

Appendix A: Assessment of Liquefaction
Vulnerability in Christchurch

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 (M_w 7.1), 22 February 2011 (M_w 6.2), 13 June (M_w 5.6 and M_w 6.0 separated by 80 minutes), and 23 December 2011 (M_w 5.8 and M_w 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 in 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.

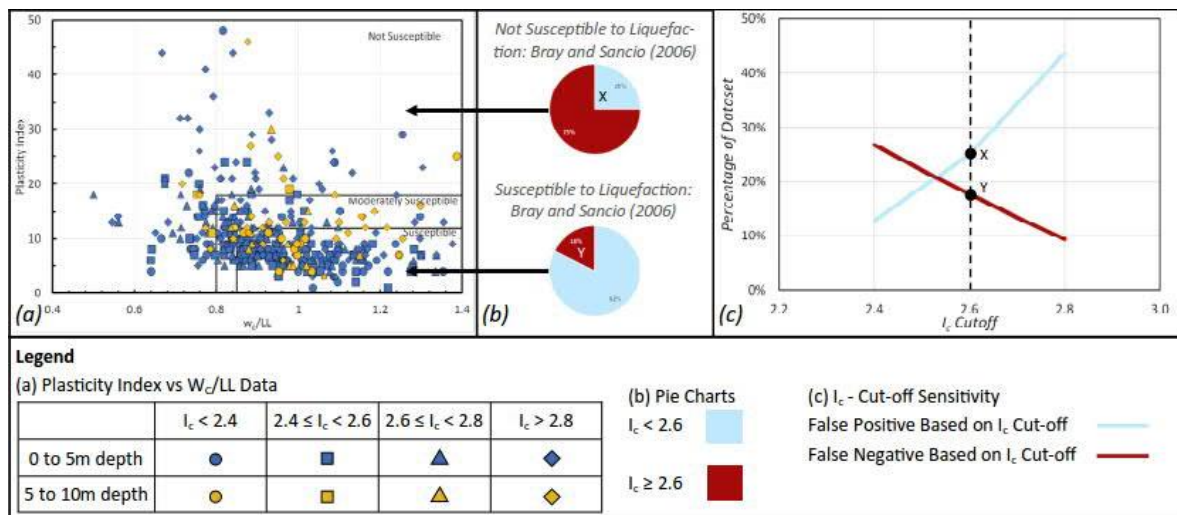


Figure A2.1: (a) Associated Atterberg Limit, w_c and I_c data obtained from geotechnical investigation data in the CGD from the Christchurch area plotted onto the Bray and Sancio (2006) liquefaction susceptibility criteria. (b & c) percentage of dataset identified as not susceptible with values below 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 cutoff 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 (**M_w**) 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_{c1NCS} 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 an 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 $FC-I_c$ 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.

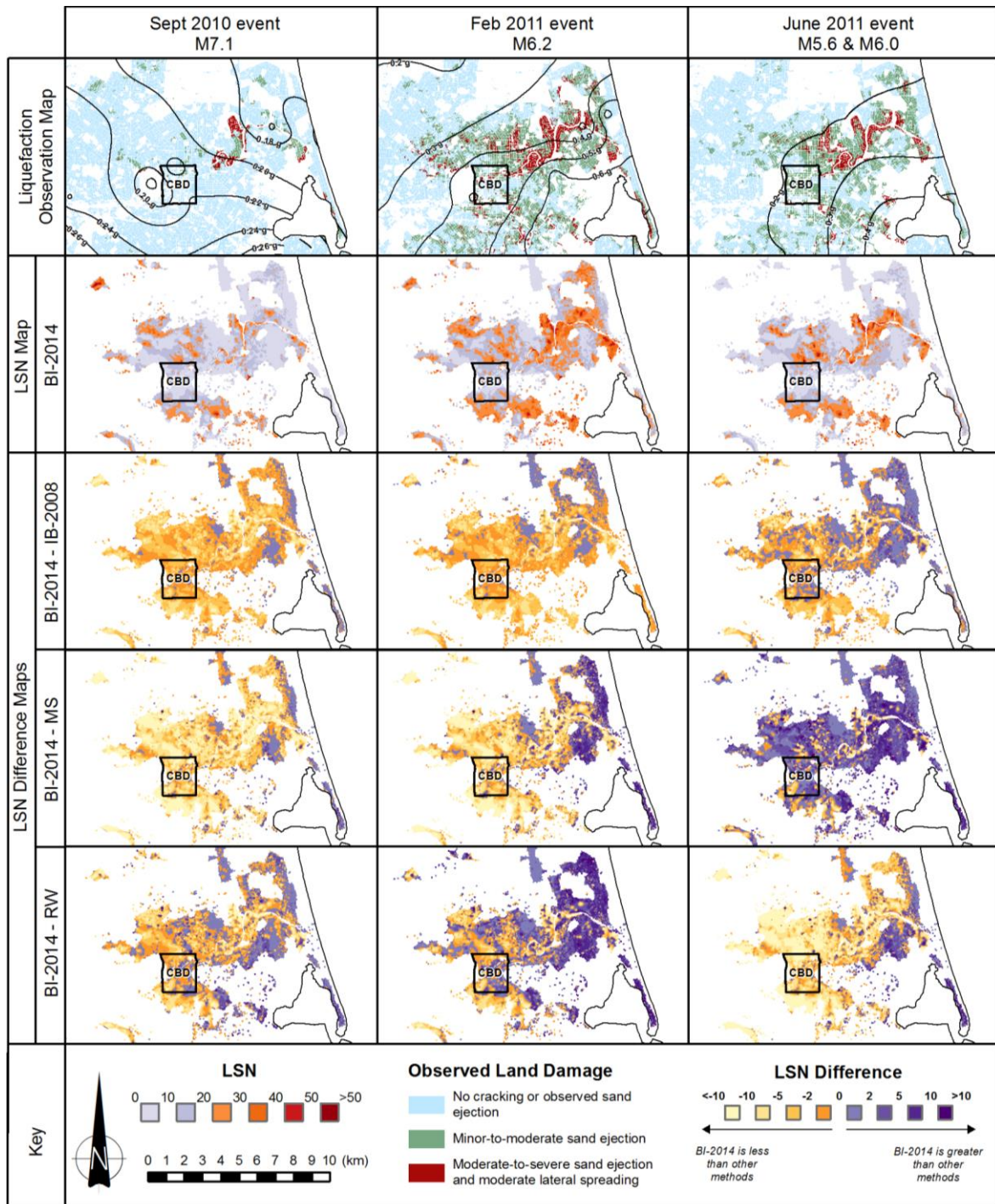


Figure A3.1: Map series of liquefaction severity observations and estimated LSN for all CPTs available in the CGD using the BI-2014, IB-2008, MS and RW simplified liquefaction evaluation methods for the September 2010, February 2011 and June 2011 earthquake events. PGA contours (Bradley and Hughes 2012) 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 Ballegooy 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)

Input parameter	Default value adopted	Comments
Soil Density	18 kN/m ³	Not sensitive to the typical variability in soil density in Christchurch (Tonkin & Taylor, 2013)
FC - I _c correlation	C _{FC} = 0.0	Appropriate upper bound value for Christchurch soils (Lees, van Ballegooy, & Wentz, 2015)
I _c - cutoff	I _c cutoff = 2.6	Appropriate value for Christchurch soils (Lees, van Ballegooy, & Wentz, 2015)
Level of earthquake shaking	M _w = 6.0, PGA = 0.3g	Critical case for 100 year return period levels of earthquake shaking using the BI 2014 methodology
Probability of Liquefaction (P _L)	P _L = 15%	Based on standard engineering design practice
Depth to Groundwater (GWD)	Surrogate median groundwater surface	Based on the GNS groundwater model (van Ballegooy, et al., 2014a) Two key assumptions associated with GWD are: <ul style="list-style-type: none"> • The groundwater profile is hydrostatic below the ground water 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 ε 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.

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

$$\text{where } 0\% \leq FC \leq 100\%$$

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- I_c 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 I_c 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 I_c 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.

