5 Information used for the ILV Assessment

5.1 Purpose and Outline

In this section of the report the sources of information used in the ILV Assessment Methodology are described.

The sources of information relied upon for the ILV assessments included:

- Geological maps, soil maps and other historical land use and drainage maps;
- Ground surface elevations, relative to sea level, estimated using DEMs. These models were derived from LiDAR surveys of the Christchurch region undertaken in 2003 and after each of the main earthquakes in the CES;
- Groundwater levels throughout Christchurch, which have been the subject of ongoing monitoring;
- Soil composition data obtained from extensive geotechnical investigations, including CPTs, subsurface drilling, and laboratory tests;
- Models estimating PGA in each of the main events in the CES;
- Aerial photography after each of the main earthquakes in the CES;
- Observed performance of land, including liquefaction, in the CES, relative to the estimated levels of earthquake shaking;
- Mapping of lateral spread caused by the CES; and
- Consequential building damage.

For the ILV Assessments this information was collected and collated into “packs” of information relating to geographic groupings of properties. The compilation of these packs is described in Sections 9.2.2 and 10.2.2. This process enabled the information to be considered on a holistic and spatial basis.

The information for the assessment of ILV was predominantly sourced from the CGD. This database is publicly available to professional engineering companies involved with Canterbury recovery, the New Zealand Government, scientific and academic institutions, EQC, local authorities and insurers.

The only information that was not sourced from the CGD is the EQC LDAT property inspection reports. The LDAT reports for individual properties are available from EQC at the request of the property owner.

5.2 Geological Maps, Soil Maps and other Historical Land Use and Drainage Maps

A range of regional and detailed geological maps is available on the CGD.

The main geological resource used in the assessment of ILV is the Geological and Nuclear Sciences (GNS) geology of the Christchurch Urban Area Map (Brown & Weeber, 1992). Additional resources available on the CGD include:

- the GNS 1:250,000 Geological Map 16 of Christchurch (Forsyth, et al., 2008);
- the GNS Christchurch Geoserver layers of geological units QMAP, faults, folds and boundaries;
- the GNS QMAPs for Aorangi, Kaikoura, Greymouth, Inactive Faults, and All New Zealand;
- Historical land use maps; and
- Historical drainage maps.
A selection of geotechnical and geological maps from the Archives NZ and Christchurch City Council are included as layers on the CGD. These maps have been referred to for additional historic information particularly regarding old swamps.

5.3 Ground Surface Elevations

Ground surface elevation DEMs derived from LiDAR surveys have been used for the assessment of ILV, both in terms of the pre- and post-CES ground surface elevations and the change in ground surface elevations. The ground return LiDAR survey points have been used to develop ‘bare earth’ DEMs of the pre-CES ground surface and after each of the four main CES earthquakes. Bare earth DEMs are models of the ground surface with vegetation and structures removed. For ILV assessment purposes, the DEMs were resolved to the following grids:

- **25m x 25m grid** to estimate the LSN values at each CPT location and to estimate the LSN value for each property for the automated ILV model (refer to Section 8.2); and
- **5m x 5m grid** to estimate the ΔLSN values for each property (refer to Section 8.2).

The accuracy of the LiDAR point elevations was evaluated by comparing them to elevations published by Land Information New Zealand (LINZ) from surveys of an extensive set of survey benchmarks within the Christchurch region (LINZ, 2013) before, between and after the earthquakes. 80% of the LiDAR point elevations were within ±0.07m of the surveyed elevations. The exception was the 2003 survey, where 80% were within ±0.15m. Further discussion about the LiDAR survey and accuracy is provided in Appendix G.

LiDAR surveys were acquired by Australian Aerial Mapping (AAM) and New Zealand Aerial Mapping (NZAM) following each of the main CES earthquakes. The LiDAR survey sources and commissioning agencies are tabulated in Table 5.1. The extents of the LiDAR survey are shown on maps in Appendix G and the accuracy and limitations of the LiDAR survey data, the DEMs and the difference DEMs are also discussed in Appendix G.

<table>
<thead>
<tr>
<th>DEM</th>
<th>Source LiDAR</th>
<th>Commissioning Agencies</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre-September 2010</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 and Waimakariri District Council</td>
</tr>
<tr>
<td></td>
<td>AAM, 6-11 Feb 2008</td>
<td>Environment Canterbury and Selwyn District Council</td>
</tr>
<tr>
<td>Post-September 2010</td>
<td>NZAM, 5 September 2010</td>
<td>Ministry of Civil Defence and Emergency Management</td>
</tr>
<tr>
<td>Post-February 2011</td>
<td>NZAM, 8-10 Mar 2011</td>
<td>Ministry of Civil Defence and Emergency Management</td>
</tr>
<tr>
<td></td>
<td>AAM, 20-30 May 2011</td>
<td>Christchurch City Council</td>
</tr>
<tr>
<td>Post-December 2011</td>
<td>NZAM 17-18 Feb, 2012</td>
<td>Earthquake Commission</td>
</tr>
</tbody>
</table>
5.4 Groundwater Surface Levels

Groundwater data used for ILV assessment was obtained from EQC monitoring wells, Environment Canterbury monitoring well records and Christchurch City Council monitoring wells. All of this data are available on the CGD (MBIE, 2012).

Data from approximately 1,000 shallow groundwater monitoring wells has been analysed. The monitoring wells are typically less than 10m deep and are therefore representative of the groundwater conditions in the upper soil layers.

These groundwater monitoring well records were used to develop four groundwater surface levels. The levels selected were the 15th, 50th and 85th percentile and the surrogate median groundwater surfaces. The 15th, 50th and 85th percentile groundwater surfaces include only groundwater monitoring wells with long term records (i.e. more than 12 months of data). The surrogate median groundwater surface includes groundwater monitoring wells with both short term (i.e. 12 months or less) and long term data. Further detail about the development of these models is provided in van Ballegoooy et al. (2014a).

The 15th, 50th and 85th percentile groundwater surfaces were used to estimate mean LSN values. The surrogate median was used to estimate median LSN values. The use of these LSN estimations is discussed in more detail in Sections 7, 8.2 and 8.3.

5.5 Geotechnical Investigation Data

The CGD (MBIE, 2012) is a geotechnical data repository used by (among others) professional geotechnical engineers in Christchurch. This database holds a collection of geotechnical investigation information uploaded by EQC, private insurers and others. As at the end of 2014, information was available from the CGD regarding approximately 18,000 CPTs, 4,000 boreholes, 1,000 groundwater monitoring wells and 6,000 laboratory tests from recovered soil samples located around the Christchurch area. The investigations available in the CGD are primarily sourced from post-CES testing and have been used in the detailed assessment of ILV, including sensitivity analyses for liquefaction susceptibility and liquefaction triggering assessments.

Only the geotechnical data publicly available from the CGD, at the time each information pack (used to assess ILV) was compiled, has been used. These packs (for Stage 1 and Stage 2 assessments) are described in more detail in Sections 9.2.2 and 10.2.2 of this report.

The number of geotechnical investigations available in the CGD continues to increase as more investigation data collected by private insurers and private property owners is uploaded to the CGD. Therefore, any geotechnical information that has become publicly available subsequent to the compilation of an ILV assessment information pack will not have been considered as part of the ILV assessment process.

It is acknowledged that in some limited cases the consideration of the additional geotechnical information (which has become available subsequent to the compilation of the ILV assessment information pack) could change the ILV assessment outcome. A review process has therefore been adopted through which EQC customers can request that the additional geotechnical investigation data be considered as part of their ILV assessment.

5.6 PGA Models of the Main CES Earthquakes

Conditional PGA contours developed for liquefaction assessment by Bradley Seismic and the University of Canterbury (referred to as the University of Canterbury model) are provided on the CGD (Bradley and Hughes, 2012). For the University of Canterbury model, PGA at each location was
estimated by combining the prediction from the empirical ground motion model of the fault rupture with the PGAs recorded at any adjacent strong motion recording stations.

The PGA contours developed by O’Rourke et al. (2012) as an interpolated geometric mean from recording stations (the 
Cornell University model) are also provided on the CGD. These were examined where significant differences were evident between the two PGA models.

The key difference between these two PGA models is that:

- The University of Canterbury model accounts for the ground motion attenuation physics, incorporating the location and magnitude of each of the main CES earthquake events;
- The Cornell University model in contrast, is a surface fitted interpolation over the geometric mean PGA values obtained from each recording station.

These two PGA models have been used for back analyses of the CPT-based liquefaction consequence parameters (in particular the LSN parameter) to compare the estimated LSN values with the observed land performance as part of the liquefaction vulnerability assessment (discussed in Sections 7 to 10).

Where the two models provide similar PGA estimates there is a higher level of confidence in the PGA estimate for a given property. Conversely, where the two models provide significantly different PGA estimates, then there is a lower level of confidence in the PGA estimate for a given property. However, the corresponding recorded land and building damage for an event also provides a good indication of which model provides the best estimate of PGA for that area.

