil is predicted to liquefy which in turn supports high values of the other parameters. But if the CTL value is low then the vulnerability calculation should be reviewed to understand why high values are being predicted from only a thin layer of soil being predicted to liquefy (e.g. very shallow ground water levels).

#### A4.3.2 One-dimensional Volumetric Reconsolidation Settlement (\( S_{VID} \))

Volumetric reconsolidation occurs when granular soils are shaken down into a more compact arrangement. Reconsolidation strains can be estimated using the method of Ishihara and Yoshimine (1992) as incorporated in Zhang et al. (2002). The \( S_{VID} \) parameter (also referred to as “calculated settlement”) is defined as:

\[
S_{VID} = \int_0^z \varepsilon_v \, dz
\]

Where \( \varepsilon_v \) is the calculated volumetric strain and \( z \) is the depth below the ground surface.

The method correlates a factor of safety (FS) against liquefaction with relative density (based on \( q_{CINCS} \)) to generate \( \varepsilon_v \). The Zhang et al. (2002) method predicts volumetric strain in layers where the liquefaction FS is less than 2. The \( S_{VID} \) indicator increases as the FS drops and the material approaches a liquefied state.

The MBIE (2012) guidelines recommends the use of the \( S_{VID} \) parameter for determining appropriate foundation solutions for properties with liquefaction susceptible soil deposits where liquefaction triggering is predicted in the soil profile at SLS and ULS levels of earthquake shaking. Sections 5 and 15 of the MBIE (2012) guidelines provide criteria for the range of foundation solutions that can be applied on TC3 properties based on the \( S_{VID} \) parameter at SLS and ULS levels of earthquake shaking. These criteria are summarised in Table A4.1.
Table A4.1: MBIE (2012) site criteria based on SLS and ULS calculated $S_{VID}$

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Foundation Solution Robustness</th>
<th>SLS Calculated $S_{VID}$ (mm)</th>
<th>ULS Calculated $S_{VID}$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TC1 criteria</td>
<td>Low</td>
<td>&lt; 15</td>
<td>&lt; 25</td>
</tr>
<tr>
<td>TC2 criteria</td>
<td></td>
<td>&lt; 50</td>
<td>&lt; 100</td>
</tr>
<tr>
<td>TC3 criteria</td>
<td>TC3 hybrid criteria</td>
<td>&lt; 50</td>
<td>&gt; 100</td>
</tr>
<tr>
<td>TC3 criteria</td>
<td>TC3 SLS&lt;100 mm criteria</td>
<td>&lt; 100</td>
<td>n/a</td>
</tr>
<tr>
<td>TC3 criteria</td>
<td>TC3 SLS&gt;100 mm criteria</td>
<td>&gt; 100</td>
<td>n/a</td>
</tr>
</tbody>
</table>

The guidelines recommend that the thresholds in provided in Table A4.1 should be applied to the $S_{VID}$ calculated from the liquefying soil layers in the top 10m of the soil profile.

The MBIE (2012) guidelines uses $S_{VID}$ for its foundation site criteria thresholds on the basis that the differential ground surface subsidence causing foundation damage is likely to be proportional to the predicted vertical settlement (i.e. $S_{VID}$). This means that more robust foundation solutions are targeted to areas where higher $S_{VID}$ values are calculated.

Studies were undertaken to assess the correlation of the calculated $S_{VID}$ at each CPT location for the September 2010, February 2011 and June 2011 earthquake events with the corresponding estimated liquefaction related ground surface subsidence (Tonkin & Taylor, 2013).

This study showed that the $S_{VID}$ parameter has a weak correlation to liquefaction related ground surface subsidence estimated from aerial LiDAR surveys. Figure A4.2 is a modified version of a similar figure in Tonkin & Taylor (2013) and demonstrates these weak correlations. Possible reasons for this include contributions from sand ejecta and lateral spreading and topographic releveling which are not considered, limitations in LiDAR accuracy, uncertainty in regional tectonic movement, and uncertainty in PGA (Tonkin & Taylor, 2013). It is also important to note that the $S_{VID}$ parameter does not take crust thickness into account, and has a low sensitivity to the groundwater level.
Figure A4.2: Relationship between $S_{VD}$ and the estimated liquefaction related ground surface subsidence for the September 2010, February 2011 and June 2011 earthquake events. This figure is a modified version of a figure presented in Tonkin & Taylor (2013). The main modification is an updated liquefaction triggering methodology from IB-2008 to BI-2014 for the calculation of $S_{VD}$ in accordance with the MBIE (2014) guideline recommendations.

While the $S_{VD}$ parameter does not correlate well with estimated ground surface subsidence it does appear to have a better correlation with observed land damage (Tonkin & Taylor, 2013; van Ballegooij et al., 2015c). In particular the results show that areas with higher calculated $S_{VD}$ values for the September 2010, February 2011 and June 2011 earthquake events shows some correlation with the areas with more severe land damage. Accordingly in Christchurch the $S_{VD}$ values work better as an index for liquefaction related damage at the ground surface rather than a predictor of liquefaction related ground surface subsidence.

The advantages and disadvantages of the $S_{VD}$ liquefaction vulnerability parameter compared with other CPT-based liquefaction vulnerability parameters are summarised in Table A4.2.
A4.3.3 Liquefaction Potential Index (LPI)

LPI is a liquefaction vulnerability parameter proposed by Iwasaki et al. (1978; 1982). LPI uses the FS from liquefaction triggering as well as a depth based weighting function and is defined as:

\[ LPI = \int_{0}^{20m} F_1 W(z) \, dz \]

Where \( W(z) = 10 - 0.5z \), \( F_1 = 1 - \text{FoS} \) for \( \text{FoS} < 1.0 \), \( F_1 = 0 \) for \( \text{FoS} > 1.0 \) and \( z \) is the depth below the ground surface in metres.

The LPI parameter assumes that the severity of liquefaction manifestation is proportional to the thickness of a liquefied layer; the amount by which FS is less than 1.0; and the proximity of the layer to the ground surface. LPI can range from 0 for a site with no liquefaction potential to a maximum of 100 for a site where FS = 0 over the entire 20m depth. The LPI parameter assumes that each liquefying soil layer contributes to some extent to the damage potential at the ground surface. The shallower and/or thicker these layers are, the greater their potential contribution to damage.

Studies were undertaken by Tonkin & Taylor (2013) and van Ballegooy et al. (2014b; 2015c) to compare the calculated LPI values for the September 2010, February 2011 and June 2011 earthquakes with the corresponding mapped land damage. Figure A4.3 is a modified version of a figure from van Ballegooy et al. (2015c) which summarises the results.

Figure A4.3 demonstrates that higher estimated LPI values show some spatial correlation with areas with more severe observed land damage but that correlation is not strong. The distribution of estimated LPI for the three land damage observation categories is also different for each event. This indicates that the correlation between LPI and land damage is event specific and produces an inconsistent response across the three events.
Figure A4.3: Maps of liquefaction severity observations (top row) and estimated LPI (second row) for the September 2010, February 2011 and June 2011 earthquake events. PGA contours from Bradley & Hughes (2012) are overlaid on the liquefaction severity observation maps (top row). Histograms of the liquefaction severity observations and their correlation with LPI are shown on the bottom row. This figure is modified from van Ballegoooy et al. (2015c).

The advantages and disadvantages of the LPI liquefaction vulnerability parameter compared with other CPT-based liquefaction vulnerability parameters are summarised in Table A4.2.

**A4.3.4 Liquefaction Potential Index- (Ishihara) – LPI<sub>ISH</sub>**

LPI<sub>ISH</sub> is a new liquefaction severity index that was developed by Maurer et al. (2015) who recognised the limitations of the LPI framework and proposed modifications to better capture the trends in the Ishihara boundary curves to include the influence of the thickness of a non-liquefied cap on the surficial liquefaction manifestations. LPI<sub>ISH</sub> is defined as:

$$LPI_{ISH} = \int_0^{20m} F_1(FS) \frac{25.56}{z} \, dz$$

Where

$$F(FS) = \begin{cases} 
1 - FS & \text{if } FS \leq 1 \cap H_1 \cdot m(FS) \leq 3 \\
0 & \text{otherwise}
\end{cases}$$

and

$$m(FS) = \exp\left(\frac{5}{25.56(1 - FS)}\right) - 1$$

where $H_2$ is defined as the depth of the non-liquefying crust, $z$ is the depth below ground level.
The LPI\textsubscript{ISH} framework accounts for the relative thickness of the liquefied stratum and the non-liquefied layers via the additional criteria on FS where FS ≤1, and a depth weighting factor which is proportional to 1/z (where z is depth), as opposed to being linear in LPI. Specific to the depth weighting factor, in the LPI\textsubscript{ISH} framework shallower liquefied layers contribute more to surficial manifestations than predicted by the LPI framework (van Ballegooiy, et al., 2014c).

Studies were undertaken by van Ballegooiy et al. (2015c) to compare the estimated LPI\textsubscript{ISH} values for the September 2010, February 2011 and June 2011 earthquakes with the corresponding mapped land damage. Figure A4.4 is a modified version of a figure from van Ballegooiy et al. (2015c) which summarises the results.

The findings of these studies on LPI\textsubscript{ISH} were similar to those of LPI. Figure A4.4 demonstrates that higher estimated LPI\textsubscript{ISH} values show some spatial correlation with areas with more severe observed land damage but that correlation is not strong. The histograms show that distribution of estimated LPI\textsubscript{ISH} for the three land damage categories is different for each event. This indicates that the correlation between LPI and land damage is event specific and produces an inconsistent response across the three events.

![Figure A4.4](image-url)
A4.3.5 Liquefaction Severity Number (LSN)

As a result of the detailed studies summarised in Tonkin and Taylor (2013) and van Ballegooy et al. (2014b), it was recognised that the existing $S_{V1D}$ and LPI vulnerability parameters were not appropriate for assessment of liquefaction vulnerability in the Christchurch area. Therefore, a new parameter was required for the assessment of liquefaction vulnerability for the Christchurch ground conditions and the assessment of the increase in liquefaction vulnerability as a result of the ground surface subsidence caused by the CES.

The Liquefaction Severity Number (LSN) is a new liquefaction vulnerability parameter developed by Tonkin & Taylor based on liquefaction damage observations from the CES to reflect the more damaging effects of shallow liquefaction on land and shallow foundations (Tonkin & Taylor, 2013; van Ballegooy, et al., 2014b). It was formulated to provide a better fit to the observed liquefaction induced damage in Christchurch than the existing $S_{V1D}$ and LPI parameters.

LSN is an index parameter which characterises the vulnerability of land to damage due to liquefaction for a given level of shaking and a given groundwater level. The LSN parameter is defined in terms of the calculated $\varepsilon_v$, integrated over the depth of the soil profile containing liquefying layers, with a depth weighting factor.

The LSN parameter is defined as:

$$LSN = 1000 \int_0^z \frac{\varepsilon_v}{z} \, dz$$

Where $\varepsilon_v$ is the calculated volumetric reconsolidation strain in the subject layer from Zhang et al. (2002) and $z$ is the depth to the layer of interest in metres below the ground surface.

LSN gives a larger weighting factor to liquefying soil layers closer to the ground surface compared to liquefying layers at depth as was supported by general observations during the land mapping work, particularly the observation that ejection of liquefied material tended to result in significant differential settlements. It considers the balance between crust thickness and severity of underlying liquefaction. LSN allows the analysis of more complex layered soil profiles such as those found in the Christchurch area. It incorporates the strength of the soil and assesses how severely the soil reacts once it becomes liquefied.

LSN uses the depth weighted calculated volumetric densification strain within soil layers as an indicator for the severity of liquefaction land damage likely at the ground surface. The published strain calculation techniques consider strains that occur where materials have a calculated triggering FS that reduces below 2.0. This means that the LSN begins to increase smoothly as factors of safety fall, rather than when the FS reaches 1.0 (i.e. the point at which liquefaction is triggered). One other aspect of LSN to note is that strains self-limit based on the initial relative density as the FS falls below 2.0, so a given soil profile has a maximum LSN that it tends towards as the PGA increases.

Studies were undertaken by Tonkin & Taylor (2013) and van Ballegooy et al. (2014b; 2015c) to compare the estimated LSN values for the September 2010, February 2011 and June 2011 earthquakes with the corresponding mapped land damage. Figure A4.5 is a modified version of a figure from van Ballegooy et al. (2015c) which summarises the results.

Figure A4.5 demonstrates a good spatial correlation between LSN and the liquefaction severity observations. Areas where high LSN values are estimated correlate well with the areas where moderate-to-severe land damage occurred. Conversely, areas where low LSN values are estimated correlate well with areas where none-to-minor land damage was observed.
The frequency histograms in Figure A4.5 also produce consistent distributions of LSN values for each of the main CES events for the three different land damage groupings (i.e. none-to-minor, minor-to-moderate and moderate-to-severe). This consistent distribution of estimated land damage is a key advantage of the LSN parameter because it demonstrates that for a given estimated LSN value there is the same likelihood of liquefaction vulnerability whether for a small earthquake with a loose soil profile or a large earthquake with a medium dense to dense soil profile.

The advantages and disadvantages of the LSN liquefaction vulnerability parameter compared with other CPT-based liquefaction vulnerability parameters are summarised in Table A4.2.

**A4.4 Comparisons of the CPT-based $S_{V1D}$, LPI and LSN Parameters with the Ishihara (1985) Criteria**

As discussed in Section A4.2, Ishihara plotted observations of the expression of liquefied material at the ground surface using the thickness of the overlying non-liquefying surface layer ($H_1$) or “crust” and the thickness of the underlying liquefied material ($H_2$). These observations were used to define boundary curves that separated those sites where liquefied material was expressed at the ground surface from sites that where it was not. These boundary curves were developed for $M_w$ 7.5 earthquakes at 0.2g, 0.3g and 0.4 to 0.5g levels of earthquake shaking.
The important points to note with the Ishihara (1985) criteria are that:

- Soil profiles which plot to the left hand side of the boundary curve are potentially vulnerable to liquefaction related ground damage; and
- Soil profiles which plot to the right hand side of the boundary curve are not potentially vulnerable to liquefaction related ground damage.

To examine how $S_{V1D}$, LPI and LSN compare with the Ishihara (1985) $H_1 - H_2$ criteria contour plots of $H_1$ versus $H_2$ for each of these parameters were developed. These contour plots are also shown on Figure A4.6. These contour plots were develop by calculating $S_{V1D}$, LPI and LSN values based on the BI-2014 liquefaction triggering method at M6 0.3g. These values were calculated using idealised soil CPT profiles that replicate a simplified two layer soil model. The analysis varied the thicknesses of non-liquefying soil crust layers ($H_1$) and the underlying thicknesses of liquefying soil layers ($H_2$). It was based on a soil with a normalised CPT clean sand equivalent tip resistance ($q_c^{1NCS}$) of 80 atm. This $q_c^{1NCS}$ value represents an appropriate equivalent to the SPT blow count of 10 blows per 300mm used by Ishihara (1985) to define the thickness of the liquefying $H_2$ soil layer.

Comparing the Ishihara (1985) criteria and the $S_{V1D}$ and LPI contours in Figure A4.6 shows that for the range of non-liquefying crust thickness of most importance in the ILV assessment (i.e. $H_1$ ranges between 0.5 to 3m), there is no single $S_{V1D}$ or LPI value which is consistent with the Ishihara (1985) criteria. That is, for the $S_{V1D}$ and the LPI parameters, there are both low and high values plotting on the left hand side of the Ishihara (1985) boundary curve.

In comparison, the LSN parameter is consistent with the Ishihara (1985) boundary curve over the important range (i.e. up to an $H_1$ of 5m). High LSN values plot on the left hand side and low LSN values plot on the right hand side of the boundary curve. Therefore both the LSN parameter and the Ishihara (1985) criteria indicate increasing vulnerability to liquefaction with decreasing non-liquefying ($H_1$) crust thickness and increasing underlying liquefying ($H_2$) layer thickness.

### A4.5 Overall Performance of the CPT-based Liquefaction Vulnerability Parameters for the Christchurch Soil Conditions

Table A4.2 summarises the comparison between the $S_{V1D}$, LPI and LSN CPT-based liquefaction vulnerability parameters, and their advantages and disadvantages specific to the assessment of liquefaction vulnerability in Christchurch soil conditions and the assessment of the increase in liquefaction vulnerability as a result of ground surface subsidence caused by the CES.
### Table A4.2: Liquefaction vulnerability tools (advantages shaded green, disadvantages shaded red)

<table>
<thead>
<tr>
<th></th>
<th>$S_{V1D}$</th>
<th>LPI(^1)</th>
<th>LSN</th>
</tr>
</thead>
<tbody>
<tr>
<td>Agreement with Ishihara (1985) ground damage criteria</td>
<td>No single $S_{V1D}$ value separates vulnerable from non-vulnerable soil profiles.</td>
<td>No single LPI value separates vulnerable from non-vulnerable soil profiles.</td>
<td>Good agreement with Ishihara (1985) criteria over the range of $H_1$ values most critical in the Christchurch area.</td>
</tr>
<tr>
<td>Role of the non-liquefying ($H_1$) crust in land performance</td>
<td>Does not incorporate the importance of the non-liquefying crust ($H_1$).</td>
<td>Gives insufficient weighting to the importance of the non-liquefying crust ($H_1$) when compared with observed land performance in the Christchurch area.</td>
<td>Gives appropriate weighting to importance of surface crust. Alternative depth weighting factors have been assessed, showing the 1/D factor is consistent with observations in the Christchurch area.</td>
</tr>
<tr>
<td>Sensitivity to very shallow groundwater levels</td>
<td>Low sensitivity to very shallow groundwater levels.</td>
<td>Moderately sensitive to very shallow groundwater levels.</td>
<td>Very sensitive (and potentially over sensitive) to very shallow groundwater levels when GWD &lt; 0.5m.</td>
</tr>
<tr>
<td>Response to ground surface subsidence (key criteria for ILV assessment)</td>
<td>Low sensitivity to ground surface subsidence. Predicts very small change in $S_{V1D}$ for the given levels of ground surface subsidence observed in the Christchurch area.</td>
<td>Only moderately sensitive to ground surface subsidence.</td>
<td>Sensitive to ground surface subsidence. Predicts the highest change in liquefaction vulnerability for given levels of ground surface subsidence.</td>
</tr>
<tr>
<td>Contributing soil layers</td>
<td>Includes contribution from soil layers which are approaching liquefaction triggering, but still with safety factor (FS) &gt; 1.</td>
<td>No contribution from soil layers which have a FS &gt;1 even though they may have generated some excess pore-water pressure and are approaching liquefaction triggering.</td>
<td>Includes contribution from soil layers which are approaching liquefaction triggering, but still with FS &gt; 1.</td>
</tr>
<tr>
<td>Treatment of loose versus dense soil layers</td>
<td>Recognises that looser soils cause more severe liquefaction effects such as ejection and ground surface subsidence.</td>
<td>Only partially addresses effect of soil density, through the safety factor.</td>
<td>Recognises that looser soils cause more severe liquefaction effects such as ejection and ground surface subsidence.</td>
</tr>
</tbody>
</table>

\(^1\) The correlations between LPI\(_{ISH}\) and the mapped land damage for the CES events was similar to the LPI correlations (refer to Sections A4.3.3 and A4.3.4). Therefore, only the LPI parameter has been evaluated in this table.
The LSN parameter was found to be the most suitable tool of all the liquefaction vulnerability parameters considered (listed in Section A4.1) for predicting land performance in the Christchurch area.

The LSN parameter combines many of the advantages of the other tools while reducing many of the disadvantages. The key advantages of the LSN liquefaction vulnerability parameter for assessing liquefaction vulnerability in Christchurch and the increase in liquefaction vulnerability as a result of ground surface subsidence caused by the CES, compared to the other parameters are:

- It is better able to analyse the complex layered soil profiles typical across Christchurch;
- It incorporates both the CPT $q_c$ and corresponding CRR of the soil and how severely the soil reacts (i.e. the $\varepsilon_v$) once it becomes liquefied;
- It considers the ratio between the non-liquefying crust thickness and the thickness and severity of the underlying liquefying soil layers;
- It applies greater weighting to the liquefaction of soil layers closer to the ground surface compared to the liquefaction of soil layers at greater depths, consistent with damage observations in Christchurch; and
- It provides the best correlations with the land damage observations from the CES events and results in consistent distributions of estimated LSN values for the none-to-minor, minor-to-moderate and moderate-to-severe land damage categories for different ground shaking intensities.

<table>
<thead>
<tr>
<th><strong>Natural plateau in liquefaction performance</strong></th>
<th>Recognises that increasing ground shaking beyond a certain level does not significantly worsen the flat land liquefaction performance of a soil profile.</th>
<th>Does not recognise natural plateau in liquefaction performance, so does not translate consistently between earthquakes of different shaking intensity.</th>
<th>Recognises that increasing ground shaking beyond a certain level does not significantly worsen flat land liquefaction performance of a soil profile.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Correlation with observed CES land damage</strong></td>
<td>Does not correlate well with the estimated liquefaction related ground surface subsidence. $S_{VD}$ correlates better with the land damage observations.</td>
<td>The distributions of estimated LPI for a given land damage severity varies significantly for each of the main CES events.</td>
<td>Correlates reasonably well with each of the main CES events and results in consistent distributions of estimated LSN for a given severity of land damage (i.e. areas with high LSN values generally correlate with areas with moderate-to-severe land damage and areas with low LSN values with areas with none-to-minor land damage).</td>
</tr>
<tr>
<td><strong>Precedent and other case histories</strong></td>
<td>Limited previous use as a standalone tool for predicting liquefaction consequences. Often needs to be paired with the non-liquefying crust thickness to provide a rational assessment.</td>
<td>Used as a predictive tool in liquefaction hazard studies over past 30 years.</td>
<td>Validated for the Canterbury earthquakes only.</td>
</tr>
</tbody>
</table>
A4.6 Liquefaction Vulnerability at SLS, ILS and ULS Levels of Ground Shaking

The vulnerability for the Canterbury region as indicated by LSN for SLS, ILS and ULS design levels of shaking is illustrated in Figure A.4.7. The higher LSN values at the ILS and ULS levels of ground shaking reflect the increasing extent and severity of predicted liquefaction vulnerability at higher levels of earthquake shaking.

![Figure A.4.7: Maps showing increasing vulnerability predicted by LSN at SLS, ILS and ULS levels of ground shaking](image)

In order to translate the estimated LSN values at the design SLS, ILS and ULS levels of ground shaking into likely land performance, the correlated back calculated LSN values with the land damage observations from the CES can be used. Van Ballegoooy et al. (2015c) showed that the distributions of calculated LSN for each land damage observation grouping was relatively consistent for the September 2010, February 2011 and June 2011 events (refer to the histograms on the third row of Figure A.4.5).

The datasets for these three events were combined into a single frequency bar chart shown in Figure A.4.8. This figure shows that at low estimated LSN values (i.e. blue areas on the maps shown in Figure A.4.7) there is a high likelihood of none-to-minor land damage and a low likelihood of moderate-to-severe land damage. Conversely, Figure A.4.8 also shows that at high LSN values (i.e. the yellow and red areas on the maps shown in Figure A.4.7) there is a low likelihood of none-to-minor land damage and a high likelihood of moderate-to-severe land damage.
Figure A4.8: Frequency bar chart showing the likelihood of none-to-minor, minor-to-moderate and moderate-to-severe land damage for different LSN bands based on correlated back calculated LSN values with the land damage observations from the September 2010, February 2011 and June 2011 events.

A5 Limitations of CPT-based Liquefaction Vulnerability Assessment Methods

While the LSN parameter is the preferred method for CPT-based liquefaction vulnerability assessment in Christchurch and the preferred method for assessing the increase in liquefaction vulnerability as a result of ground surface subsidence due to the CES, it is subject to a range of uncertainties. These uncertainties include:

- Earthquake motion characteristics;
- Geological spatial variability;
- Soil profile complexities;
- Groundwater saturation and pressure complexities; and
- Soil behaviour characteristics.

These uncertainties and complexities cannot be perfectly captured by current investigation and analysis tools. For this reason, liquefaction analysis in engineering practice is based on correlations that aim to err on the side of conservatism. That is, the analysis over-predicts liquefaction triggering more often than it under-predicts. The analysis used to estimate the LSN values face these same challenges, so it cannot fully predict the liquefaction vulnerability of a property in an earthquake.

The result of this uncertainty is that the LSN value does not predict a particular liquefaction consequence in a specified event. Instead, it represents a prediction of a range of possible consequences in a specified event.

If a group of 100 properties were considered that all had calculated LSN values of exactly 20 and they were all subjected to identical earthquake shaking, it is unlikely that the performance of all
100 properties would be identical. It is likely that a range of liquefaction damage would be observed with a few properties with none-to-minor land damage, the majority with minor-to-moderate land damage and some with moderate-to-severe land damage.

If a group of 100 properties with LSN of 40 were considered, it is likely that a greater number of properties would have moderate-to-severe liquefaction related land damage and very few would have none-to-minor liquefaction related land damage when compared to the group with lower calculated LSN values.

As such the LSN parameter, like any other available liquefaction vulnerability assessment tool, should be considered as an indicator of land performance indicating the likelihood of particular levels of liquefaction related damage occurring. Liquefaction analysis cannot provide a black-and-white prediction of the exact level of land damage that will occur. This demonstrates the importance of the application of engineering judgement when considering modelled LSN values as part of a liquefaction vulnerability assessment.

While the LSN parameter is the preferred method for CPT-based liquefaction vulnerability assessment in Christchurch, the CTL and $S_{vd}$ sensitivity to PGA curves can be used to further understand the proportion of the upper soil profile predicted to liquefy at different levels of earthquake shaking and the associated accumulation of volumetric strain which provides an idea of the volume of excess pore water that is likely to be generated. This provides an indication of the differential ground surface settlement that can be expected.

In Christchurch engineering judgement can be applied by manually reviewing the results of the estimated LSN values with reference to the performance of the land during the CES relative to the estimated event specific levels of earthquake shaking for each main CES event.

For example, if the estimated LSN values at the design SLS levels of ground shaking are high but no land damage has been observed during the CES at levels of ground shaking greater than SLS (M6 0.19g), then engineering judgement would be used to over-ride this assessment. This would indicate that the LSN parameter is not appropriately capturing the behaviour of the soil profile and would lead an engineer to base their assessment of liquefaction vulnerability on the land performance during the CES.

Finally, the CPT-based liquefaction vulnerability parameters discussed in this appendix have all been specifically developed to assess the liquefaction vulnerability of “level” ground. These parameters have not been developed to predict lateral spreading damage. While many areas that are vulnerable to lateral spreading also have high LSN values. There are also areas which are not vulnerable to liquefaction that have low LSN values due to a relatively thick and stiff non-liquefying crust. Despite the low LSN values these areas may still be vulnerable to lateral spreading.

A6 References

Boulanger, R. W. & Idriss, I. M. 2014. CPT and SPT based liquefaction triggering procedures. (Report No. UCD/CGM-14/01). Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, CA.


Ministry of Business, Innovation and Employment, 2015. Repairing and rebuilding houses affected by the Canterbury earthquakes - Section 15.3 Updated guidance on site ground improvement, Christchurch.