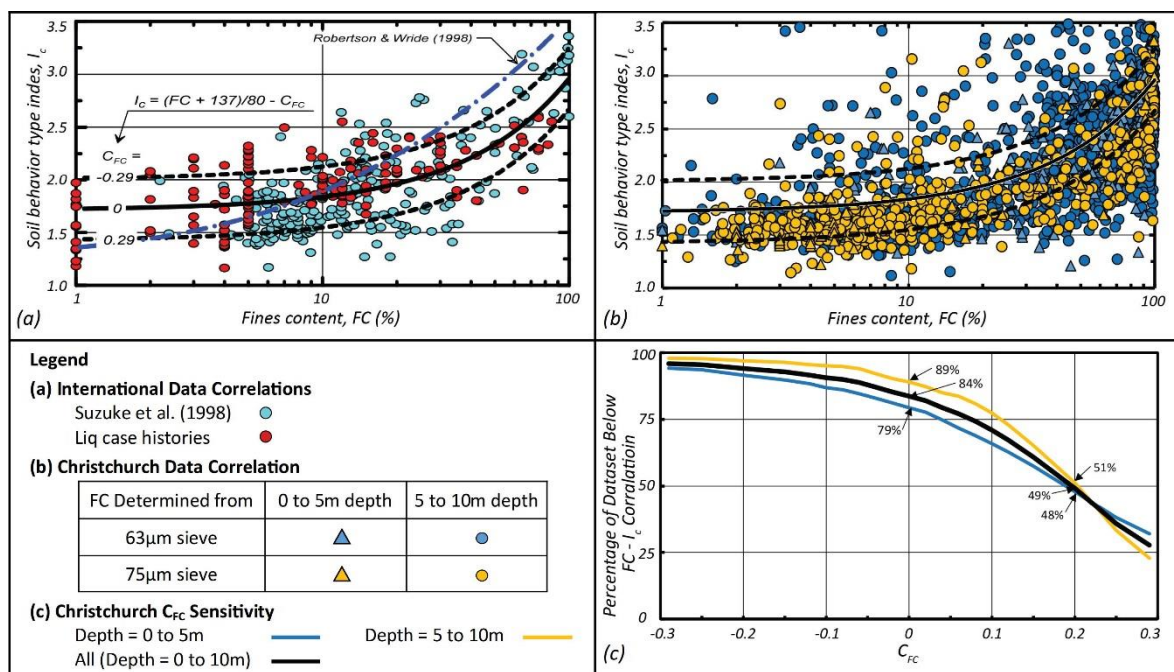


Figure A3.2: (a) Plots of RW and BI-2014 FC- I_c correlations overlaid on the international liquefaction case history data. (b) FC vs. median I_c for data in the Christchurch area with the BI-2014 FC- I_c correlations with a C_{FC} of -0.29 , 0 and 0.29 overlaid. (c) Percentage of the FC- I_c dataset below the BI-2014 FC- I_c 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 I_c cutoff from the default parameters of $C_{FC} = 0$ and I_c 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 I_c cutoff is discussed in Section A3.2.4.

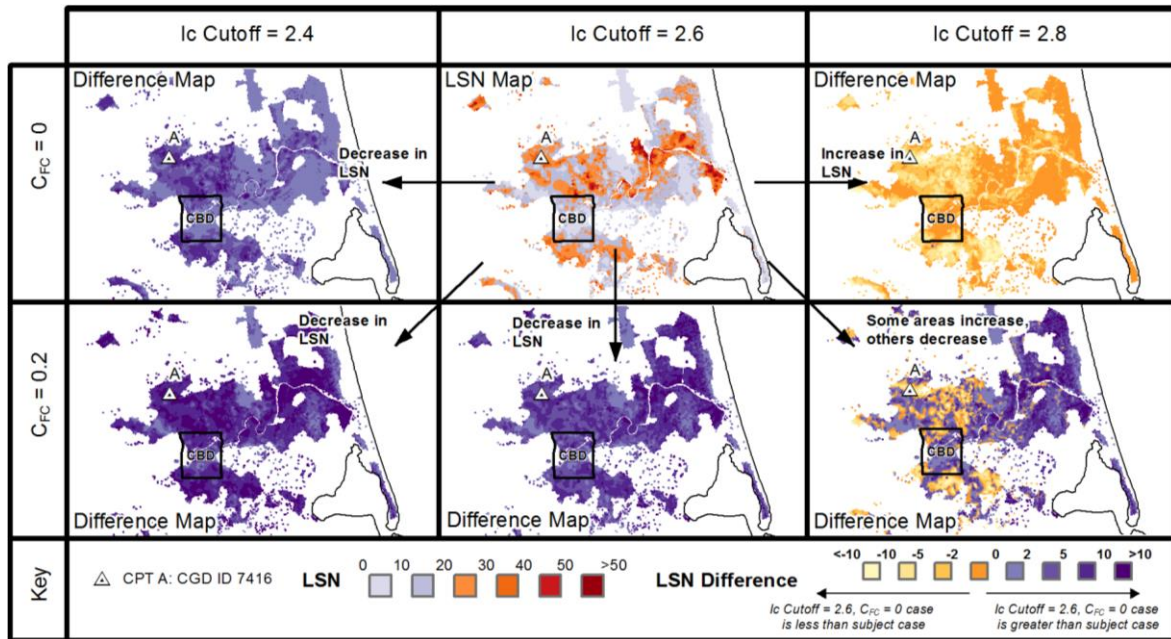


Figure A3.3: Estimated LSN map at $C_{FC}=0$ and I_c cutoff = 2.6 for M6 0.3g levels of ground shaking (the central map in the top row). The other maps show the differences of the estimated LSN for LSN at $C_{FC}=0$ and I_c cutoff=2.6 with LSN at $C_{FC}=0$ and 0.2 (top and bottom rows respectively) and a I_c cutoff = 2.4, 2.6 and 2.8 (left, centre and right hand columns respectively). Figure reproduced from Leeves et al. (2015).

A3.2.3 Soil Behaviour Type Index (I_c) Cutoff

As discussed in Section A2, the I_c 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 I_c 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 I_c cutoff value. This is because most of the soil layers in that area have I_c 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 I_c cutoff value. This is because the soil layers in that area typically have I_c 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 M_w of 7.5. This applies when undertaking liquefaction triggering analysis on deep or soft soil site (Class D).

Work undertaken by van Ballegooy et al. (2015c) has demonstrated that in Christchurch soils the critical magnitude for the assessment of liquefaction triggering using BI-2014 is $M_w = 6.0$ for PGA with equivalent return period levels of shaking compared to the $M_w = 7.5$ 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

Design Case	Annual Probability of Exceedance	PGA at $M_w = 7.5$	PGA at $M_w = 6.0$
SLS	1 in 25 years	0.13g	0.19g
ILS	1 in 100 years	0.20g	0.30g
ULS	1 in 500 years	0.35g	0.52g

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.

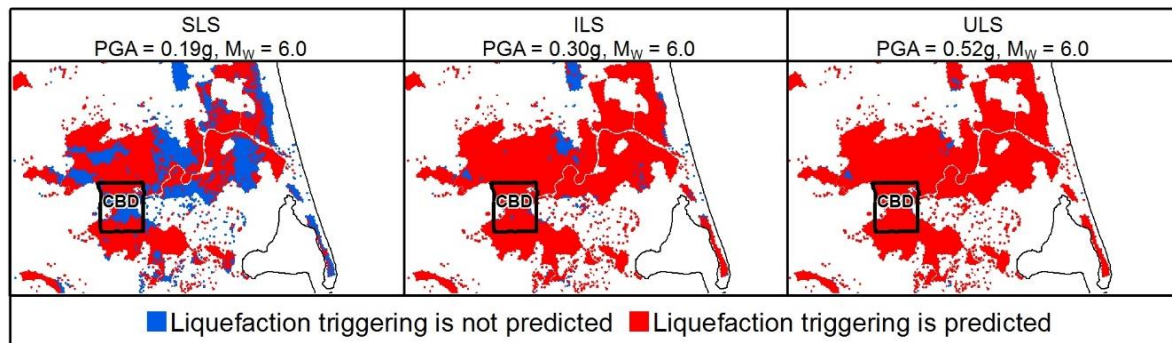


Figure A3.4: Maps showing areas where liquefaction triggering is predicted for SLS, ILS and ULS levels of earthquake shaking assuming an I_c cutoff = 2.6, $C_{FC} = 0$ and $P_L = 15\%$.