5.7 Aerial Imagery

High resolution aerial photographs of areas of Christchurch affected by liquefaction have been used for the assessment of ILV. The images were primarily used to confirm the extent of any liquefaction ejecta (Section 5.8), assess the extent of landform changes due to human development and understand the influence of dense vegetation on LiDAR survey data (Section 5.3).

The aerial photographs were acquired on 5 September 2010, 24 February 2011, 14 – 15 June 2011, 16 June 2011 and 24 December 2011 by NZ Aerial Mapping. The acquisition dates and commissioning agencies for each photograph set are shown in Table 5.2.

<table>
<thead>
<tr>
<th>Photograph Set</th>
<th>Acquisition Date(s)</th>
<th>Commissioning Agencies</th>
</tr>
</thead>
<tbody>
<tr>
<td>Post-September 2010</td>
<td>5 September 2010</td>
<td>Ministry of Civil Defence and Emergency Management</td>
</tr>
<tr>
<td>Post-June 2011</td>
<td>14-15 June 2011</td>
<td>The Earthquake Commission</td>
</tr>
<tr>
<td></td>
<td>16 June 2011</td>
<td>The Earthquake Commission</td>
</tr>
<tr>
<td>Post-December 2011</td>
<td>24 December 2011</td>
<td>The Earthquake Commission</td>
</tr>
</tbody>
</table>

The images were acquired as soon as possible after each of the main CES earthquakes. The post-September 2010 and post-June aerial photography was obtained in light intensity conditions that were not ideal for aerial photography, but nevertheless the data are still useful records of where liquefaction ejecta occurred.
5.8 Land Damage Observations

Land damage mapping was undertaken following the September 2010, February 2011, June 2011 and December 2011 events through observational mapping, aerial photography and aerial LiDAR. The following information is available on the CGD:

- **Aerial Photography**: as discussed in Section 5.7;
- **Rapid inspections**: On foot rapid inspections were undertaken of liquefaction and lateral spreading in areas with significant damage following each earthquake. These have been collated into layer maps on the CGD for the September 2010 (property observations), February 2011 (road and property observations), June 2011 (road observations) and December 2011 (limited road and property observations as well as aerial surveys) events; and
- **Ground Cracking**: Ground cracks were mapped post the September 2010 and February 2011 events. Crack mapping was limited in the September event where mapping was generally undertaken on specific properties for insurance claims. Ground crack mapping following the February event was carried out at 1:10,000 to 1:2,000 in various areas. Horizontal ground movement was also derived from the LiDAR surveys and checked against ground surveys.

For the purpose of simplicity the land damage observation maps presented in this report have been provided as three colour maps representing the broad land damage categories of none-to-minor, minor-to-moderate and moderate-to-severe defined in Section 3.7.3 and Appendix B. However, six colour versions that provide more granularity for the same data set have been used in the ILV Assessment Methodology. Examples of these maps are provided in the regional maps in Appendix K.

The relationship between the simplified three colour land damage maps and the more granular six colour land damage maps is shown in Table B1.1 in Appendix B.

In addition to the aerial and ground based land damage maps available on the CGD, the individual property LDAT inspection reports undertaken for approximately 60,000 properties in the areas affected by liquefaction land damage have also been used. These reports contain detailed qualitative and quantitative observations and locations of land damage including observed ejecta (including under timber floors), ground cracking, ground undulations, ground surface subsidence, lateral spreading and building damage. These reports are accompanied with photographs taken at the time of inspection.

5.9 Lateral Spread Mapping

The total horizontal movement of the ground surface between each of the main CES earthquakes and over the CES has been derived from a correlation process between respective LiDAR survey data sets. By subtracting the associated horizontal tectonic component, a map layer was developed giving an approximation of the magnitude and direction of the horizontal movement due to liquefaction related effects on the flat land across Christchurch. The estimated horizontal movements of the land around the lower reaches of the Avon River are given in Appendix F. The map showing the estimated horizontal movements over the wider Christchurch area is available on the CGD.

The horizontal movement mapping is used in the ILV assessment process to assist in the identification of properties where the observed damage may be in whole or in part the result of lateral spreading. For further detail on the development of these horizontal movement maps refer to Bevan et al (2012).
6 ILV Assessment Assumptions

6.1 Purpose and Outline

In undertaking this engineering assessment it has been necessary to make a number of assumptions. This section of the report explains those assumptions as they relate to:

- Earthquake seismicity (Section 6.2);
- Return period levels of earthquake shaking (Section 6.3);
- Anthropogenic and climate change influences on groundwater levels (Section 6.4);
- Changes in soil behaviour (Section 6.5);
- Vulnerability to lateral spreading (Section 6.6);
- Impact from other forms of land damage (Section 6.7); and
- Long access ways are not considered in the assessment of ILV (Section 6.8).

6.2 Earthquake Seismicity

Changes in seismicity (the level of shaking expected in an earthquake of a given return period) have occurred in the Christchurch area as a result of the rupturing of the Darfield fault in the September 2010 earthquake. As a result of these changes in regional seismicity, properties that may not have been materially vulnerable to liquefaction for a given return period level of ground shaking prior to the CES, may now be materially vulnerable to liquefaction for the same return period level of ground shaking because the level of shaking for that return period has increased.

EQC is only entitled to compensate for loss arising from physical changes to insured residential land. EQC has advised T+T that, for something to be considered a ‘physical change’ a change to the structure or materials of the residential land is required. While the seismicity of Christchurch as a whole has increased as a result of the CES, this results from large scale effects on faults rather than changes to the structure or materials of specific residential land. As a result, any increase in vulnerability due to changes in seismicity does not result from a physical change to the land which is covered by the EQC Act 1993.

To avoid taking into account increases in liquefaction vulnerability resulting from changes to seismicity which are unrelated to a physical change to the insured land, the ILV Assessment Methodology assumes a consistent level of seismicity both before and after the CES.

EQC has asked T+T to assess ILV based on the post-CES level of seismicity. This enables consideration of the current (post-earthquake) level of vulnerability of the land with the current seismic hazard. It is noted that had the ILV assessments been conducted on the basis of the pre-CES level of seismicity, less vulnerability and less change in vulnerability would have been estimated. As a result, fewer properties would have satisfied both engineering criteria and therefore fewer properties would have been eligible for ILV.

Prior to the CES the design 25, 100 and 500 year return period levels of earthquake shaking in Christchurch were PGAs of 0.06g, 0.12g and 0.25g, respectively, for a $M_{\text{w}}$ 7.5 earthquake based on the New Zealand Loadings Code (NZS1170.5).

In Christchurch, the seismic hazard is now higher than it was before the CES (McVerry, et al., 2012). This is because the stress relief from the rupture of the various faults has been transferred to adjacent faults in the Canterbury region which, as a result of this transfer, now have a higher probability of rupturing. The currently elevated seismic hazard is expected to reduce with time provided that no major earthquakes occur. If one does occur, then the seismic hazard would again be expected to re-set to a higher value.
The MBIE (2012) guidelines specify the post-CES 25 and 500 year return period levels of earthquake shaking for liquefaction assessment as a PGA of 0.13g and 0.35g, respectively, paired with a $M_W$ 7.5 earthquake (which incorporates the elevated seismicity post-CES). These PGA values are intended by MBIE to be used as interim values as part of the foundation design of residential buildings as part of the recovery from the CES while the seismicity is elevated. The 100 year return period level of earthquake shaking (derived from interpolation between the MBIE 25 and 500 year return period levels of earthquake shaking) is estimated to be 0.2g paired with a $M_W$ 7.5 earthquake.

As the seismicity is expected to decrease over time (in the absence of a further significant earthquake event) these ground motions are an upper bound estimate of the expected levels of earthquake shaking over the next 50 to 100 year period.

The increased levels of seismicity mean that a large number of properties which were not materially vulnerable to liquefaction damage at 25, 100 and 500 year return period levels of earthquake shaking prior to the CES are now vulnerable at the same respective return period levels of earthquake shaking. However, the faults contributing to the seismicity in Christchurch are spatially distributed throughout the Canterbury region and most of the contribution to seismicity comes from faults which are of regional significance and are mainly located beyond the urban residential areas of Christchurch. This is discussed further in Appendix D.

6.3 Return Period Levels of Earthquake Shaking for ILV Assessment

In order to assess the two engineering criteria, a return period level of shaking and corresponding ground motions needs to be assumed. EQC has asked T+T to assess vulnerability based on an up to 100 year return period level of shaking and the associated ground motion values of $M_W = 6.0$ and PGA = 0.3g (here on referred to as M6 0.3g).

That request is based on engineering advice discussed briefly below.

6.3.1 Rationale for Adopting the 100 Year Return Period Level of Shaking Scenario

To assess liquefaction vulnerability (as well as change in liquefaction vulnerability), it is necessary to first select the return period level of earthquake shaking that is to be considered.

Based on engineering advice from T+T EQC has asked T+T to assess ILV at up to a 100 year return period level of earthquake shaking. If, based on the post-CES ground levels, a property is expected to experience material damage from liquefaction at a 100-year return period level of shaking or less, then it is considered to be materially vulnerable (Criterion 1).