Appendix B: Forms of Flat Land Damage

- Table B1.1 – EQC Land Damage Categories for the Flat Land Areas in Christchurch
- Figure B1.1 - Maps of the liquefaction related land damage for the four main earthquake events in the CES
- Table B1.2 – Detailed land damage descriptions
- Figures B1.2 to B1.3 – Photos of land with none-to-minor liquefaction related land damage
- Figures B1.4 to B1.11 - Photos of land with minor-to-moderate liquefaction related land damage
- Figures B1.12 to B1.14 - Photos of land with moderate-to-severe liquefaction related land damage
Table B1.1: EQC Land Damage Categories covered by EQC for the Flat Land Areas in Christchurch

<table>
<thead>
<tr>
<th>Type</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Damage that can be easily observed</strong></td>
<td></td>
</tr>
<tr>
<td>Land cracking caused by lateral spreading&lt;sup&gt;1&lt;/sup&gt;</td>
<td>Lateral spreading is the horizontal movement of land, typically toward watercourses. This horizontal movement results in the stretching and cracking of the non-liquefied soil crust overlying liquefied soils. Surface damage can include minor or major cracks in the land and tilting of ground.</td>
</tr>
<tr>
<td>Land cracking caused by oscillation movements&lt;sup&gt;1&lt;/sup&gt;</td>
<td>Backwards and forwards ground movement during earthquake shaking can result in oscillation cracking. Cracks resulting from oscillation are typically minor and isolated.</td>
</tr>
<tr>
<td>Undulating land</td>
<td>Undulating land is caused by the uneven subsidence of the ground surface as a result of topographic re-levelling resulting from the liquefaction of underlying soil layers, the ejection of sand and silt at the ground surface, and the uneven volumetric densification of the underlying liquefied soil layers.</td>
</tr>
<tr>
<td>Local ponding</td>
<td>Local ground surface subsidence or lowering of the land caused by topographic re-levelling resulting from the liquefaction of underlying soil layers, the ejection of sand and silt at the ground surface, and the uneven volumetric densification of the underlying liquefied soil layers. This results in water forming ponds on the ground surface for extended periods following rainfall events in locations where it did not pond before the CES.</td>
</tr>
<tr>
<td>Local ground surface subsidence causing drainage issues</td>
<td>In some areas where residential land has subsided due to topographic re-levelling resulting from the liquefaction of underlying soil layers, the ejection of sand and silt at the ground surface, and the uneven volumetric densification of the underlying liquefied soil layers it has resulted in a decrease in the drainage capacity of the land. This reduction in drainage capacity can be caused by uneven subsidence of the land beneath public services resulting in the water in the drains not flowing properly or the reduced depth to groundwater decreasing the soakage capacity of the ground.</td>
</tr>
<tr>
<td>Groundwater springs</td>
<td>New groundwater springs have emerged and are now flowing over the ground surface where this was not happening before the CES. The spring usually occurs at a specific location on residential land.</td>
</tr>
<tr>
<td>Inundation by ejected sand and silt</td>
<td>Water, sand and silt is ejected to the ground surface through cracks and penetrations in the non-liquefying crust. The ejected sand and silt may be deposited in isolated mounds, or over large areas on the residential properties including under the dwellings.</td>
</tr>
<tr>
<td><strong>Damage involving an increased vulnerability as a result of ground surface subsidence</strong></td>
<td></td>
</tr>
<tr>
<td>Increased liquefaction vulnerability (ILV)</td>
<td>Throughout Christchurch the ground surface has subsided but the groundwater table has typically remained at the same level. Therefore, the ground surface in some areas is materially closer to the groundwater table than prior to the CES. In some areas this reduces the non-liquefying crust thickness (depending on ground conditions). As a result there has been a material increase in the future liquefaction vulnerability of some residential properties.</td>
</tr>
<tr>
<td>Increased flooding vulnerability (IFV)</td>
<td>Throughout Christchurch the ground surface has subsided. As a result, there has been a material increase in the future vulnerability to flooding of some sites situated near waterways and in overland flow paths.</td>
</tr>
</tbody>
</table>

<sup>1</sup> Cracking of the non-liquefying crust also results in increased liquefaction vulnerability. However once the cracks are repaired in accordance with the repair methods listed in the Guide to the Settlement of Canterbury Flat Land Claims (EQC, 2013) then the crust integrity will be reinstated back to the pre-CES
crust integrity and hence pre-CES liquefaction vulnerability. This guide is available at
http://www.eqc.govt.nz/canterbury-earthquakes/land-claims/guide-to-settlement-of-cantebury-flat-
land-claims.

Figure B1.1: Map showing the inferred levels of earthquake shaking and the observed land damage for
urban residential properties in Christchurch after the (a) 4 September 2010, (b) 22 February 2011, (c) 13
June 2011 and (d) 23 December 2011 earthquakes

In Figure B1.1 the contour lines for the June 2011 and December 2011 are the estimated PGA
contour lines for the main earthquake events on those dates. These do not capture the influence
of the PGAs associated with the foreshocks of these events which are relevant to the liquefaction
related damage observed. This is discussed in more detail in Section 3.7.3 of the report.
<table>
<thead>
<tr>
<th>Simplified land damage categories</th>
<th>Land damaged observation categories in Appendix K</th>
<th>CRITERIA / DESCRIPTION</th>
</tr>
</thead>
</table>
| None to Minor                    | None Observed                                   | • No observed cracks, undulation/deformations at the ground surface, and,  
|                                  | Minor                                           | • No signs of ejected liquefied material at the ground surface, and,  
|                                  |                                                 | • No apparent lateral movement. |
| Minor to Moderate                | Moderate                                        | • Minor to moderate quantities of ejected liquefied material on ground surface (generally <25% of site covered with ejected material), and/or,  
|                                  |                                                 | • Small cracks from ground oscillations (<50 mm) may be present, but little to no vertical displacement across cracks, and,  
|                                  | Major                                           | • No apparent lateral movement. |
| Moderate to Severe               | Severe                                          | • Large quantities of ejected liquefied material on ground surface (generally >25% of site covered with ejected material), and/or,  
|                                  |                                                 | • Severe observed ground surface subsidence, and/or,  
|                                  |                                                 | • Small cracks from ground oscillations (<50 mm) may be present, but little to no vertical displacement across cracks, and,  
|                                  |                                                 | • Limited evidence of lateral movement. |
| Moderate to Severe               | Very Severe                                     | • Moderate to major lateral spreading (<1 m cumulative), and/or,  
|                                  |                                                 | • Large cracks extending across the ground surface, with horizontal and/or vertical displacement (>50 mm, but generally <200 mm), and,  
|                                  |                                                 | • Ejection of liquefied material at the ground surface may also be observed. |
|                                  |                                                 | • Extensive lateral spreading (≥1 m cumulative), and/or,  
|                                  |                                                 | • Large open cracks extending through the ground surface, with very severe horizontal and/or vertical displacements (≥200 mm), and,  
|                                  |                                                 | • Ejection of liquefied material at the ground surface may also be observed. |
Figures B1.2 to B1.3: Photos of land with none-to-minor liquefaction related land damage.
None to minor land damage

Even lawns and undamaged pavers

Undamaged asphalt driveway

Undamaged concrete driveway and kerbing

Aerial shot of Broomfield area showing no liquefaction ejecta on the roads or Broomfield Common

View of Aston Drive with little damages

View of Centaurus Road with little damage

*Figure B1.2: Photos of land with none-to-minor liquefaction related land damage.*
### None to minor land damage

<table>
<thead>
<tr>
<th>Description</th>
<th>Image</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flat lawn</td>
<td><img src="image1" alt="Flat lawn" /></td>
</tr>
<tr>
<td>Slightly undulating lawn and driveway</td>
<td><img src="image2" alt="Slightly undulating lawn and driveway" /></td>
</tr>
<tr>
<td>Slightly undulating lawn</td>
<td><img src="image3" alt="Slightly undulating lawn" /></td>
</tr>
<tr>
<td>Undulating lawn</td>
<td><img src="image4" alt="Undulating lawn" /></td>
</tr>
<tr>
<td>Undulating lawn</td>
<td><img src="image5" alt="Undulating lawn" /></td>
</tr>
<tr>
<td>Aerial photo of North New Brighton showing little damage</td>
<td><img src="image6" alt="Aerial photo of North New Brighton showing little damage" /></td>
</tr>
</tbody>
</table>

*Figure B1.3: Photos of land with none-to-minor liquefaction related land damage.*
Figures B1.4 to B1.11: Photos of land with minor-to-moderate liquefaction related land damage.
Figure B1.4: Photos of land with minor-to-moderate liquefaction related land damage.
<table>
<thead>
<tr>
<th>Minor to moderate land damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquefaction ejecta around washing line pole</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Minor to moderate land damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foundation damage</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Minor to moderate land damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquefaction ejecta on lawn</td>
</tr>
</tbody>
</table>

Figure B1.5: Photos of land with minor-to-moderate liquefaction related land damage.
Figure B1.6: Photos of land with minor-to-moderate liquefaction related land damage.
Minor to moderate land damage

Liquefaction ejecta at a New Brighton Road junction

Aerial photo of liquefaction ejecta at Cashmere High School

Minor to moderate land damage

Liquefaction ejecta on lawn

Liquefaction ejecta on lawn

Minor to moderate land damage

Liquefaction ejecta on lawn

Isolated area of liquefaction ejecta

Figure B1.7: Photos of land with minor-to-moderate liquefaction related land damage.
Minor to moderate land damage

Damage to asphalt

Liquefaction ejecta on lawn

Minor to moderate land damage

Liquefaction ejecta in piles on road

Undulating paving bricks

Minor to moderate land damage

Isolated area of liquefaction ejecta

Foundation damage

Figure B1.8: Photos of land with minor-to-moderate liquefaction related land damage.
Minor to moderate land damage

Foundation damage

Liquefaction ejecta on lawn

Minor to moderate land damage

Liquefaction ejecta in many places on lawn

Large area of liquefaction ejecta on lawn

Minor to moderate land damage

Foundation and brickwork damage

Liquefaction ejecta and brickwork damage

Figure B1.9: Photos of land with minor-to-moderate liquefaction related land damage.
Figure B1.10: Photos of land with minor-to-moderate liquefaction related land damage.
Figure B1.11: Photos of land with minor-to-moderate liquefaction related land damage.
Figures B1.12 to B1.14: Photos of land with moderate-to-severe liquefaction related land damage
Figure B1.12: Photos of land with moderate-to-severe liquefaction related land damage.
Moderate to severe land damage

Aerial photo showing liquefaction ejecta on Seabreeze Close

Liquefaction ejecta in turning circle of Seabreeze Close

Moderate to severe land damage

Outside of dwelling showing level liquefaction ejecta reached on brickwork and windows

Large amounts of liquefaction ejecta

Moderate to severe land damage

Large amounts of liquefaction ejecta and tilting dwelling

Large amounts of liquefaction ejecta around property

Figure B1.13: Photos of land with moderate-to-severe liquefaction related land damage.
Large amounts of liquefaction ejecta

Large amounts of liquefaction ejecta

Large amounts of liquefaction ejecta inside dwelling

Close up photo of road damage seen in aerial photo of Woolston

s) Foundation damage, tilting dwelling

Figure B1.14: Photos of land with moderate-to-severe liquefaction related land damage.
Appendix C: The Evolution of the ILV Methodology
C1  Purpose and Outline

The purpose of this appendix is to summarise key aspects of the history of the development of the Increased Liquefaction Vulnerability (ILV) Assessment Methodology and in particular:

- The background to the recognition of the phenomenon of increased liquefaction vulnerability as a result of ground surface subsidence in the Christchurch area (Section C2);
- The recognition of ILV as a form of land damage covered under the EQC Act and the development of the ILV Assessment Framework (Section C3); and
- The implementation of the ILV Assessment Framework with the ILV Assessment Methodology (Section C4).

C2  Background to Increased Liquefaction Vulnerability in the Christchurch Area

The earthquake shaking from the Canterbury Earthquake Sequence 2010-2011 (CES) triggered minor-to-severe liquefaction induced ground surface deformations resulting in land damage throughout the Christchurch area. The land damage included liquefaction ejecta, liquefaction-induced differential settlement, and lateral spreading resulting in extensive residential building damage.

It was observed that the majority of the areas affected by severe liquefaction induced land damage coincided with lower lying areas near the Avon River where the groundwater surface is close to the ground surface. Conversely, areas with less liquefaction-induced land damage were typically areas with higher elevation and deeper groundwater levels, indicating a correlation between liquefaction damage and the depth to the groundwater surface and hence the non-liquefying crust thickness.

Comparison of LiDAR survey information taken before and after the CES showed significant ground subsidence occurred as a result of the CES due to liquefaction-induced volumetric densification, liquefaction ejecta, lateral spreading and tectonic subsidence. Of the 140,000 flat land residential properties in Christchurch, approximately 85% have subsided following the CES (Rogers et al., 2015). Of these:

- 60,000 have estimated total ground surface subsidence of more than 0.2m;
- 12,000 have estimated total ground surface subsidence of more than 0.5m; and
- 500 have estimated total ground surface subsidence of more than 1m.

Following the September 2010 and February 2011 events, residents in low lying suburbs were reporting that their land was performing differently during smaller aftershocks relative to the pre-CES performance. However, there was no evidence that the earthquakes were having an effect on the soil strength and stiffness in Christchurch (as discussed in Section 6.5).

It was following the April 2011 aftershock and subsequently the June 2011 event that it became apparent that land performance was deteriorating. The shaking intensity from these events was lower than the September and February events yet the land damage was more severe in certain parts of Christchurch. In the most affected parts of Christchurch, the land was less able to support the weight of buildings than it was prior to the CES. This allowed the houses to partially sink into the ground in subsequent CES earthquakes (Russell et al. 2015). This was particularly noticeable in low lying areas where the greatest amount of subsidence had occurred. It was at this time that the concept of “crust thinning” was first proposed.

Due to the ground subsidence and seasonal increases in groundwater levels, the depth to groundwater during the April 2011 aftershock and June 2011 event was closer to the ground surface than it had been in both the September 2010 and February 2011 events. This formed the basis of a
hypothesis that, in areas where the upper soil layers are susceptible to liquefaction, reduced depth to the groundwater surface due to ground surface subsidence was effectively reducing the thickness of the non-liquefying crust – i.e. “crust thinning” (Russell et al., 2015). In turn this “crust thinning” resulted in increased vulnerability to liquefaction-induced land damage.

In the low lying areas of Christchurch with the most significant levels of ground surface subsidence this hypothesis was consistent with land damage observations. However in areas away from the rivers and less affected by ground surface subsidence, a more complex picture emerged. In these areas, ground surface subsidence did not always correlate with deteriorating land performance through the CES. That is, “crust thinning” was not always directly correlated to change in depth to groundwater.

It became apparent that the complex ground conditions encountered in the Christchurch area were contributing to this complex picture. This is because the thickness of the non-liquefying crust and the thickness of the underlying liquefying soil layers is also dependent on the geological composition of the upper soil layers. The complexity of these ground conditions is demonstrated in Table C2.1 which summarises the geological units encountered in the Christchurch urban area. Areas where the near surface soils below the groundwater table are not susceptible to liquefaction have not experienced “crust thinning” despite having subsided as a result of the CES.

### Table C2.1: Summary of Geological Units in the Christchurch Urban Area

<table>
<thead>
<tr>
<th>Time period and location</th>
<th>Relevant Geological Formation</th>
<th>Description</th>
<th>Liquefaction Implications</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt;10,000 years before present (BP). WEST to EAST.</td>
<td>Riccarton Gravel</td>
<td>The top of the Riccarton Gravel formed the land surface when the sea level was approximately 30m lower than today. The old land surface is 10m above sea level near Christchurch Airport and 35m below sea level at New Brighton, an overall slope of about 1 in 300 towards the east. The Riccarton Gravel formed as an alluvial fan of the braided Waimakariri River.</td>
<td>‘Hard’ base to the Christchurch geology. Extremely low potential for liquefaction.</td>
</tr>
<tr>
<td>10,000 to 6,500 years BP. CENTRAL and EAST.</td>
<td>Base of Christchurch Formation</td>
<td>Sea level rose to the current level and the coast line regressed from east of New Brighton to an arc through Cashmere, Riccarton and Belfast. Fine grained forested coastal soils and estuarine deposits form the capping layer that confines the present day Riccarton Gravel aquifer.</td>
<td>Soils are generally soft and loose; some layers have potential for liquefaction.</td>
</tr>
<tr>
<td>Time period and location</td>
<td>Relevant Geological Formation</td>
<td>Description</td>
<td>Liquefaction Implications</td>
</tr>
<tr>
<td>--------------------------</td>
<td>-------------------------------</td>
<td>-------------</td>
<td>--------------------------</td>
</tr>
<tr>
<td>6,500 to 2,500 years BP. CENTRAL and EAST.</td>
<td>Christchurch Formation</td>
<td>The coast line moved eastward from Riccarton to Wainoni due to the build-up of sand and silt supplied by the Waimakariri River and trapped in coastal currents between Pegasus Bay and Banks Peninsula. Dense to very dense sands are indicative of offshore sand bank and surf beach deposits (as per present day New Brighton), while silt and sand layers with minor organics indicate estuarine and back beach environments similar to the present day estuary. Narrow lenses of gravel deposited by deltas of the prehistoric Avon and Heathcote Rivers persist through the Christchurch Formation in places (e.g. Waltham/St Martins).</td>
<td>Dense to very dense sands often have a low potential for liquefaction and are 5 to 10m thick, commencing a few meters below sea level. Interbedded silt and sand layers and very low plasticity silts can create problems with the assessment of liquefaction and therefore appropriate engineering responses to mitigate liquefaction triggering and consequence are required.</td>
</tr>
<tr>
<td>10,000 years BP to present. WEST.</td>
<td>Yaldhurst Member of the Springston Formation</td>
<td>The alluvial fan of the Waimakariri River has steadily built out onto the old Riccarton Gravel land surface, and then onto the coastal deposits of the Christchurch Formation. Gravel with minor layers of sand and silt form the fan, which slopes at about 1 in 250 towards the east from the airport to Hagley Park. The boundary between Riccarton Gravel and overlying Yaldhurst Member gravel is generally indistinct and the aquifer is unconfined.</td>
<td>Low potential for liquefaction because of the predominantly gravel materials and groundwater some 3 to 10m below surface due to the relatively steeply sloping fan surface.</td>
</tr>
<tr>
<td>2,500 years BP to present. CENTRAL.</td>
<td>Yaldhurst Member of the Springston Formation</td>
<td>The alluvial fan has built out across the coastal plain as ‘fingers’ of medium dense gravel and sand forming flood channels, and flood plains formed by loose overbank silt and sand layers. The channels are self-levée forming and tend to sit above the overall flood plain level. Several low lying areas have been surrounded by flood plain and channel deposits to form wetlands, or historic wetlands, since in-filled, e.g. Hendersons Basin, Spreydon, Riccarton, Papanui/Cranford, St Albans, Marshlands. The combined alluvial and coastal plain from Hagley Park to New Brighton slopes eastward at about 1 in 1500.</td>
<td>High potential for liquefaction in near surface sand and silts. Actual risk of surface damage varies from area to area depending on groundwater depth, which is controlled by relative ground surface elevation (e.g. higher level flood channels and lower lying plains or wetlands). Lenses and layers of gravel deposited in layers a few metres either side of sea level have a generally low risk of liquefaction. Caution is required in assessing rapid changes in gravel thickness across individual development sites. Relatively low liquefaction potential may be present in some low lying areas due to the predominance of plastic fine grained soils through the profile.</td>
</tr>
<tr>
<td>Time period and location</td>
<td>Relevant Geological Formation</td>
<td>Description</td>
<td>Liquefaction Implications</td>
</tr>
<tr>
<td>--------------------------</td>
<td>-------------------------------</td>
<td>-------------</td>
<td>---------------------------</td>
</tr>
<tr>
<td>2,500 years BP to present. EAST.</td>
<td>Christchurch Formation</td>
<td>Eastward progradation of the coast line has continued, depositing dense beach sands, medium dense dunes and loose estuarine deposits Dune deposits from Shirley to Linwood have been eroded by the Avon and Heathcote Rivers and thin layers of sandy alluvium re-deposited in river channels and on flood plains within a few metres of sea level.</td>
<td>High potential for liquefaction in recent near surface coastal and alluvial sand and silt. Groundwater is generally 1 to 2m below surface due to the very flat slope of the coastal plain (e.g. tidal influence on the Avon River extends to Manchester Street). Low potential for liquefaction in dense beach sand, typically 5m below surface.</td>
</tr>
</tbody>
</table>

In various documentation, “crust thinning” was initially referred to as Category 8 (or Cat 8) land damage. The terminology was later amended to Increased Liquefaction Vulnerability (ILV). From this point forward in this appendix it will be referred to as ILV.

**C3 Recognition of ILV Land Damage and the Development of the ILV Assessment Framework**

In early 2012, EQC recognised ILV as a form of natural disaster damage for the purposes of the EQC Act 1993, where it was having a material effect on the use and amenity of the land. The primary challenge arising from the recognition of this form of land damage was how to define and assess the material effects of ILV. The reason for this is that, unlike most other forms of land damage recognised by the EQC (listed in Appendix B), the effects of ILV are not immediately apparent. This is because this form of land damage is dependent upon the occurrence of a future earthquake event in order for the change in vulnerability to be realised.

At this time, it was recognised that there was insufficient information available in order to undertake an assessment of ILV throughout the Christchurch area. As a result of this, EQC commissioned and collated one of the most extensive databases of geotechnical investigation information and land and dwelling performance observations ever assembled. This database and other relevant information has been made publicly available on the Canterbury Geotechnical Database (CGD), which was launched in April 2012. The information sources used in the assessment of ILV are discussed in Section 5.

Concurrently with the collation of the geotechnical investigations and land and building performance information, a detailed literature review of the widely used CPT-based liquefaction vulnerability parameters was undertaken. As a result of this literature review and the analysis of the available geotechnical information, it was recognised that none of these tools were appropriate for the assessment of ILV in the Christchurch area. Therefore, a new liquefaction vulnerability parameter called the Liquefaction Severity Number (LSN) was developed for the assessment of ILV. Refer to Section 7 and Appendix A for further detail about the assessment of liquefaction vulnerability using CPT-based parameters.

Based on the information gathered during 2012 and legal and engineering advice, an initial ILV land damage assessment framework presented in Figure C3.1 was approved by EQC in January 2013. The purpose of this framework was to provide a consistent approach for the assessment of ILV.
The ILV assessment framework was established to consider the engineering considerations associated with the assessment of ILV. Therefore, it can only be used to assess the engineering criteria as discussed in Section 2.4. The engineering criteria are as follows:

- The residential land has a material vulnerability to liquefaction damage after the CES at 100 year return period levels of earthquake shaking (Criterion 1); and
- The vulnerability to liquefaction damage of the residential land in future earthquakes has materially increased at up to 100 year return period levels of earthquake shaking as a result of ground surface subsidence of the land caused by the CES (Criterion 2).

The ILV land damage assessment framework shown in Figure C3.1 did not provide the mechanism to assess criterion 3 – namely that “...any increase in vulnerability to liquefaction damage of the residential land has caused the value of the property (the residential land and associated buildings combined) to decrease.”

C4 Refinement and Development of the ILV Assessment Framework with the ILV Assessment Methodology

Following the approval of the ILV land damage assessment framework by the EQC, the assessment of ILV was operationalised. As the undertaking of an assessment of this nature and scale had never been attempted before, the ILV Assessment process has developed in an iterative manner as assessments were undertaken and understanding of ILV land damage increased. Where this occurred all properties previously assessed were reviewed to ensure consistency with the updated understanding. The ILV Assessment Methodology described in the Report is the final methodology applied to all residential properties in Christchurch.
Initially it was envisioned that an automated approach could be adopted to ILV assessment. The automated ILV model was developed for this purpose. The intention was that the automated model would be used to confirm approximately 90% of the ILV qualifications with the remaining 10% requiring manual review (i.e. the green box in Figure C3.1). However, it became apparent that the automated ILV model was unable to account for the full complexity of ground conditions requiring assessment for ILV (Table C2.1 demonstrates the complexity of the ground conditions in Christchurch).

Accordingly, in July 2013 it was determined that a manual ILV assessment would be required for each of the 139,390 urban residential properties in the Christchurch area with the level of manual engineering assessment on each property being proportional to the complexity of the assessment. The automated ILV model was used as an input into the manual ILV assessment process. Refer to Section 8.2 for more information about the automated ILV model.

The manual assessment of ILV was undertaken using a two stage process. Stage 1 was used to assess straightforward cases and Stage 2 was used to assess more complex cases. The Stage 1 ILV assessments started in June 2013 and were completed by September 2014. The Stage 2 ILV assessments started in February 2015 and were completed by April 2015. Further discussion about the ILV assessment process is provided in Sections 8, 9 and 10.

As discussed in Section 1.4, the ILV Assessment Methodology has been developed in collaboration with a number of other parties with an interest in the ground surface subsidence caused by the CES. A key outcome of this collaborative process was the incorporation of the mean LSN value into the automated ILV model in February 2014. As shown in Figure C3.1, previously the automated ILV model used only the median LSN value. Refer to Appendix H for further discussion about the mean and median LSN.

Another significant change to the ILV Assessment Methodology was the adoption of the Boulanger and Idriss (2014) liquefaction triggering methodology in July 2014. Previously the Idriss and Boulanger (2008) liquefaction triggering methodology was applied, however the April 2014 update to this methodology incorporated 50 case histories from the CES and provided an improved correlation with land damage observations.

Adopting this revision to the liquefaction triggering methodology meant that the indicator LSN and ΔLSN values for the two ILV engineering criteria (refer to Section 2.4) required revision. Originally, using the Idriss and Boulanger (2008) liquefaction triggering methodology, the LSN and ΔLSN indicator values were 20 and 6 respectively. Due to the changes in the Boulanger and Idriss (2014) liquefaction triggering methodology the LSN and ΔLSN values were revised to 16 and 5 respectively. As a result of these revisions the Stage 1 ILV assessment process effectively needed to be started again. Further detail about the differences between the various liquefaction triggering methodologies can be found in Appendix A.

The final refinement relates to the EQC Act’s requirement that EQC must determine whether an insured property has suffered natural disaster damage in each natural disaster event. EQC must therefore be satisfied that a physical change has occurred resulting in a material increase in vulnerability to liquefaction that has affected the amenity and value of the insured property in one or more of the main earthquake events. However, for the reasons listed in Section 2.7.3, the manual assessment methodology developed for the CES can only practically be undertaken by considering ground surface subsidence-induced changes to liquefaction vulnerability across the CES, and then (after the assessment process is completed) considering which individual events are likely to have contributed to that change. Accordingly, the assessment of the effects of a particular event was removed from Step c in Figure C3.1.

For completeness, it is noted that Step b1 (assessment of depth to median groundwater) in Figure C3.1 was not directly applied in final ILV assessment methodology. This is because low LSN values
and low $\Delta LSN$ values are estimated when the depth to groundwater is greater than 3m, which in practice makes Step b1 superfluous. Similarly, the 100mm threshold for non-tectonic subsidence in Step d3 in Figure C3.1 was intended as a filter to determine which properties would require manual review. However, when the decision was made to complete a manual assessment for all ILV properties this step also became superfluous.

The main refinements to the implementation of the ILV eligibility assessment process are summarised in Table C4.1 below.