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 Ballegooy et al. (2014b) demonstrated that it provides an appropriate way 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.

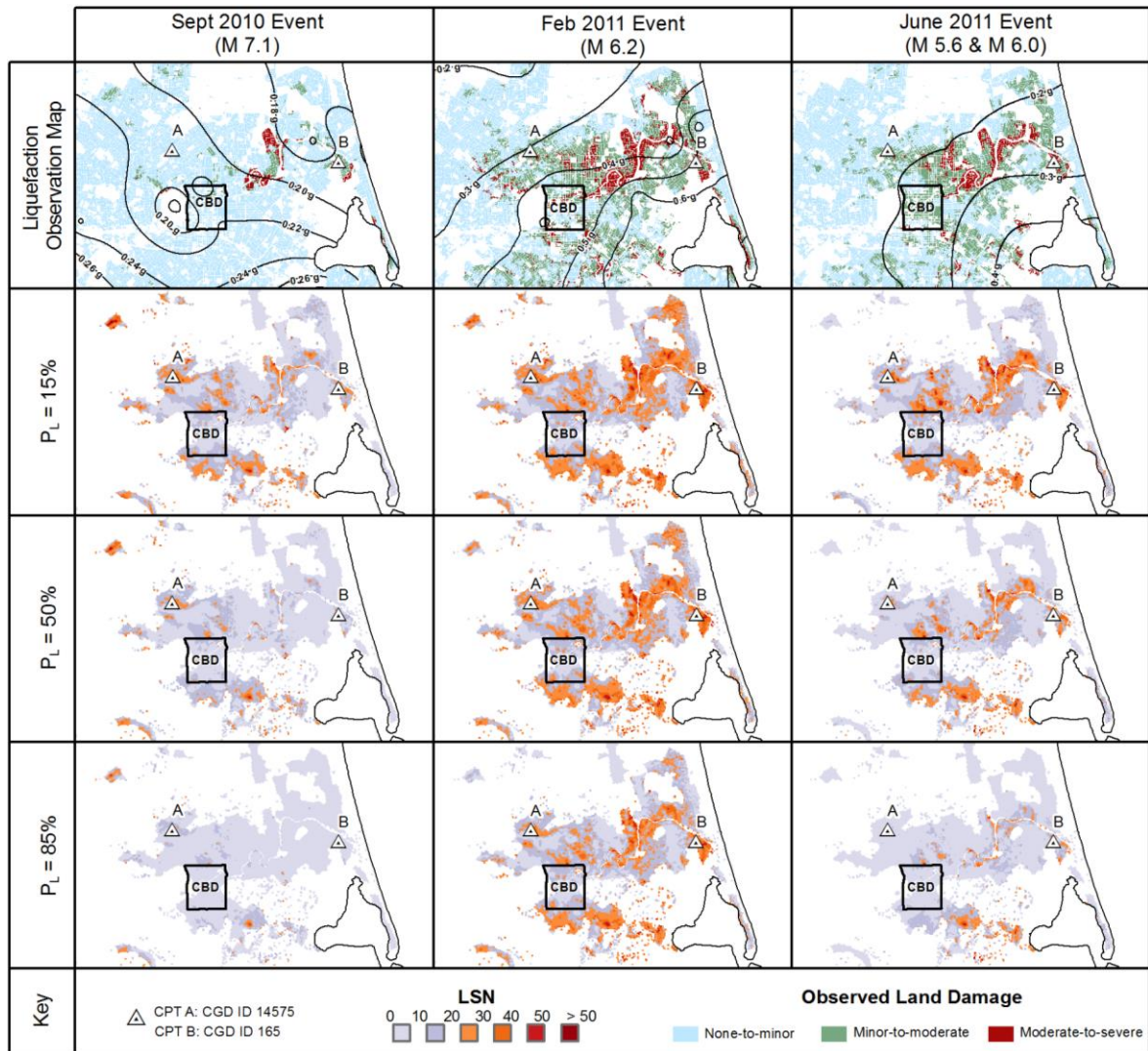


Figure A3.5: Maps of liquefaction severity observations across the CES overlaid with PGA contours (top row). Maps of estimated LSN for the September 2010, February 2011 and June 2011 earthquake events using $P_L = 15\%$ (2nd row), $P_L = 50\%$ (3rd row) and $P_L = 85\%$ (bottom row). Figure reproduced from Lacrosse et al. (2015).

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.

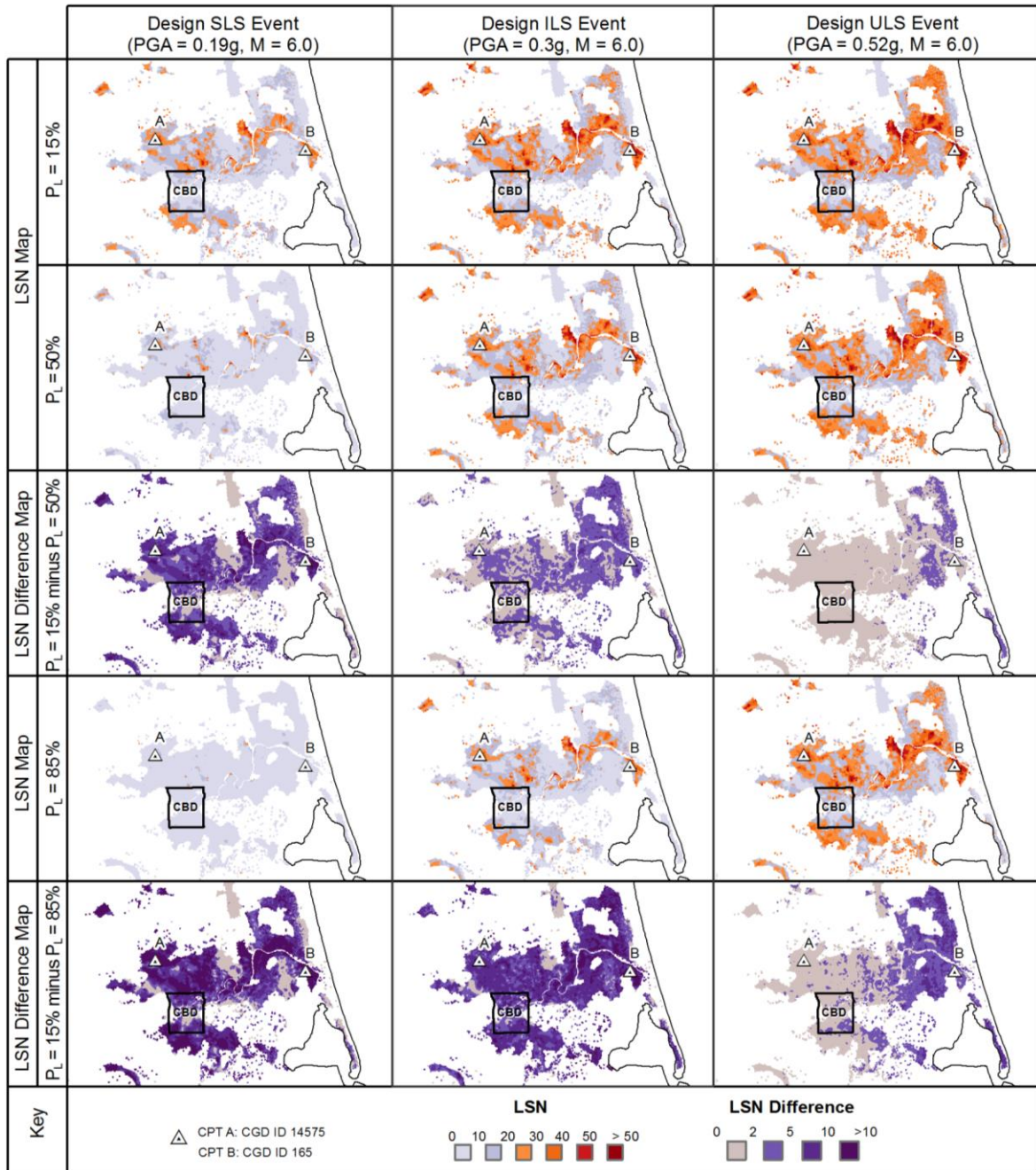


Figure A3.6: Estimated LSN maps at $P_L = 15\%$, 50% and 85% (rows 1, 2 and 4 respectively) for the design SLS, ILS and ULS ground motions (left, center and right hand columns respectively). Rows 3 and 5 show difference maps of the estimated LSN for $LSN_{P_L=15\%} - LSN_{P_L=50\%}$ and $LSN_{P_L=15\%} - LSN_{P_L=85\%}$ respectively. Reproduced from Lacrosse et al. (2015).