Land is not separately insured outside of New Zealand. As a result, there is no significant international practice in relation to recognition of liquefaction vulnerability that can be drawn on for the purposes of selecting a return period level of shaking.

Accordingly, the selection of the up to 100 year return period level of earthquake shaking was based on the following engineering considerations:

- Design parameters for land hazard identification in New Zealand; and
- Design parameters for buildings in New Zealand.

**Design parameters for land hazard identification in New Zealand**

The approach taken for the assessment of natural hazards for the development of land in New Zealand has been considered. Section 106 of the Resource Management Act (1991) and Section 71 of the Building Act (2004) include a test of whether land is likely to be subject to damage from a natural hazard. For this application, “likely” has been determined to be a 1 in 100 year return period event (DBH, 2008), rather than an extreme event with a low probability of occurrence.
Liquefaction is not currently a hazard listed in either the Resource Management Act 1991 or the Building Act 2004. However, adopting an up to 1 in 100 year return period is consistent with the approach taken for other natural hazards which relate to land in both of these Acts. This also applies to other district and regional planning documents around the country.

While these assessment periods have generally been applied to other forms of natural disasters (flood, land slippage and coastal erosion), there is no reason that a similar period should not be adopted for liquefaction.

In addition, when requested for geotechnical advice by developers or a person seeking to obtain a resource consent to subdivide the land, it is best practice for an engineer to examine the property with regards to liquefaction vulnerability for a 100 year return period level of earthquake shaking to assess its suitability for residential purposes.

**Design parameters for buildings in New Zealand**

The approach taken for the design of residential buildings in the New Zealand Building Code has also been considered. The New Zealand Building Code is performance based. That is, the expected performance of buildings is prescribed at 25 year and 500 year return period levels of earthquake shaking. At 25 year return period levels of earthquake shaking the performance expectation is that building damage should only be minor and non-structural. At 500 year return period levels of earthquake shaking the performance expectation is that buildings should not collapse, thereby preventing loss of life. However, the building is expected to sustain structural damage. Engineers can utilise any design that meets these performance expectations.

In eastern Christchurch, where the estimated ground motions were greater than 500 year return period levels of earthquake shaking for some of the main CES earthquakes (described in Section 3.7.2), the performance of both the land and the buildings showed that no residential buildings on TC1, TC2 and TC3 land collapsed due to liquefaction related land damage. Accordingly, despite widespread triggering of liquefaction at the 500 year return period levels of earthquake shaking, the TC1, TC2 and TC3 land performance was demonstrated to achieve the residential building performance expectations to a satisfactory level.

This is consistent with international experience that liquefaction damage for single and two storey light weight residential structures is not a life risk. As a result, it is unlikely that liquefaction vulnerability at the 500 year return period levels of earthquake shaking would govern the foundation design of single storey or two storey residential buildings.

With the exception of some of the residential Red Zone areas, in areas where ground motions were around 25 year return period levels of earthquake shaking, there was no evidence to suggest that liquefaction triggering caused any structural damage to residential buildings (Rogers, et al., 2015; van Ballegooy, et al., 2015d). Hence the land performance at the 25 year return period levels of earthquake shaking also satisfactorily achieved the building performance expectations.

Therefore, in evaluating appropriate return period levels of earthquake shaking for the assessment of liquefaction vulnerability for land, neither the 25 nor the 500 year return period levels of shaking assist in that assessment. This is because the land in Christchurch has demonstrated that it has already adequately supported the residential buildings and achieved the New Zealand Building Code performance expectations. Accordingly, a return period somewhere between the 25 and 500 year return period level of earthquake shaking is appropriate to assess material liquefaction vulnerability.

Based on a consideration of the above factors, the selection of a 1 in 100 year return period levels of shaking provides an appropriate basis on which to assess liquefaction vulnerability.
6.3.2 Rationale for Assuming Ground Motion of M6 0.3g

As discussed above, the MBIE guidelines (2012) specify post-CES 25 and 500 year return period levels of earthquake shaking for liquefaction assessment as part of foundation design residential buildings as a PGA of 0.13g and 0.35g, respectively, paired with a M\(_{W}\) 7.5 earthquake. The 100 year return period level of earthquake shaking (estimated from interpolation between the MBIE 25 and 500 year return period levels of earthquake shaking) is 0.2g paired with a M\(_{W}\) 7.5 earthquake.

However, the critical 100 year return period level of earthquake shaking for evaluating liquefaction vulnerability for the Boulanger and Idriss (2014) liquefaction triggering assessment method is for lower magnitude higher PGA ground motions (as discussed and summarised in Appendix A). Therefore an appropriate equivalent 100 year return period lower magnitude, higher PGA case is required for undertaking ILV land damage assessment.

A site specific Probabilistic Seismic Hazard Analysis (PSHA) for Christchurch soil sites was undertaken by Bradley Seismic Ltd (refer to Appendix D). This analysis indicates a mean 100 year return period M\(_{W}\) value of 5.8 based on the modified seismic activity in the immediate vicinity of Christchurch. This was rounded to 6.0 and has been adopted as the M\(_{W}\) value for the ILV land damage assessment process.

Magnitude scaling is the most common approach to finding an equivalent PGA value for different magnitude events, but the Magnitude Scaling Factor (MSF) in the Boulanger and Idriss (2014) liquefaction triggering assessment method is a function of both the normalised CPT tip resistance (q\(_{c}\)) and M\(_{W}\).

The difficulty with scaling the 0.2g at M\(_{W}\) 7.5 level of shaking to an equivalent PGA at M\(_{W}\) 6.0 value (using the Boulanger & Idriss (2014) MSF) is that, due to spatially differing soil characteristics, different scaling factors would be required for virtually every site and every soil layer throughout Christchurch. For the loose to medium dense soils with lower q\(_{c}\) values (i.e. the soils in Christchurch which have the greatest liquefaction potential), the Idriss and Boulanger (2008) MSF equations result in significantly higher MSF values when compared to the Boulanger and Idriss (2014) MSF equations.

This is demonstrated by applying the Idriss and Boulanger (2008) and Boulanger and Idriss (2014) MSF equations for a M\(_{W}\) 6.0 earthquake for a number of selected CPTs from the TC2, TC3 and residential Red Zone areas shown in Figure 6.1. The CPT q\(_{c}\) and I\(_{c}\) traces are shown in the first two left hand columns of Figure 6.2 and the estimated MSF traces are shown in the third column from the left. The coloured traces in the graphs on the two right hand columns represent the estimated MSF values using the Boulanger and Idriss (2014) method. The black dashed lines represent the Idriss and Boulanger (2008) MSF which is a constant value (i.e. independent of soil properties).
Comparison of the MSF traces in Figure 6.2 for the MW 6.0 earthquake show that the Boulanger and Idriss (2014) estimated values for some soil layers are higher than the Idriss and Boulanger (2008) values. For other soil layers the opposite is true. Closer inspection of the MSF traces shows that, where the Boulanger and Idriss (2014) estimated values for some soil layers are higher than the Idriss and Boulanger (2008) values, the corresponding CPT $q_c$ values are higher and are unlikely to liquefy for the ranges of ground motion values of relevance for residential design purposes. Therefore, estimated MSF values for the soil layers which have an assessed liquefaction triggering factor of safety greater than 1.5, based on the MBIE specified 500 year return period levels of earthquake shaking, using the Boulanger and Idriss (2014) liquefaction triggering methodology, have been filtered out so that the soil layers of greatest interest could be further examined.

The filtered MSF traces are shown in the right hand column of Figure 6.2. The filtered results show that the Boulanger and Idriss (2008) MSF is generally an upper bound for loose to medium-dense soils and is likely to overestimate the equivalent PGA value for the soil layers of interest when compared to the Boulanger and Idriss (2014) MSF.

In order to avoid these scaling issues, Bradley Seismic Ltd undertook a PSHA for Christchurch soil sites (refer to Appendix D). Based on the post-CES seismic activity in the immediate vicinity of Christchurch, the mean $M_w$ values and associated PGA values for the 100 year return period level of earthquake shaking have been estimated as an $M_w$ 5.8 and PGA of 0.19g. However this is significantly lower than the scaled MBIE guideline (2012) 25 year return period design value.
Figure 6.2: Estimated MSF values for a $M_W$ 6.0 earthquake based on the Idriss and Boulanger (2008) and Boulanger and Idriss (2014) equations for selected CPT profiles (from locations shown in Figure 6.1) from the TC2, TC3 and residential Red Zone areas in Christchurch.
The MBIE guidelines are the industry standard with respect to the assessment of seismic hazard for single storey residential buildings in Christchurch. Typically a PSHA is only undertaken for structures of significance such as hospitals and other large buildings. Therefore, to avoid inconsistency between ILV land damage assessments and the MBIE guidance (2012) on assessment of liquefaction vulnerability, the Idriss and Boulanger (2008) MSF relationship has been applied to the M7.5 0.2g case. This gives an equivalent value of M6 0.3g.

It is recognised that this is a conservative assessment resulting in an upper estimate of the most likely 100 year return period levels of earthquake shaking for Christchurch based on the most recent scientific analysis.