**Table C4.1: Main refinements to the ILV eligibility assessment process**

<table>
<thead>
<tr>
<th>Feature</th>
<th>Initial framework (Figure C3.1)</th>
<th>Updated assessment process (Figure 4.2 in the Report)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Assessing ILV for a 1 in 100 year earthquake event.</td>
<td>Assess ILV at exactly a 1 in 100 year event (refer to Figure C3.1).</td>
<td>Assess ILV at a 1 in 100 year event, or any more frequent event where a greater change in vulnerability occurs (refer to Figure 4.2 in the Report).</td>
</tr>
<tr>
<td>Manual review process.</td>
<td>90% of ILV eligibility decisions expected to be made via an automated model, with 10% of properties assessed individually by engineers (refer to Figure C3.1).</td>
<td>Every ILV eligibility assessment includes manual review by experienced engineering staff (refer to Figure 4.2 in the Report).</td>
</tr>
<tr>
<td>Interpolation of LSN values between geotechnical investigation locations.</td>
<td>Calculate LSN at single point location on the property only (refer to Figure C3.1).</td>
<td>Calculate LSN over entire ILV assessed area (refer to Figure 4.2 in the Report).</td>
</tr>
<tr>
<td></td>
<td>Interpolate change in LSN based on change in ground elevation at investigation locations (refer to Figure C3.1).</td>
<td>Impose measured subsidence of property being assessed on to surrounding investigations before interpolating (refer to Figure 4.2 in the Report).</td>
</tr>
<tr>
<td></td>
<td>Where investigation included pre-drill (to clear services), incomplete soil profile data skews interpolated LSN values (refer to Figure C3.1).</td>
<td>Overwrite pre-drill layers with known LSN data from surrounding investigations (refer to Figure 8.1 in the Report).</td>
</tr>
<tr>
<td>Natural seasonal and year-to-year variations in the groundwater level.</td>
<td>Assess thresholds using the median LSN value, calculated based on the long-term median groundwater level (refer to Figure C3.1).</td>
<td>Assess both median and mean LSN values, calculated across a wide range of groundwater levels (refer to Section 7.5 in the Report).</td>
</tr>
<tr>
<td>Liquefaction triggering methodology for the automated ILV model.</td>
<td>Automated ILV model applied using Idriss and Boulanger (2008) liquefaction triggering methodology. The original indicator values were as follows:  - $LSN = 20$ for the assessment of Criterion 1; and  - $\Delta LSN = 6$ for the assessment of Criterion 2. (refer to Figure C3.1)</td>
<td>Automated ILV model applied using Boulanger and Idriss (2014) liquefaction triggering methodology. The revised indicator values are as follows:  - $LSN = 16$ for the assessment of Criterion 1; and  - $\Delta LSN = 5$ for the assessment of Criterion 2. (refer to Appendix A)</td>
</tr>
<tr>
<td>Ground surface subsidence assessment case</td>
<td>Assess ILV based on the change in ground surface elevation over the CES and for each individual event (refer to Figure C3.1).</td>
<td>Assess ILV based on the change in ground surface elevation over the entire CES (refer to Section 2.7.3 in the Report).</td>
</tr>
</tbody>
</table>
C5 References


Appendix D: Probabilistic Seismic Hazard Analysis for Christchurch Soil Sites

- Report prepared by Bradley Seismic Ltd.

Seismic hazard analysis for urban Christchurch accounting for the 2010-2011 Canterbury earthquake sequence


Brendon A. Bradley\(^1,2\)

\(^1\)Director, Bradley Seismic Limited
\(^2\)Senior Lecturer, Department of Civil and Natural Resources Engineering, University of Canterbury

7 July 2014
Christchurch, New Zealand
Disclaimer:
This report has been commissioned by Tonkin and Taylor Limited on behalf of the New Zealand Earthquake Commission to provide information on the seismic hazard in Christchurch, New Zealand. The author accepts no liability for the use of the material contained in this document by third parties, and retains the right to utilize the results presented herein elsewhere.
Table of Contents

Executive Summary ................................................................. 4
1. Introduction ........................................................................... 5
2. Canterbury earthquake sequence activity ................................ 5
   2.1. Observed magnitude-frequency distribution ....................... 5
   2.2. Observed spatial distribution .............................................. 6
   2.3. Observed temporal distribution .......................................... 7
   2.4. Predicted future seismicity associated with the Canterbury earthquake sequence 9
   2.5. Modelling Canterbury earthquake sequence seismicity in the NZ ERF ........ 10
3. Probabilistic seismic hazard analysis (PSHA) ............................. 11
   3.1. Adopted earthquake rupture forecast (ERF) models ............... 11
   3.2. Adopted ground motion prediction equations (GMPE) ............. 11
   3.3. Generic site considered .................................................... 11
   3.4. Seismic hazard analysis results .......................................... 12
4. Conclusions ........................................................................... 18
5. References ............................................................................. 19
Executive Summary

This report presents seismic hazard estimates for urban Christchurch based on state of the art methods. The on-going Canterbury earthquake sequence is directly incorporated in the adopted earthquake rupture forecast (ERF) based on a first-principles analysis of the observed earthquake activity rates and empirical models based on the Gutenberg-Richter and modified-Omori aftershock laws.

The seismic hazard analyses performed suggest that the PGA seismic hazard averaged over the next 50 years is 40-50% higher than that prior to the Canterbury earthquake sequence. However, the magnitudes which dominate this seismic hazard are 0.3-0.5 $M_w$ units lower than the pre-2010 hazard.

Using a commonly adopted magnitude scaling factor, $MSF$, magnitude-correct PGA values, $PGA_{7.5}$, are determined from the seismic hazard analysis results in this study. Comparison with the provisional design values in the MBIE guidelines suggests that those values are a factor of 2-3 greater than the values obtained here based on rigorous first-principles analysis, and therefore an urgent reassessment of the MBIE values is warranted.
1. Introduction

The 2010-2011 Canterbury earthquake sequence produced significant ground motion shaking in urban areas of Christchurch, and associated damage. Although the activity rates associated with this sequence have reduced significantly there is still an on-going seismic hazard associated with this sequence that is not adequately accounted for in prior forecasts. The intention of this document is to illustrate the magnitude of the increased seismic hazard resulting from this on-going sequence.

Section 2 of this document examines the size, spatial, and temporal distribution of earthquake activity in the Canterbury earthquake sequence in order to develop a model to forecast the hazard posed by the on-going decay of this sequence over the typical 50 year design life of future structures and infrastructure. Section 3 presents the seismic hazard analyses that are performed using this aftershock sequence model in additional to the pre-2010 knowledge of the seismic hazard posed to the Canterbury region. Section 4 provides an explicit comparison between the seismic hazard values obtained in this study in comparison to the MBIE provisional design guidelines for liquefaction assessment.

2. Canterbury earthquake sequence activity

In this section the size, spatial, and temporal distributions of the Canterbury earthquake sequence are examined in order to develop the necessary models to represent the future predicted seismicity from this sequence into future NZ earthquake rupture forecasts.

In order to examine the earthquake activity from the on-going Canterbury earthquake sequence, the moment tensor catalogue compiled by John Ristau was obtained from GeoNet (www.geonet.org.nz; last accessed 1 July 2014). The analyses performed herein were also undertaken based on the alternative GNS catalogue of event locations and Richter magnitudes, with results found to be comparable. Based on examining the spatial distribution of events within the Canterbury region, geographical bounds of Lat=[-43.75,-43.25] and Lon=[172,173] were selected for further analysis. These bounds consider essentially all of the seismicity occurring during the Canterbury earthquake sequence in the vicinity of the urban Christchurch region, and in particular, all events of engineering significance ($M_w \geq 4$).

2.1. Observed magnitude-frequency distribution

Figure 1 illustrates the magnitude-frequency (i.e. size) distribution of all events within the region from the moment tensor catalogue. For comparison, the Gutenberg-Richter distribution for $b = 1$ is also illustrated, as given by:

$$\log_{10} N = a - bM_w$$

where $a$ and $b$ are empirical constants. Typically $b \approx 1$, and $a$ is the activity rate (which varies in both space and time, as elaborated upon subsequently). It can be seen in Figure 1 that the Gutenberg-Richter scaling holds well for $M_w > 3.5$, largely because the moment tensor catalogue becomes incomplete for $M_w < 3.5$. Therefore $M_w = 3.5$ is used as the cutoff magnitude for the subsequent analyses.
2.2. Observed spatial distribution

Figure 2 illustrates the spatial distribution of $M_w \geq 3.5$ events within the considered region from 1 September 2010-present. It can be seen that there is a significant clustering of events. To the South-west of the Christchurch urban area is a significant clustering of events associated with structural complexity between the rupturing faults on the 4 September 2010 and 22 February 2011 (Beavan et al. 2012). Significant clustering also exists to the South and South-east of the urban Christchurch area, in the vicinity of the locations of the 22 February, 13 June, and 23 December 2011 earthquakes (Beavan et al. 2012). The spatial distribution of future seismicity associated with this sequence is accounted for by using a spatially variable $\alpha$-value in Equation (1) based on the assumption that the future seismicity will be well represented based on this recent seismicity.
Figure 2: Spatial distribution of $M_w \geq 3.5$ earthquake events in the Canterbury earthquake sequence. The considered region of Lat=[-43.75,-43.25] and Lon=[172,173] is shown in the black polygon.

2.3. Observed temporal distribution

Figure 3 illustrates the cumulative number of events in the moment tensor catalogue with time. It can be seen that there is relatively little seismicity prior to the Canterbury earthquake sequence. The sequence, and its aftershock decay is dominated by 4 key events (4 Sept 2010, 22 Feb 2011, 13 June 2011, 23 Dec 2011), which is the reason for the complex aftershock activity rate with time.

The observed temporal earthquake activity rate is modelled with a modified-Omori law comprised of the 4 different sub-sequences as in Shcherbakov et al. (2012). The modified-Omori law for each sub-sequence is given by:

$$ r = \frac{dN}{dt} = \frac{1}{\tau (1 + \frac{t}{c})^{p}} $$

where $r$ is the activity rate (the derivate of the number of events, $N$, with time, $t$); and $\tau, c, p$ are empirical constants. Note that Equation (2) can be integrated analytically to obtain:

$$ N = \frac{c}{\tau(1-p)} \left[ \frac{1}{\left(1 + \frac{t}{c}\right)^{p-1}} - 1 \right] $$

For the four different sub-sequences which comprise the overall Canterbury earthquake sequence, the total activity rate can be expressed as $r = \Sigma r_i$, thus:
\[
    r(t) = \sum_{i=1}^{4} \frac{H(t - t_i)}{t_i \left(1 + \frac{t - t_i}{c_i}\right)^p_i}
\]

where the subscript \(i\) is used to represent each sub-sequence, \(t_i\) is the time (in days) for the start of each sub-sequence, and \(H(t - t_i)\) is the Heaviside ‘step’ function, such that \(H = 1\) for \(t - t_i \geq 0\) and \(H = 0\) otherwise.

Table 1 provides the numerical values of the parameters of the modified Omori law describing the temporal decay of earthquake activity in the study region, while Figure 3 illustrates the adequacy of these parameters for modelling the observed activity. It can be seen that a satisfactory fit was obtained with constant values of the parameters \(c\) and \(p\) for all 4 sub-events. This simplicity is desired to prevent ‘over-fitting’ of this empirical model to the specific dataset (because of the potential for poor extrapolation).

Table 1: modified Omori-law parameters in Equations (2)-(4).

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>(t_i)</td>
<td>0</td>
<td>171</td>
<td>282</td>
<td>475</td>
</tr>
<tr>
<td>(\tau)</td>
<td>0.06</td>
<td>0.13</td>
<td>0.23</td>
<td>0.145</td>
</tr>
<tr>
<td>(c)</td>
<td>2.8</td>
<td>2.8</td>
<td>2.8</td>
<td>2.8</td>
</tr>
<tr>
<td>(p)</td>
<td>1.2</td>
<td>1.2</td>
<td>1.2</td>
<td>1.2</td>
</tr>
</tbody>
</table>
2.4. Predicted future seismicity associated with the Canterbury earthquake sequence

Based on the modified Omori law utilized in the previous section, Figure 4 illustrated the extrapolated prediction for the earthquake activity associated with the Canterbury earthquake sequence over the 50 years from 1 July 2014. As one might expect, it can be seen that the activity rate decays over this period. The model predicts approximately 63 $M_w > 3.5$ events in the region considered over the coming 50 years due to this aftershock sequence alone. That is, this modelling includes only earthquake events resulting from the temporal decay of this specific earthquake sequence, and not any future sequences, which are discussed subsequently. Taking into account the Gutenberg Richter distribution (with $b \approx 1$ as shown in Figure 1), this implies approximately 1.99 $M_w \geq 5$ events over the next 50 years in the region considered. Figure 4 also illustrates that over approximately the last 1 year the observed activity rate is slightly less than the modelled rate, however, this discrepancy is minor and not statistically significant.
Figure 4: Illustration of the predicted rate of aftershocks in the region over the 50 year period from 1 July 2014-1 July 2064

2.5. Modelling Canterbury earthquake sequence seismicity in the NZ ERF

In the NZ national seismic hazard model (or earthquake rupture forecast, ERF) of Stirling et al. (2012), there are a total of 55 point sources (for 5 Latitudes and 11 Longitudes values on a grid of 0.1 degrees) to represent the background seismicity in the region considered. Two options were considered for representing the spatial distribution of seismicity: (i) spatially uniform; and (ii) spatially clustered. In the spatially uniform model the expected 1.99 Mw>5 events in the next 50 years can be divided by 50 years and the 55 sources to obtain a rate of $\lambda = 7.24\text{e}-3$ Mw>5 events per point source per year. In the spatially clustered model, the activity rates were assigned to each of the 55 point sources based on the spatial clustering of the earthquake sequence to date as shown in Figure 2 (specifically based on an inverse distance weighting scheme, with distance weighting exponent of 2.0). The spatially clustered model is based on the assumption that past seismicity is the best indicator of future seismicity, while the uniform model could be based on the assumption that stress transfer could lead to the more ‘quiet’ regions in the sequence to date becoming more active. Both of these two approaches were considered in the subsequent seismic hazard analyses. It was found that the spatially clustered model, which is arguably more realistic, provides approximately 10% larger ground motion intensities and therefore only the resulting using this model are explicitly documented here.

The obtained spatially variable activity rates for $M_w \geq 5$ events in the Canterbury earthquake sequence are added on top of the existing background seismicity rates for these point sources in the Stirling et al. (2012) model. The reason for the addition (rather than...
replacement, for example) of these activity rates is that the activity rates in the Stirling et al. (2012) model account for the ‘background’ rate of earthquake in this region, and therefore does not account for aftershock sequences.

3. **Probabilistic seismic hazard analysis (PSHA)**

3.1. **Adopted earthquake rupture forecast (ERF) models**

The seismic analyses performed herein utilize two models for the earthquake rupture forecasts. The first is the model of Stirling et al. (2012), which was completed in mid-2010, and represents a national consensus model as at this time. The second model is obtained from this first model with two specific modifications: (i) the earthquake activity rates in the Canterbury region are modified to account for the aftershock activity in the Canterbury earthquake sequence based on the documented methodology in the previous section; and (ii) the earthquake source depths are modified from a minimum of 10km, to 5km to account for the un-conservative nature of the former assumption, as discussed at length in Bradley (2012b).

The earthquake activity from the Canterbury earthquake sequence is considered using a time-independent Poissonian model, as is conventional in NZ PSHA. Therefore, the time dependent activity rates determined in the previous section are averaged over the 50 year time window considered.

3.2. **Adopted ground motion prediction equations (GMPE)**

A total of four different GMPEs are utilized in ground motion prediction for the analyses performed herein in a logic tree. The NZ-specific Bradley (2010, 2013) GMPE is given 70% weighting as the only NZ-specific GMPE which has been extensively validated against observations from the Canterbury earthquake sequence (Bradley 2012a, Bradley 2012c, Bradley and Cubrinovski 2011). The remaining three models from the NGA-West project considered are: Abrahamson and Silva (2008), Campbell and Bozorgnia (2008), Boore and Atkinson (2008), each of which is prescribed a logic tree weight of 10% each. It is noted that in addition to the Bradley model bring NZ-specific, it has also been modified to accurately model small magnitude earthquakes, which as shown, dominate the seismic hazard in Christchurch. The other three models, have been documented to over-predict ground motions from small magnitude events (i.e. $M_w \leq 6$) and thus are given an appropriate weight.

3.3. **Generic site considered**

A generic site at location: Lat=$-43.53$; Lon=$172.6203$ is used for the seismic hazard analyses presented herein. This location represents the centre of the urban Christchurch region. Several locations in the urban Christchurch region were considered, but the differences observed were very small, and therefore only results for this single location are presented.

The generic site is considered to have a 30-m averaged shear wave velocity of $V_{s30} = 200m/s$ and a basin-depths of $Z_{1.0} = 500m$ and $Z_{2.5} = 1.0km$, based on recent research by the author in the development of a seismic velocity model for the entire Canterbury region (Lee et al. 2014, McGann et al. 2014). Sensitivity analyses illustrates that the PSHA results were not overly sensitive to these parameters within their reasonable ranges.
3.4. Seismic hazard analysis results

Based on the modified seismic activity in the immediate vicinity of Canterbury resulting from the Canterbury earthquake sequence, seismic hazard analyses were performed to obtain the peak ground acceleration (PGA) seismic hazard.

Figure 5 illustrates the obtained seismic hazard curves for the two different earthquake rupture forecasts considered (i.e. pre- and post-Canterbury earthquake sequence models). The seismic hazard curve values for return periods (i.e. the inverse of the annual exceedance rate) of 25, 100, 500, and 2500 years are annotated with markers.

Figure 5: Seismic hazard curve comparison for peak ground acceleration (PGA) based on both the pre-Canterbury earthquake sequence model of Stirling et al. (2012) and the model developed here. Both analyses use the same set of GMPEs. The values for return periods of 25, 100, 500, and 2500 years are annotated with markers.

Because the seismic hazard curves shown in Figure 5 represent the aggregate seismic hazard resulting from all potential earthquake sources then it is insightful to understand the relative contributions of each of the seismic sources. Figure 6 and Figure 7 illustrate the seismic hazard deaggregation plots for the four return periods of interest with the pre- and post-Canterbury earthquake sequence ERF’s, respectively. It can be seen that for all cases, the PGA hazard is dominated by small magnitude, near source seismicity. This is particularly the case for the post-Canterbury earthquake sequence model, because of the increased rate of near-source seismicity due to the on-going aftershock sequence.
Table 2 summarizes the PGA hazard values and mean magnitudes from deaggregation for these four return periods of interest. It can be seen that the PGA hazard values are approximately 40-50% larger for the results in this study (accounting for the Canterbury earthquake sequence) as compared to the values based on the pre-Canterbury earthquake sequence seismicity. However, it can also be seen that the mean magnitudes are 0.3-0.5 $M_w$ units smaller. As already noted, this is because the only reason for the increased seismicity is the elevated rate of near-source seismicity, which is Gutenberg-Richter distributed.

### Table 2: Summary of PGA values and mean magnitude values from probabilistic seismic hazard analysis.

<table>
<thead>
<tr>
<th>Model</th>
<th>Return period (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>25</td>
</tr>
<tr>
<td>Stirling et al. (2012)</td>
<td></td>
</tr>
<tr>
<td>ERF</td>
<td></td>
</tr>
<tr>
<td>PGA (g)</td>
<td>0.061</td>
</tr>
<tr>
<td>$M_w$ mean</td>
<td>6.35</td>
</tr>
<tr>
<td>This study</td>
<td></td>
</tr>
<tr>
<td>PGA</td>
<td>0.085</td>
</tr>
<tr>
<td>$M_w$ mean (g)</td>
<td>5.92</td>
</tr>
</tbody>
</table>

It is important to emphasise that the results shown for the “Stirling et al. (2012) ERF” case are not expected to be directly compatible with the NZS1170.5:2004 values for several reasons:

1. The seismic hazard values underlying NZS1170.5:2004 are based on the use of the Stirling et al. (2002) ERF, where as the results presented here represent the Stirling et al. (2012) ERF.
2. The NZS1170.5:2004 values make use of the McVerry et al. (2006) GMPE only, where as the results presented here make use of a logic tree with significantly more robust GMPEs.
3. The NZS1170.5:2004 values use a ‘magnitude-weighting’ to modify the directly predicted PGA values, however the magnitude weighting function is very different to that used in contemporary geotechnical design. Through the Canterbury Earthquakes Royal Commission it was apparent that the use of the this ‘magnitude-weighting’ was adopted to account for the fact that the McVerry et al. (2006) GMPE significantly over-predicts ground motions from small magnitude earthquakes (i.e. $M_w < 6$), which are very important for Christchurch (particularly so following this earthquake sequence).
4. The values presented here are ‘direct’ results obtained from probabilistic seismic hazard analyses, and not ‘codified’ values within a functional methodology adopted for design codes. The “Z” value in NZS1170.5:2004 is not intended to represent the design PGA, but rather the response spectra for a vibration period of $T = 0.5s$, and therefore the fact that the Z value is numerically equal to the design PGA for rock sites is based on the assumed spectra shape (McVerry 2003).
Figure 6: Deaggregation plots for seismic hazard analysis with increased seismicity from the Canterbury earthquake sequence.
Figure 7: Deaggregation plots for seismic hazard analysis prior to the Canterbury earthquake sequence (i.e. Stirling et al. (2012))
4. **Comparison with other seismic hazard analysis values for the Christchurch region**

This specific study has been commissioned to provide input into geotechnical analysis and design and therefore in this section a brief comparison is made with existing guidelines in this field.

Table 3 illustrates the three performance limit states that have been provisionally recommended by the Ministry of Business, Innovation and Employment (MBIE) for earthquake geotechnical design in the Christchurch urban region. It is noted that the MBIE-recommended design levels all correspond to a $M_w = 7.5$ event as compared to the more realistic representation of the dominant magnitudes for the PGA seismic hazard shown in Table 2.

**Table 3: Provisional performance limit states adopted by MBIE**

<table>
<thead>
<tr>
<th>Performance limit state</th>
<th>Return period (years)</th>
<th>Design PGA (g)</th>
<th>Design magnitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>SLS</td>
<td>25</td>
<td>0.13</td>
<td>7.5</td>
</tr>
<tr>
<td>ILS</td>
<td>100</td>
<td>0.20</td>
<td>7.5</td>
</tr>
<tr>
<td>ULS</td>
<td>500</td>
<td>0.35</td>
<td>7.5</td>
</tr>
</tbody>
</table>

While magnitude scaling factors, $MSF$, are potentially a function of soil properties in addition to earthquake magnitude, for the purposes of comparison herein, the model of Idriss and Boulanger (2008) is utilized, which is given by:

$$MSF = 6.9 * \exp \left( -\frac{M_w}{4} \right) - 0.058 \leq 1.8$$  \hspace{1cm} (5)

from which the magnitude-corrected PGA, $PGA_{7.5}$, can be obtained from:

$$PGA_{7.5} = PGA * \frac{1}{MSF}$$  \hspace{1cm} (6)

Figure 8 compares the $PGA_{7.5}$ values for the three different performance criteria: MBIE, Stirling et al. (2012); and this study. It is re-iterated that the Stirling et al. (2012) values are based on pre-Canterbury earthquake sequence activity and are shown here only for reference. It can be seen that the MBIE suggested values are 2-3 times greater than the values presented in ‘this study’.

As far as the author is aware, no documentation of the scientific basis behind the numerical values adopted by MBIE in Table 3 has been provided. However, it is noted that this study has adopted a rigorous first-principles approach to determine the seismic hazard in the Christchurch region, and on this basis it is argued that the MBIE-adopted values are significantly conservative and warrant urgent review.
Figure 8: Comparison of the magnitude-corrected $PGA_{7.5}$ design values from MBIE, compared to those in this study.
5. Conclusions

This report has developed seismic hazard estimates for urban Christchurch based on state of the art methods. The on-going Canterbury earthquake sequence has been directly incorporated in the adopted earthquake rupture forecast (ERF) based on a first-principles analysis of the observed earthquake activity rates and empirical models based on the Gutenberg-Richter and modified-Omori aftershock laws.

The seismic hazard analyses performed suggest that the PGA seismic hazard averaged over the next 50 years is 40-50% higher than that prior to the Canterbury earthquake sequence. However, the magnitudes which dominate this seismic hazard are 0.3-0.5 $M_w$ units lower than the pre-2010 hazard.

Using a commonly adopted magnitude scaling factor, $MSF$, magnitude-correct PGA values, $PGA_{7.5}$, were determined from the seismic hazard analysis results in this study. Comparison with the provisional design values in the MBIE guidelines suggests that those values are a factor of 2-3 greater than the values obtained here based on rigorous first-principles analysis, and therefore an urgent reassessment of the MBIE values is warranted.
6. References


Bradley, B. A., (2012c). "Strong ground motion characteristics observed in the 4 September 2010 Darfield, New Zealand earthquake", *Soil Dynamics and Earthquake Engineering*, 42, 32-46. 10.1016/j.soildyn.2012.06.004


Campbell, K. W., Bozorgnia, Y., (2008). "NGA Ground Motion Model for the Geometric Mean Horizontal Component of PGA, PGV, PGD and 5% Damped Linear Elastic Response Spectra for Periods Ranging from 0.01 to 10 s", *Earthquake Spectra*, 24, 139-171.


The data presented in this Appendix support the assumption that the estimated resistance of soil to liquefaction was essentially the same prior to the 2010-2011 Canterbury Earthquake Sequence (CES) as it has been estimated post-CES. This assumption is discussed in detail in Section 6.5.

There are 25 locations in Christchurch where Cone Penetration Tests (CPTs) were performed after the CES which are within 10m of CPTs that were performed before the CES. This set of adjacent pairs of pre-CES and post-CES CPT profiles enables one to examine if the CPT results were altered significantly by the CES. The locations of these 25 CPT sets are shown as blue and green circles on Figure E1.1. Additionally, CPTs were undertaken at 8 locations in the eastern suburbs of Christchurch following the September 2010 earthquake, and additional CPTs were pushed again at these 8 locations at various times (from 3 to 6 times) following later main earthquake events. The locations of these 8 CPT pairs are shown as purple circles in Figure E1.1.

Comparison of adjacent CPT tip resistance ($q_c$) and soil behaviour type index value ($I_c$) traces at each location are shown in the attached plots in:

- **Annex E1** for the twenty five locations where pre and post-CES CPTs were undertaken within 10m of each other; and
- **Annex E2** for the locations where CPT were pushed at various times following the main earthquake events.

Inspection of each CPT pair and CPT set shows that while there are minor variations between the CPT traces at each location there is no inherent bias towards either an increase or a decrease in $q_c$ with time. The minor variations in $q_c$ between the pre- and post-CES values between adjacent CPTs is typically due to spatial variability in ground conditions over short distances. This variability in ground conditions is reflected in the corresponding $I_c$ traces.
Figure E1.1: Locations where repeat CPTs have been undertaken
Based on these CPTs it is not possible to conclude that there has been a change in \( q_c \) as a result of the CES. To examine further whether the CES has changed soil properties, pre and post-CES \( q_c \) values for all the CPT pairs and CPT sets (where the corresponding \( I_c \) value is less than 2.6) have been plotted as cumulative frequency graphs for 0 to 5m and 5 to 10m as shown in Figures E1.2 and E1.3, respectively.

Figure E1.2 shows the cumulative frequency graphs for 0 to 5m and 5 to 10m horizontal distance corresponding to pre and post-CES CPT \( q_c \) values for the 25 locations in Christchurch described above (i.e. CPT pairs A to AH in Annex E1). There is little difference between the cumulative distributions for the pre and post-CES values and insufficient difference to substantiate a change in soil properties.

**Figure E1.2**: Pre and post CES cumulative frequency graphs of \( q_c \) for varying depths from CPT locations in Figure E1.1 filtered for soil layers with \( I_c < 2.6 \).

Figure E1.3 shows the cumulative frequency graphs for 0 to 5m and 5 to 10m depth below ground surface corresponding to post-September 2010, post-February 2011, post-June 2011, and post-December 2011 CPT \( q_c \) values for the 8 locations in Christchurch described above (i.e. CPT sets AI to AP in Annex E2).
Figure E1.3: Post-September 2010, post-February 2011, post-June 2011 and post-CES cumulative frequency graphs of $q_c$ for varying depths from CPT locations in Figure E1.1 filtered for soil layers with $I_c < 2.6$.

Inspection of the CPT sets in Annex E2 shows that several of the post-December 2011 CPT $q_c$ traces have higher values compared to the other CPT $q_c$ traces. Further inspection of the corresponding $I_c$ traces indicates that these CPTs were undertaken in soils with different soil behaviour type indices. This highlights the potential for spatial variability of ground conditions over relatively short distances in the Christchurch area. To reduce the error associated with comparing $q_c$ values from significantly different soil types, the post-September 2010 and post-December 2011 $q_c$ data were filtered manually to exclude portions of the CPT $q_c$ traces where the post-September 2010 and post-December 2011 CPT $I_c$ values were sufficiently different. Table E1.1 lists the depth ranges for the 8 sets of CPT traces where the $q_c$ values were excluded.