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 ground water 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 straight forward 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 $I_c < 1.8$.

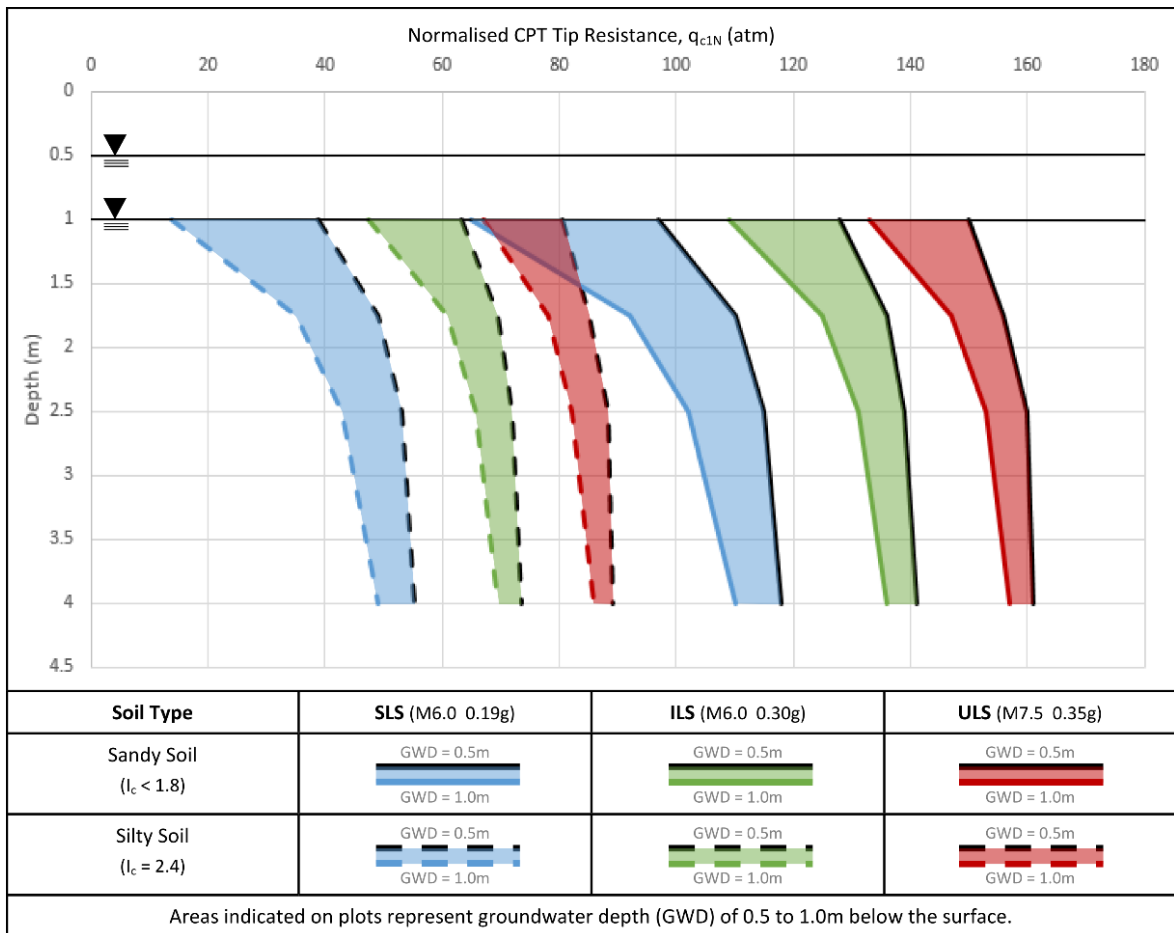


Figure A3.7: Liquefaction triggering curves for low various levels of shaking, with sandy and silty soils and GWD of 0.5m and 1m assuming the BI-2014 liquefaction triggering method and with $C_{FC} = 0$ and $P_L = 15\%$. Trends of increasing resistance to liquefaction in soils with higher strength, silty 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 liquefying material at $P_L = 15\%$ whereas CPT_2360 is indicating only isolated lenses of liquefying material.

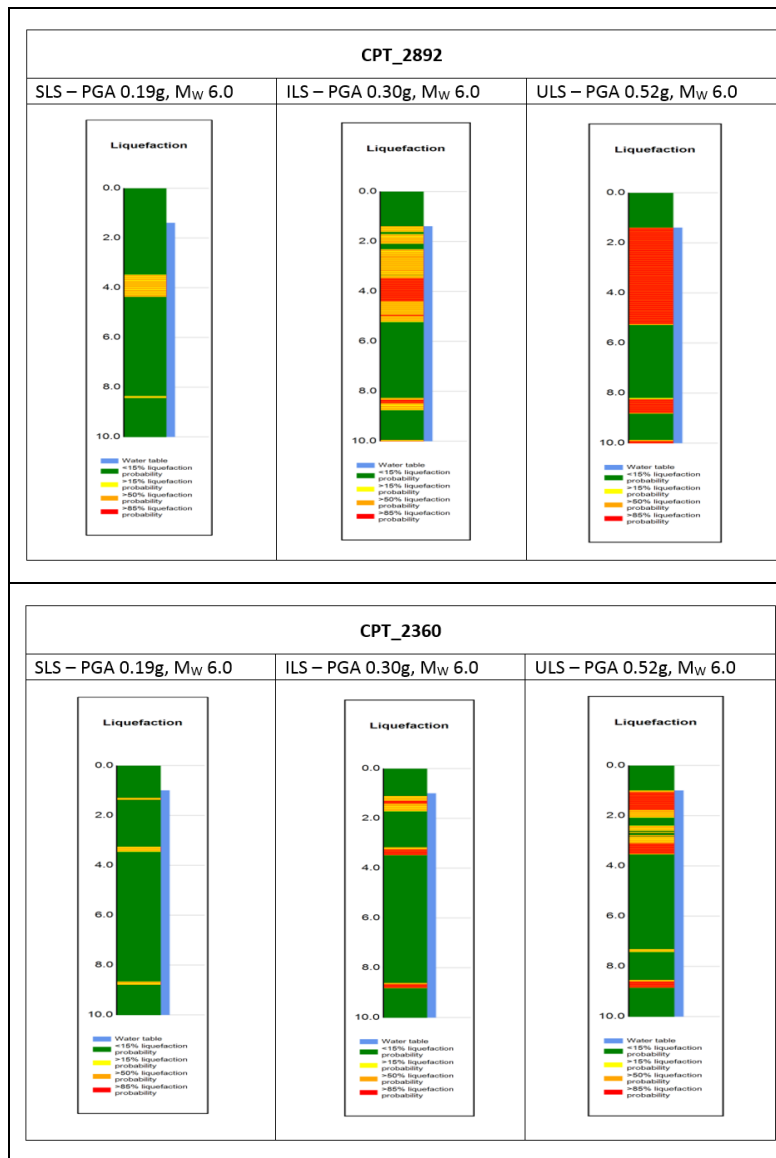


Figure A3.8: Liquefaction triggering analysis for two example CPTs, for various strengths of ground shaking. Green shading indicates that liquefaction is unlikely, with yellow, orange and red shading indicating $P_L = 15\%$, $P_L = 50\%$ and $P_L = 85\%$ respectively.