6.4 Anthropogenic and Climate Change Influences on Groundwater Levels

Anthropogenic (man-made) influences and climate change induced sea level rise and climate change in future weather patterns will influence future groundwater levels and as a result will influence future liquefaction vulnerability (Russell, et al., 2015; Quilter, et al., 2015). However these changes in liquefaction vulnerability are not related to any physical change to the land as a result of the CES.

In order to assess the two engineering criteria, a groundwater level needs to be assumed. EQC has asked T+T to use the current groundwater levels (i.e. 15th, 50th and 85th percentile regional groundwater surfaces) for the assessment of ILV and that the assessment should not take into account any future changes in groundwater levels as result of anthropogenic influences, climate change and sea level rise.

The engineering basis for EQC’s decision is set out briefly below.

6.4.1 Rationale for Assuming Groundwater Levels Excluding an Allowance for Future Anthropogenic Influences to Groundwater Levels

Anthropogenic changes to groundwater levels can be caused by:

- Changes in groundwater extraction patterns - either by pumping to lower groundwater levels in low lying areas or for extraction purposes for urban and rural use;
- Changes to drainage networks in low lying areas;
- Further land development increasing stormwater runoff and decreasing stormwater infiltration; and
- Changing stormwater management approaches such as construction or removal of soakage pits and ponds.

It is uncertain what anthropogenic influences will occur in the future development of Christchurch and how these will affect the groundwater levels. These changes do not form part of any physical change to the land occurring as a result of the CES. Therefore such potential effects have not been taken into account when determining appropriate groundwater levels for the assessment of ILV.

6.4.2 Rationale for Assuming Groundwater Levels Excluding an Allowance for the Effect of Climate Change on Sea Level Rise

Sea level rise will result in an increase to the groundwater levels in the low lying areas of eastern Christchurch. Studies undertaken by Quilter et al. (2015), based on a simplified modelling approach, indicate that the groundwater level is controlled by sea level up to the eastern side of the CBD (i.e. approximately from Fitzgerald Ave eastwards). Sea level rise is likely to affect groundwater levels in this area. If sea level rise was taken into account and applied to both pre-CES and post-CES ground surface levels more properties would qualify with Criterion 1 and therefore the potential pool of
properties that could then be considered by Criterion 2 would increase. Ultimately this would affect the number of properties eligible for ILV.

A problem with using an elevated groundwater surface to take into account sea level rise is that a timeframe would also need to be defined. While there is consensus within the scientific community that sea level rise will occur, the rate at which it will occur is less certain. In the near term, effects are likely to be gradual, involving changes of an order of magnitude within the next 20 years that are within the range of uncertainty of the ILV assessments.

Rising sea levels and corresponding rise in groundwater levels are also likely to result in anthropogenic intervention in low lying areas (e.g. more drainage and pumping in low lying areas). These anthropogenic changes to groundwater levels due to sea level rise would result in additional uncertainty in selecting appropriate groundwater levels.

Given this uncertainty these potential effects have not been taken into account when determining appropriate groundwater levels for assessment of ILV.

6.4.3 Rationale for Assuming Groundwater Levels Excluding an Allowance for the Effect of Climate Change on Future Weather Patterns

It is unknown how the influence of climate change on future weather patterns will affect groundwater levels in Christchurch outside of those areas that are controlled by sea level or over what timeframe this will occur.

If climate change results in a decrease in the annual rainfall in the Canterbury region and Southern Alps, then the median groundwater level may decrease and thus liquefaction vulnerability might decrease. If climate change results in an increase in the annual rainfall in the Canterbury region and the Southern Alps, the groundwater level is likely to be higher and thus liquefaction vulnerability is likely to increase. Which of these two scenarios will occur is unclear.

In addition, weather related climate change is likely to result in anthropogenic intervention (e.g. more groundwater extraction) if the climate change results in prolonged periods without rainfall. These anthropogenic changes to groundwater levels due to climate change effects would result in further uncertainty in selecting an appropriate groundwater level. At this point in time, there is no way of reliably predicting what (if any) intervention will occur.

Given the slow and gradual nature of any changes and this uncertainty, these potential effects have not been taken into account when determining appropriate groundwater levels for assessment of ILV.

6.5 Change in Soil Behaviour

Almost all of the geotechnical investigation data available in the CGD, which has been used for ILV assessment purposes, was obtained following the 22 February 2011 earthquake and is therefore considered to be representative of the post-CES soil strength and stiffness characteristics.

For the liquefaction vulnerability assessments, it has been assumed that the estimated resistance of soil to liquefaction, often described as the Cyclic Resistance Ratio (CRR) was the same pre-CES as it has been estimated post-CES.

This assumption is necessary given the limited availability of geotechnical investigation data prior to the CES. This assumption is reasonable based on the current state of practice with respect to the assessment of liquefaction triggering and estimation of the CRR in soil deposits, and the likelihood that any change in CRR will not be material to the ILV assessments.

However, there are three hypotheses within the literature that are potentially at odds with this assumption. These hypotheses are:
Liquefaction of soils can result in an increase in density of the soil which increases the CRR of the soil from the pre-earthquake condition in a process known as Densification; and

Liquefaction of soils results in new soil fabric resulting from reconsolidation after liquefaction. This new soil fabric is known to have a very low resistance to liquefaction. Soil ageing (discussed in the next point) is related to micro changes in the soil fabric.

Liquefaction of soils results in a breaking of mechanical and chemical bonds between the soil particles which decreases the CRR of the soil from the pre-earthquake condition. Over time these mechanical and chemical bonds reform resulting in an increase in CRR of the soil in a process known as Soil Ageing.

The purpose of this section is to address each of these hypotheses with comparison to the data available on the CGD and demonstrate that it is reasonable to assume that the pre and post-CES CRRs are the same.

6.5.1 The Densification of Soil due to Ground Shaking

Each time a soil has liquefied from an earthquake, it may reconsolidate which can result in some minor volumetric densification (i.e. it becomes slightly denser) and accompanying ground surface subsidence as seen in the liquefaction related subsidence maps shown in Figure 3.8c. As a result, the post-CES CRR of some liquefied soil layers may be greater than the pre-CES CRR. This is potentially more likely to occur in the deeper soil layers where there is greater confinement and is less likely to occur in the upper soil layers where there is less confinement.

At present, there is no practical and reliable way of quantifying this increase in CRR. Nor is it considered possible to determine the boundary between liquefying soil layers, which have subsequently had an increase in CRR, and liquefying soil layers, which have not had an increase in CRR.

To test the effects of these influences, CPTs that were pushed in Christchurch in close proximity to one another pre and post-CES have been analysed. There are 25 locations in Christchurch where 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 6.3. 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 6.3.
Comparison of adjacent CPT traces at each location are shown in Appendix E. 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 (consistent with the expected natural soil variability discussed in Section 3.6). This variability in ground conditions is reflected in the corresponding $I_c$ traces in Appendix E.

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 6.4 and 6.5, respectively.

Figure 6.4 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 Appendix E). 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 6.4: Pre and post CES cumulative frequency graphs of $q_c$ for varying depths from CPT locations in Figure 6.3 filtered for soil layers with $I_c < 2.6$.

Figure 6.5 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 Appendix E). Note that due to spatially variable ground conditions some filtering of the data presented in Figure 6.5 has been undertaken. The full details of this are discussed in Appendix E.

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 6.5, because the CPTs that were used with each corresponding post-February, post-June or post-December 2011 CPTs 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.

6.5.2 The Effects of Soil Ageing

There is ongoing research in the area of ‘soil ageing’, which suggests that mechanical and chemical processes that take place following the sedimentation of sand deposits results in an increase in the soil strength and therefore CRR over time (Leon, et al., 2006; Green, et al., 2008; Hayati & Andrus, 2009). The published literature suggests that liquefaction triggering resets the soil ageing clock back to zero in the liquefied soil layers (Green, et al., 2008; Maurer, et al., 2014b). The current levels of scientific understanding of soil ageing and its effect on the soil strength and CRR are still in early development. Proposed methods to account for this, such as strength gain factors (often annotated as $K_{DR}$), are not widely accepted in engineering practice.

Published studies have typically considered very old (Pleistocene age) sand deposits (Lewis, et al., 1999; Arango & Migues, 1996) or a limited number of laboratory tests and undisturbed samples of recent Holocene deposits (Seed, 1979). Two published studies (Hayati & Andrus, 2009; Robertson, 2000) have considered both undisturbed samples from naturally deposited soils and reconstituted samples for laboratory tests ranging in age from <1 year to 35 million years to investigate the increase in soil strength and CRR over time.

A plot of strength gain factors versus time from recently published research papers is shown in Figure 6.6.
While all published studies show an increase in the strength gain factor over time, the rate of increase and the reference age for naturally deposited or liquefied soils varies between all studies. These differences are likely due to the natural variability of the materials studied as well as different methods of preparation and testing used by the various researchers. Given the variability of the studies the selection of an appropriate strength gain factor for Christchurch soils is challenging. Furthermore, as stated above the results in Figure 6.4 and Figure 6.5 show that there is no systematic bias in pre-CES $q_c$ values being lower than the post-CES $q_c$ values. Therefore, the underlying premise of an initial loss of soil strength has not been demonstrated with this data.