Table E1.1: Length of CPT $q_c$ trace excluded from Figure E1.3

<table>
<thead>
<tr>
<th>CPT Set</th>
<th>Length of CPT $q_c$ trace excluded</th>
</tr>
</thead>
<tbody>
<tr>
<td>AI</td>
<td>0 – 7.2m</td>
</tr>
<tr>
<td>AJ</td>
<td>2.3 – 5.2m</td>
</tr>
<tr>
<td>AK</td>
<td>Nil</td>
</tr>
<tr>
<td>AL</td>
<td>4.0 – 6.0m</td>
</tr>
<tr>
<td>AM</td>
<td>2.4 – 5.0m</td>
</tr>
<tr>
<td>AN</td>
<td>3.1 – 6.0m</td>
</tr>
<tr>
<td>AO</td>
<td>3.5 – 4.7m</td>
</tr>
<tr>
<td>AP</td>
<td>3.4 – 4.8m</td>
</tr>
</tbody>
</table>

The resulting cumulative distribution plots show little difference for depths of 0 to 5m below ground surface. This is the depth range of soil deposits that tend to have the largest effect on potentially damaging ground deformations. Greater variability in the cumulative distributions is apparent for the 5 to 10m depth, reflecting a more rapid change in subsurface conditions for the deeper deposits at the locations of the 8 sets of CPT traces.

Note that there are three different cumulative frequency curves for the post-September 2010 data in Figure E1.3, because the CPTs that were used with each corresponding post-February,
post-June or post-December 2011 CPT’s were different for each event. Therefore, three slightly different post-September 2010 cumulative frequency curves were generated for comparison with the post-February, post-June, and post-December 2011 cumulative frequency curves.

There is little difference between the cumulative distributions for the post-September 2010, post-February 2011, post-June 2011 and post-December 2011 values, and insufficient difference to substantiate a change in soil properties.

Collectively, these figures demonstrate that there is a lack of systemic bias in the pre-CES values being higher or lower than the post-CES values. This supports the assumption that the CES has not changed the resistance of soil to liquefaction in the Christchurch area.
Annex E1 – Comparison of CPT Pairs Before and After the CES

The following CPT pairs (A to AH) have been categorised according to the time the data was collected relative to the preceding earthquake. The CPT data is displayed according to the legend provided below.

Legend

- Pre-CES
- Post-CES
<table>
<thead>
<tr>
<th>CPT Set A</th>
<th>CPT Set B</th>
<th>CPT Set C</th>
<th>CPT Set D</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>q_c (MPa)</strong></td>
<td><strong>I_c</strong></td>
<td><strong>q_c (MPa)</strong></td>
<td><strong>I_c</strong></td>
</tr>
<tr>
<td>CPT_8567 14/6/01</td>
<td>CPT_5158 11/9/12</td>
<td>CPT_8288 19/9/00</td>
<td>CPT_44429 27/2/13</td>
</tr>
<tr>
<td>CPT_8425 16/8/00</td>
<td>CPT_37186 11/12/13</td>
<td>CPT_8159 2/4/01</td>
<td>CPT_44211 18/8/14</td>
</tr>
<tr>
<td>Depth (m)</td>
<td>Depth (m)</td>
<td>Depth (m)</td>
<td>Depth (m)</td>
</tr>
</tbody>
</table>

**CPT Set A**
- q_c (MPa) and I_c graphs showing depth vs. q_c and I_c values.
- Depth range from 0 to 10 m.

**CPT Set B**
- q_c (MPa) and I_c graphs showing depth vs. q_c and I_c values.
- Depth range from 0 to 10 m.

**CPT Set C**
- q_c (MPa) and I_c graphs showing depth vs. q_c and I_c values.
- Depth range from 0 to 10 m.

**CPT Set D**
- q_c (MPa) and I_c graphs showing depth vs. q_c and I_c values.
- Depth range from 0 to 10 m.
<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>CPT Set E</th>
<th>q_c (MPa)</th>
<th>l_c</th>
<th>CPT Set F</th>
<th>q_c (MPa)</th>
<th>l_c</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPT_8566 14/6/01</td>
<td>0</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>0</td>
</tr>
<tr>
<td>CPT_14934 20/12/12</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CPT Set G</td>
<td>q_c (MPa)</td>
<td>l_c</td>
<td>CPT Set H</td>
<td>q_c (MPa)</td>
<td>l_c</td>
<td></td>
</tr>
<tr>
<td>CPT_8089 13/11/00</td>
<td>0</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>0</td>
</tr>
<tr>
<td>CPT_22436 4/3/13</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### CPT Set M

<table>
<thead>
<tr>
<th>q_c (MPa)</th>
<th>lc</th>
</tr>
</thead>
<tbody>
<tr>
<td>[Graph]</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>[Graph]</td>
</tr>
</tbody>
</table>

- CPT_8280 30/8/00
- CPT_37608 16/10/13

### CPT Set N

<table>
<thead>
<tr>
<th>q_c (MPa)</th>
<th>lc</th>
</tr>
</thead>
<tbody>
<tr>
<td>[Graph]</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>[Graph]</td>
</tr>
</tbody>
</table>

- CPT_8434 12/9/00
- CPT_18573 31/1/14

### CPT Set O

<table>
<thead>
<tr>
<th>q_c (MPa)</th>
<th>lc</th>
</tr>
</thead>
<tbody>
<tr>
<td>[Graph]</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>[Graph]</td>
</tr>
</tbody>
</table>

- CPT_8192 10/11/00
- CPT_19136 7/12/12

### CPT Set P

<table>
<thead>
<tr>
<th>q_c (MPa)</th>
<th>lc</th>
</tr>
</thead>
<tbody>
<tr>
<td>[Graph]</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>[Graph]</td>
</tr>
</tbody>
</table>

- CPT_8201 7/11/00
- CPT_1434 4/8/11
<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>CPT Set Q</th>
<th>CPT Set R</th>
<th>CPT Set S</th>
<th>CPT Set T</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$q_c$ (MPa)</td>
<td>$I_c$</td>
<td>$q_c$ (MPa)</td>
<td>$I_c$</td>
</tr>
<tr>
<td></td>
<td><img src="image1" alt="Graph" /></td>
<td><img src="image2" alt="Graph" /></td>
<td><img src="image3" alt="Graph" /></td>
<td><img src="image4" alt="Graph" /></td>
</tr>
<tr>
<td></td>
<td><img src="image5" alt="Graph" /></td>
<td><img src="image6" alt="Graph" /></td>
<td><img src="image7" alt="Graph" /></td>
<td><img src="image8" alt="Graph" /></td>
</tr>
<tr>
<td></td>
<td><img src="image9" alt="Graph" /></td>
<td><img src="image10" alt="Graph" /></td>
<td><img src="image11" alt="Graph" /></td>
<td><img src="image12" alt="Graph" /></td>
</tr>
<tr>
<td></td>
<td><img src="image13" alt="Graph" /></td>
<td><img src="image14" alt="Graph" /></td>
<td><img src="image15" alt="Graph" /></td>
<td><img src="image16" alt="Graph" /></td>
</tr>
<tr>
<td></td>
<td><img src="image17" alt="Graph" /></td>
<td><img src="image18" alt="Graph" /></td>
<td><img src="image19" alt="Graph" /></td>
<td><img src="image20" alt="Graph" /></td>
</tr>
<tr>
<td>CPT Set AC</td>
<td>CPT Set AD</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>------------</td>
<td>------------</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>q_c (MPa)</strong></td>
<td><strong>l_c</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><img src="image1.png" alt="Graph" /></td>
<td><img src="image2.png" alt="Graph" /></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Depth (m)</strong></td>
<td><strong>Depth (m)</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><img src="image3.png" alt="Graph" /></td>
<td><img src="image4.png" alt="Graph" /></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CPT Set AE</th>
<th>CPT Set AF</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>q_c (MPa)</strong></td>
<td><strong>l_c</strong></td>
</tr>
<tr>
<td><img src="image5.png" alt="Graph" /></td>
<td><img src="image6.png" alt="Graph" /></td>
</tr>
<tr>
<td><strong>Depth (m)</strong></td>
<td><strong>Depth (m)</strong></td>
</tr>
<tr>
<td><img src="image7.png" alt="Graph" /></td>
<td><img src="image8.png" alt="Graph" /></td>
</tr>
</tbody>
</table>

Explanation:
- The table lists the CPT data for different sets (AC, AD, AE, AF).
- Each set includes two columns: **q_c (MPa)** and **l_c**.
- The graphs illustrate the variation of **q_c (MPa)** and **l_c** with depth (m).

**Revision Details**
- **CPT Set AC** includes data from CPT_8075 13/11/00 to CPT_8095 26/10/00.
- **CPT Set AD** includes data from CPT_8082 21/11/00 to CPT_5579 18/9/12.
- **CPT Set AE** includes data from CPT_7833 14/11/00 to CPT_37134 11/4/12.
- **CPT Set AF** includes data from CPT_8095 26/10/00 to CPT_30132 2/5/13.
<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>CPT Set AG</th>
<th>CPT Set AH</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( q_c ) (MPa)</td>
<td>( I_c )</td>
</tr>
<tr>
<td>[Graphs]</td>
<td>[Graphs]</td>
<td>[Graphs]</td>
</tr>
<tr>
<td>CPT_8305 18/9/00</td>
<td>CPT_8276 25/8/00</td>
<td></td>
</tr>
<tr>
<td>CPT_12373 11/12/12</td>
<td>CPT_27458 24/7/13</td>
<td></td>
</tr>
</tbody>
</table>
Annex E2 – Comparison of CPT Sets Before, During and After the CES

The following CPT sets (AI to AN) have been categorised according to the time the data was collected relative to the preceding earthquake. The CPT data is displayed according to the legend provided below.

Legend

- Post-September 2010 Earthquake
- Post-February 2011 Earthquake
- Post-June 2011 Earthquake
- Post-December 2011 Earthquake
### CPT Set AK

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>q_c (MPa)</th>
<th>l_c</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>4</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>5</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>6</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>7</td>
<td>7</td>
<td>7</td>
</tr>
<tr>
<td>8</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td>9</td>
<td>9</td>
<td>9</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
<td>10</td>
</tr>
</tbody>
</table>

### CPT Set AL

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>q_c (MPa)</th>
<th>l_c</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>4</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>5</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>6</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>7</td>
<td>7</td>
<td>7</td>
</tr>
<tr>
<td>8</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td>9</td>
<td>9</td>
<td>9</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
<td>10</td>
</tr>
</tbody>
</table>
Appendix F: Changes in Vulnerability to Lateral Spreading
F1 Introduction

In parts of Christchurch in close proximity to streams and rivers the four main earthquakes in the Canterbury Earthquake Sequence (CES) caused extensive liquefaction related lateral spreading. This lateral spreading resulted in significant damage to buildings, bridges, underground services and roads. The severity and extent of liquefaction related lateral spreading varied for each main earthquake in the CES.

Figure F1.1 presents the lateral spreading observations and free field liquefaction observations (i.e. flat land away from the influence of rivers and streams) recorded over the four main earthquakes during the CES.

Figure F1.1 shows that the majority of the lateral spreading observations were in close proximity to the rivers and streams in the Christchurch area, in particular:

- The Avon River and its tributaries which flow from west to east generally in the north eastern area of the Central Business District (CBD); and

---

1 Observations of ground cracking in some areas is attributable to mechanisms other than lateral spreading such as ground oscillation damage.
2 The land damage observations were undertaken on behalf of the EQC for the purpose of settling land damage claims on residential properties. As such the non-residential land damage was not comprehensively mapped. Hence, there are non-residential areas where lateral spreading and free field liquefaction occurred that are not presented on this figure.
The Heathcote River which flows from west to east at the foothills of the Port Hills to the south of CBD. Kaiaipoi and the northern suburbs were also significantly affected by liquefaction related lateral spreading. The majority of the lateral spreading in Kaiaipoi and the northern suburbs occurred during the September 2010 earthquake, where the seismic demand was higher compared with the subsequent CES earthquakes.

F1.1 Purpose and Outline

This appendix provides a review of lateral spreading prediction methods commonly used by engineering practitioners in New Zealand and also presents observations of lateral spreading which occurred during the CES. It demonstrates that the potential for lateral spreading in the Christchurch area very likely has not increased as a result of the physical changes to the land that occurred as a result of the CES. This conclusion forms the basis of a key assumption in the ILV Assessment Methodology, which is discussed in Section 6.6.

The ILV Assessment Methodology does not incorporate lateral spreading vulnerability into the assessment of Criterion 1 on the basis that this type of liquefaction vulnerability very likely has not increased. Therefore, land that is unlikely to be vulnerable to free field liquefaction related damage but is vulnerable to liquefaction related lateral spreading damage does not satisfy Criterion 1 and hence, does not qualify for ILV land damage as discussed in Section 8.4.

This appendix is structured as follows:

- Section F2 provides background information about lateral spreading including a description of the process and a summary of the measurements and observations of liquefaction related lateral spreading as a result of the CES;
- Section F3 discusses the effect of the physical changes to the land as a result of the CES on the future vulnerability to liquefaction related lateral spreading in the Christchurch area. This is discussed with reference to commonly used lateral spreading prediction models. Analyses of horizontal movements estimated from lateral spreading crack measurement observations for each earthquake as well as the LiDAR surveys demonstrates that the severity of lateral spreading likely reduced in the latter earthquakes during the CES relative to the seismic demand of each earthquake; and
- Section F4 summarises the main conclusions from this appendix.

F2 Background to Lateral Spreading

F2.1 Liquefaction Related Lateral Spreading Process

In areas with soils that are susceptible to liquefaction, significant damage to structures and lifelines can be caused by liquefaction related lateral spreading and lateral stretching (as observed in such areas through the CES). Lateral spreading occurs in areas with gentle slopes or areas with nearly level ground with a free-face in close proximity (such as a road cutting, old river terrace or river bank).

For the purposes of this appendix, lateral spreading is defined as the horizontal movement of ground towards the free-face or downslope as a result of the liquefaction of shallow underlying soil deposits. Liquefaction primarily occurs as a result of earthquake shaking of loose sands and soils. The liquefaction process and the methods used to assess liquefaction triggering and vulnerability are discussed in detail in Appendix A.

The schematic diagram presented in Figure F2.1 is a three dimensional depiction of liquefaction related lateral spreading. Typically, liquefaction related lateral spreading occurs at a site underlain
by liquefying soil material when either the ground is sloping or when the land is flat and in close proximity to a free-face. In the Christchurch area where the land is relatively flat and incised by river and stream channels, lateral spread areas associated with flat land and a free-face was predominant during the CES and caused severe land, infrastructure and building damage.

Figure F2.1: Schematic of liquefaction related lateral spreading (Deterling (2015), originally from Varnes 1978).

There is an important distinction between lateral spreading and lateral stretching as they are referred to in this appendix. Lateral stretch (also referred to as lateral strain) is a measure of the difference between the horizontal movement of two observation points over a given length, whereas lateral spread is a measure of the global horizontal movement of a block of land.

Figure F2.2 demonstrates these two concepts. Dwelling A has laterally spread on average 0.5m but it has not sustained lateral stretch across the building footprint. Dwelling B has laterally spread on average 2.25m and sustained 0.5m of lateral stretch across the building footprint.

Ground cracks are typically a manifestation of lateral stretching and occur when a block of land sustains less global horizontal movement relative to the block in front of it. It is important to note that lateral stretch is typically non-uniform, generally resulting in an irregular crack pattern. Also, the ground cracks are not always visible at the ground surface.
Figure F2.2: Simplified cross-section showing large-scale lateral spreading and localised lateral stretching of gently-sloping land.

F2.2 Lateral Spreading during the CES

The white and black dashed box in Figure F1.1 indicates the study area that is the focus of this appendix. This study area has been chosen because it covers the general areas in Christchurch with the most severe and extensive liquefaction related lateral spreading. Figure F2.3 is an expanded view of this study area showing the ground surface elevation prior to the CES with estimated horizontal movement vectors derived from LiDAR survey data overlaid. Large arrows indicating the general direction of significant horizontal movements are shown. This figure also shows the river cross-section locations that are discussed further below.

Figure F2.3: Ground surface elevation prior to the CES with LiDAR derived liquefaction related horizontal movement vectors overlaid.
The estimated horizontal movement vectors shown on Figure F2.3 are derived from LiDAR survey data that was obtained pre and post-CES. They represent the estimated liquefaction related horizontal movement as a result of the CES, calculated by taking the estimated total horizontal movement and subtracting the estimated tectonic horizontal movement.

The horizontal movement vectors are generated using a sub-pixel correlation method developed by Imagin’ Labs Corporation and the California Institute of Technology (Beavan et al., 2011). The horizontal movement estimates were generated at an 8m grid spacing in most areas with LiDAR survey coverage, but are shown in Figures F2.3, F2.4 and F2.5 at a 56m grid spacing for presentation purposes.

The accuracy of the LiDAR derived horizontal movement is generally half of the accuracy of the vertical movement estimates (which is approximately ±0.2m for 90 to 95% of the LiDAR survey area). This means the LiDAR derived horizontal movement accuracy is approximately ±0.4m for 90 to 95% of the LiDAR survey area.

Comparison of the ground surface elevation and the liquefaction related horizontal movement vectors shows that lateral spreading tends to occur in a downslope direction (i.e. from higher elevation areas to lower elevation areas). It is noted that the slopes in the study areas which have large horizontal movements have relatively gentle slopes typically in the order of 1 to 2%. This is typical of most of the land affected by liquefaction related land damage in the Christchurch area. In most cases the direction of horizontal movement is towards a (i.e. the river banks). A notable exception in this study area is the horizontal movement to the north of the river between cross sections 5 and 6. This is indicating movement away from the river and towards a low point in this area and is attributable to the soils near the river bank having a higher resistance to liquefaction.

Figure F2.4 presents the same horizontal movement vectors as shown in Figure F2.3 overlaid onto the liquefaction related ground surface subsidence during the CES. The ground surface subsidence is derived by calculating the difference between the estimated total ground surface subsidence as a result of the CES and the estimated vertical tectonic movement.

Figure F2.4 demonstrates that significant ground surface subsidence (i.e. the red and pink areas) is generally associated with more significant lateral spreading zones (i.e. areas with larger horizontal movement vectors). As discussed in Appendix A, the ground surface subsidence is attributable to a combination of volumetric densification, volume loss due to liquefaction ejecta, localised topographic relevelling and slumping associated with down slope lateral movement.

---

3 The estimated total ground surface subsidence is generated from LiDAR difference DEMs which are discussed in detail in Appendix G.
In addition to the horizontal movement estimates from the LiDAR surveys, independent methods that have been used to assess the severity, extent and direction of lateral spreading caused by the CES including estimation of horizontal movements derived from satellite imagery, GPS survey benchmarks, ground crack observation surveys (by summing up the crack widths along transects), land damage observations and river channel profile surveys. Each of these methods examines and measures different aspects of the lateral spreading caused by the CES as discussed below:

- **Satellite imagery** has been used to estimate the median horizontal movement for 8 x 8m cells within the Avon River area for the February 2011 earthquake (Martin & Rathje, 2014). The horizontal movement estimates derived from the satellite imagery are consistent with the estimated horizontal movements from the LiDAR surveys for the February 2011 earthquake. The difference between adjacent cells can be used to estimate the lateral stretching;

- **GPS surveys of benchmarks** provide an estimate of the horizontal movement at point locations. This method can only be used to estimate lateral spread at the GPS benchmarks locations. They are typically not of sufficient density to assess the severity, extent and direction of the lateral spreading nor to estimate lateral stretch caused by the CES. Comparisons of the measured horizontal movement of the benchmark points for each of the main CES earthquakes show the measurements are consistent with the surrounding estimated horizontal movements from the LiDAR surveys and satellite imagery;

- **Ground crack (measurement) observation surveys** have been undertaken by Robinson et al. (2011, 2012 & 2013), Robinson (2015) and Cubrinovski & Robinson (2015) following the September 2010 and February 2011 earthquakes. These surveys estimate the lateral stretch along a transect as a summation of the observed crack widths. This is the conventional manner in which most lateral spread case histories were developed previously.
The results indicate that this survey method tends to under-estimate the total lateral movement relative to the other horizontal movement estimation methods. This is because absolute displacements are estimated from the LiDAR surveys whereas relative displacements (calculated from the cumulative sum of the crack widths) are relative to a reference point which is typically 150 to 200m away from the free-face. It is likely that for some of the ground crack surveys, the reference point along the transects has also moved, resulting in smaller displacements (since it assumes zero movement of the reference point) compared to the LiDAR surveys.

- **Land damage observations** based on property inspections are useful for understanding the spatial distribution of lateral spreading damage. While this data set does not provide an estimate of the magnitude of horizontal movement, it does map the areas visually affected by lateral spreading damage. The areas with the greatest horizontal movements (estimated from the LiDAR surveys and satellite imagery) for both the September 2010 and February 2011 earthquakes coincide with the mapped areas of lateral spreading damage (i.e. the severe and very severe land damage categories) as shown in Figure F2.5; and

- **River channel profiles** also provide an estimate of lateral spreading based on the relative movement of pre and post-CES river bank profiles. Figure F2.6 shows the pre and post-CES channel profiles in the locations shown on Figures F2.3 and F2.4. This method only provides an estimate of horizontal movement of the river bank which can be influenced by localised ground slumping of the river bank that can over-estimate the amount of horizontal movement relative to the other horizontal movement estimation methods.

Both the LiDAR and satellite imagery methods provide estimates of median horizontal movement for each 8 x 8m cell. Lateral stretch can be inferred from these survey methods as the difference between the horizontal movement estimates on two adjacent cells. However, in reality, the distribution of lateral spread is highly non-uniform and larger concentrations of lateral stretch occur at discrete locations (typified by ground cracks). Therefore, the LiDAR and satellite imagery methods are more useful for understanding the patterns and extent of horizontal movement rather than precise measurements of lateral spread.

The GPS surveys of benchmarks provide the most precise measurement of lateral spread at point locations but are typically collected at insufficient density to provide any meaningful interpretation of lateral spread patterns, nor the assessment of lateral stretching. Therefore, this method is only useful for understanding the accuracy and limitations of the other survey methods. Similarly, the ground crack (measurement) observation surveys do not capture all of the accumulated lateral stretch and the river channel profile methods often incorporate localised river bank slumping. As such, in isolation, none of the survey methods used to estimate the lateral spreading caused by the CES provide a complete picture and hence each the methods should be considered in conjunction with one another.

Based on analysis and examination of each of these data sets it can be seen that they are complimentary and largely consistent with one another in assessing the lateral spread which occurred in the Christchurch area as a result of the CES.
Figure F2.5: Land damage observations with LiDAR derived horizontal movement vectors overlaid for the September 2010 and February 2011 earthquakes.
Figure F2.6: Pre and post-CES river and floodplain cross sections derived from a combination of direct river invert cross section survey measurements and LiDAR data. Cross section locations are shown on Figures F2.3 and F2.4.

Where the land is relatively flat, ground extension due to lateral spreading tends to extend a long way from the free-face (up to 400m in some areas as a result of the CES as shown by the horizontal movement vectors in Figures F3.3, F3.4 and F3.5). The horizontal ground movement accumulates over many sets of parallel cracks, so the land can be conceptualised as a series of blocks moving different distances, as shown in Figure F2.1. This geometry of lateral spreading tended to include large lengths of riverbank within the one block of ground movement (typically 0.5 - 2 km along the length of riverbank).

Geotechnical engineers and engineering geologists working in the field during the CES observed that the horizontal movements associated with lateral spreading in the Christchurch areas
initiated after the earthquake shaking had finished (i.e. they occurred post-seismically). In some instances these horizontal movements were continuing up to fifteen minutes after the earthquake shaking finished.

F3 The Effect of the CES on Lateral Spreading Vulnerability

F3.1 Lateral Spread Assessment Methods

As discussed in Section F2 liquefaction related lateral spreading is a highly complex process which is dependent upon a number of variables such as:

- The elevation difference between the base of the free-face (i.e., a road cutting, old terrace or a river bank) and the elevation of the land at the point of interest (referred to as the free-face height \( H \) from herein);
- The distance \( L \) from the base of the free-face to the point of interest;
- The earthquake ground motions including Peak Ground Accelerations (PGA) and earthquake magnitude \( (M_w) \);
- The thickness, relative density and location of liquefying layers within the soil profile; and
- Additional topographic and geological boundary conditions.

This complexity means that the development of lateral spreading assessment methods is particularly challenging and there are limitations associated with the methods that are currently available. It is important to understand these limitations and the degree of accuracy associated with the results from the methods.

The methods currently recommended for use in engineering practice in New Zealand (NZTA, 2014; NZGS, 2010) for the assessment of liquefaction related lateral spreading can be broadly categorised into the following two groups:

- Empirically based methods e.g. Youd et al. (2002) and Tokimatsu & Asaka (1998); and

Other widely used methods are published in Saygili & Rathje (2008), Bray & Travasarou (2007), Zhang et al. (2004), Makdisi & Seed (1978) and Seed & Martin (1966).

The methods and their key limitations are discussed under the subheadings that follow.

F3.1.1 Empirically Based Methods

Empirically based methods for the assessment of liquefaction related lateral spreading predict horizontal movement as a function of ground motion, geometry (i.e. \( H \) and \( L \)) and the liquefying soil layer parameters. These methods are derived from field observations and in-situ geotechnical investigation data. The three empirical methods listed above were developed by Tokimatsu & Asaka (1998), Youd et al. (2002) and Zhang et al. (2004). The two most recent methods are discussed below.

Youd et al. (2002) used multiple linear regression to analyse earthquake ground motion parameters, geometry and soil characteristics to develop an empirical relationship to predict horizontal movement associated with lateral spreading. The Youd (2002) model for displacement for free-face conditions is:

\[
\log D_H = -16.713 + 1.532M - 1.406\log R^* - 0.012R + 0.592\log W + 0.540\log T_{15} \\
+ 3.413\log(100 - F_{15}) - 0.795\log(D50_{15} + 0.1mm)
\]

where \( D_H \) is the horizontal movement in metres and \( W \) is the free-face ratio in percent and is equal to 100\( (H/L) \). The other variables in the equation are related to ground motion parameters.
(M is the earthquake magnitude and R is the distance from the earthquake epicentre in km) and soil parameters (T₁₅ is the thickness of the liquefying soil layer in m, F₁₅ is the fines content of the T₁₅ layer in percent and D₅₀ is the median sediment grain size by weight of the T₁₅ layer in mm). These parameters can be derived from seismic hazard studies and geotechnical investigations respectively.

The Zhang et al. (2004) method is derived from an estimate of the induced cyclic shear stress (estimated from CPT or SPT data) which is then empirically corrected for geometric effects such as H and L. The Zhang model for free-face conditions is as follows:

\[ LD = 6 \left( \frac{L}{H} \right)^{-0.8} LDI \]

where LD is horizontal movement in meters and LDI is the lateral displacement index (an empirical approximation of the magnitude of cyclic shear stress) and is calculated by:

\[ LDI = \int_0^{z_{\text{max}}} \gamma_{\text{max}} \, dz \]

where \( z_{\text{max}} \) is typically taken as 2H and \( \gamma_{\text{max}} \) = maximum cyclic shear strain which is derived from liquefaction triggering assessments (refer to Appendix A) coupled with the Zhang et al (2004) maximum cyclic shear strain empirical equations.

The following are some of the key limitations associated with using empirically based methods for the assessment of lateral spreading:

- These empirical correlations are based on a limited case history database of lateral spreading observations. In particular, the CPT case history database is limited. The Youd (2002) model is based on SPT data only and modifications are required to use CPT data. The Zhang (2004) model is based on only 6 CPT case history sites; and
- Back analysis of observations of lateral spreading from the CES have demonstrated significant differences between the observed and the predicted horizontal movements using empirical methods.

The results from back analysis of the Youd (2002) model are variable. Robinson et al. (2013) and Bowen et al (2012) concluded that the model was significantly over-predicting, whereas Deterling (2015) concluded that the model was significantly under-predicting. It is noted that these studies were undertaken at different river cross-sections in different locations throughout Christchurch with different geological characteristics.

The results from the back analysis of the Zhang (2004) method are also variable. Deterling (2015) found that back analysis of the Zhang (2004) model indicates that predicted horizontal movements are within the same order of magnitude as those observed for sites within the specified range of L/H between 4 and 40 and that generally the Zhang (2004) model is an improvement over other models that were back analysed (such as Youd et al. (2002)). However, for L/H values less than 10, the Zhang model tended to under-predict displacements. Conversely, Robinson (2015) found that back analysis of the Zhang (2004) model resulted in relatively poor correlation with the observed horizontal movements.