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.

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_{1VD}**) 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).

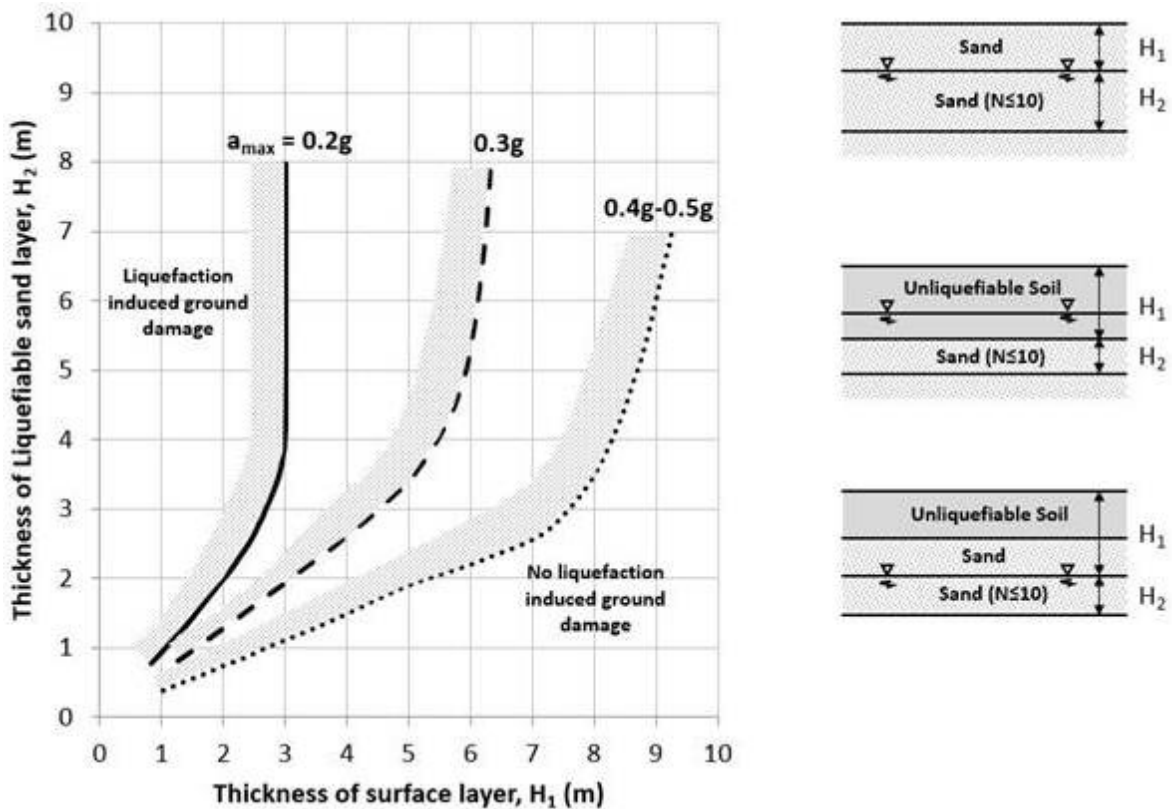


Figure A4.1: Relationship between thickness of a liquefying layer, H_2 (m) and thickness of a non-liquefiable overlying layer, H_1 (m) at sites for which surface manifestation of level-ground liquefaction has been observed. Figure reproduced from Ishihara (1985).

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_1) 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.

$$CTL = \int_0 (FS < 1) dz$$

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_{V1D} , 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_{V1D})

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_{V1D} parameter (also referred to as “calculated settlement”) is defined as:

$$S_{V1D} = \int_0 \epsilon_v dz$$

Where ϵ_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_{c1NCs}) to generate ϵ_v . The Zhang et al. (2002) method predicts volumetric strain in layers where the liquefaction FS is less than 2. The S_{V1D} indicator increases as the FS drops and the material approaches a liquefied state.

The MBIE (2012) guidelines recommends the use of the S_{V1D} 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_{V1D} 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_{V1D}

Criteria		Foundation Solution Robustness	SLS Calculated S_{V1D} (mm)	ULS Calculated S_{V1D} (mm)
TC1 criteria		Low ↓ High	< 15	< 25
TC2 criteria			< 50	< 100
TC3 criteria	TC3 hybrid criteria		< 50	> 100
	TC3 SLS<100 mm criteria		< 100	n/a
	TC3 SLS>100 mm criteria		> 100	n/a

The guidelines recommend that the thresholds in provided in Table A4.1 should be applied to the S_{V1D} calculated from the liquefying soil layers in the top 10m of the soil profile.

The MBIE (2012) guidelines uses S_{V1D} 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_{V1D}). This means that more robust foundation solutions are targeted to areas where higher S_{V1D} values are calculated.

Studies were undertaken to assess the correlation of the calculated S_{V1D} 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_{V1D} 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 re-leveling 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_{V1D} parameter does not take crust thickness into account, and has a low sensitivity to the groundwater level.