Given that:

- Strength gain factors are not currently widely used in engineering practice;
- Studies used to estimate strength gain due to soil ageing provide a wide range of estimates to adopt; and
- Investigation into the CGD CPT dataset has not demonstrated a loss of soil strength following the CES;

it is reasonable not to account for the effects of soil ageing in the ILV Assessment Methodology.

### 6.6 Vulnerability to Lateral Spreading has not Increased

The potential for lateral spreading from future earthquake shaking has been judged to have not increased as a result of the physical changes to the land caused by the CES. Therefore, when assessing Criterion 1, it has been assumed that lateral spreading vulnerability does not need to be considered when determining whether a property is materially vulnerable to liquefaction damage. That is, only vulnerability to “free field” liquefaction damage (that is, liquefaction damage unrelated to lateral spreading) is assessed in Criterion 1.
While the prediction of lateral spreading vulnerability is highly complex, the models recommended for use in engineering practice all generally predict an inverse relationship between lateral spreading vulnerability and the elevation between the ground surface and the base of adjacent river channels and old river terraces, all other factors being held constant.

As a result of the lateral spreading and ground surface subsidence caused by the CES, the elevation difference between the ground surface and the base of adjacent river channels and old river terraces remained the same or reduced (refer to Appendix F). Therefore, given that the elevation difference has not increased (and in most instances has decreased) and that the soil properties have not changed (refer to Section 6.5), the potential for lateral spreading has not increased and in most instances has decreased as a result of the changes to the land caused by the CES (refer to Appendix F for further analyses).

This assumption is supported by the liquefaction related horizontal movement measurements from the LiDAR surveys, satellite imagery, GPS based benchmark surveys, ground crack observation surveys, land damage observations and river channel profile surveys following each of the four main CES earthquakes. Analyses presented in Appendix F show that the horizontal movements indicate a reduction in the amount of lateral spreading for each of the three main earthquakes subsequent to the September 2010 event relative to the seismic demand of each earthquake.

6.7 Impacts of Other Forms of Land Damage

In addition to ILV damage, the CES caused other forms of land damage in Christchurch. These are described in Appendix B of this report.

Some of these other forms of land damage may also increase the vulnerability of land to liquefaction if unrepaired. In particular, crust disturbance, such as cracking, could provide a path for the ejection of liquefied soil and thereby increase the land vulnerability to liquefaction damage.

However, observable damage is compensated for separately by EQC by paying the cost of repairing the cracks in accordance with approved repair methodologies. 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 essentially reinstated back to the pre-CES crust integrity and hence pre-CES liquefaction vulnerability.

As a result, the loss of crust integrity as a result of cracking has not been taken into account as part of the ILV assessment process. This is because it has been assumed that the compensation provided for ground cracking will be used to re-instate the crust to its pre-CES integrity.

6.8 Long Access Ways are not Considered in the Assessment of ILV

The ILV engineering assessment documented in this Report was undertaken over the residential land comprising:

- The land under the main residential building (the building footprint); and
- The land in an area up to 8 m, in a horizontal line, of the main residential building, within the property boundary.

For most properties in urban residential Christchurch, this area of residential land covers all of the land holding, including any driveway and land around appurtenant structures (such as the garage and sheds). An example of this is presented in Figure 6.7a.
The ILV assessment and, in particular, analysis of whether liquefaction vulnerability or a change in liquefaction vulnerability was material (as discussed in Sections 7.3 and 7.4) were determined in relation to the use and amenity of this land as a platform for a residential building.

All outcomes of the ILV assessment methodology recorded in this report, including the maps and statistics presented in Section 11, relate solely to the assessments carried out in relation to the above areas.

It is acknowledged that the “residential land” insured by the EQC Act may also include areas of land which are not otherwise included in the area of land that constitutes the building footprint and land within 8 m, in a horizontal line, of the main residential building. These additional areas may include, for example, the main access way within 60 m, in a horizontal line, of the residential building, and land up to 8 m, in a horizontal line, around appurtenant structures. An example of this situation in relation to a long access way is presented in Figure 6.7b.

It is possible that, in a limited number of cases, additional areas, such as a long access way, may experience a change in vulnerability to liquefaction damage when the building platform and area around the residential building does not. Where, on the balance of probabilities, it can be concluded that a change has occurred to such an additional area (and the land holding does not otherwise satisfy the Engineering Criteria), the additional area will be separately assessed against the Engineering Criteria and Criterion 3.

The assessment of these additional areas against the Engineering Criteria and Criterion 3 will take account that, unlike the building footprint and area surrounding the residential building, these areas generally cannot be used as a platform for a residential building. Accordingly, a change in liquefaction vulnerability may not be material for land that can only be used as an access way, even though that change may be material for land that can be used to support a residential building.
7 Estimating Liquefaction Vulnerability and Assessing Criterion 1 and Criterion 2

7.1 Purpose and Outline

To undertake an assessment of ILV within the Christchurch area it was necessary to develop a methodology that could use the extensive CPT and borehole data gathered following the CES, together with LiDAR DEMs obtained both before and after the CES, to estimate liquefaction vulnerability and change in liquefaction vulnerability due to ground surface subsidence as a result of the CES in conjunction with land damage observations and other information to undertake ILV assessments.

It was also necessary to determine what level of liquefaction vulnerability, and change in vulnerability, should be regarded as material for the purposes of assessing each of the engineering criteria.

This section of the report describes:

- The estimation of liquefaction vulnerability including the development of the CPT-based LSN parameter and why it is the preferred tool for estimating liquefaction vulnerability;
- The approach taken to define a material liquefaction vulnerability (that is, Criterion 1 of the engineering criteria), and why the performance of land with LSN value of 16 or greater was chosen as an indicator of this level of vulnerability;
- The approach taken to define a material change in liquefaction vulnerability caused by the CES (that is, Criterion 2 of the engineering criteria), and why a difference in performance of land implied by a difference of 5 LSN units or greater between the pre-CES and post-CES LSNs (∆LSN) was chosen as an indicator of this level of change in vulnerability; and
- The concepts of the median and mean LSN parameters and why EQC has asked T+T to assess ILV considering both of these parameters for the purposes of assessing material liquefaction vulnerability and material change in liquefaction vulnerability.

7.2 Estimating Liquefaction Vulnerability

Following the CES there were a number of different tools available for the estimation of liquefaction vulnerability in the Christchurch area, including:

- Land damage observations (from aerial imagery, land damage mapping and LDAT property inspection reports);
- CPT-based liquefaction vulnerability parameters;
- Composition of near surface soils; and
- Liquefaction related ground surface subsidence.

Of these tools land damage observations provide the best evidence of liquefaction vulnerability. However, land damage observations only reflect the liquefaction vulnerability of a particular property under event specific conditions. These event specific conditions include the level of estimated ground shaking, the estimated groundwater level at the time of the earthquake event and the estimated ground surface elevation to estimate the groundwater depth.

For example, while ILV is assessed at up to 100 year return period levels of ground shaking, most of eastern Christchurch experienced ground shaking levels considerably in excess of the 100 year return period levels of ground shaking. In such areas, liquefaction related damage does not necessarily mean that the land is vulnerable in a lower level event.
Given that:

- The ILV Assessment Methodology requires specific assumptions for each of these conditions (refer to Section 6); and
- The observed land damage in many instances resulted from levels of ground shaking significantly above the M6 0.3g ILV assessment level;

it was necessary to consider additional tools to estimate liquefaction vulnerability.

The most widely used means of assessing liquefaction vulnerability are CPT-based liquefaction vulnerability parameters. This section summarises the development of the CPT-based LSN parameter and describes why it is the preferred tool for estimating liquefaction vulnerability at the M6 0.3g level of earthquake shaking.

It is important to reiterate that the LSN parameter is one of several tools available for the assessment of liquefaction vulnerability. Each ILV assessment has involved the reconciliation of the LSN parameter with the other liquefaction vulnerability tools. In particular, this process of reconciliation has focused on land damage observations. Sections 8.3, 9, and 10 describe this reconciliation process in detail.

### 7.2.1 The LSN Parameter

As discussed in Section 6, to provide detailed understanding of the observed liquefaction in Christchurch and to predict potential liquefaction vulnerability of future earthquakes, EQC commissioned and collated one of the most extensive databases of geotechnical investigation information and land and dwelling performance observations ever assembled. Subsequent to the EQC’s collation of this database, private insurers and engineers have also contributed a large amount of geotechnical investigation information. A critical component of this dataset was the set of more than 18,000 CPTs (as at the end of 2014) in urban Christchurch.

A process involving detailed literature review, expert engagement and expert review was undertaken to select an analysis tool which was suitable to assess liquefaction vulnerability using information contained in the CPT dataset in Christchurch. The following range of existing numerical tools were reviewed:

- The Ishihara (1985) criteria;
- The estimated one-dimensional volumetric densification settlement ($S_{VD1D}$) using the empirical equations developed by Zhang et al. (2002); and
- The Liquefaction Potential Index (LPI) parameter developed by Iwasaki et al. (1982).