**F3.1.2 Newmark Sliding Block Methods**

Newmark sliding block methods model land movement as a block of soil that slides on a defined failure surface when subjected to ground motions that approximate those experienced during an earthquake. The land movement is estimated by integrating acceleration (\( a \)) twice with respect to time over the parts of an earthquake acceleration-time history which initially exceed the minimum yield acceleration (\( a_{\text{yield}} \)) required to initiate sliding by overcoming the friction resistance.
between the block and the failure surface it slides on until the velocities of the sliding block and underlying ground coincide.

There are a number of different approaches that have been developed to apply the Newmark sliding block method. These methods can be used to estimate horizontal movement due to liquefaction induced lateral spreading due to inertial loading. The following are some of the key limitations associated with using a Newmark-based method for the assessment of lateral spreading:

- It was developed using seismic landslide case history and not liquefaction related lateral spreading case histories. Therefore its use in the assessment of lateral spreading is beyond the scope of the original research; and
- The Newmark based methods all assume an inertial mechanism causing the lateral spreading that is inconsistent with the post-seismic observations of liquefaction related lateral spreading that were made during the CES (i.e. the lateral spreading occurred after the earthquake shaking had stopped as discussed in Section F2).

F3.1.3 Application of Lateral Spread Assessment Methods to the CES

The limitations and variability of both the empirically based and the Newmark sliding block methods mean that they are not ideal tools to assess lateral spreading vulnerability throughout Christchurch. However, the empirically based methods are still useful to gain insight into the factors that influence lateral spread.

Inspection of the equations shows that the empirically based methods are in the general form:

\[
\text{Lateral movement} = \text{function of (}\frac{H}{L}, \text{ground motion parameters and soil properties})^4
\]

As discussed in Section 6.3, the 100 year return period ground motions are assumed to be a constant when assessing the change in liquefaction vulnerability as a result of ground surface subsidence from the CES. Similarly as discussed in Section 6.5, the soil properties in the Christchurch area have not changed as a result of the CES. Appendix E provides copies of CPT pairs that were pushed pre and post-CES.

Therefore, the primary influence on the estimate of horizontal movement that has changed as a result of the CES is the H/L ratio which has decreased (because the free-face height has decreased as a result of the CES as discussed below). Therefore, the amount of future estimated horizontal movement is also expected to decrease for the same level of earthquake shaking.

Similarly, for the Newmark sliding block methods the \(a_{\text{yld}}\) increases as a result of decreasing free-face height. This is because the flatter a slope has a smaller driving force requirement to induce lateral spreading.

While the LiDAR survey data demonstrates that the ground surface has subsided, the river channels have not subsided, and in most locations the river bed is now at higher elevations than it was prior to the CES due to lateral spreading and sedimentation from liquefaction ejecta entering waterways (Hughes et al., 2015).

Figure F2.6 shows the pre and post-CES river and floodplain cross sections. These cross sections are derived from a combination of direct river invert cross section survey measurements commissioned by the Christchurch City Council and LiDAR data available on the Canterbury

---

4 Note that the Zhang model is presented as \((L/H)^{0.8}\) which is equal to \((H/L)^{0.8}\)
Geotechnical Database. As indicated on the cross sections in Figure F2.6, the raising of the river bed and the subsidence of the surrounding flood plain has resulted in all cases in a decrease in the free-face height (H) at all cross section locations (i.e. \( H_{\text{post}} < H_{\text{pre}} \)).

The river cross section survey was undertaken in 2008, March 2011 (i.e. following the February 2011 earthquake) and in September 2011 (i.e. following the June 2011 earthquake). These surveys indicated that the June 2011 earthquake resulted in very minor changes to the river channel relative to the changes caused by the September 2010 and February 2011 earthquakes.

The river channel was not surveyed following the December 2011 earthquake on the basis that the June 2011 earthquake had not significantly changed the river channel. Also, the LiDAR survey data indicate that even less horizontal movement occurred in the December 2011 earthquake relative to the June 2011 earthquake. Therefore, while the river cross-sections in Figure F2.6 do not include changes due to the December earthquake, these changes are likely to be very small and hence the September 2011 river channel shape is a reasonable representation of the post-CES river channel.

In summary, the methods currently recommended for use in engineering practice for the assessment of liquefaction related lateral spreading all indicate an inverse relationship between the elevation difference between the base of the free-face and the elevation of the land at the point of interest, if all other parameters are held constant. The elevation difference between the base of the free-face and the elevation of the land at the point of interest has decreased due to ground surface subsidence and lateral spreading caused by the CES. Accordingly, on the basis that the soil properties have not changed (refer to Section 6.5 and Appendix E), the potential for horizontal movement due to lateral spreading has not increased over the CES (and in most cases decreased) for a given set of earthquake ground motions.

**F3.2 Observed Horizontal Movements for each of the Main CES Earthquakes**

Another means to investigate effect of changes to the land as a result of the CES on the vulnerability to lateral spreading in the Christchurch area is to analyse and track the estimated liquefaction related horizontal movements for each of the main earthquakes from the various lateral spreading measurement and observation methods listed in Section F2.2.

The ground crack (measurement) observation surveys undertaken by Robinson (2015) show that repeat measurements of horizontal movements at cross section transects in the Avonside and Dallington areas (marked with an “S” on Figure F2.5) were larger in the September 2010 earthquake compared to the February 2011 earthquake, even though the seismic demand was higher in the February 2011 earthquake.

Based on the examination of the Cone Penetration Test (CPT) data in these areas, the September 2010 earthquake provided sufficient seismic demand to liquefy the full soil profile in this area. Therefore, even though the seismic demand in the February 2011 earthquake was higher, the extent and thickness of liquefied soil is likely to have been similar for both earthquakes (as indicated in Figure 2.5). This indicates that the reduction in the lateral spreading in these areas for the February 2011 earthquake relative to the September 2010 earthquake is attributable to the reduction in the free-face height as a result of lateral spreading from the September 2010 earthquake (refer to Figures F2.5 and F2.6).

Conversely, in most other areas adjacent to the Avon River and its tributary streams, repeat lateral spread measurements presented in Robinson (2015) show that the measured horizontal movement for the February 2011 earthquake was larger than the measured horizontal movement for the September 2010 earthquake. These areas are marked with an “F” on Figure F2.5. This is primarily attributable to the larger seismic demand from the February 2011 earthquake triggering
a greater thickness and extent of liquefying soils (based on examination of the CPT in these areas) resulting in a greater potential for lateral spreading to occur (as indicated in Figure 2.5 by visually comparing the mapped liquefaction related land damage extents in the September 2010 and February 2011 earthquakes). While the horizontal movements were generally larger in the February 2011 earthquake, when considered relative to the seismic demand, the horizontal movements were generally similar to or smaller than the September 2010 earthquake.

The horizontal movement estimates derived from the LiDAR survey data for the September 2010 and February 2011 earthquakes are consistent with the ground crack (measurement) observation surveys undertaken by Robinson (2015).

Bouziou (2015) undertook spatial statistical analysis on the liquefaction related horizontal movement vectors derived from the LiDAR surveys (shown in Figures F2.2 and F2.3) for the September 2010, February 2011 and June 2011 earthquakes. For this study Bouziou (2015) selected a subset of the LiDAR data and removed areas that contribute to LiDAR error such as densely vegetated zones and areas with multi-storey building damage. In this work, Bouziou (2015) demonstrated that over the assessment area, on average, the most severe liquefaction related horizontal movement occurred in the September 2010 earthquake and reduced in subsequent earthquakes.

To further examine the spatial analyses, the work undertaken by Bouziou (2015) has been expanded upon by:

- Inclusion of the estimates of liquefaction related horizontal movements during the December 2011 earthquake;
- Further refinement of the study area through the removal of additional areas with erroneous data in parks and rivers where reliable estimates of the horizontal movements from the LiDAR surveys cannot be made; and
- Sub-categorisation of the horizontal movement for the various mapped land damage observation categories for each of the main CES earthquakes during the CES as shown in Table B.2 in Appendix B, Figure F2.5 and Figures K1.5 to K1.8 in Appendix K.

The refined study area is presented in Figure F3.1. Areas in blue show the common area where LiDAR surveys were undertaken after each of the main earthquakes during the CES. Areas in red were eliminated from the study by Bouziou (2015) to remove the areas with erroneous data. The areas in orange are the additional areas where erroneous data that have been eliminated for the horizontal movement analysis presented in this section.
As discussed in Section 5.8, land damage observations for each of the four main earthquakes during the CES were recorded as one of six different land damage severity categories. The six land damage severity categories with a brief description of the types of land damage they represent are provided in Table B.2 in Appendix B.

Figure F3.2 shows the cumulative frequency distributions of the liquefaction related horizontal movement of the study area shown in Figure F3.1 for the six land damage severity categories for the four main earthquakes in the CES as shown in Figures K1.5 to K1.8 in Appendix K. The “none observed”, “minor” and “moderate” land damage categories include properties which sustained “no apparent lateral movement” based on ground and aerial photo mapping for that earthquake.

While the “major” land damage category is described as having “limited visual evidence of lateral movement” (refer to Table B.2 in Appendix B) it is possible that these properties have sustained horizontal movement due to stretching of the land or block movement that was not visually apparent on an individual property basis. This is primarily because a lot of these properties were covered with liquefaction ejecta which made is difficult to see the underlying evidence of any ground cracking and/or lateral stretching. The “severe” and “very severe” land damage categories include properties mapped with moderate to extensive lateral spreading.
Figure F3.2: Plots showing the cumulative frequency distribution of the estimated liquefaction related horizontal movement for the six land damage observation categories during the four main earthquakes in the CES.

Figure F3.2 shows that the amount of liquefaction related horizontal movement increases with increasing mapped land damage severity.

The population of properties which sustained “none observed”, “minor and “moderate” land damage have very similar cumulative frequency distributions of liquefaction related horizontal movement. For each of the four main earthquakes, 80% of the properties moved less than 0.4m (i.e. within the accuracy limits for 95% of the LiDAR survey area).

The population of properties that sustained “major”, “severe” and “very severe” land damage show that in the September 2010 and February 2011 earthquakes more horizontal movement occurred compared to the June 2011 and December 2011 earthquakes. The most severe horizontal movement occurred on those properties which sustained “very severe” mapped land damage. This was less than 1m for 80% of the properties mapped with this type of land damage following the September 2010 earthquake and less than 0.6m for 80% of the properties mapped with this type of land damage following the February 2011 earthquake.

While for all land damage severity categories the typical liquefaction related horizontal movements were largest in September 2010, smaller in February 2011 and smaller again in the June 2011 and December 2011 earthquakes, care should be taken in appropriately interpreting the results presented in Figure E3.2. As discussed earlier, the lateral spreading in many areas increased in the February 2011 earthquake compared to the September earthquake, due to the higher seismic demand. While the typical horizontal movements for each land damage category decreased, many of the properties which were mapped as a less severe land damage category changed to a more severe category following the February 2010 earthquake, making comparisons between events difficult to interpret.

Therefore, further analyses were undertaken on the horizontal movements on the subset of 1,200 properties categorised as sustaining “severe” or “very severe” land damage during the September 2010 earthquake and again in the February 2011 earthquake (i.e. the areas marked with “S” on
Figure E2.5). The cumulative frequency distributions of horizontal movements for these 1,200 properties for the September 2010, February 2011, June 2011 and December 2011 earthquakes are shown in Figure E3.3. This figure demonstrates that properties, which sustained lateral spreading in the September 2010 earthquake (most of which are in the areas marked “S” on Figure E2.5), have statistically moved less horizontally in each subsequent earthquake. This conclusion is consistent with the lateral spreading crack measurement surveys made by Robinson (2015) and is also consistent with the cross section data shown in Figure F2.6. This Figure shows a reduction in H progressively through the CES as a result of ground surface subsidence reducing the potential for horizontal movements to occur.

![Figure E3.3](image)

**Legend:**
- **September 2010**
- **February 2011**
- **June 2011**
- **December 2011**

**Figure F3.3:** Cumulative frequency distribution for properties categorised as sustaining “severe” or “very severe” land damage observation categories for each of the four main CES earthquakes.

### F3.3 Conclusion

On the basis that the elevation difference between the base of the free-face and the elevation of the land at the point of interest has remained the same or reduced as a result of ground surface subsidence and lateral spreading caused by the CES, the potential for lateral spreading very likely has not increased and in most instances likely has decreased. This is supported by the liquefaction related horizontal movement estimates from the LiDAR surveys, satellite imagery, GPS based benchmark surveys, ground crack observation surveys (by summing up the crack widths along transects), land damage observations and river channel profile surveys following each of the four main CES earthquakes. These horizontal movements show a reduction in the amount of lateral spreading for each of the three main earthquakes subsequent to the September 2010 earthquake relative to the seismic demand for each earthquake.

### F4 References


New Zealand Transport Agency (NZTA) 2014. *Bridge manual (SP/M/022)*, New Zealand Transport Authority.


Appendix G: Accuracy and Limitations of LiDAR data
G1 Introduction

LiDAR surveys were flown before and after each of the main earthquake events in the 2010-2011 Canterbury Earthquake Sequence (CES) for the purpose of assessing the ground surface subsidence caused by each main earthquake event. A suite of Digital Elevation Models (DEMs) of the ground surface were developed from position data points collected during the LiDAR surveys that were flown before and after each of the main earthquake events.

The substantial amount of position data points collected during each LiDAR survey were acquired as a LiDAR survey point cloud (referred to herein as LiDAR points). The LiDAR points were classified as either ground classified points or non-ground (i.e. LiDAR points reflected off vegetation and structures) classified points. While all LiDAR points may be used in the development of DEM, only ground classified points were used in the development of the bare earth DEMs (referred to herein as DEM). A DEM consists of cells of a set size (i.e. 1m x 1m, 5m x 5m, 25m x 25m, etc). Each DEM cell is constructed by taking the average of the ground classified points within the DEM cell. From herein reference to a DEM is typically in respect to a DEM with 5m x 5m cells unless specifically stated otherwise.

To estimate the change in vertical ground surface elevation due the CES, difference DEMs were obtained by subtracting the DEM from an earlier DEM. The DEMs for each main earthquake event and difference DEMs were constructed for the purpose of ILV assessment, to establish the pre- and post-CES ground surface elevation and to determine the change in ground surface elevation for each main earthquake event as well as over the CES.

It was also necessary to understand the accuracy and limitations of the DEMs (and hence the difference DEMs), which are governed mainly by:

- Measurement error of the LiDAR points;
- Localised error due to interpolation in areas with low density of ground classified points; and
- Spatial resolution (granularity) of the DEM and the accuracy and appropriateness in representing the ground surface elevation.

G1.1 Purpose and Outline

The purpose of this appendix is to summarise the verification of the LiDAR points and DEMs against independent survey information. Additionally, this appendix provides a discussion of the errors and uncertainty in the DEMs and difference DEMs pertinent to the ILV assessment process.

The appendix is structured as follows:

- A brief background of the LiDAR surveys undertaken in Christchurch before and after each of the main earthquake events in the CES is provided in Section G1.2;
- The verification of the acquired LiDAR points is discussed in Section G2;
- The LiDAR acquisition process is discussed in Section G3;
- The development and verification of the DEMs is discussed in Section G4;
- Discussion of the development of difference DEMs, to determine the change in ground surface elevation for each main earthquake event as well as over the CES, is provided in Section G5;
- Discussion of the limitations and error bands in the difference DEMs is provided in Section G6; and
A summary of the most pertinent points, in Sections G2 to G6, relevant to the assessment of ILV land damage is provided in Section G7.

A technical specification providing a full discussion of the calibration and verification of the LiDAR points and DEMs (CERA, 2014) is available on the Canterbury Geotechnical Database (CGD).

**G1.2 Background**

LiDAR is an aerial survey method using laser scanning technology on board an aircraft that is capable of collecting millions of position data points (horizontal and vertical positions) across the surveyed area. A LiDAR survey of Christchurch, commissioned between 2003 and 2008, provided a baseline DEM. LiDAR surveys were also flown after each of the main earthquakes during the CES. Each LiDAR survey was typically undertaken one month after each main earthquake, providing time for ejected sand and silt to be removed from most properties and streets. This enabled the measurements to record the ground surface level relative to the Lyttelton vertical Datum 1937 (refer to Annex G1). The main exception was the post-September 2010 LiDAR survey that was acquired the day after the 4 September 2010 earthquake. The extents of the LiDAR surveys were generally scoped to cover the areas expected to have been affected by tectonic and liquefaction related ground surface subsidence. Therefore, the LiDAR survey coverage was different for each earthquake as shown in Figure G1.1.

LiDAR was acquired by AAM Brisbane (AAM) Pty and New Zealand Aerial Mapping (NZAM) Ltd. The LiDAR sources and commissioning agencies are given in Table G1.1. The two suppliers classified the acquired points as either ground classified points or non-ground classified points (i.e. structures and vegetation that were judged to be higher than 0.13m above the surrounding ground) in order to enable the DEMs to be developed.

**Table G1.1 LiDAR Source and Commissioning Agencies**

<table>
<thead>
<tr>
<th>DEM</th>
<th>LiDAR Source</th>
<th>Commissioning Agencies</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre-CES</td>
<td>AAM, 6-9 Jul 2003</td>
<td>Christchurch City Council</td>
</tr>
<tr>
<td></td>
<td>AAM, 21-24 Jul 2005</td>
<td>Environment Canterbury &amp; Waimakariri District Council</td>
</tr>
<tr>
<td>Post-Sept 2010</td>
<td>NZAM, 5 Sep 2010</td>
<td>Ministry of Civil Defence and Emergency Management</td>
</tr>
<tr>
<td>Post-Feb 2011</td>
<td>NZAM, 8-10 Mar 2011</td>
<td>Ministry of Civil Defence and Emergency Management</td>
</tr>
<tr>
<td>Post-June 2011</td>
<td>AAM, 20-30 May 2011</td>
<td>Christchurch City Council</td>
</tr>
</tbody>
</table>
Metadata supplied by each LiDAR source indicates the LiDAR points have a horizontal accuracy of 0.55m. It also includes specification of a vertical accuracy of ±0.15m for the 2003 LiDAR points and ±0.07m for the post-September 2010 LiDAR points. These accuracy limitations exclude Global Positioning Survey (GPS) network error and the approximations within the New Zealand Quasigeoid 2009 (NZGeoid2009) reference surface (with an expected vertical accuracy of ±0.06m).
While the 2003 LiDAR survey has sparser ground classified points than the LiDAR surveys undertaken post-September 2010, all of the LiDAR surveys contain sufficiently large quantities of LiDAR points for the purposes of developing DEMs. DEMs were then developed by averaging the ground classified point elevations within each DEM cell to form a DEM comprising 5m x 5m and 25m x 25m cells of average LiDAR point elevations for each survey. Note the difference between DEMs comprising 5m x 5m and 25m x 25m cells is discussed in Section G4.1. The DEMs were developed for EQC and published for more general use on the CGD, where each DEM was rendered to create a colour banded elevation model. Significant waterways and coastal marine areas were clipped from the elevation model in the CGD map layer.

G2 Verification of LiDAR Point Cloud Elevations

In order to verify the accuracy of the supplied LiDAR points, a comparison was made between the LiDAR points and elevations of surveyed benchmarks within the Christchurch region before and after each of the main earthquakes within the CES. Land Information New Zealand (LINZ) provide elevations of benchmarks that have been surveyed using GPS-based equipment and precise levelling methods. The locations of the LINZ benchmarks that were surveyed using each method are shown in Figures G2.1 and G2.2 respectively.

![Figure G2.1: Locations of LINZ benchmarks surveyed using GPS-based equipment.](image-url)
The accuracy of the LiDAR points relative to the LINZ benchmarks were estimated by subtracting the mean elevations of the LiDAR points around each LINZ benchmark from the surveyed elevation of the LINZ benchmark (referred to as the “approximate error”). The error is approximate because LINZ benchmark elevations typically have a vertical accuracy of ± 0.03m.

The approximate errors of the LiDAR points for different LiDAR surveys are plotted as a cumulative frequency distributions in Figure G2.3. This shows that the median for all LiDAR surveys, with the exception of the September 2011 survey was similar, with the difference between respective medians being approximately ±0.02m. The September 2011 LiDAR points appears to be offset by approximately 0.05m based on comparison against surveyed benchmark elevations. An adjustment for this was made during processing and in developing the September 2011 DEM for ILV assessment purposes. The July 2003 LiDAR survey was found to have a larger standard deviation. The larger standard deviation for the 2003 LiDAR points is considered to be a result of the lower precision of LiDAR equipment used in the 2003 LiDAR survey and the fewer LiDAR points acquired per unit area.
Figure G2.3 shows that for the 2003 LiDAR survey approximately 80% of the LiDAR point elevations are within the specifications provided by each LiDAR source (i.e. ±0.15m). Similarly for the post-September 2010 LiDAR surveys, approximately 80% of the LiDAR point elevations are within the specifications provided by each LiDAR source (i.e. ±0.07m).

G3 LiDAR Acquisition and Processing

The mechanism of obtaining LiDAR surveys has an impact on the data accuracy. While this affects the accuracy of the LiDAR points and hence the DEM, the impact is more clearly visible in the difference DEMs (provided and discussed in Section G6). The LiDAR points for each LiDAR survey is acquired using a laser beam sweeping from one side to the other and back again as the plane flies forwards (Figure G3.1). This provides an approximately 940m wide zig-zag swath of LiDAR points from the terrain passing beneath the plane. As the plane flies back and forth in a grid pattern swaths from adjacent flight paths usually overlap so the terrain near swath edges will be rescanned (twice and occasionally three or more times). The sweep angle of the scanner is fixed so the aircraft height, flying speed and wind speed control the average LiDAR point density, with
dynamic corrections made due to pitch (front-back), roll (side-to-side) and yaw (bearing) of the aircraft.

![LiDAR illustration](image)

**Figure G3.1:** An illustration of LiDAR point acquisition process showing the 40° sweep angle for one scan of the laser beam beneath the aircraft.

The travel time of the light impulses can be more precisely measured than the sweep angle. Similarly, the aircraft position is more precise than its pitch, roll and yaw angles, so the ground vertical elevations of the LiDAR points are more precisely measured than their horizontal positions.

The post-acquisition point classification uses a mixture of automated and manual classification methods. Some LiDAR points are automatically classified as ‘non-ground’ when their elevations are 0.13m or more above those classified as ground. Additionally, post-acquisition processing includes manual identification of vegetation and structures and removal of the non-ground classified points (refer to Section G3.2 for illustration and further discussion).

The artefacts of the LiDAR point acquisition and post-acquisition processing are LiDAR error bands, which align with flight paths. As noted above these bands can be clearly seen on difference DEM maps and are discussed in more detail in Section G6.

### G4   DEM Development, Characteristics and Verification

#### G4.1   Development of DEM from LiDAR Points

The relationship between a 5m x 5m and 25m x 25m DEM cells and the LiDAR points are illustrated in Figure G4.1. This shows the plan and cross-section of a typical area with vegetation cover, a building and sloping ground surface.
As shown in Figure G4.1, in areas where there is a large amount of vegetation, buildings etc, no ground classified points or very few ground classified points are obtained, resulting in DEM cell elevations being estimated by interpolation of adjacent DEM cells (resulting in a high likelihood of localised error). In areas where there is non-flat land, step changes in the ground surface and retaining walls, the distribution of LiDAR point elevations developed from each of the LiDAR surveys can vary over the DEM cell resulting in a difference between the average elevation and the actual ground surface elevation.

For a 5m x 5m DEM cell, this is likely to have higher interpolation error but the average DEM elevation is less likely to vary from the actual ground surface. A 25m x 25m DEM cell would have less interpolation error but the average DEM elevation is more likely to vary from the actual ground surface elevation in areas of non-flat land.

Figure G4.1: Plan and cross-section of an area with vegetation and building illustrating the removal of non-ground returns to create a DEM.
From this point onwards only the 5m x 5m DEMs are analysed in relation to the verification of the DEM.

**G4.2  Ground Classified Point Density**

In addition to lower density of ground classified points due to vegetation and structures (as discussed in Section G4.1), there were different quantities of ground classified points contributing to the elevations within the DEM cells for each of the LiDAR surveys. The quantities of ground classified points within 40,000 5m x 5m square cell samples from the July 2003, May 2011 and September 2011 DEMs are shown on cumulative frequency graphs in Figure G4.2b. The 40,000 5m x 5m DEM cells come from a 1km² area from a suburban area shown on Figures G2.1 and G2.2.

The ground classified point quantities are affected by the number of LiDAR acquisition sweeps (discussed in Section G3), which varied between 1 and 3 over any given area. Figure G4.2 clearly indicates that a quarter of the 5m x 5m cells for the July 2003 DEM have elevations interpolated from neighbouring cells because there are no ground classified points. The quantities are mostly an order of magnitude smaller than they are for the other two LiDAR surveys analysed. Hence the DEM based on the 2003 LiDAR survey will have more interpolation error (discussed in Section G4.1) relative to the DEMs from the other LiDAR surveys. The hard surfaces generally provided a greater number of ground classified points than within a suburban area as shown in Figure G4.2a and hence less interpolation error can be expected in the DEMs in such areas.

Comparison of the ground classified point elevations from a hard, reasonably flat, surface with the DEM cell elevations provides an indication of the measurement error or noise within the ground classified point data. The standard deviations of the differences between the ground classified point elevations with the DEM elevation for each cell are plotted in Figure G4.3a for the same DEM cells used to generate Figure G4.2. For the hard, reasonably flat surfaces, Figure G4.3a suggests that the median standard deviation is approximately 0.03m to 0.04m for the May and September 2011 LiDAR survey and approximately 0.1m for the July 2003 LiDAR survey. The lack of ground classified point density and surface undulations (illustrated in Figure G4.1) considerably increases the median standard deviation within the 1km² suburban area.
Figure G4.3: Standard deviations of the elevation difference between the ground classified points and the average return elevation within 5m x 5m cells for the July 2003, May 2011 and September 2011 LiDAR surveys for a) hard surfaces (Christchurch International Airport runway) and b) cell within a 1km suburban area shown on Figures G2.1 and G2.2.

**G4.3 Verification of the DEM**

For the verification of the DEM the LINZ benchmarks (shown in Figures G2.1 and G2.2) were not suitable because the benchmarks are only at discrete points and spatially biased to lower elevation areas (i.e. footpaths and kerbs). As a result, they are not representative of the wider DEM. Therefore, three topographical survey sources were used to evaluate the accuracy of the elevations within the DEM relative to the survey data. These were street and subdivision topographical surveys collected before the September 2010 earthquake, a large scale topographical survey in a residential area commissioned by CERA in August 2011 and a topographical survey of residential properties commissioned by EQC in January 2014.

- The July 2003 DEM was compared against topographical survey elevations for roads within Christchurch City and two fields that were subdivided between 2003 and 2010. The extents of the survey locations are shown in Figure G4.4.
- The September 2011 DEM was calibrated against an extensive topographical survey carried out in August 2011. This had significantly larger extent and is shown in Figure G4.5. The survey included properties as well as roads.
- The February 2012 DEM was compared against a January 2014 topographical survey of 17 residential properties. The location of these properties is shown in Figure G4.6.
Figure G4.4: Clusters of topographical survey data classified as fields and road that were used to verify the July 2003 DEM.

Figure G4.5: Clusters topographical survey data that were used to verify the September 2011 DEM.
Figure G4.6: Properties with topographical surveys that were used to verify the February 2012 DEM.

The relative errors for three of these DEMs (i.e. the DEM cell elevation minus the average topographical survey elevation for the same corresponding area) are plotted as cumulative frequency distributions in Figure G4.7. All three cumulative frequency distributions are based on 2590 5m x 5m cell samples, using a random selection from the considerably greater quantities of survey locations available for the July 2003 and August 2011 DEMs.