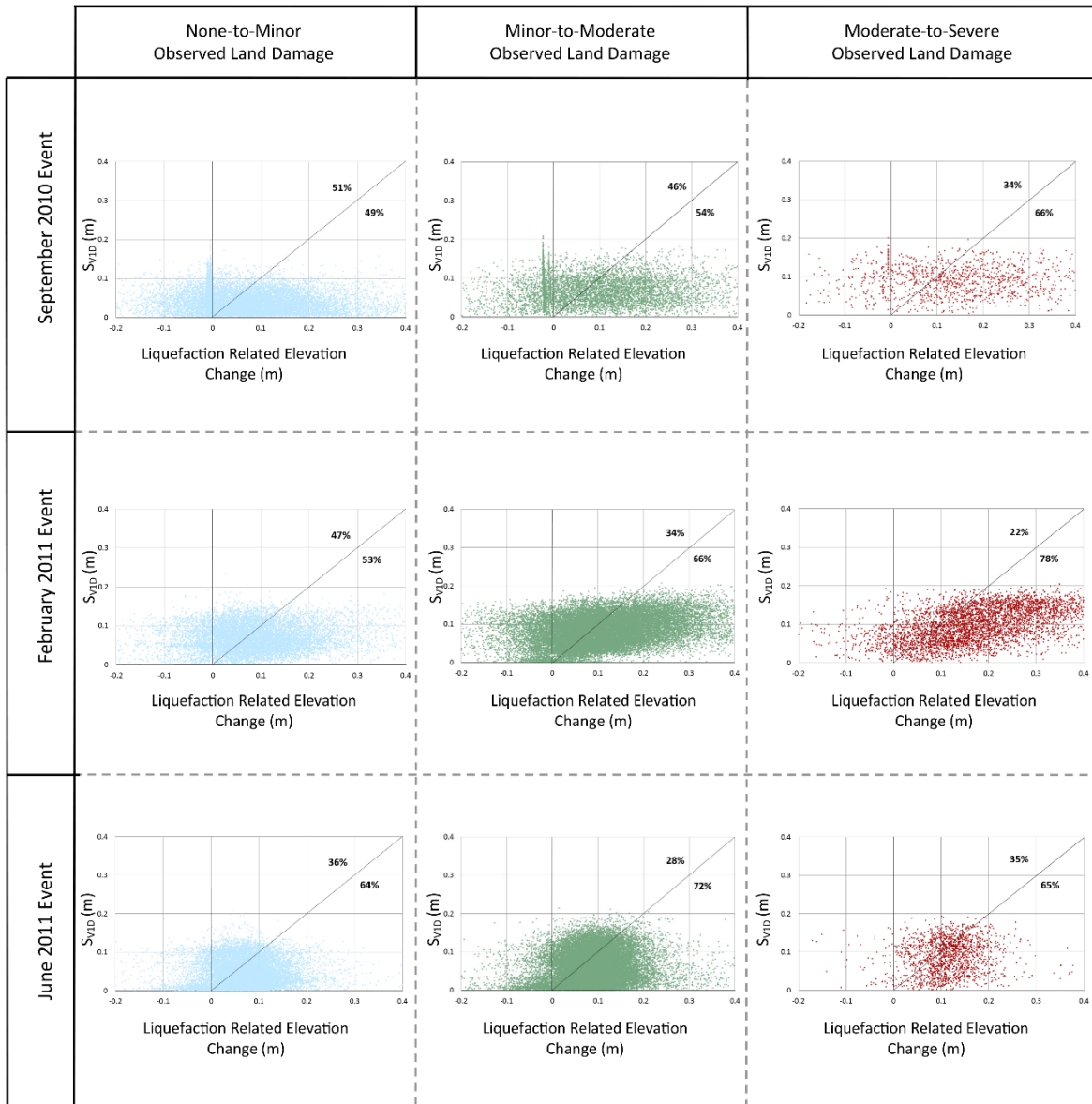


Figure A4.2: Relationship between S_{V1D} and the estimated liquefaction related ground surface subsidence for the September 2010, February 2011 and June 2011 earthquake events. This figure is a modified version of a figure presented in Tonkin & Taylor (2013). The main modification is an updated liquefaction triggering methodology from IB-2008 to BI-2014 for the calculation of S_{V1D} in accordance with the MBIE (2014) guideline recommendations.

While the S_{V1D} 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 Ballegooy et al., 2015c). In particular the results show that areas with higher calculated S_{V1D} 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_{V1D} 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_{V1D} 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 $\text{FS} = 0$ 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.

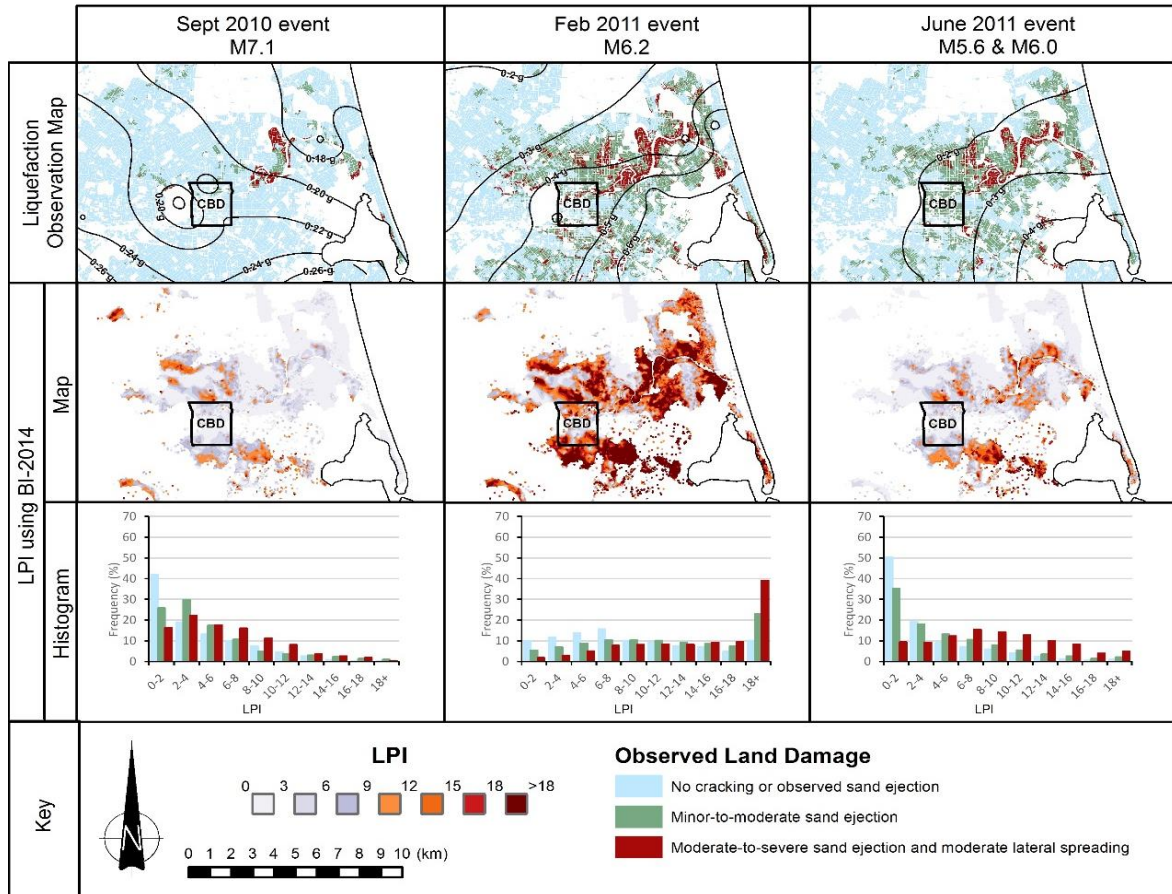


Figure A4.3: Maps of liquefaction severity observations (top row) and estimated LPI (second row) for the September 2010, February 2011 and June 2011 earthquake events. PGA contours from Bradley & Hughes (2012) are overlaid on the liquefaction severity observation maps (top row). Histograms of the liquefaction severity observations and their correlation with LPI are shown on the bottom row. This figure is modified from van Ballegooy 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_{ish}

LPI_{ISH} 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_{ISH} 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_1 is defined as the depth of the non-liquefying crust, z is the depth below ground level.

The LPI_{ISH} framework accounts for the relative thickness of the liquefied stratum and the non-liquefied layers via the additional criteria on FS where $FS \leq 1$, 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_{ISH} framework shallower liquefied layers contribute more to surficial manifestations than predicted by the LPI framework (van Ballegooy, et al., 2014c).

Studies were undertaken by van Ballegooy et al. (2015c) to compare the estimated LPI_{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 Ballegooy et al. (2015c) which summarises the results.

The findings of these studies on LPI_{ISH} were similar to those of LPI. Figure A4.4 demonstrates that higher estimated LPI_{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_{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.