Each of these parameters were assessed to have particular advantages and disadvantages for application in the Christchurch area.

The Ishihara (1985) criteria are internationally recognised and widely used in engineering practice to assess the potential for liquefaction damage at the ground surface due to liquefaction of underlying soil layers. However, it assumes a simple 2-layer soil model of a non-liquefying crust overlying a liquefying soil layer. This means it is difficult to apply to the complex layered soils (i.e. multiple interbedded layers of liquefying and non-liquefying soils) found in Christchurch. Nevertheless, the Ishihara (1985) criteria are useful for understanding the influence of the non-liquefying crust thickness on liquefaction vulnerability.

While it is recognised that the Ishihara (1985) criteria are based on a limited number of case studies, it is one of the standard assessment tools for liquefaction vulnerability. In the case of the CES, the Ishihara (1985) criteria has proved robust in predicting where liquefaction damage will and will not manifest at the ground surface in the limited situations where it has been able to be applied.
Results from CPT-based liquefaction vulnerability parameters (i.e. $S_{V1D}$ and LPI) were compared to observed damage in each of the earthquakes to assess how well they predicted liquefaction vulnerability in Christchurch. As a result of these detailed studies (summarised in Appendix A) it was recognised that none of the existing CPT-based liquefaction vulnerability assessment tools were appropriate for assessing ILV in Christchurch and hence a new liquefaction vulnerability parameter was required that was specifically validated for Christchurch ground conditions.

The liquefaction vulnerability parameter developed is called the Liquefaction Severity Number (LSN). LSN is an estimated parameter that expresses in a numerical index the vulnerability of land to liquefaction related damage at a particular level of shaking. The higher the LSN, the greater the vulnerability of land to liquefaction-related damage. For example, an LSN of 20 predicts a greater probability of liquefaction related damage occurring than an LSN of 15, for the same level of shaking.

LSN 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 application of assessing ILV in Christchurch, compared to the alternative parameters are:

- It is better able to analyse the complex layered soil profiles typical across Christchurch (described in Section 3.6);
- It incorporates both the CPT $q_c$ and corresponding CRR of the soil and how severely the soil reacts (i.e. the expected volumetric densification) once it becomes liquefied;
- 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 factors in the non-liquefying crust thickness and the thickness and severity of the underlying liquefying soil layers, thereby providing a more consistent result across a wide range of soil profiles and ground conditions.

Further detail comparing the suitability of the different available numerical tools for assessing liquefaction vulnerability and the development of the LSN parameter is provided in Appendix A.

### 7.2.2 Uncertainties in CPT-based Liquefaction Vulnerability

While the LSN parameter has been adopted as the preferred tool for CPT-based liquefaction vulnerability assessment for ILV purposes, it is subject to a range of uncertainties. These uncertainties include:

- Earthquake motion characteristics;
- Geological spatial variability;
- Soil profile complexities;
- Groundwater pressure and saturation 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 value faces 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 estimated 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 estimated LSN values.

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

7.3 Material Liquefaction Vulnerability (Criterion 1)

In order to assess whether or not a property is vulnerable to material liquefaction damage (Criterion 1 in Section 2.4) it was necessary to determine what level of vulnerability to liquefaction represents the difference between land which is not materially vulnerable to liquefaction damage (i.e. the land is unlikely to suffer material liquefaction related land damage) and land which is materially vulnerable to liquefaction damage (i.e. the land is likely to suffer material liquefaction related land damage).

Figure 3.2 indicates that at M6 0.3g levels of ground shaking, liquefaction triggering is predicted to occur in almost all TC3 and residential Red Zone areas (note no inferences can be made regarding the TC2 areas because there is insufficient CPT data available). Therefore, all of this land has some degree of liquefaction vulnerability (because the estimated LSN will not be 0 if liquefaction triggering is predicted to occur). However, not all of this liquefaction vulnerability is material.

The level of vulnerability which can regarded as material, from an engineering perspective, is related to the use and amenities of the land as a building platform and for other related purposes.

Having considered a range of factors, land can be considered materially vulnerable to liquefaction where:

- The land is more likely to suffer moderate-to-severe liquefaction related land damage than none-to-moderate liquefaction related land damage at up to 100 year return period levels of ground shaking; and

- The vulnerability of the land to liquefaction means that based on specific engineering assessment of geotechnical investigation data an enhanced building foundation (over and above a TC2 foundation) is likely be required when applying the objectives set out in MBIE (2012) Guidelines. This requirement is dependant on the liquefaction vulnerability at 25 year return period levels of earthquake shaking.

Below these levels of liquefaction vulnerability, enhanced TC3 foundation systems would not be required and TC2 foundation systems would be sufficient. That is, any limited liquefaction vulnerability would be adequately addressed by standard engineering solutions (including TC2 foundations).

In addition, in order to use the LSN parameter to assess whether or not a property is vulnerable to material liquefaction damage (Criterion 1 in Section 2.4) it was necessary to determine the value of the LSN parameter which represents this level of vulnerability.
For the purposes of the ILV assessment an LSN value of 16 is considered to be generally representative of the transition between land which is materially vulnerable and land which is not materially vulnerable to liquefaction damage. To determine the LSN value representative of this level of vulnerability, the following have been considered:

- Land and residential building performance across the CES;
- International literature on liquefaction vulnerability; and

Due to the inherent limitations of the LSN parameter (discussed in Section 7.2.2 and Appendix A) the LSN value of 16 is one of several indicators, rather than a strict threshold value, in the determination of whether or not a property is considered vulnerable to a material level of liquefaction damage (Criterion 1).

### 7.3.1 Land and Residential Building Performance

From an engineering perspective, material liquefaction vulnerability is best assessed by considering:

- The likelihood of moderate-to-severe liquefaction related land damage; and
- The likelihood of significant liquefaction related building damage.

Vulnerability to none-to-minor and minor-to-moderate land damage can be accommodated by conventional engineering techniques (i.e. a TC2 foundation), and is within an allowable level of risk for land at the 100 year return period level of shaking. Increases in the risk of damage being sustained should therefore not alter the practical advice that would be given by an engineer in relation to the uses of the land, in particular as a platform to support a residential building.

Vulnerability to significant liquefaction related building damage is more difficult to predict, as the performance of the building in an event causing moderate-to-severe land damage will depend on a number of factors that are independent from the vulnerability of the land to liquefaction. These include foundation design, building shape and building weight. However, as described in Section 3.7.5, data sets are available for the CES that provide some indication, in the Christchurch area, what level of liquefaction vulnerability gave rise to significant building damage across the portfolio of residential buildings in the CES, and therefore could be expected to perform similarly in future up to 100 year levels of earthquake shaking.

To assess the value of LSN for which moderate-to-severe land damage is more likely to occur than none-to-minor and minor-to-moderate land damage, the predicted land performance based on the CES earthquakes was considered. This was achieved by estimating event specific LSN values for each property using the estimated PGA and groundwater levels for the September 2010, February 2011 and June 2011 earthquake events. These LSN values were estimated in the TC3 and residential Red Zone areas because there was sufficient density of CPTs to estimate LSN values for each residential property. The LSN value for each property was then correlated with the corresponding observed land damage for each respective event (Tonkin & Taylor (2013) and van Ballegooy et al., (2014b; 2015c) describe how this was done).

The correlations showed that the frequency distribution of calculated LSN for each land damage observation grouping was relatively consistent with the correlations for each of the three earthquake events. These datasets were combined and box plots and histograms of the data are shown in Figure 7.1.

The plots in Figure 7.1 show that an LSN value of less than 16 characterises properties with none-to-minor liquefaction related land damage and approximately half of the properties with minor-to-moderate liquefaction related land damage. Whereas an LSN value of greater than 16 characterises
properties with moderate-to-severe liquefaction related land damage and approximately half of the properties with minor-to-moderate liquefaction related land damage.

Figure 7.1: (a) Box and whisker plot showing the distribution of land damage observations correlated against LSN for the September 2010, February 2011 and June 2011 earthquake events (b) histogram showing the distribution of land damage observations correlated against LSN.

These analyses were not undertaken for the December 2011 event. This is because, at the time these analyses were originally prepared for publication, the aerial photography for the December 2011 event had not been analysed in sufficient detail to develop the land damage map shown in Figure 3.6. Review of the subsequent observed land damage and estimated PGA values for the December 2011 event indicates that there is no reason to believe that the incorporation of the December 2011 event analyses would result in a different distribution that would alter the LSN indicator values.

The moderate-to-severe land damage observation classification includes land also affected by lateral spreading damage. The LSN parameter is not intended to incorporate the potential for lateral spreading damage. It should only be used as an index to estimate free-field liquefaction vulnerability. However, the inclusion of the lateral spreading damage in the moderate-to-severe land damage category for the back analysis shown in Figure 7.1 is unlikely to bias materially the distribution of observed land performance at different LSN values, and this will not change the LSN indicator value of 16. This is because:

- A very small proportion of properties have sustained lateral spreading damage without also sustaining free field liquefaction damage; and
- The majority of properties which have sustained significant lateral spreading damage, do not have a competent non-liquefying crust and, in the absence of a free face or slope, would be likely to sustain moderate-to-severe free-field liquefaction damage.