Figure G4.7: Accuracy of the July 2003, September 2011 and February 2012 DEMs as a cumulative frequency plot of the DEM subtracted from the topographical survey elevations.
The July 2003 DEM has a smaller standard deviation relative to the other post-September 2011 DEMs because the topographical surveys were predominantly on roads whereas the September 2011 and February 2012 have large portions of locations on residential properties, which are typically not as flat and have less ‘hard’ surfaces and have areas with non-classified ground points.

There were only 340 5m x 5m cells over road areas within the topographical survey used to validate the February 2012 DEM. The errors for these and the same quantities of road locations randomly selected from the September 2011 DEM and from the July 2003 DEM are plotted as cumulative frequency distributions in Figure G4.8. The minor difference for the July 2003 DEM most likely only reflects the sample quantity, but a more significant decrease in the standard deviation was noted for the September 2011 DEM. The small decrease for the February 2012 DEM was most likely because the ‘road’ locations are kerbs and footpaths rather than road centrelines with larger areas of similar elevations.

![Figure G4.8: Cumulative frequency plot of the DEMs subtracted from the topographical survey elevations for road areas.](image)

Figure G4.8 shows that for the July 2003 DEM approximately 65% have a difference between the DEM and topographical survey of less than ±0.1m and approximately 95% have a difference of less than ±0.2m. In contrast Figure G4.8 shows that for the September 2011 and February 2012 DEM approximately 70% to 85% have a difference between DEM and topographic survey of less than ±0.1m and approximately 90% to 95% have a difference of less than ±0.2m. The differences between the DEM and topographic survey are expected to be larger for the portions of the DEMs covering residential properties, particularly in built up areas which are heavily vegetated.

**G5 Difference DEM**

The DEMs have a number of uses but, of particular importance was the means to quantify the change in ground surface elevation over the CES for the purpose of assessing criterion 2 for ILV land damage. Elevation changes were estimated by subtracting a later DEM from an earlier DEM to derive a difference DEM.
However, subtracting one DEM (consisting of uncertainty and errors discussed in Sections G2 to G4) from another can increase the error compared to when only one of the DEMs is used and in other areas the errors could cancel, reducing the overall error. This is not only because the resulting difference DEM incorporates errors from both DEMs but the errors may become smaller or larger portion depending on the level of independency between the variables.

The errors and uncertainty in the LiDAR points and DEM include errors due to the accuracy of the LiDAR equipment (found to be of particular importance when comparing the July 2003 and September 2010 DEMs), varied quantity of ground classified points available between LiDAR surveys and the lower level of precision in the modelled ground surface elevation in areas with buildings, vegetation and non-level ground.

The lack of spatial independence between the various errors means that a statistical analysis of the error in the difference DEMs is difficult. Therefore, a qualitative assessment of the difference DEMs was required in each location for ILV assessment purposes. The various difference DEM for each main earthquake event is presented in Section G6. Additionally, a discussion of error bands due to flight paths and errors due to topographic features are also highlighted and discussed.

G6 Analysis of Error Bands in DEM Difference Maps

Flat land DEM elevation difference maps for July 2003 and September 2010; September 2010 and May 2011; May 2011 and September 2011; and September 2011 and February 2012 are provided in Figures G6.1 to G6.4. The figures show the change in ground surface elevation as a result of the September 2010, February 2011, June 2011 and December 2011 earthquake events respectively. The indicative flight path orientation for July 2003 and the flight path orientations for all the other LiDAR surveys are also shown on each of the difference DEM maps.

All of the difference DEM show distinct bands of apparent greater uplift or subsidence that are parallel to and centred on individual flight paths or between flight paths used to acquire the LiDAR points. A number of the ground surface elevation maps also show patches that match vegetation changes within individual land parcel areas or ground surface altered due to construction or earthworks (which can be readily identified on aerial photographs).

The vertical change in ground surface elevation between the July 2003 DEM and September 2010 DEM is provided in Figure G6.1. Clear linear features can be seen in Figure G6.1 from intersecting pairs of difference DEM bands. The flight path for the September 2010 LiDAR survey was NNW-SSE. No flight paths were available for the July 2003 LiDAR survey, but an indicative flight path of approximately WSW-ENE is suggested by the banding in Figure G6.1.
Figure G6.1: Vertical elevation difference map (Difference DEM) for the July 2003 and September 2010 DEMs (showing the change in elevation typically caused by the 4th September 2010 earthquake) with superimposed September 2010 survey flight paths and indicative July 2003 survey flight paths.

The vertical change in ground surface elevation between the September 2010 DEM and May 2011 DEM is provided in Figure G6.2. The banding in Figure G6.2 is less clear than the banding in Figure G6.1 but nevertheless bands parallel to the September 2010 flight path (NNW-SSE) are identifiable. Comparison of difference DEM shown in Figure G6.1 and Figure G6.2 shows a significant zone beneath three of the central NNW-SSE flight paths, which appears to have more ground surface subsidence than areas beneath the adjacent flight lines in the July 2003 minus September 2010 DEM map and less ground surface subsidence in the September 2010 minus May 2011 DEM map. This indicates that as an average the September 2010 DEM elevations in this area are approximately 100mm lower than the actual ground surface.
The vertical change in ground surface elevation between the May 2011 DEM and September 2011 DEM is provided in Figure G6.3. A number of patches where there is change in ground surface elevation due to areas with construction or earthworks can be seen on Figure G6.3. In addition there is a west to east zone aligning with the post-June 2011 flight paths which appears to have approximately 100mm more ground surface subsidence. The post-June 2011 LiDAR survey was undertaken in three phases (refer to Table G1.1), due to prolonged adverse weather conditions, and this zone coincides with the area where two of the post-June 2011 LiDAR survey results for two phases were stitched together.
Figure G6.3: Vertical elevation difference map (Difference DEM) for May 2011 and September 2011 DEMs (showing the changes in elevation typically caused by the 13 June 2011 earthquake) with superimposed September 2011 and May 2011 flight paths. Note a 0.05m offset has been subtracted from the September 2011 DEM as discussed in Section G2.

The vertical change in ground surface elevation between the May 2011 DEM and September 2011 DEM is provided in Figure G6.4. Features are less easily distinguished and attributed to one DEM when the flight paths are nearly parallel as illustrated in Figure G6.4.
Figure G6.4: Vertical elevation difference map (Difference DEM) for the September 2011 and February 2012 DEMs (showing the changes in elevation typically caused by the 23 December earthquake with superimposed September 2011 and February 2012 flight paths. Note a 0.05m offset has been subtracted from the September 2011 DEM as discussed in Section G2.

The flight path directions and relative significance of the visually discernible error band features attributable to the DEMs from the analysis of the difference DEMs are summarised in Table G6.1.

Table G6.1. Error band features identified in the DEMs by comparing pairs of DEMS

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Flight path direction</td>
<td>WSW-ENE</td>
<td>NNW-SSE</td>
<td>W-E</td>
<td>WWSW-ENE</td>
<td>W-E</td>
</tr>
<tr>
<td>Error bands aligned with flight paths</td>
<td>Moderate</td>
<td>Significant</td>
<td>Not observed</td>
<td>Moderate</td>
<td>Minor</td>
</tr>
</tbody>
</table>

In addition to the LiDAR error bands aligned to the flight lines the difference DEMs shown in Figures G6.1 to G6.4, large uniform patches of minor elevation change often represent seasonal vegetation changes in individual paddocks/fields, subdivision works, and occasionally construction or deconstruction of structures where ground classified points would be available for one of the DEMs but not the other. These are localised, readily identifiable and usually attributable to anthropogenic activity.

The error bands observed in Figures G6.1 to G6.4 are most likely artefacts of the LiDAR acquisition and post-processing by the suppliers. The LiDAR acquisition mechanism and processing was discussed in Section G3.1. The LiDAR acquisition and post-processing appears to produce:

- Elevations along a flight path that are on average higher or lower than those acquired from adjacent swaths; and
Elevations on one side of a flight path that are on average higher than those on the other side of the same flight path.

The discussion presented in Section G6 to this point has been centred on error bands observed in the difference DEM for each earthquake over the CES. While some of the error still exists when assessing the difference DEM over the CES, the percentage of error in the ground surface subsidence is generally smaller relative to the larger total ground surface subsidence due to CES (i.e. the difference DEM for the July 2003 and February 2012 DEMs). The error bands that are noted in Table G6.1 for the intermediate DEMs (i.e. September 2010, May 2011 and September 2011) are not noted to feature in the overall difference DEM. Additionally, as the total ground surface subsidence due to the CES is typically larger than the ground surface subsidence due to individual events, the error due to alignment of flight paths is a smaller percentage of the overall ground surface subsidence. Therefore, for ILV assessment purposes using the total subsidence over the CES is the most reliable approach, which requires the least amount of manual adjustment to assess the likely ground surface subsidence caused by the CES for each property. The total difference DEM for the CES is shown in Figure G6.5. While the percentage of error is small for total difference DEM, error bands (typically in the WWS – NNE alignment) are still visible in Figure G6.5. Additionally, localised areas where the ground surface elevation change is influenced by anthropogenic activity (mostly between July 2003 LiDAR survey and September 2010 LiDAR survey) are illustrated in the Figure G6.5 below.

**Figure G6.5:** Vertical elevation Changes (Difference DEM) between the July 2003 and February 2012 (and September 2011 where there was no coverage from the February 2012 LiDAR survey) DEMs (showing the changes in elevation typically caused by the CES.

### G7 Discussion and Conclusions

The limitations and errors within the LiDAR point clouds and the DEMs need to be understood when using the difference DEMs to assess the likely ground surface subsidence caused by the CES
for ILV assessment purposes. The LiDAR point clouds have errors that were quantified by comparing the elevations of the returns against reference benchmark elevations. These were shown to have low mean and median approximate errors (Figure G2.3), suggesting reasonable accuracy as a whole, however the standard deviation of the error was greater for the July 2003 LiDAR points than for the subsequent LiDAR surveys. The greater standard deviation is most likely because of a lower density of LiDAR points that were acquired and lower level of precision in the equipment that was used to carry out the LiDAR survey in 2003.

Comparisons of the DEMs within roads and properties against topographical surveys showed that the hard surfaces provide smaller standard deviations of errors for roads than for properties, reflecting the differing roughness of the two types of terrain. The errors can only be qualitatively judged because the topographical surveys used for the comparison had differing portions of their survey locations on roads and properties.

Figure G4.8 shows that for the July 2003 DEM approximately 65% of the difference between the DEM elevation and topographic survey is less than ±0.1m and approximately 95% of the difference is less than ±0.2m. In contrast Figure G4.8 shows that for the September 2011 DEM and February 2012 DEM approximately 70% to 85% of the difference between the DEM elevation and the topographic survey less than ±0.1m and approximately 90% to 95% of the difference is less than ±0.2m. The differences between the DEM and topographic survey are expected to be larger for the portions of the DEMs covering residential properties, particularly in built up areas which are heavily vegetated.

The difference DEMs derived from subtracting one DEM from another carries the uncertainty and error from the source DEMs. However, it is difficult to quantify the error through statistical analysis due to the lack of spatial independence of the errors. The error in the difference DEM are more readily identifiable through qualitative assessment and engineering judgement both at a regional and local scale. Error bands due to flight paths can be identified as discussed in Section G6 and potential over or under measurement can be assessed through qualitative assessment (for instance the example shown in Figures G6.1 and 6.2). Similarly through qualitative assessment error due to topographic features, construction, de-construction, earthworks or changes in vegetation can be identified and allowed for in the manual ILV assessment process.

As noted in Section G6, the error becomes smaller relative to the total ground surface subsidence over the CES (i.e. the percentage error becomes smaller). In addition, error bands in the intermediate DEMs do not influence the difference DEM over the CES. Therefore, for ILV assessment purposes using the total subsidence over the CES is the most reliable approach requiring the least amount of manual adjustment to allow for errors when assessing the most likely ground surface subsidence caused by the CES.

G8 References

Canterbury Earthquake Recovery Authority (CERA) 2014. Verification of LiDAR acquired before and after the Canterbury Earthquake Sequence Technical Specification 03, 30 April 2014 available https://canterburygeotechnicaldatabase.projectorbit.com


Annex G1: Vertical Datums

Vertical datums are commonly used for heights and vertical elevations in Christchurch and Canterbury. While the errors introduced by using some ostensibly similar but practically different datums often has little consequence, the relatively high accuracy of the LiDAR makes these differences more significant.

The two datums normally used for construction are Lyttelton Vertical Datum 1937 (abbreviated as either LVD37 or MSL) and Christchurch Drainage Datum (CDD, where historically the C stood for the Christchurch Drainage Board, but sometimes abbreviated as CCD for Christchurch City Datum because the CDB no longer exists). The CDD is 9.043m below LVD37 (as defined in Annex 1 of the Christchurch City Council Waterways, wetlands and drainage guide). Most elevations within the collated data in the Canterbury Geotechnical Database use the LVD37 datum.

The relationships between the vertical datums used in Christchurch and Canterbury are illustrated in Figure 1. The vertical datum for the whole of New Zealand is the NZ Vertical Datum 2009 (NZVD2009), however in practice most heights in Canterbury are given relative to either the Lyttelton Vertical Datum 1937 (LVD37) or the Christchurch Drainage Datum (CDD).

![Figure 1: Relationships between the LVD37 and CDD levelling datums and other vertical datums.](image)

The NZVD2009 is nominally 0.47 ± 0.09m higher than the Lyttelton Vertical Datum 1937 (LVD37), with the offset varying with location. LINZ has suggested that the offset within much of Christchurch City is near 0.523 m, as shown in Figure 1. This larger offset, which lowers elevations observed using GPS-based equipment by 0.053m so they are similar to those observed using precise levelling, has not been adopted as a standard by either LINZ or the Christchurch City Council. The offset variation between LVD37 and the NZ Geodetic Datum 2000 (NZGD2000) is also shown in Figure 1.

Surveyed elevations are almost always tied to benchmarks within the LINZ Geodetic survey control coordinates network (described in Section G2), so the 0.053m offset correction is still appropriate for surveys carried out before the establishment of NZVD2009.
Appendix I: The LSN Calculation and Interpolation Process
I1. Introduction

Cone Penetration Tests (CPTs) can be used in the assessment of the liquefaction vulnerability. The CPTs available from the Canterbury Geotechnical Database (CGD) can vary in length (some long, some short, some with predrill) and hence need to be standardised before corresponding Liquefaction Severity Number (LSN) values can be interpolated to estimate, on a regional basis, an LSN value for each property for given levels of shaking.

As of the end of 2015, approximately 18,000 CPT investigations have been undertaken across Christchurch. Of these 18,000 CPT approximately 15,000 have been undertaken in a manner that provides a sufficient length of soil profile for the purposes of estimating LSN values at the test location, illustrated by green dots in Figure I1.1. The remaining 3,000 CPT illustrated by yellow and red dots in Figure I1.1 are missing portions of information, generally as a result of predrilling at the surface to avoid services or termination of the test prior to reaching the required depth.

In order for the estimated LSN values from a CPT to be used, a sufficient length of soil profile is required. A process of the LSN slicing has been developed to make use of the estimated vertical LSN slice increments (hereinafter referred to as LSN slices) available in the 3,000 CPT that have insufficient length of soil profile, without reducing the accuracy of the estimated LSN values by introducing arbitrary from nearby CPT for the missing portions of the CPT profiles.

A number of limitations are applied to the slicing process to ensure only relevant LSN slices from nearby CPT investigations is used to fill in incomplete portions of the CPT Profiles. These limitations are addressed in Section I3 below.

Figure I1.1: CPT test locations

I1.1 Purpose and Outline

This appendix presents a brief summary of the estimation of LSN values, CPT slicing and the interpolation process used to assess on a regional basis the liquefaction vulnerability of each
property in Canterbury for both the pre-CES and post-CES ground surface elevations for the purposes of building an automated IVL assessment model.

This appendix is laid out in the following sections:

- Section I2 outlines the method used to estimate the Liquefaction Vulnerability parameter at specific CPT test locations;
- Section I3 outlines the process used to slice LSN values from nearby CPT pairs which are missing portions of information on the soil profile to obtain complete CPT profiles for use in the LSN analysis;
- Section I4 outlines the interpolation process used to estimate property specific LSN values from the CPT LSN values; and
- Section I5 outlines the minor extrapolation process used at the boundary of the interpolation grid.

I2. Overview of the Estimation of LSN

CPT investigations provide measurements of cone tip resistance ($q_c$) and skin friction ($f_s$) recorded at regular intervals below the ground surface ($z$) within the CPT profile. These soil parameters are used to estimate the resistance to liquefaction (CRR) throughout the soil profile and comparing it to the seismic demand (CSR) to determine whether liquefaction is likely to trigger under specific levels of ground shaking. Further detail about these terms is provided in Appendix A.

The soil layers are then assessed to estimate the LSN value for that CPT location. Section I2.1 outlines the liquefaction triggering procedure and lists the assumptions used to undertake the liquefaction triggering procedures on an automated basis for regional assessment purposes. Section I2.2 outlines the LSN estimation procedure and similarly lists the assumptions used to undertake the assessment on an automated basis for regional assessment purposes.

I2.1 Assessment of Liquefaction Triggering

Liquefaction triggering, expressed as a factor of safety (FS) is assessed using the Boulanger and Idriss (2014) method summarised in Figure I2.1 below. Descriptions of the parameters illustrated in Figure I2.1 and a more in depth description of the estimation methodology can be found in Boulanger and Idriss (2014) *CPT and SPT based liquefaction triggering procedures*. 
The Boulanger and Idriss (2014) liquefaction triggering method requires the Fines Content (FC) for the soil profile at each CPT location. For these regional-scale analyses, the FC is inferred from the CPT profile using the empirical function of the Soil Behaviour Type Index ($I_c$) and a fitting parameter ($C_{FC}$) using the expression:

$$\text{FC} \% = \begin{cases} 0 & \text{if } I_c + C_{FC} < 1.71 \\ 80(I_c + C_{FC}) - 137 & \text{if } 1.71 \leq I_c + C_{FC} < 2.96 \\ 100 & \text{if } 2.96 \leq I_c + C_{FC} \end{cases}$$

$I_c$ is based on the normalised CPT tip resistance ($Q$) and normalized friction ratio ($F$) as recommended by Robertson and Wride (1998) and Youd et al. (2001). The $I_c$ is also used as a limit where, if $I_c$ exceeds the specified $I_c$ cut-off value, the soil is not considered susceptible and the liquefaction triggering assessment for these soil layers is not undertaken.

The Boulanger and Idriss (2014) liquefaction triggering procedure has a Cyclic Resistance Ratio (CRR) fitting parameter ($C_o$) that was used to fit CRR predictions to experimental results with probabilities of liquefaction of $P_L = 15\%$, $P_L = 50\%$, and $P_L = 85\%$

The total vertical stress ($\sigma_{vc}$) and effective vertical stress parameters ($\sigma_{vc}'$) are given by the expressions:

$$\sigma_{vc} = \gamma z \text{ and } \sigma_{vc}' = \sigma_{vc} - u$$

where $\gamma$ is the soil density and $u$ is the static porewater pressure defined by:

$$u = \begin{cases} 0 & \text{if } z < \text{GWD} \\ (z - \text{GWD}) \times 9.81 & \text{if } z \geq \text{GWD} \end{cases}$$

where GWD is the depth to the ground water below the ground surface.
The $\gamma$, $C_{FC}$, $I_{c}cutoff$, earthquake magnitude ($M_{w}$), Peak Ground Acceleration (PGA), $P_{L}$ and GWD parameters used for the automated LSN assessment in Christchurch for ILV assessment purposes are summarised in Table I2.1.

Each of the input parameters listed in Table I2.1 and the reason for why the associated values have been adopted are discussed in Appendix A.

**Table I2.1: Input Parameters used for the automated regional liquefaction triggering assessment for ILV assessment purposes**

<table>
<thead>
<tr>
<th>Input parameter</th>
<th>Default value adopted</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil Density ($\gamma$)</td>
<td>18 kN/m$^3$</td>
<td>Not sensitive to the typical variability in soil density in Christchurch (Tonkin &amp; Taylor, 2013)</td>
</tr>
<tr>
<td>Fitting parameter $C_{IC}$</td>
<td>$C_{IC} = 0.0$</td>
<td>Appropriate upper bound value for Christchurch soils (Lees, et al., 2015)</td>
</tr>
<tr>
<td>$I_{c}cutoff$</td>
<td>$I_{c}cutoff = 2.6$</td>
<td>Appropriate value for Christchurch soils (Lees, et al., 2015)</td>
</tr>
<tr>
<td>Level of earthquake shaking</td>
<td>$M_{w} = 6.0$, PGA = 0.3g</td>
<td>Critical case for 100 year return period levels of earthquake shaking using the BI 2014 methodology</td>
</tr>
<tr>
<td>Probability of Liquefaction ($P_{L}$)</td>
<td>$P_{L} = 15%$</td>
<td>Based on standard engineering design practice</td>
</tr>
</tbody>
</table>
| Depth to Groundwater (GWD)             | Surrogate median groundwater surface for the post-CES ground surface elevation and offsets from the surrogate median groundwater surface for the pre-CES ground surface elevation | Based on the GNS groundwater model (van Ballegooiy, et al., 2014a) Two key assumptions associated with GWD are:  
  - The groundwater profile is hydrostatic below the ground water surface; and  
  - The soils are fully saturated below the groundwater surface. |

### I2.2 Assessment of Liquefaction Vulnerability

The LSN parameter was developed to assess the liquefaction vulnerability of residential land in Canterbury in future earthquakes and was validated against the CES land damage observations. The LSN is defined as:

$$LSN = 1000 \int_{GW\overline{D}}^{10m} \frac{\varepsilon_{v}(z)}{z} dz$$

where $\varepsilon_{v}(z) = \text{the volumetric densification strain at depth, } z$, based on Zhang et al. (2002), which is a function of the FS (described in Section I2.1) and the normalised clean sand CPT tip resistance ($q_{c1N_{CS}}$).

Extensive studies have been undertaken on assessing the vulnerability of land to liquefaction damage (summarised in Appendix A). These studies show that liquefaction triggering of soil layers more than 10m below the ground surface provides a negligible contribution to liquefaction damage at the ground surface. Therefore, the regional LSN models are based on the top 10m of the soil profile only.

### I3. LSN Slicing Methodology

In order for the information from a CPT to be used, a sufficient soil profile is required. A process of the LSN slicing has been developed to make use of the data available in the 3,000 CPT on the CGD
that do not contain a sufficient length of profile, without reducing the accuracy of the estimated LSN values by introducing arbitrary LSN slices from neighbouring CPT for the missing portions of the CPT profiles.

The general LSN slicing methodology applied is as follows:

1. The LSN value is estimated for each CPT as described in Section I2 and is then broken down into 16 contributing slices in the upper 10m of the soil profile. The contribution from slices below 10m is not considered;
2. All CPT data is taken into account and the LSN slices are estimated for each of the slices.
3. LSN slices which are in the pre-drill part of the CPT and extend deeper than the GWD or which stop short of 10m are replaced with “NULL” values;
4. All LSN slices above the median groundwater table are assigned an LSN value of 0 (regardless of whether they are in the pre-drill part of the CPT) as they are unlikely to liquefy;
5. CPT containing any NULL values are identified. The LSN slice layers from surrounding CPT in a geologically similar area (based on the areas shown in Figure I3.1) and within 50m are used to replace the NULL value with a LSN slice;
6. A proportional distance weighting is used if more than one CPT in a geologically similar area are within 50m and able to contribute to the LSN slice value (i.e. the LSN slice values from nearby CPT have a higher weighting compared to those from CPT that are further away);
7. CPT which are shorter than a depth of 5m are not extended using the LSN slice methodology (i.e. all CPT are used to contribute towards LSN slice values but only CPT greater than 5m deep are extended to a 10m depth if neighbouring CPT LSN slices are is available); and
8. The LSN slice values of each slice down the CPT profile are summed to give a single overall adjusted LSN value at each CPT location with missing data from the soil profile.

The LSN slicing methodology is demonstrated using the schematic drawing in Figure I3.2.
Figure 13.1: Geological areas used as slice interpolation zones.
Figure I3.2: Schematic example of the LSN slicing method
I4. LSN Interpolation Process

The estimated LSN values at each CPT location need to be interpolated in order to obtain, on a regional basis, LSN values specific to each property. The LSN interpolation is based on the Natural Neighbour (NN) method with inverse distance weighting (Shepard’s basic formulae). The general methodology used to apply these methods is as follows.

1. The location of the complete set of CPT and their corresponding LSN values are plotted. Interpolation boundaries are then applied along major watercourses, geological units and other obvious locations as shown on Figure I4.1;
2. Each of the sub-areas that is produced is interpolated separately using natural neighbours and the result is clipped back to its defined boundary;
3. The results are then mosaicked to create a single continuous raster of continuous LSN values.

CPT profiles less than 5m deep or with a pre-drill depth greater than 2m are excluded from the analyses.

Following the interpolation process, a minor extrapolation at the perimeter of the grid is carried out. This is described in Section I5 below.

Figure I4.1: Spatial distribution of CPTs and interpolation extents.

I5. LSN Extrapolation Process

The extrapolation process is used around the perimeter of the site investigation regions to include areas that are within 50m of a site investigation location.

1. All CPT that are less than 50m from the boundary are selected.
The estimated LSN values of each CPT are extrapolated for each grid cell up to 50m beyond the boundary as shown in Figure I5.1 below. No extrapolation is carried out across any defined break line.

Where more than one CPT fall within 50m of each other, Inverse Distance Weighting (IDW) is applied to estimate the LSN value at that point.

The interpolated raster is then overlaid on the extrapolated raster to produce the final raster.

A series of figures showing the extrapolation process is set out in Figure I5.1.
16. References

Boulanger, R. W. & Idriss, I. M. 2014. CPT and SPT based liquefaction triggering procedures. (Report No. UCD/CGM-14/01). Center for Geotechnical Modelling, Department of Civil and Environmental Engineering, University of California, Davis, CA.


Appendix J: Liquefaction Vulnerability Parameter Sensitivity Analyses

Table J1.1 – Classifications used for CPT and borehole log review

CPT Examples J1 – J11
This appendix presents examples of each of the classifications used for the review of CPT and boreholes during the Stage 2 ILV qualification process. The different classifications are presented in Table J1.1 followed by the examples.