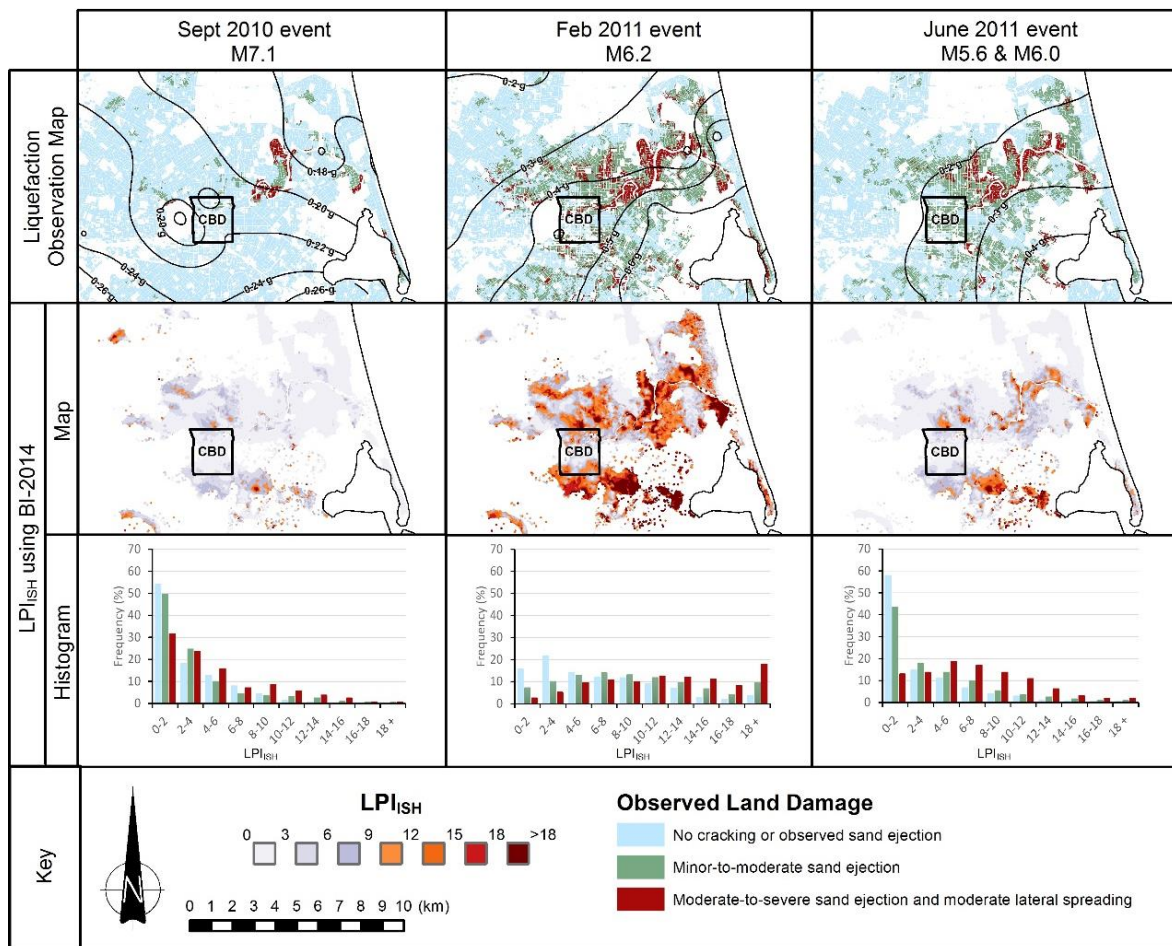


Figure A4.4: Maps of liquefaction severity observations (top row) and estimated LPI_{ISH} (second row) for the September 2010, February 2011 and June 2011 earthquake events. PGA contours from Bradley & Hughes (2012) are overlaid on the liquefaction severity observation maps (top row). Histograms of the liquefaction severity observations and their correlation with LPI_{ISH} are shown on the bottom row. This figure is modified from van Ballegooy et al. (2015c).

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 ϵ_v , integrated over the depth of the soil profile containing liquefying layers, with a depth weighting factor.

The LSN parameter is defined as:

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

Where ϵ_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.

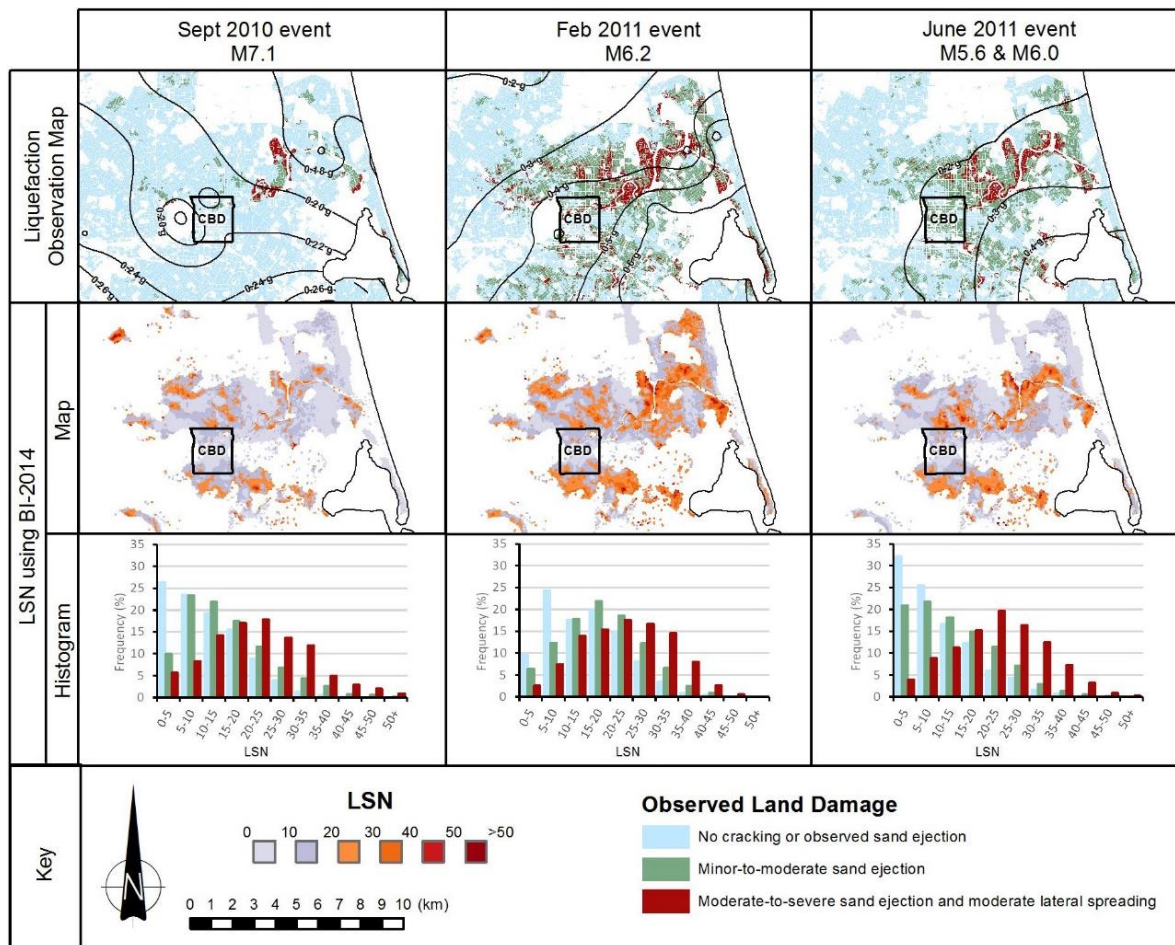


Figure A4.5: Maps of liquefaction severity observations (top row) and estimated LSN (second row) for the September 2010, February 2011 and June 2011 earthquake events. PGA contours from Bradley & Hughes (2012) are overlaid on the liquefaction severity observation maps (top row). Histograms of the liquefaction severity observations and their correlation with LSN are shown on the bottom row. This figure is modified from van Ballegooy et al. (2015c).

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 developed 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_{c1NCS}) of 80 atm. This q_{c1NCS} 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.

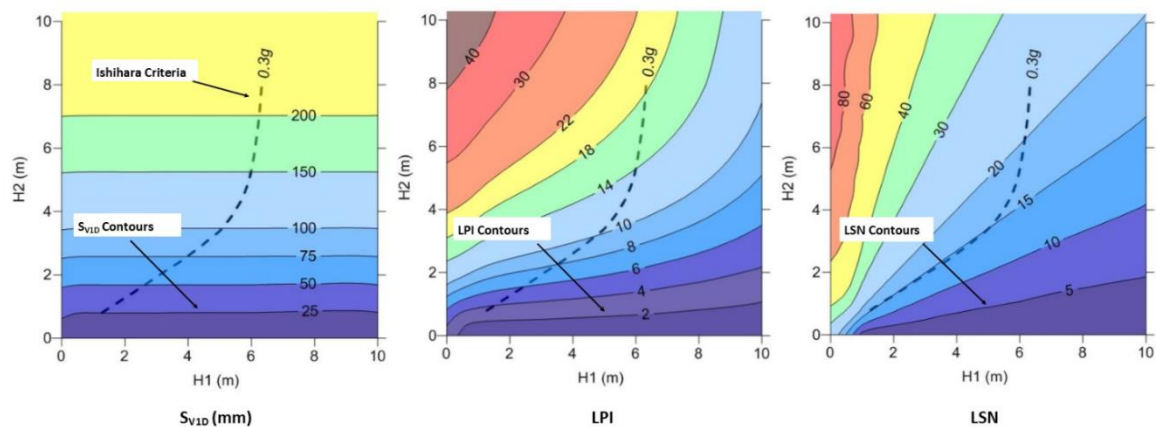


Figure A4.6: Comparison of estimated CPT-based liquefaction vulnerability parameter (S_{V1D} , LPI and LSN) values to the Ishihara (1985) land damage boundary curve. The liquefaction vulnerability parameters have been estimated for M6 0.3g levels of earthquake shaking assuming a soil strength of $q_{c1NCS} = 80$ atm, and a simplified 2-layer soil profile