To assess the LSN value for which more significant liquefaction related building damage is likely to occur similar analyses were undertaken for the residential buildings in the TC3 and residential Red Zone areas for three groups of BDR (less than 0.3, between 0.3 and 0.5 and greater than 0.5). The

---

3 The box and whisker plots and what they represent are defined in the glossary.
BDR dataset is discussed in Section 3.7.5 with the spatial distribution of BDR shown in Figure 3.9. The results from these analyses are shown in Figure 7.2.

It is important to note that the likelihood of significant liquefaction related building damage is not as reliable an indicator of material vulnerability to liquefaction as the land damage indicator. This is because the performance of the building in an earthquake event that results in liquefaction consequences is dependent on a number of factors that are independent of the vulnerability of the land to liquefaction. These factors include foundation design, building shape and building weight. Notwithstanding these limitations, the available BDR data and land damage observations indicate a spatial correlation between BDR and land damage. Therefore there is merit in also considering this dataset for the establishment of LSN indicator values.

![Figure 7.2: (a) Box and whisker plot showing the distribution of BDR for the residential dwellings in the TC3 and residential Red Zone areas correlated against LSN for the September 2010, February 2011 and June 2011 earthquake events (b) histogram showing the distribution of BDR correlated against LSN.](image)

Similarly to the land damage correlations (shown in Figure 7.1), Figure 7.2 shows that the LSN values greater than 16 characterise properties with a distribution of higher BDR values whereas LSN values of less than 16 characterise properties with a distribution of lower BDR values.

The information on land damage and BDR demonstrate that an LSN value of 16 is an appropriate indicator of material liquefaction vulnerability. Of the two data sets considered, the land damage data set is the most relevant.

### 7.3.2 International Literature on Land Vulnerability

As noted in Section 7.2, the Ishihara (1985) criteria are internationally recognised and widely used in engineering practice to assess the potential for liquefaction damage at the ground surface due to liquefaction of underlying soil layers. Notwithstanding the limitations discussed in Section 7.2, the Ishihara (1985) criteria are a useful tool for comparison with estimated LSN values which are greater than and less than 16.

To develop his criteria, 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 a material level of liquefaction related ground damage; and
- Soil profiles which plot to the right hand side of the boundary curve are not potentially vulnerable to a material level of liquefaction related ground damage.

It is noted that the LSN contour of 15 in Figure 7.3 runs parallel with the Ishihara \( H_1 - H_2 \) boundary curve (up to a non-liquefying \( H_1 \) crust thickness of 5m) and that it provides results that are reasonably consistent with the land damage criteria proposed by Ishihara (1985).

For LSN values of less than 15, the Ishihara (1985) criteria suggests that no material damage would be expected regardless of the thickness of the underlying \( H_2 \) liquefying soil layer. Whereas, for LSN values greater than 15 the Ishihara (1985) criteria indicate material liquefaction damage would be expected. Van Ballegoooy et al. (2015a) provides further detail about the similarity between the Ishihara (1985) criteria and LSN.

![Figure 7.3: The Ishihara (1985) ground damage threshold for M7.5 0.3g compared to contours of LSN for ground shaking of M6 0.3g.](image)

It is noted that the LSN contours in Figure 7.3 are based on M6 0.3g levels of earthquake shaking but the Ishihara (1985) boundary curve was developed from land damage observation data from earthquakes of approximately M7.5 0.3g levels of earthquake shaking. Therefore the absolute values from the LSN contours should not be directly compared to the Ishihara (1985) boundary curve.

If an equivalent Ishihara (1985) boundary curve was to be developed for M6 0.3g levels of earthquake shaking, it is expected that it would plot slightly to the left of the M7.5 0.3g boundary curve. This indicates that an LSN indicator value of 16 at M6 0.3g levels of earthquake shaking is not inconsistent with the Ishihara (1985) criteria.
7.3.3 The MBIE (2012) Guidelines for Rebuilding on Liquefiable Soils in Christchurch

The cumulative frequency graphs of estimated LSN for M6 0.3g levels of earthquake shaking based on post-CES ground surface levels are shown in Figure 7.4. This figure shows that between 50 to 60% of the TC3 properties have an M6 0.3g LSN value of more than 16 based on post-CES ground surface levels. As noted in Section 3.8, approximately 45% of the TC3 properties require more robust foundation solutions (i.e. enhanced TC3 foundations or TC2 foundations in conjunction with ground improvement) to manage the more severe liquefaction vulnerability on those properties.

![Cumulative frequency graphs of estimated LSN for TC3 and residential Red Zone properties for M6 0.3g levels of earthquake shaking based on post-CES groundwater levels.](image)

While the MBIE criteria for designing residential foundation systems is based on SLS (25 year return period) and ULS (500 year return period) levels of earthquake shaking, the majority of the population of properties which require enhanced TC3 foundations largely coincides with the population of TC3 properties where the 100 year return period (M6 0.3g) LSN value is greater than 16 based on post-CES ground surface levels.

Conversely, Figure 7.4 also shows that between 40 and 50% of the TC3 properties have an M6 0.3g LSN value of less than 16 based on post-CES ground surface levels. As noted in Section 3.8 approximately 55% of the TC3 properties can use TC2 foundations.

Therefore, an LSN value of more than 16 is generally consistent with the MBIE criteria for when enhanced TC3 foundations are required and hence implies that the level of liquefaction vulnerability is considered to be material.

As discussed in Section 3.8, due to the relatively low additional cost coupled with significant benefits, it is current engineering practice to specify TC2 foundation solutions on soils that are susceptible to liquefaction. Therefore TC2 foundation solutions are considered to be standard conventional designs that do not take particular account of material liquefaction vulnerability.

Conversely, enhanced TC3 foundation solutions generally include extensive ground improvement and represent a considerable additional cost over and above the TC2 foundation solutions. Enhanced TC3 foundation solutions or TC2 foundations with ground improvement are required when the estimated liquefaction vulnerability is material.
It is important to reiterate that, due to the inherent limitations of the LSN parameter (discussed in Section 7.2.2 and Appendix A) the LSN value of 16 is one of several indicators, rather than a strict threshold value, in the determination of whether or not a property is considered vulnerable to a material level of liquefaction damage (Criterion 1).

7.4 Material Change in Liquefaction Vulnerability (Criterion 2)

In order to determine whether or not a property has had a material change in vulnerability to liquefaction damage as a result of ground surface subsidence caused by the CES (Criterion 2 in Section 2.4) it was necessary to determine the magnitude of change between the pre-CES and post-CES liquefaction vulnerability of a property which represents the difference between land which has not had a material change in vulnerability to liquefaction damage and land which has materially changed in vulnerability to liquefaction damage.

Land can be considered to have materially increased liquefaction vulnerability where:

- The observed and measured changes are such that, having regard to uncertainty associated with the liquefaction analysis, it is more likely than not that a change in vulnerability has occurred; and
- The measured change is such that, from an engineering perspective, the use and amenities of the land as a building platform and for other related purposes, can be said to have adversely changed.

In order to use the LSN parameter as an indicator to assess whether or not a property has had a material change in liquefaction vulnerability as a result of ground surface subsidence caused by the CES (Criterion 2 in Section 2.3) it was necessary to establish a change in LSN from the pre-CES to the post-CES ground surface elevation (i.e. ΔLSN) value that is representative of this material change in liquefaction vulnerability.

For the purposes of the ILV Assessment Methodology a ΔLSN value of 5 is considered to be generally representative of this material change in liquefaction vulnerability.

The selection of a ΔLSN value of 5 as an indicator of this transition takes into account both an assessment of the materiality of change in vulnerability, and that the LSN parameter has a number of inputs which have measurement uncertainty and analysis uncertainty. These input and analysis uncertainties result in an uncertainty in the calculation of LSN at any given CPT location. The ΔLSN value used as an indicator for material change in liquefaction vulnerability needs to reflect this level of uncertainty.

It is important to note that the ΔLSN value of 5 is one of several tools, rather than a strict threshold value, in the determination of whether or not a property has had a material change in liquefaction vulnerability (Criterion 2). The application of the ΔLSN value in conjunction with these other tools in the determination of Criterion 2 is described in Section 8.

This sub-section describes:

- The uncertainty of the measurement of the input parameters and liquefaction triggering analyses; and
- The materiality of different levels of change in liquefaction vulnerability.

7.4.1 Uncertainty in Measurement of Input Parameters

The level of uncertainty associated with the liquefaction analysis is dependent on the particular parameter being considered. In some cases, independent observations can be used in the manual process to reduce the level of uncertainty.
The uncertainty of LSN is dependent on the measurement uncertainty of the input parameters and liquefaction triggering analyses which are part of the LSN calculations. A ΔLSN of 5 is considered to be the minimum level of estimated change which provides confidence, given those uncertainties, that on the balance of probabilities a change has occurred.