**Table J1.1: Classifications used for CPT and borehole review**

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Example</th>
</tr>
</thead>
<tbody>
<tr>
<td>✔</td>
<td>Post-CES median LSN at M6 0.3g greater than or equal to 16 and ΔLSN greater than or equal to 5</td>
<td>J1</td>
</tr>
<tr>
<td>✗</td>
<td>Post-CES median and mean LSN at M6 0.3g less than or equal to 14; or (Post-CES median and mean LSN at M6 0.3g between 14 and 16 and ΔLSN less than 5.)</td>
<td>J2</td>
</tr>
<tr>
<td>Mean✓</td>
<td>Post-CES mean LSN at M6 0.3g greater than or equal to 16 and mean ΔLSN greater than or equal to 5</td>
<td>J3</td>
</tr>
<tr>
<td>NC</td>
<td>Post-CES median and mean LSN at M6 0.3g greater than or equal to 16 and ΔLSN less than 4.</td>
<td>J4</td>
</tr>
<tr>
<td>M</td>
<td>Post-CES LSN at M6 0.3g greater than or equal to 16 and ΔLSN between 4 and 5; or (Post-CES median and mean LSN at M6 0.3g between 14 and 16 and ΔLSN greater than or equal to 5.)</td>
<td>J5.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>J5.2</td>
</tr>
<tr>
<td>PD</td>
<td>Predrill on CPT affects LSN result</td>
<td>J6</td>
</tr>
<tr>
<td>?</td>
<td>CPT data quality is questionable</td>
<td>J7</td>
</tr>
<tr>
<td>SC</td>
<td>Stress Change – ΔLSN at M6 0.3g is less than 5; but ΔLSN is greater than or equal to 5 for any M6 PGA less than 0.3g where the corresponding post-CES LSN is greater than 16</td>
<td>J8</td>
</tr>
<tr>
<td>S</td>
<td>CPT depth is less than 5m</td>
<td>J9</td>
</tr>
<tr>
<td>✓h</td>
<td>LSN &gt; 16 and ΔLSN &gt; 5 at M6 0.3g but LSN is hypersensitive to very small changes in depth of groundwater (typically where groundwater is shallower than 0.5m)</td>
<td>J10</td>
</tr>
<tr>
<td>GW</td>
<td>Significant difference between the mean and median groundwater levels noted</td>
<td>J11</td>
</tr>
</tbody>
</table>
**Table:**

<table>
<thead>
<tr>
<th>CGD ID: 15566</th>
<th>LSN Pre Sept = 24</th>
<th>LSN Post Dec = 36</th>
<th>ΔLSN = 11.7</th>
</tr>
</thead>
</table>

**Key:**
- **Pre Sept** GWD 1.3m  
  Predrill Depth = 0m  
  CFC = 0  
  Ic Limit = 2.6
- **Post Dec** GWD 0.8m  
  CPT Depth = 10m  
  CPT Rig: Geoprobe66

**Investigation Date:** 25/01/2013

**Supervising Company:** Tonkin & Taylor Ltd

**Drilling Company:** RDCL

**Graphs:**
- **LSN response to PGA**
- **CTL response to PGA**
- **Calculated Settlement (S) response to PGA**
- **LSN response to GWD**
- **CTL response to GWD**
- **Calculated Settlement (S) response to GWD**

**Notes:**
- LSN > 16
- ΔLSN > 5
Key:
- **Pre Sept GWD 3.2m**
  - Predrill Depth = 0m
- **Post Dec GWD 2.9m**
  - CPT Depth = 19.92m
- CFC = 0
- Ic Limit = 2.6

Investigation Date: 30/11/2012
Supervising Company: Coffeys
Drilling Company: Coffey Information
CPT Rig: PG200

**LSN response to PGA**

**CTL response to PGA**

**Calculated Settlement (S) response to PGA**

**LSN response to GWD**

**CTL response to GWD**

**Calculated Settlement (S) response to GWD**
J3

<table>
<thead>
<tr>
<th>CGD ID: 49423</th>
<th>LSN Pre Sept = 32</th>
<th>LSN Post Dec = 36</th>
<th>ΔLSN = 3.4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Key:</td>
<td>Investigation Date: 3/11/2014</td>
<td>Supervising Company: Geotechnics LTD</td>
<td>Drilling Company: Geotechnics</td>
</tr>
<tr>
<td></td>
<td>Predrill Depth = 0m</td>
<td>CPT Depth = 9.64m</td>
<td>CPT Rig: Georig220</td>
</tr>
<tr>
<td>Pred Sept GWD 1.2m</td>
<td>CPT Depth = 9.64m</td>
<td>ic Limit = 2.6</td>
<td></td>
</tr>
<tr>
<td>Post Dec GWD 0.8m</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CFC = 0</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Mean difference is: (14 + 6 + 2) ÷ 3 = 7.3
Therefore Mean ΔLSN > 5

Mean difference is: (14 + 6 + 2) ÷ 3 = 7.3
Therefore Mean ΔLSN > 5
<table>
<thead>
<tr>
<th>CGD ID: 36085</th>
<th>LSN Pre Sept = 17</th>
<th>LSN Post Dec = 20</th>
<th>ΔLSN = 2.8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Key:</td>
<td></td>
<td>Investigation Date: 5/10/2013</td>
<td></td>
</tr>
<tr>
<td>Pre Sept GWD 1.3m</td>
<td>Predrill Depth = 0m</td>
<td>Supervising Company: Cook Costello Ltd</td>
<td></td>
</tr>
<tr>
<td>Post Dec GWD 1.1m</td>
<td>CPT Depth = 19.04m</td>
<td>Drilling Company: GEOCIVIL</td>
<td></td>
</tr>
<tr>
<td>CFC = 0</td>
<td>Ic Limit = 2.6</td>
<td>CPT Rig: Geomil LWC 200</td>
<td></td>
</tr>
</tbody>
</table>

- LSN > 16
- ΔLSN < 5
Key:
- **Pre Sept** GWD 0.8m, Predrill Depth = 0m
- **Post Dec** GWD 0.4m, CPT Depth = 10.23m
- CFC = 0, Ic Limit = 2.6

Investigation Date: 25/07/2014
Supervising Company: Tonkin & Taylor Ltd
Drilling Company: Lankelma

CPT Rig: Lankelma Geomil 200kN (UK18)

---

**LSN response to PGA**

**CTL response to PGA**

**Calculated Settlement (S) response to PGA**

**LSN response to GWD**

**CTL response to GWD**

**Calculated Settlement (S) response to GWD**
CGD ID: 25870  LSN Pre Sept = 13  LSN Post Dec = 18  ΔLSN = 4.9

Key:
- Pre Sept GWD 1.6m  Predrill Depth = 0m
- Post Dec GWD 1.3m  CPT Depth = 20m
- CFC = 0  Ic Limit = 2.6

Investigation Date: 9/04/2013
Supervising Company: Golder Associates
Drilling Company: GCL
CPT Rig: Georig220

ΔLSN between 4 and 5

LSN response to PGA

CTL response to PGA

Calculated Settlement (S) response to PGA

LSN response to GWD

CTL response to GWD

Calculated Settlement (S) response to GWD
<table>
<thead>
<tr>
<th>CGD ID: 1428</th>
<th>LSN Pre Sept = 35</th>
<th>LSN Post Dec = 49</th>
<th>ΔLSN = 14</th>
</tr>
</thead>
<tbody>
<tr>
<td>Key:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Pre Sept GWD 0.8m</td>
<td>Predrill Depth = 1.2m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Post Dec GWD 0.5m</td>
<td></td>
<td>CPT Depth = 10.54m</td>
<td></td>
</tr>
<tr>
<td>CFC = 0</td>
<td></td>
<td>Ic Limit = 2.6</td>
<td></td>
</tr>
</tbody>
</table>

- Investigation Date: 10/06/2011
- Supervising Company: Tonkin & Taylor Ltd
- Drilling Company: Geotech Drilling Ltd
- CPT Rig: N/A

### Graphs

**LSN response to PGA**

![LSN response to PGA](image)

**CTL response to PGA**

![CTL response to PGA](image)

**Calculated Settlement (S) response to PGA**

![Calculated Settlement (S) response to PGA](image)

**LSN response to GWD**

![LSN response to GWD](image)

**CTL response to GWD**

![CTL response to GWD](image)

**Calculated Settlement (S) response to GWD**

![Calculated Settlement (S) response to GWD](image)
Unusual LSN graph (hyperbolic shape). Upon further investigation, reveals unreliable raw CPT data.
<table>
<thead>
<tr>
<th>Key:</th>
<th>Predrill Depth = 0m</th>
<th>CPT Depth = 3.9m</th>
<th>CFC = 0</th>
<th>Ic Limit = 2.6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre Sept GWD 1m</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Post Dec GWD 0.7m</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>CGD ID: 45974</th>
<th>LSN Pre Sept = 2</th>
<th>LSN Post Dec = 2</th>
<th>ΔLSN = 0</th>
</tr>
</thead>
<tbody>
<tr>
<td>Investigation Date:</td>
<td>3/05/2014</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Supervising Company:</td>
<td>Cook Costello Ltd</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Drilling Company:</td>
<td>GEOCIVIL</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CPT Rig:</td>
<td>Geomil LWC 200</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Depth ≤ 5m**

**LSN response to PGA**

**CTL response to PGA**

**Calculated Settlement (S) response to PGA**

**LSN response to GWD**

**CTL response to GWD**

**Calculated Settlement (S) response to GWD**
<table>
<thead>
<tr>
<th>CGD ID: 37940</th>
<th>LSN Pre Sept = 17</th>
<th>LSN Post Dec = 49</th>
<th>ΔLSN = 32</th>
</tr>
</thead>
<tbody>
<tr>
<td>Investigation Date:</td>
<td>27/02/2014</td>
<td>Supervising Company:</td>
<td>Davis Ogilvie</td>
</tr>
<tr>
<td>Predrill Depth = 0m</td>
<td>CPT Depth = 20.2m</td>
<td>Drilling Company:</td>
<td>Fugro</td>
</tr>
<tr>
<td>CFC = 0</td>
<td>Ic Limit = 2.6</td>
<td>CPT Rig:</td>
<td>20 Tonne Fugro CPT truck</td>
</tr>
</tbody>
</table>

**Key:**
- **Pre Sept GWD 0.3m**
- **Post Dec GWD 0.1m**

Example 10 - "Hypersensitive" CPT (ΔLSN is >> 5 because DGW is very shallow)

**LSN response to PGA**

**CTL response to PGA**

**Calculated Settlement (S) response to PGA**

**LSN response to GWD**

**CTL response to GWD**

**Calculated Settlement (S) response to GWD**
**Key:**
- **Pre Sept** GWD 1.3m  Predrill Depth = 0m
- **Post Dec** GWD 0.9m  CPT Depth = 20.12m
- CFC = 0  Ic Limit = 2.6

**Investigation Date:** 8/05/2013

**Supervising Company:** Cook Costello Ltd

**Drilling Company:** GEOCIVIL

**CPT Rig:** Geomil LWC 200

**Median GW is greater than 85th percentile model or less than 15th percentile**
Appendix K: Maps used for the Regional Assessment of ILV
Appendix K1 - Maps used for Regional Assessment of ILV

- Figure K1.1 – MBIE Technical Category Map
- Figure K1.2 – Ground Surface Elevation Map Post-June 2011/Post-Dec 2011 DEM
- Figure K1.3 – Total Elevation Change – Pre-Sept 2010 to Post-June/Post-Dec 2011
- Figure K1.4 – Liquefaction Related Elevation Change – Pre-Sept 2010 to Post-June/Post-Dec 2011
- Figure K1.5 to K1.8 – Land Damage Observations After the September 2010, February 2011, June 2011 and December 2011 Events
- Figure K1.9 to K1.11 – Depth to Ground Water Contour: 15th, 50th and 85th Percentile Surface
- Figure K1.12 – Depth to Ground Water Contour: Difference Between the 15th and 85th Percentile
- Figures K1.13 to K1.17 – Median Normalised CPT Tip Resistance ($q_{\text{CN}}$) at 1m intervals: Depth 0m to 5m
- Figures K1.18 to K1.22 – Median Soil Behaviour Type Index ($I_c$) at 1m intervals: Depth 0m to 5m
- Figure K1.23 – Depth to First Hard Layer Greater than 0.5m Thick
- Figures K1.24 to K1.26 – Sensitivity of LSN to PGA: PGA = 0.28 to 0.32g; $M_w = 6.0$
- Figures K1.27 to K1.29 – Sensitivity of LSN to $P_L$: $P_L = 15^\text{th}$, 50th and 85th percentile
- Figure K1.30 to K1.31 – Liquefaction Susceptibility Criteria Banded by the Soil Behaviour Type Index: Depth 0m to 5m and Depth 5m to 10m
- Figure K1.32 to K1.33 – Comparison of Fines Content Banded by the Soil Behaviour Type Index: Depth from 0m to 5m and 5m to 10m
- Figures K1.34 to K1.39 – Sensitivity of LSN to $C_{\text{FC}}$ and $I_c$ Cutoff: $C_{\text{FC}} = 0$ and 0.2; $I_c$ Cutoff = 2.4, 2.6 and 2.8
LEGEND

Worked example locations

Post December 2011 DEM Extent

Post June 2011 Ground Elevation (m RL)

High : 24

Low : 0

EARTHQUAKE COMMISSION
CHRISTCHURCH CITY & KAIPOI
Ground Surface Elevation Map
Post-June 2011/Post-Dec 2011 DEM

Notes: NZ Mainland Road Centerlines, NZ Coastlines and Island Polygons sourced from Land Information New Zealand under CC-BY.
EARTHQUAKE COMMISSION
CHRISTCHURCH CITY & KAIAPOI
Total Elevation Change - Pre-Sept 2010 to Post-June/Post-Dec 2011 Event


LEGEND

- Worked example locations
- Post December 2011 Difference DEM Extent

Elevation Change (m)

- < -1.0
- -1.0 to -0.5
- -0.5 to -0.4
- -0.4 to -0.3
- -0.3 to -0.2
- -0.2 to -0.1
- -0.1 to +0.1
- +0.1 to +0.2
- +0.2 to +0.3
- +0.3 to +0.4
- +0.4 to +0.5
- +0.5 to +1.0
- +1.0

SCALE (AT A3 SIZE)
Prepared by Tonkin & Taylor Ltd.
EARTHQUAKE COMMISSION
CHRISTCHURCH CITY & KAIAPOI
Liquefaction Related Elevation Change - Pre-Sept 2010 to Post-June/Post-Dec 2011 Event


Kaiapoi & Northern Suburbs

LEGEND
- Worked example locations
- Post December 2011 Difference DEM Extent

Elevation Change (m)
- < -1.0
- -1.0 to -0.5
- -0.5 to -0.4
- -0.4 to -0.3
- -0.3 to -0.2
- -0.2 to -0.1
- -0.1 to +0.1
- +0.1 to +0.2
- +0.2 to +0.3
- +0.3 to +0.4
- +0.4 to +0.5
- +0.5 to +1.0
- +1.0

EARTHQUAKE COMMISSION
52020-0200-LCC129
K1.4
DRAWN
7/08/2015
Time: 9:53:53 a.m.
Prepared by Tonkin & Taylor Ltd.
Ref: ARCFILE
DRAWN
CHECKED
APPROVED
FIGURE No.
Rev.
1
2
3
4
5
0 1 2 3 4 (km)
Land Damage Observations After the September 2010 Event

Legend:
- Worked example locations
- Land Damage Observation Category:
  1 - None observed
  2 - Minor
  3 - Moderate
  4 - Major
  5 - Severe
  6 - Very Severe
- PGA contours (g)
  - Major
  - Minor
- Station Location and Maximum PGA Readings (g)
  - Vertical acceleration
  - Horizontal acceleration
- Piezometer Locations:
  - ECAN monitoring well
  - EQC monitoring well
  - CCC monitoring well
  - Drawdown analysis piezometer

Notes:
New Zealand Mainland Road Centerlines, NZ Coastlines and Island Polygons sourced from Land Information New Zealand under CC-BY.

Prepared by Tonkin & Taylor Ltd.

Scale: A3 SCALE 1:60,000

Date: 6/08/2015 Time: 4:18:32 p.m.

EARTHQUAKE COMMISSION
CHRISTCHURCH CITY & KAIAPOI
Land Damage Observations After the September 2010 Event

Kaiapoi & Northern Suburbs
0.5 m
0.25 m
0 m
0.5 m
0.25 m
0 m
0.5 m
0.25 m
0 m

LEGEND

Worked example locations

Difference (m)

0 - 0.25
0.25 - 0.5
0.5 - 0.75
0.75 - 1.0
> 1.0

Monitoring Wells

EQC Monitoring Well
> 12 Data Readings
ECan Monitoring Well
< 12 Data Readings
CCC Monitoring Well

Other Analysis Data

Coast
Transitional
Tidal
Non Tidal

Drawdown Points

Notes:
Enter data sourced from Land Information New Zealand under CC-BY.
http://creativecommons.org/licenses/by/3.0/nz/

Christchurch City
Kaiapoi & Northern Suburbs

A3 SCALE 1:60,000

Prepared by Tonkin & Taylor Ltd.

Road Database supplied by Terralink International Ltd.
Contains data sourced from Land Information New Zealand under CC-BY.
http://creativecommons.org/licenses/by/3.0/nz/
LEGEND

- Worked example locations

Soil Behaviour Type Index (Ic)
- < 1.8
- 1.8 - 2.0
- 2.0 - 2.1
- 2.1 - 2.2
- 2.2 - 2.3
- 2.3 - 2.4
- 2.4 - 2.5
- 2.5 - 2.6
- 2.6 - 2.8
- > 2.8

Notes: NZ Mainland Road Centerlines, NZ Coastlines and Island Polygons sourced from Land Information New Zealand under CC-BY.

A3 SCALE 1:60,000
LEGEND

Worked example locations

Soil Behaviour Type Index (Ic)

- < 1.8
- 1.8 - 2.0
- 2.0 - 2.1
- 2.1 - 2.2
- 2.2 - 2.3
- 2.3 - 2.4
- 2.4 - 2.5
- 2.5 - 2.8
- 2.6 - 2.8
- > 2.8

Notes: NZ Mainland Road Centerlines, NZ Coastlines and Island Polygons sourced from Land Information New Zealand under CC-BY.
LEGEND

Worked example locations

Depth

- 0 - 2
- 2 - 4
- 4 - 6
- 6 - 8
- 8 - 10
- 10+

Notes: NZ Mainland Road Centerlines, NZ Coastlines and Island Polygons sourced from Land Information New Zealand under CC-BY.
LEGEND

- Worked example locations

LSN

- < 14
- 14 - 16
- 16 - 18
- 18 - 25
- 25 - 40
- > 40

Notes: NZ Mainland Road Centerlines, NZ Coastlines and Island Polygons sourced from Land Information New Zealand under CC-BY.

EARTHQUAKE COMMISSION
CHRISTCHURCH CITY & KAIAPOI
Sensitivity of LSN to PGA:
PGA = 0.28; Mw = 6.0

Notes:
- NZ Mainland Road Centerlines, NZ Coastlines and Island Polygons sourced from Land Information New Zealand under CC-BY.
LEGEND

- Worked example locations

LSN

- < 14
- 14 - 16
- 16 - 18
- 18 - 25
- 25 - 40
- > 40

Notes: NZ Mainland Road Centerlines, NZ Coastlines and Island Polygons sourced from Land Information New Zealand under CC-BY.

Date: 20/07/2015 Time: 12:00:20 p.m.

Scale: A3 1:60,000

Christchurch City

Kaiapoi & Northern Suburbs

EARTHQUAKE COMMISSION
CHRISTCHURCH CITY & KAIAPOI
Sensitivity of LSN to PGA:
PGA = 0.32; Mw = 6.0

Prepared by Tonkin & Taylor Ltd.

Ref:
ARCFILE

DRAWN
ACAM 05/15

CHECKED
APPROVED

FIGURE No.
Rev.

NZ Mainland Road Centerlines, NZ Coastlines and Island Polygons sourced from Land Information New Zealand under CC-BY.
Comparison of Fines Content Banded by the Soil Behaviour Type Index: Depth from 0 m to 5 m

Legend
- Worked example locations
- Calculated Cfc Values Based on Laboratory Testing and CPT Traces
  - < -0.2
  - -0.2 - -0.1
  - -0.1 - 0
  - 0 - 0.1
  - 0.1 - 0.2
  - > 0.2
  - Paired but Ic > 2.6
  - Unpaired Laboratory Fines Content Tests

Notes: NZ Mainland Road Centerlines, NZ Coastlines and Island Polygons sourced from Land Information New Zealand under CC-BY.
Comparison of Fines Content Banded by the Soil Behaviour Type Index: Depth from 5 m to 10 m

Legend
- Worked example locations
- Calculated Cr/C Values Based on Laboratory Testing and CPT Traces
  - <-0.2
  - -0.2 - -0.1
  - -0.1 - 0
  - 0 - 0.1
  - 0.1 - 0.2
  - > 0.2
  - Paired but Ic > 2.6
  - Unpaired Laboratory Fines Content Tests

Notes: NZ Mainland Road Centerlines, NZ Coastlines and Island Polygons sourced from Land Information New Zealand under CC-BY.
LEGEND

- Worked example locations

LSN

- < 14
- 14 - 16
- 16 - 18
- 18 - 25
- 25 - 40
- > 40

Notes: NZ Mainland Road Centerlines, NZ Coastlines and Island Polygons sourced from Land Information New Zealand under CC-BY.

A3 SCALE 1:60,000

Date: 20/07/2015 Time: 1:28:03 p.m.

Prepared by Tonkin & Taylor Ltd.

Ref: ARCFILE

DRAWN

CHECKED

APPROVED

FIGURE No. Rev.

NZ Mainland Road Centerlines, NZ Coastlines and Island Polygons sourced from Land Information New Zealand under CC-BY.
LEGEND

- Worked example locations

LSN

- < 14
- 14 - 16
- 16 - 18
- 18 - 25
- 25 - 40
- > 40

EARTHQUAKE COMMISSION

CHRISTCHURCH CITY & KAIPOI

Sensitivity of LSN to $C_{FC}$ and $I_{C}$ Cutoff: $C_{FC} = 0.2$; $I_{C}$ Cutoff = 2.4

Notes: NZ Mainland Road Centerlines, NZ Coastlines and Island Polygons sourced from Land Information New Zealand under CC-BY.
EARTHQUAKE COMMISSION
CHRISTCHURCH CITY & KAIAPOI

Sensitivity of LSN to $C_{FC}$ and $I_C$ Cutoff:
$C_{FC} = 0; I_C$ Cutoff = 2.6

LEGEND

- Worked example locations

LSN
- < 14
- 14 - 16
- 16 - 18
- 18 - 25
- 25 - 40
- > 40

Notes: NZ Mainland Road Centerlines, NZ Coastlines and Island Polygons sourced from Land Information New Zealand under CC-BY.

Christchurch City

Kaiapoi & Northern Suburbs
LEGEND

- Worked example locations

LSN
- < 14
- 14 - 16
- 16 - 18
- 18 - 25
- 25 - 40
- > 40

Notes:
- NZ Mainland Road Centerlines, NZ Coastlines and Island Polygons sourced from Land Information New Zealand under CC-BY.

EARTHQUAKE COMMISSION
CHRISTCHURCH CITY & KAIAPOI
Sensitivity of LSN to $C_{FC}$ and $I_{C}$ Cutoff:
$C_{FC} = 0.2; I_{C} \text{ Cutoff } = 2.6$
Sensitivity of LSN to $C_{FC}$ and $I_C$ Cutoff:

- $C_{FC} = 0$; $I_C$ Cutoff = 2.8

LEGEND

- Worked example locations
- LSN
  - < 14
  - 14 - 16
  - 16 - 18
  - 18 - 25
  - 25 - 40
  - > 40

Notes: NZ Mainland Road Centerlines, NZ Coastlines and Island Polygons sourced from Land Information New Zealand under CC-BY.
EARTHQUAKE COMMISSION
CHRISTCHURCH CITY & KAIAPOI
Sensitivity of LSN to $C_{FC}$ and $I_C$ Cutoff:
$C_{FC} = 0.2$; $I_C$ Cutoff = 2.8

LEGEND

- Worked example locations
- LSN

$< 14$
$14 - 16$
$16 - 18$
$18 - 25$
$25 - 40$
$> 40$

Notes: NZ Mainland Road Centerlines, NZ Coastlines and Island Polygons sourced from Land Information New Zealand under CC-BY.
Appendix K2 – Additional Maps Relevant To the ILV Assessment Process

- Figure K2.1 – Overall Differential Foundation Settlement
- Figure K2.2 – Building Damage Ratio (BDR)
- Figure K2.3 – Liquefaction Severity Number
- Figure K2.4 – Change in Liquefaction Severity Number
- Figure K2.5 – Automated ILV Model (BI2014)
- Figure K2.6 – Liquefaction Vulnerability
CBD

LEGEND
BDR
0 - 0.2
0.2 - 0.5
0.5 - 0.75
> 0.75

Christchurch City

Notes: NZ Mainland Road Centerlines, NZ Coastlines and Island Polygons sourced from Land Information New Zealand under CC-BY.
Appendix L: Summary of Worked Example 5
North New Brighton
L1 Worked Example 5 - North New Brighton

L1.1 Introduction

The Stage 2 properties in this worked example required further manual assessment because the liquefaction vulnerability analyses of the CPT in this area indicates high levels of variability at M6 0.3g levels of earthquake shaking. This variability is with respect to both the assessment of material liquefaction vulnerability and the assessment of material change in liquefaction vulnerability.

The purpose of this worked example is to demonstrate an area where there are subtle changes in ground conditions resulting in variable predicted performance of the land making the application of the ILV criteria challenging. The assessment process for the Stage 2 properties in this worked example required the evaluation of the significant factors for each property. It also required the assessment of whether the ILV qualification is more appropriately based on the individual property assessment using the predicted behaviour from CPT profiles within each property, or whether the ILV qualification is more appropriately based on the assessment of a block of land. This is similar to the appropriate approach to liquefaction assessment for foundation design purposes in this area as described in Russell et al. (2015b).

Due to the very high density of CPT undertaken in this area and large number of Stage 2 properties included into this worked example, the assessment process has been broken down into four smaller areas (areas A, B, C and D) as shown in Figure L1.1. While these areas have been discussed separately, they need to be considered concurrently because each area performs differently and contrast between these areas helps to improve understanding of why some areas qualify for ILV and other do not.

Figure L1.1: The location of the Stage 2 properties requiring further manual assessment in North New Brighton. Areas A, B, C and D indicate the location of the four sub areas presented in the Stage 2 ILV assessment example.
L2  Stage 1 Assessment

L2.1  Area A
Review of the automated ILV model indicates that three of the Stage 2 properties in Area A are unlikely to qualify for ILV. While the other five properties are indicated as likely to qualify for ILV based on the automated model, CPT analyses adjacent to or on the properties are indicating variable results.

L2.2  Area B
Review of the automated ILV model indicates that all except one of the Stage 2 properties in Area B are unlikely to qualify for ILV.

L2.3  Area C
Review of the automated ILV model indicates that the Stage 2 properties in Area C are unlikely to qualify for ILV.

L2.4  Area D
Review of the automated ILV model indicates that three of the Stage 2 properties in Area D are likely to qualify for ILV, six are marginal and the remaining nine properties are unlikely to qualify for ILV.

L3  Stage 2 Assessment

L3.1  Summary of stage 2 assessment

L3.1.1  Area A
As a result of the Stage 2 ILV assessment, all of the Stage 2 properties in Area A have been assessed as qualifying for ILV based on engineering judgement.

L3.1.2  Area B
As a result of the Stage 2 ILV assessment, this automated result has been overturned and all of these properties have been assessed as qualifying for ILV based on engineering judgement.

L3.1.3  Area C
As a result of the Stage 2 ILV assessment, both of these properties have been assessed as qualifying for ILV based on engineering judgement.

L3.1.4  Area D
As a result of the ILV Stage 2 assessment, the four Stage 2 properties to the east have been assessed as qualifying for ILV whereas the fourteen properties to the west have been assessed as not qualify for ILV based on engineering judgement.

The full example pack including discussion and analysis of all the information listed in Section 10.2.2 along with the regional, specific and local analyses (described in Sections 10.2.3, 10.2.4 and 10.2.5 respectively) is included in the supplementary information titled “Worked Example Material”. This full pack demonstrates the complete process for determining the ILV decision of these properties. A summary of the pertinent reasons for this ILV assessment outcome are as follows:
L3.2 Regional Assessment (Task 1)

The land requiring Stage 2 ILV assessment in the four areas (areas A, B, C and D) is located approximately 650m to the north east of the Avon River and approximately 1km to the west of New Brighton beach. The land slopes gently downwards in a south westerly direction towards the Avon River and is at a typical elevation of between 3 to 4m RL.

No land damage was observed in September 2010 (equivalent to M6 0.24 levels of earthquake shaking) apart from some at the southern end where land damage was minor-to-moderate. The land damage was moderate-to-severe in the west (where the groundwater is shallower) and minor-to-moderate in the east (where the groundwater is deeper and the soil is denser) in the February 2011 earthquake (equivalent to M6 0.5g levels of earthquake shaking) and minor-to-moderate in June 2011 (equivalent to M6 0.25g levels of earthquake shaking) and December 2011 (equivalent to M6 0.36g levels of earthquake shaking).

It is noted that both the June 2011 and December 2011 events were preceded by large foreshocks 80 minutes prior to the main earthquake. These foreshocks will have resulted in some development of excess pore water pressure in the soil layers with the lowest resistance to liquefaction triggering (Quigley et al., 2015) which may have resulted in more severe and extensive liquefaction triggering in the soil profile and corresponding damage at the ground surface than would otherwise be anticipated for the estimated levels of ground shaking for the two earthquake events. Accordingly, the land damage observations for these two events relative to the estimated levels of ground shaking should be interpreted with caution.