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)

	S_{V1D}	LPI¹	LSN
Agreement with Ishihara (1985) ground damage criteria	No single S _{V1D} value separates vulnerable from non-vulnerable soil profiles.	No single LPI value separates vulnerable from non-vulnerable soil profiles.	Good agreement with Ishihara (1985) criteria over the range of H ₁ values most critical in the Christchurch area.
Role of the non-liquefying (H₁) crust in land performance	Does not incorporate the importance of the non-liquefying crust (H ₁).	Gives insufficient weighting to the importance of the non-liquefy crust (H ₁) when compared with observed land performance in the Christchurch area.	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.
Sensitivity to very shallow groundwater levels	Low sensitivity to very shallow groundwater levels.	Moderately sensitive to very shallow groundwater levels.	Very sensitive (and potentially over sensitive) to very shallow groundwater levels when GWD < 0.5m.
Response to ground surface subsidence (key criteria for ILV assessment)	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.	Only moderately sensitive to ground surface subsidence.	Sensitive to ground surface subsidence. Predicts the highest change in liquefaction vulnerability for given levels of ground surface subsidence.
Contributing soil layers	Includes contribution from soil layers which are approaching liquefaction triggering, but still with safety factor (FS) > 1.	No contribution from soil layers which have a FS >1 even though they may have generated some excess pore-water pressure and are approaching liquefaction triggering.	Includes contribution from soil layers which are approaching liquefaction triggering, but still with FS > 1.
Treatment of loose versus dense soil layers	Recognises that looser soils cause more severe liquefaction effects such as ejection and ground surface subsidence.	Only partially addresses effect of soil density, through the safety factor.	Recognises that looser soils cause more severe liquefaction effects such as ejection and ground surface subsidence.

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.

Natural plateau in liquefaction performance	Recognises that increasing ground shaking beyond a certain level does not significantly worsen the flat land liquefaction performance of a soil profile.	Does not recognise natural plateau in liquefaction performance, so does not translate consistently between earthquakes of different shaking intensity.	Recognises that increasing ground shaking beyond a certain level does not significantly worsen flat land liquefaction performance of a soil profile.
Correlation with observed CES land damage	Does not correlate well with the estimated liquefaction related ground surface subsidence. S_{V1D} correlates better with the land damage observations.	The distributions of estimated LPI for a given land damage severity varies significantly for each of the main CES events.	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).
Precedent and other case histories	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.	Used as a predictive tool in liquefaction hazard studies over past 30 years.	Validated for the Canterbury earthquakes only.

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 ϵ_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.

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 A4.7. 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.

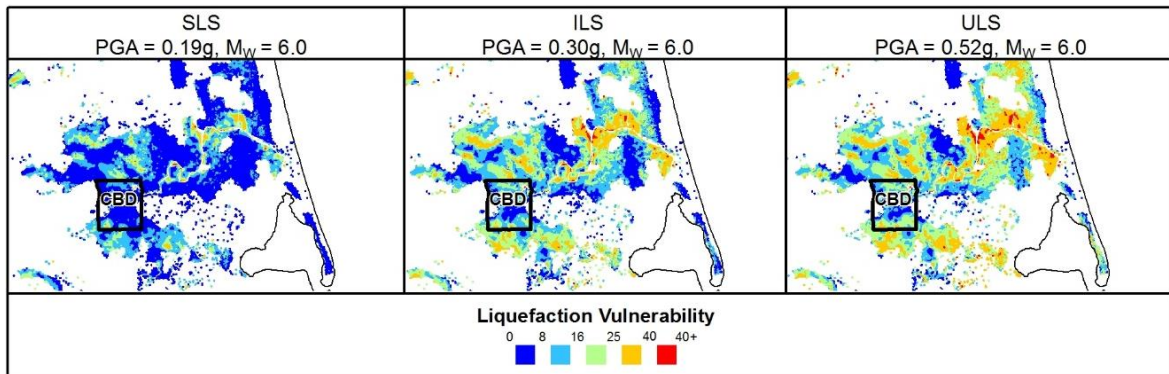


Figure A4.7: Maps showing increasing vulnerability predicted by LSN at SLS, ILS and ULS levels of ground shaking

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 Ballegooy 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 A4.5).

The datasets for these three events were combined into a single frequency bar chart shown in Figure A4.8. This figure shows that at low estimated LSN values (i.e. blue areas on the maps shown in Figure A4.7) there is a high likelihood of none-to-minor land damage and a low likelihood of moderate-to-severe land damage. Conversely, Figure A4.8 also shows that at high LSN values (i.e. the yellow and red areas on the maps shown in Figure A4.7) there is a low likelihood of none-to-minor land damage and a high likelihood of moderate-to-severe land damage.

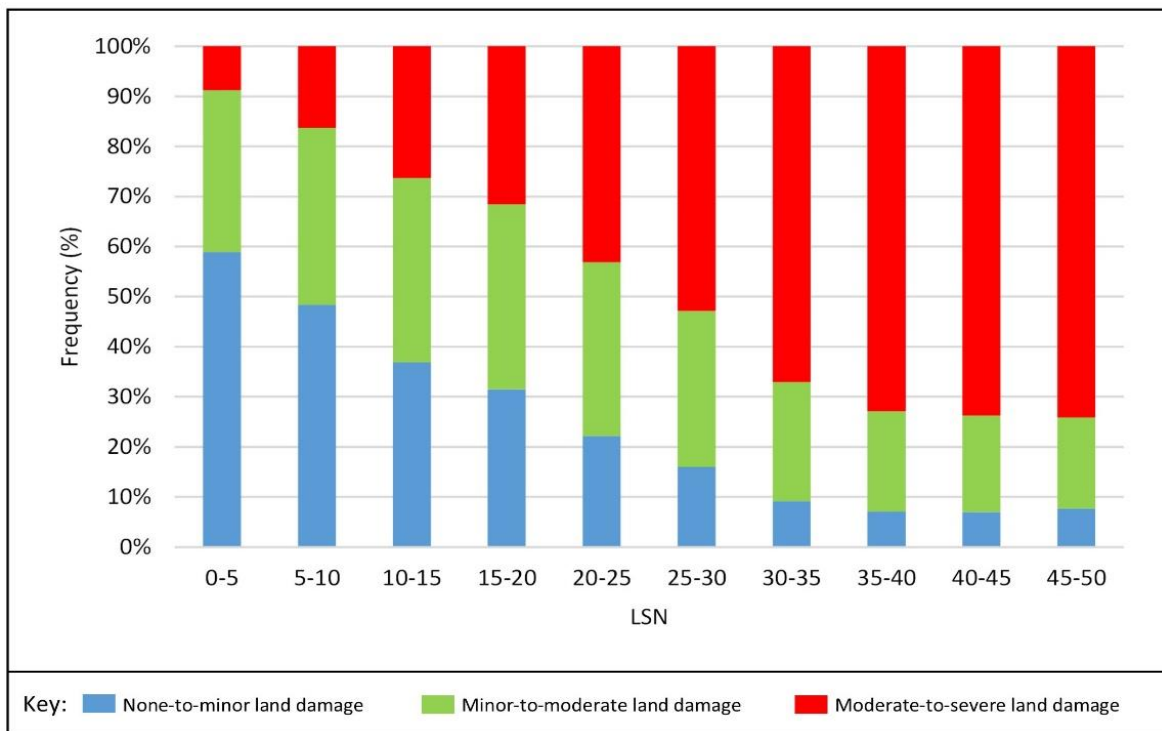


Figure A4.8: Frequency bar chart showing the likelihood of none-to-minor, minor-to-moderate and moderate-to-severe land damage for different LSN bands based on correlated back calculated LSN values with the land damage observations from the September 2010, February 2011 and June 2011 events.

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.

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