Ground surface subsidence is variable but typically 0.2m to 0.3m for the properties towards the east and generally 0.5m to 1m for the properties towards the west. Review of the estimated vertical tectonic movement indicates that this ground surface subsidence is predominantly attributable to liquefaction related effects. An error band continues across the southern part of the area. However, the effect from the error band is considered to be relatively minor.

Despite the relatively large distance from the Avon River, ground cracking observations and horizontal ground movements measured using LiDAR data indicate that the area has been affected by lateral spreading in a south west direction (i.e. towards the Avon River). Similar to the foreshocks prior to the June 2011 and December 2011 events, the complex interactions between lateral spread and liquefaction induced subsidence in this area also affect the land damage observations and accordingly the land damage observations for the June and December 2011 events should be interpreted with caution.

Groundwater surface maps indicate that the median groundwater surface is typically 1 – 2m below the ground surface. There are some areas where the groundwater is shallower, between 0 to 1m below the ground surface, to the west and east of the properties requiring Stage 2 ILV assessment.

Geological maps indicate that the area is predominantly underlain by sand of fixed and semi-fixed dunes. This is supported by the regional $I_1$ map which indicates the presence of sandy material. Evidence of the sand dunes is apparent in the ground surface elevation maps although anthropogenic land form changes appear to have removed these features in most areas.

The regional $q_{c,NN}$ map indicates variability in the density of the subsoils in this area. This may be partly attributable to the land development as the tops of the sand dunes were trimmed off exposing denser sands below and the fill material being pushed into the interdune depressions resulting in localised areas of looser sand deposits. Even though density of the subsoils is variable, there is a spatial trend with relatively looser soils to the west of the stage 2 properties (Area A) and relatively denser soils to the east of the stage 2 properties.
L3.3 Area A

L3.3.1 Local and Specific Assessment (Tasks 2 and 3)

Figure L3.1a shows that the land slopes gently downwards towards the south west corner of Area A. The depth to groundwater map shows that the groundwater surface is approximately 1m below ground level beneath the Stage 2 properties.

Figure L3.1: A series of maps used when assessing Area A using the Stage 2 qualification process. (a) post-CES ground surface elevation; (b) post-CES depth to groundwater surface; (c) September 2010 observed land damage; (d) worst observed land damage across the CES; (e) total ground surface subsidence across the CES; (f) Stage 1 ILV qualification results.
Inspection of Figure L3.1c shows that some Stage 2 properties in the southern end of Area A had minor-to-moderate observed land damage following the September 2010 earthquake (equivalent to M6 0.24). The worst observed land damage across the CES is either minor-to-moderate or moderate-to-severe for all properties in Area A (refer to Figure L3.1d).

Ground surface subsidence of the Stage 2 properties in Area A is typically between 0.5 and 1 m (refer to Figure L3.1e). The majority of the properties surrounding the Stage 2 properties were qualified for ILV during the Stage 1 assessment process (refer to Figure L3.1f). These general observations of land damage and ground surface subsidence for the Stage 2 properties are consistent with the adjacent properties which were assessed as qualifying for ILV using the Stage 1 process.

Figure L3.2 shows that the CPT-based liquefaction vulnerability analyses within this area are generally classified as ✓. One of the Stage 2 properties within the area has two CPT-based liquefaction vulnerability analyses classified as ✗. Two Stage 2 properties have four NC CPT within the property boundaries. As discussed in Section 8, the automated model has variable prediction including No, Marginal and Yes qualifications for ILV in this area.

The plots of \( q_c \) and \( I_c \) vs depth for all the CPTs in Area A, in Figure L3.3 indicate that the subsoils are typically sands steadily increasing in density with depth, with the exception of a loose slightly silty layer at about 3 to 4 m depth.
As discussed above with reference to Figure L3.3 the majority of CPT are classified as ✓ within Area A. The CPT-based liquefaction vulnerability analysis classified as × or NC fit well within the overall envelope of the $q_c$ and $I_c$ vs depth plots and are generally similar to the ✓ CPT. It appears that the differences between the CPT classified as ✓, × or NC depends on minor variation in the sand and silt layers. As discussed in Russell et al. (2015b) this variability could reasonably be expected over a residential property and hence if additional CPT were to be obtained around the relatively few × and NC CPT, it is reasonable to expect that some of them would be given a ✓ classification.

**L3.3.2 ILV Qualification Assessment (Task 4)**

The Stage 2 properties within this area have been assessed as qualifying for ILV based on the following:

- Review of the land damage observations provides no reason to differentiate between the performance of the surrounding properties that were assessed as qualifying for ILV during the Stage 1 ILV assessment and the Stage 2 properties;
• For this particular area, interrogation of the subsoil conditions indicates that the differences between the CPT that were classified as ✗ and NC and those classified as ✓ is due to minor variation in the sand and silt layers;
• The majority of the CPT in this area are classified as ✓ and this classification reconciles well with the land performance, the ground surface subsidence which occurred and the soil conditions in this area;

Accordingly these stage 2 properties have been marked with a ✓ on Figure L3.2 to indicate that based on engineering judgement both engineering criteria have been satisfied in accordance with the objectives outlined in Section 2.6 and therefore these three properties qualify for ILV.
L3.4 Area B

L3.4.1 Local and Specific Assessment (Tasks 2 and 3)

Review of Figure L3.4a shows that the land in Area B is relatively flat with a gentle slope from the northwest to the southeast. This is reflected in the depth to groundwater map (Figure L3.4b) which shows the groundwater is shallower along the eastern boundary of Area B.

Figure L3.4: A series of maps used when assessing Area B using the Stage 2 qualification process. (a) post-CES ground surface elevation; (b) post-CES depth to groundwater surface; (c) September 2010 observed land damage; (d) worst observed land damage across the CES; (e) total ground surface subsidence across the CES; (f) Stage 1 ILV qualification results.
Figure L3.4c shows that the land damage observations in this area were none-to-minor in the September event (equivalent to M6 0.24g) and that the worst observed land damage over the CES was typically minor-to-moderate (Figure L3.4d).

Figure L3.4e shows that the estimated ground surface subsidence across the CES is typically 0.5 to 1m. The notable exception to this is the middle property of the three eastern most Stage 2 properties which has estimated subsidence of approximately 0.2 to 0.4m. Given that the land damage observations for this property across the CES are moderate-to-severe it is likely that this attributable to LiDAR measurement noise rather than a true measure of ground surface subsidence.

CPT classifications within this area are generally ✓, with some ✗ and NC dispersed throughout the area (refer to Figure L3.5). Three of the eastern properties have CPT classified as ✗ on the properties. The cluster of Stage 2 properties in the north west of the area have variable CPT classifications within or adjacent to the properties.

![Figure L3.5: The location and classification of the CPT and boreholes and the ILV qualification of the properties for the Stage 2 manual assessment pack in Area B.](image)

Based on the automated ILV model (refer to “Worked Example Material”) most of the Stage 2 properties are unlikely to qualify for ILV and are classified as either Automated Marginal or Automated No. The exception to this is the north western most Stage 2 property which is classified as Automated Yes.

The plots of $q_c$ and $I_c$ vs depth in Figure L3.6 indicate that the subsoils are typically sands increasing in density with depth. The density of the top 3 to 4m of these typically sand soils is generally less than the liquefaction triggering envelope for a soil with $I_c < 1.8$ (i.e. clean sands) and therefore liquefaction triggering of the upper 3 to 4m soils is likely at M6 0.3g levels of earthquake shaking. Between 3 and 5m there is an increase in density such that the sands below this depth are unlikely to liquefy at M6 0.3g levels of earthquake shaking. In summary, the CPTs in this area indicate that it is typically underlain by clean sands and that variations in liquefaction...
triggering and calculated LSN values are typically controlled by smaller variations in the soil density as well as small variations in the layer sequencing.

![Figure L3.6: Plots of \( q_c \) and \( l_c \) vs depth for CPT grouped by CPT classifications for the CPT identified in Figure L3.5.](image)

Similar to area A, the CPT-based liquefaction vulnerability analysis classified as \( \times \) or NC fit well within the overall envelope of the \( q_c \) and \( l_c \) vs depth plots and are generally similar to the \( \checkmark \) CPT. It appears that the differences between the CPT classified as \( \checkmark \), \( \times \) or NC depends on minor variations in the density of the sandy soils in this area (similar to the observations made in Russell et al., 2015b).

### L3.4.2 ILV Qualification Assessment (Task 4)

The Stage 2 properties within this area have been assessed as qualifying for ILV based on the following:

- Review of the land damage observations provides no reason to differentiate between the performance of the surrounding properties that were assessed as qualifying for ILV during the Stage 1 ILV assessment and the Stage 2 properties.
• The measured ground surface subsidence is generally 0.5 to 1m. The exception to this is the middle property of the three eastern most Stage 2 properties which has measured ground surface subsidence of approximately 0.2 to 0.4m. Based on the consistency of the land damage observations in this area this is attributed to LiDAR survey noise rather than a true measure of ground surface subsidence.

• For this particular area, interrogation of the subsoil conditions indicates that the differences between the CPT that were classified as × and NC and those classified as ✓ is due to minor variations in the density of the sandy soils in this area;

• The majority of the CPT in this area are classified as ✓ and this classification reconciles well with the observed land performance and soil conditions; and

• These general observations for the Stage 2 properties are consistent for the properties which were assessed as qualifying for ILV with the Stage 1 properties. Accordingly these seven properties have been marked with a ✓ on Figure L3.5 to indicate that based on engineering judgement both engineering criteria have been satisfied and therefore the stage 2 properties in Area B all qualify for ILV.
L3.5 Area C

L3.5.1 Local and Specific Assessment (Tasks 2 and 3)

Review of Figure L3.7a shows that the land in Area C slopes from the northern corner to the southern corner. Similar to Area B, this is reflected in the depth to groundwater (refer to Figure L3.7b) which shows a localised area of shallower groundwater along the west boundary.

![Maps showing various assessments](image)

Figure L3.7: A series of maps used when assessing Area C using the Stage 2 qualification process. (a) post-CES ground surface elevation; (b) post-CES depth to groundwater surface; (c) September 2010 observed land damage; (d) worst observed land damage across the CES; (e) total ground surface subsidence across the CES; (f) Stage 1 ILV qualification results.
Figure L3.7c shows that the land damage observations in this area were none-to-minor in the September event (equivalent to M6 0.24g) and that the worst observed land damage over the CES was typically minor-to-moderate (Figure L3.7d). It is noted that the land to the east of Area C transitions from minor-to-moderate to none-to-minor as the worst observed land damage over the CES.

Figure L3.7e shows that the ground surface subsidence is typically 0.2 to 0.4m in the southwest of the area. In the north eastern area, coinciding with the land at a higher elevation, it is generally 0.5 to 1m. This correlation between ground surface subsidence and elevation is potentially attributable to localised topographic re-levelling and/or the lateral spreading that was occurring at a regional level.

The majority of the properties in Area C did not qualify for ILV based on the Stage 1 assessment process (refer to Figure L3.7f). This correlates with the majority of the CPT in Area C being classified as × (refer to Figure L3.8). The exception to this is the group of CPTs in the north western corner of the area which are predominantly classified as ✓. These ✓ CPT are correlated with the properties that qualified for ILV during Stage 1.

Review of the plots of q_c and I_c vs depth in Figure L3.9 shows that, similar to the plots produced for Area B, the site is typically underlain by clean sand material which increases in density with depth. The key difference between these two plots is that Area C typically has denser material in the top 4m of the soil profile.

Based on the liquefaction triggering envelopes shown on the plots, only some parts of the sandy material in the top 4m of Area C is only marginally likely to liquefy at M6 0.3g levels of ground shaking. This is the main reason for the different predominant CPT classifications for Area C (×) compared with Area B (✓).
Closer inspection of Figure L3.9 shows that the top 4m of the CPT which were classified as $\checkmark$ in the north west corner of Area C are significantly looser than those which were classified as $\times$ indicating a distinct spatial difference between the soils profiles in the northwest corner compared to the rest of Area C.

**Figure L3.9:** Plots of $q_c$ and $I_c$ vs depth for CPT grouped by CPT classifications for the CPT identified in Figure L3.8.

**Legend**

- NO
- YES
- NC
- SC

Liquefaction Triggering Envelope at M6 0.3g

Groundwater Depth
- $I_c < 1.8$
- $I_c = 2.4$

Median depth to groundwater
- 15% Spatial value
- 50% Spatial value
- 85% Spatial value

L3.5.2 **ILV Qualification Assessment (Task 4)**

The analysis shows that the properties that were already considered as not qualifying for ILV as part of the stage 1 process were appropriately assessed (i.e. the stage 2 process has confirmed the stage 1 ILV decisions). On the other hand, the Stage 2 properties within this area have been assessed as qualifying for ILV based on the following:

- Analysis of the plots of $q_c$ and $I_c$ vs depth shows that there is a significant difference in the density of the CPT classified as $\checkmark$ and those classified as $\times$;
- The $\checkmark$ CPT are contained within a localised area with consistent results indicating that there is a zone of looser soil in the north west of Area C.
Accordingly the stage 2 properties have been marked with a ☑ on Figure L3.8 to indicate that based on engineering judgement both engineering criteria have been satisfied and therefore these properties qualify for ILV.
L3.6 Area D

L3.6.1 Local and Specific Assessment (Tasks 2 and 3)

Review of Figure L3.10a shows that the land in Area D is generally at a higher elevation than the land in the other areas. There is a low point in the south eastern corner and this is reflected by a localised shallower depth to the groundwater surface (refer to Figure L3.10b) in this area.

![Maps](image)

**Figure L3.10**: A series of maps used when assessing Area D using the Stage 2 qualification process. (a) post-CES ground surface elevation; (b) post-CES depth to groundwater surface; (c) September 2010 observed land damage; (d) worst observed land damage across the CES; (e) total ground surface subsidence across the CES; (f) Stage 1 ILV qualification results.
Land damage observations are similar to those in the other areas with none-to-minor observations for the September event (equivalent to M6 0.24g) and typically minor-to-moderate land damage observations across the CES (refer to Figures L3.10c and L3.10d).

Ground surface subsidence of the northern properties is varied from 0.2 to 0.4m with isolated zones of 0.5 to 1m (refer to Figure L3.10e). For the eastern and western stage 2 properties, ground surface subsidence is typically 0.4 to 1m.

The majority of the properties in Area D did not qualify for ILV based on the Stage 1 assessment process (refer to Figure L3.11). A significant number of properties were assessed as requiring assessment with the Stage 2 ILV process in Area D. This is attributable to the marginality of the CPT classifications in this area. However there are some spatial patterns within the CPT classifications with M and ✓ CPT tending to group around the north eastern boundary of the area adjacent to the properties which were previously qualified for ILV in the Stage 1 process. The predominant CPT classification within the rest of Area D is NC.

The results of the automatic model reflect the marginality of CPT classifications with assessments of automated yes, automated no and automated marginal distributed throughout the area (refer to “Worked Example Material”).

Review of the plots of q<sub>c</sub> and I<sub>c</sub> vs depth (refer to Figure L3.12) indicate that typically the top 3 to 4m of the soil profiles in this area are clean sands overlying approximately 2m of loose silty material. The soils from 6 to 10m are typically dense sands which are unlikely to liquefy at M6 0.3g levels of ground shaking.

The key distinction between the CPT which are classified as ✓ and M relative to the CPT classified as NC is the density of the top 1 to 2m of the soil profile. The top 1 to 2m is critical because this is
where the change in the groundwater surface is occurring. Within the top 1 to 2m of the soil profile the CPT classified as ✓ tend to have looser soils with \( q_c \) values which are less than the liquefaction triggering envelope for M6 0.3g levels of ground shaking. Whereas the CPT classified as NC tend to have relatively denser soils (compared with the CPT classified with a ✓) with \( q_c \) values which plot around the threshold of liquefaction triggering for M6 0.3g levels of ground shaking. For the CPT classified as NC this “thresholding” is resulting in a small calculated \( \Delta LSN \) in areas with significant levels of ground surface subsidence (up to 1m) and at higher levels of earthquake shaking these NC CPTs would all become a ✓.

![Figure L3.12: Plots of \( q_c \) and \( I_c \) vs depth for CPT grouped by CPT classifications for the CPT identified in Figure L3.11.](image)

**L3.6.2 ILV Qualification Assessment (Task 4)**

The Stage 2 properties in the centre and west of this Area D have been assessed as not qualifying for ILV on the basis that:

- The higher elevation results in a deeper groundwater surface and hence a thicker non-liquefying crust relative to the properties to the east;
- The classification of CPT in this area is predominantly NC with a reasonably clear spatial pattern;
• Inspection of the $q_c$ and $I_c$ plots shows that the CPT classification of NC results from the density of the soils in this area. The CPT classified as NC tend to be denser around the critical depth where groundwater change is occurring (i.e. 1 to 2m) with $q_c$ values which plot both around the threshold of liquefaction triggering for M6 0.3g levels of ground shaking. This “thresholding” is resulting in a small $\Delta$LSN in areas with significant levels of ground surface subsidence (up to 1m). Therefore, despite the significant settlement in this area, the NC classification is judged to be appropriate;

• Accordingly the northern and western stage 2 properties have been marked with a ☐ on Figure L3.11 to indicate that based on engineering judgement Criterion 2 is unlikely to have been satisfied and therefore these properties do not qualify for ILV.

• The stage 2 properties to the east of this Area D have been assessed as qualifying for ILV on the basis that:
  – The lower elevation results in a shallower groundwater surface and hence a thinner non-liquefying crust relative to the properties to the west;
  – The classification of CPT in this area is predominantly ✓ and M with a reasonably clear spatial pattern;
  – Inspection of the $q_c$ and $I_c$ vs depth plots show that there is a distinction between the CPT classified as ✓ and M relative to the CPT classified as NC. Within the top 1 to 2m of the soil profile, the CPT classified as ✓ and M tend to have looser soils with $q_c$ values which are less than the liquefaction triggering envelope for M6 0.3g levels of ground shaking.

• Accordingly the eastern stage 2 properties in Area D have been marked with a ☑ on Figure L3.11 to indicate that based on engineering judgement both engineering criteria have been satisfied and therefore these eastern stage 2 properties qualify for ILV.

L4 References

Russell, J., van Ballegooy, S., Torvelainen, E. & Gulley, R. 2015b. Consideration of ground variability over an area of geological similarity as part of liquefaction assessment for foundation design. 6th International Conference on Earthquake Geotechnical Engineering, Christchurch.
Appendix M: Classification Process for Properties that do not Qualify for ILV but are Materially Vulnerable to Liquefaction
M1 Introduction

The outcome of the ILV assessment process resulted in two data sets, these being:

- Properties that qualify for ILV (i.e. properties which have satisfied both Criterion 1 and Criterion 2); and
- Properties that do not qualify for ILV (i.e. properties which have not satisfied either Criterion 1 and/or Criterion 2)

The two data sets are spatially shown in Figure M1.1.

Figure M1.1: ILV assessment results after the completion of the Stage 2 process. Note the white areas on the map represent the non-urban and non-residential land in Christchurch.

Table M1.1 below shows the categorisation of ILV land damage following Stage 2 qualifications divided into their respective TC1, TC2, TC3 and residential Red Zone areas.

---

1 Criterion 1 and 2 are defined in Section 2 of the Report.
Table M1.1: ILV Land Damage Qualification following Stage 2 by MBIE Technical Category and CERA Residential Red Zone

<table>
<thead>
<tr>
<th>Technical Category</th>
<th>ILV assessment results following the completion of the Stage 2 Process</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Number of properties which qualify for ILV</td>
</tr>
<tr>
<td>TC1</td>
<td>0</td>
</tr>
<tr>
<td>TC2</td>
<td>510</td>
</tr>
<tr>
<td>TC3</td>
<td>4,386</td>
</tr>
<tr>
<td>Red Zone</td>
<td>5,021</td>
</tr>
<tr>
<td>Total</td>
<td>9,917</td>
</tr>
</tbody>
</table>

The property counts are based on the QPID database (maintained by Quotable Value Ltd) which existed at the time of the CES. The number of properties/QPIIDs does not necessarily represent the number of claims.

For properties that do not qualify for ILV, these can be further classified as follows:

- **Properties materially vulnerable to liquefaction (LV):** where the residential land has a material vulnerability to liquefaction damage after the CES, at M6 0.3g levels of earthquake shaking (i.e. satisfy Criterion 1). However the vulnerability of the residential land to liquefaction damage in future earthquakes has not materially increased at up to M6 0.3g levels of earthquake shaking as a result of ground surface subsidence of the land caused by the CES (i.e. does not satisfy Criterion 2); and

- **Properties not materially vulnerable to liquefaction (NV):** where the residential land does not have a material vulnerability to liquefaction damage after the CES at M6 0.3g levels of earthquake shaking (i.e. it does not satisfy Criterion 1).

**M1.1 Purpose and Outline**

During the Stage 1 ILV assessment process (refer to Section 9 of the Report) the NV and LV status were not recorded for the Stage 1 properties that did not qualify for ILV. However during the Stage 2 ILV assessment process, the NV and LV classifications for properties that did not qualify for ILV were recorded.

The classification of properties, which did not qualify for ILV, as NV and LV was undertaken for the purpose of assisting EQC when communicating with property owners whose land may have had material liquefaction related damage as a result of the CES, but did not qualify for ILV land damage.

Therefore, the purpose of this appendix is to document the process which was used for classifying the properties which do not qualify for ILV into NV and LV classifications. This appendix outlines the process and then the results from the assessment process.

**M2 NV and LV Classification Process**

The classification of properties as NV and LV was undertaken in 3 steps as shown in the flow chart in Figure M2.1 and described in the following sections. The classification approach summarised in Figure M2.1 was only carried out on properties that did qualify for ILV during the Stage 1 assessment process. During the Stage 2 assessment process the NV and LV classifications were recorded for each Stage 2 property.
M2.1 Step 1: Create Preliminary NV and LV Map using the Automated Estimation of LSN Values

In this step a preliminary NV and LV map was created by classifying all residential flat land parcels not qualifying for ILV as either NV or LV based on the estimated property LSN values obtained from the automated ILV model (discussed in Section 8.2 of the Report). The indicator value used to determine if a property was NV or LV for this was the same as was used in the ILV assessment process (i.e. those properties with an automated LSN value of 16 or greater were given a preliminary classification of LV, while those properties with an automated LSN value of less than 16 were given a preliminary classification as NV).

M2.2 Step 2: Override Preliminary NV and LV Classifications Based on the Stage 2 NV, LV and ILV Classifications

In addition to the process described Step 1, all of the Stage 2 properties that were manually assessed during the Stage 2 ILV assessment as not qualifying for ILV were classified as NV or LV using the approach set out in Sections 8, 9 and 10 of the Report.

The results from the Stage 2 assessment were then overlaid on the results from the automated estimation of LSN values described in Step 1 to produce a combined NV, LV and ILV map. The results of the combined NV, LV and ILV map showed that in most areas, there was generally close alignment between the results from the estimated LSN values obtained from the ILV model (refer to Section M2.1) and the Stage 2 manual results.

In a small number of areas the Stage 2 assessment suggested that the results from the estimated property LSN values should be over-ridden. In such cases adjacent Stage 1 properties which were not qualified for ILV were identified as requiring a potential manual review of the preliminary NV or LV classification. These areas were highlighted on the map for further consideration during the detailed review, as discussed in Step 3 (refer to Section M2.3).

The automated NV and LV map was then manually overridden by the Stage 2 Assessments with NV and LV classifications.
**M2.3 Step 3: Detailed Review and Refinement of the Preliminary NV and LV Map**

As part of the manual assessment process the combined NV, LV and ILV map was reviewed by a team of engineers. Particular attention was given to the boundaries between NV, LV and ILV properties and properties adjacent to the properties which were assessed using the Stage 2 ILV assessment process. Information used for the detailed review and refinement of the combined NV and LV map is listed in Section 5 of the Report.

Using the information above, engineering judgement was used to determine whether or not a property should be classified as NV or LV in accordance with the *engineering criteria* set out in Section 2.4 of the Report, the objectives in Section 2.6 of the Report and the assumptions set out in Section 6 of the Report. In order to be classified as LV, the engineers undertaking the assessment would determine that, on the balance of probabilities, the property satisfied *Criterion 1*.

The results of this detailed review were entered directly into a GIS database using an online mapping tool created specifically for this purpose.

A final review was then undertaken by a senior technical review team using three comparison maps, which showed the NV and LV classifications overlaid with the following information:

- Map 1 – September 2010 PGA contours and September 2010 land damage (refer to Figure K1.5 in Appendix K);
- Map 2 – February 2011 PGA contours and February 2011 land damage (refer to Figure K1.6 in Appendix K); and
- Map 3 – TC1, TC2, TC3 and Red Zone map (refer to Figure K1.1 in Appendix K).

**M2.3.1 Review of Maps 1 and 2**

For properties identified as NV where:

- Minor-to-moderate or moderate-to-severe land damage was mapped in the September 2010 and February 2011 events; and
- The levels shaking were less than M6 0.3g for those events;

reviews were undertaken to understand why those properties were not classified as LV. Often this was because either the mapped land damage was minor (confirmed by detailed review of the aerial photography) and was not considered material or the mapped land damage was as a result of lateral spreading.

Similarly for properties identified as LV where:

- None-to-minor land damage was mapped in the September 2010 and February 2011 events; and
- The levels shaking were greater than M6 0.3g for those events;

reviews were undertaken to understand why those properties were not classified as NV. There were not many properties in this category. For properties where this did occur, typically the estimated PGA values were found to be less certain and potentially over estimating the levels of ground shaking that actually occurred.

**M2.3.2 Review of Map 3**

In a similar manner, TC2 properties classified as LV and TC3 properties classified as NV were also manually reviewed to check whether this was appropriate.
In the case of the LV classification of the TC2 properties, the main reasons for this is the observed land damage indicators and the geotechnical data indicated that these properties are materially vulnerable to liquefaction. Most of those properties are located in the north eastern suburbs of Christchurch.

In the case of the NV classification of the TC3 properties, the main reasons for this is the geotechnical data, the land damage maps and the aerial photography demonstrate that these areas are not materially vulnerable to liquefaction damage at M6 0.3g levels of ground shaking. Most of the observed liquefaction related damage in these areas is caused by levels of ground shaking which were greater than M6 0.3g or is attributable to lateral spreading.

As part of the review process, the NV and LV classification of some properties assessed as not qualifying for ILV was changed from NV to LV and vice versa. This did not occur frequently and was typically restricted to boundary areas of NV & LV properties.

**M3 Results**

The spatial distribution of NV and LV properties is shown in Figure M3.1. When comparing this figure to Figure M1.1, it can be seen that a significant number of properties that did not qualify for ILV in the northern, central, eastern and southern areas of Christchurch, have been classified as LV.

![Figure M3.1: ILV assessment results after the completion of Stage 2 also showing the NV and LV properties. Note the white areas on the map represent the non-urban and non-residential land in Christchurch.](image)

Table M3.1 shows the categorisation of NV, LV and ILV results divided into their respective TC1, TC2, TC3 and residential Red Zone areas.
### Table M3.1: NV, LV and ILV classification by MBIE Technical Category and CERA Residential Red Zone

<table>
<thead>
<tr>
<th>Technical Category</th>
<th>Number of properties which qualify for ILV</th>
<th>Number of properties materially vulnerable to liquefaction (LV)</th>
<th>Number of properties not materially vulnerable to liquefaction (NV)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TC1</td>
<td>0</td>
<td>0</td>
<td>23,267</td>
</tr>
<tr>
<td>TC2</td>
<td>510</td>
<td>1,867</td>
<td>78,645</td>
</tr>
<tr>
<td>TC3</td>
<td>4,386</td>
<td>9,693</td>
<td>13,773</td>
</tr>
<tr>
<td>Red Zone</td>
<td>5,021</td>
<td>1,272</td>
<td>956</td>
</tr>
<tr>
<td>Total</td>
<td>9,917</td>
<td>12,832</td>
<td>116,641</td>
</tr>
</tbody>
</table>

The property counts are based on the QPID database (maintained by Quotable Value Ltd) which existed at the time of the CES. The number of properties/QPIDs does not necessarily represent the number of claims.