The significant variables that contribute to the uncertainty of LSN are listed below:

1. Earthquake ground motions (MW and PGA);
2. Liquefaction triggering to calculate LSN;
3. Post-liquefaction volumetric strain (εv) to calculate LSN;
4. Depth to groundwater estimates;
5. CPT measurement accuracy; and
6. Spatial variation and interpolation of LSN values at CPT locations.

Each of these variables and their associated uncertainty are discussed in turn below.

**Earthquake ground motions**

The estimation of the regional 100 year return period ground motions in the Class D sub-soils in Christchurch have inherent uncertainty. This is the most significant source of uncertainty in the calculation of LSN. However these regional values are specified in the MBIE guidelines and are assumed as a prescribed value as detailed in Section 5.3. Therefore, the level of uncertainty associated with the regional 100 year return period ground motions has not been evaluated.

However, throughout the Class D subsoils in Christchurch, the site specific ground response varies. Therefore, the actual level of 100 year return period ground shaking at a specific site is likely to vary spatially from the MBIE specified 100 year return period value.

Figure 7.5a shows the corresponding distribution in the difference of the LSN values estimated at each CPT location. This figure demonstrates that 90% of the LSN differences at all the CPT locations, for the uncertainty in a site specific PGA of ±0.02g, are within ±2 LSN points.

**Liquefaction triggering to calculate LSN**

The Boulanger and Idriss (2014) liquefaction triggering methodology requires selection from a range of input parameters. These parameters include Ic cutoff, FC estimation, PL and soil density. As discussed in Sections 3.3 and 3.4, default upper bound parameters (consistent with engineering design practice) have typically been assumed.

Detailed studies (summarised in Appendix A) show that these parameters vary from site to site and also between soil layers giving rise to reasonably large uncertainty. However, during the detailed manual assessment process (described in Sections 8, 9 and 10), if the predicted liquefaction triggering assessment and associated LSN values were inconsistent with the observed land performance, then these variations were considered and factored in accordingly. The manual assessment process therefore reduces the amount of uncertainty associated with liquefaction triggering.

Lees et al. (2015) showed that the Ic cutoff parameter in Christchurch soils can vary within a soil layer from 2.4 to 2.6. Lees et al. (2015) also showed that the Cc parameter for estimating FC from Ic typically varies between 0 and 0.3 for the Christchurch soils. While the manual assessment process associated with the ILV Assessment Methodology reduces the uncertainty associated with liquefaction triggering it is not entirely eliminated.

Figure 7.5 (c, d & e) show the distribution in the difference of the estimated LSN values at each CPT location when allowing for Ic cutoff variability (2.4 to 2.6), Cc (0 to 0.1) and PL (15% to 50%). Figure 7.5c shows that approximately 60% of the LSN differences at all the CPT locations for the Ic cutoff
range of 2.4 to 2.6 are less than 2 LSN points and 85% of the LSN differences are less than 5 LSN points. Similarly, Figure 7.5d shows that approximately 85% of the LSN differences for the $C_t$ range of 0 to 0.1 are less than 5 LSN points. Figure 7.5e shows a much larger distribution of the LSN differences at all the CPT locations for the $P_l$ range of 15% to 50% with approximately 90% of these differences being less than 6 LSN points.

In addition, the following are also sources of uncertainty associated with liquefaction triggering:

- The liquefaction triggering assessments assume that the groundwater pressures below the depth of the groundwater surface increase hydrostatically and that the soil is fully saturated below this level. However, there are areas in Christchurch where the groundwater comprises a system of perched layers within the complex soil structure and hence the water pressures do not increase hydrostatically which affects the liquefaction triggering calculations; and
- In some areas, the soil below the groundwater surface has been found to be partially saturated which also affects liquefaction triggering.

Post–liquefaction Volumetric Strain ($\varepsilon_v$) to calculate LSN

The LSN parameter uses the Zhang et al. (2002) post-liquefaction volumetric densification strain ($\varepsilon_v$) empirical equations as discussed in Appendix A. These empirical equations have inherent uncertainty and transitioning between layers of soils of different density can further compound this uncertainty.

Depth to groundwater

The depth to groundwater estimates have the following uncertainties:

- 30 to 35% of the area of the pre- and post-CES DEM used to represent the ground surface has a deviation greater than 0.1m and 45 to 65% of the area of the DEM has a deviation greater than ±0.05m from the ground surface. Refer to Appendix G for further discussion about accuracy of the LiDAR data;
- The level of the groundwater surface is based on an interpolation of groundwater levels in wells at large spacings. However the groundwater level throughout Christchurch is relatively flat so it is possible to develop groundwater surfaces with wide spaced well data with reasonable confidence.
- The groundwater surface in urban Christchurch is estimated to have an uncertainty in the order of 0.1 to 0.2m. Therefore, the depth to groundwater (estimated by the DEM elevation minus the groundwater surface elevation) has a measurement error in the order of 0.2m to 0.3m. However, with the manual review process discussed in Sections 8, 9 & 10 it is reasonable to reduce this uncertainty estimate to ±0.2m.

Figure 7.5b shows the distribution in the difference of the estimated LSN values at each CPT location when allowing for a GWD uncertainty of ±0.2m. It shows that approximately 70% of the LSN differences at all the CPT locations for GWD uncertainty of ±0.2m are within ±2 LSN points and approximately 90% are within ±4 LSN points.

CPT measurement accuracy

CPTs are used for the calculation of LSN. Some variability in CPT test results can be expected as the CPTs have been carried out by a range of contractors with a range of equipment in Christchurch over a number of years. In identical soils two CPTs are expected to have slightly varying results due to measurement error.

CPTs which had a valid calibration certificate were done in accordance with American Society for Testing and Materials (ASTM) D5778-12 (ASTM, 2012). This standard requires a baseline zero calibration to be completed at the beginning and end of every CPT sounding. ASTM D5778-12 allows
for a 2% tolerance in load cell drift at the beginning and end of every CPT test which translates to an error of approximately 2% for both \( q_c \) and \( f_s \). Therefore the CPT \( q_c \) and \( f_s \) are likely to vary by 2% as a result of this measurement error.

Figure 7.5f shows the distribution in the difference of the estimated LSN values at each CPT location when allowing for a 2% uncertainty in \( q_c \). It shows that approximately 80% of the LSN differences at all the CPT locations for \( q_c \) uncertainty are within ±1 LSN points. Close to 100% of the LSN differences are within ±2 LSN points.

It is noted that in some cases lower LSN values are estimated at slightly lower \( q_c \) values (as shown by the negative difference in the histogram in Figure 7.5f). This is because for some CPTs slightly lower \( q_c \) values result in a small increase in \( I_c \), resulting in some soil layers having an \( I_c \) value that increases from just below to just above the \( I_c \) cutoff value of 2.6, which are therefore assessed as not susceptible to liquefaction.

**Spatial variation and interpolation of LSN values at CPT locations**

The estimated LSN value at each CPT location is spatially variable and therefore the resolution of LSN interpolated between each CPT will also be variable. In some areas in Christchurch this spatial variation in estimated LSN is relatively small and does not contribute significantly to the uncertainty associated with LSN. However, in other areas this spatial variation is more significant and accordingly needs to be managed with the manual assessment process.

The worked example summarised in Appendix L demonstrates this spatial variability in LSN, the effect it has on interpolation in the automated ILV model and how it is managed in the ILV Assessment Methodology.
Figure 7.5: shows the distribution in the difference of the estimated LSN values for all CPTs available on the CGD at the end of July 2015 for: (a) the difference in LSN estimated for PGA = 0.3 and 0.28g, (b) the difference in LSN estimated for median GWD and Median GWD + 0.2m, (c) the difference in LSN estimated for Ic cutoff = 2.4 and Ic cutoff = 2.6, (d) the difference in LSN estimated for CFC = 0.0 and CFC = 0.1, (e) the difference in LSN estimated for PL = 15% and PL = 50% and (f) the difference in LSN estimated for qc and 0.98qc.

Summary of Uncertainties

It is important to note that not all of the uncertainties which are listed above translate into LSN uncertainty at a particular site. For example Leeves et al. (2015) showed that the LSN sensitivity to Ic cutoff is greater in western Christchurch whereas sensitivity to CFC is greater in eastern Christchurch. Similarly, the groundwater surface is reasonably flat in eastern Christchurch and as a result is associated with less uncertainty. Conversely, in western Christchurch there is more uncertainty associated with the groundwater surface due to its upwards gradient towards the west. This variability is applicable to most of the uncertainties discussed above.

Therefore, it is not appropriate to attempt to combine all of these uncertainties together. However, it is reasonable to assume that a given site will be affected by some of them.

In the ILV Assessment Methodology, manual assessment has been included as an integral component to address the uncertainties of the inputs into LSN. It includes a review of the dataset of the CES including land damage observations, geological and topographic assessments, detailed specific analysis of the geotechnical information, review of laboratory test data, and further sensitivity analyses of liquefaction vulnerability assessments.
However, while manual review can be used to mitigate some of this uncertainty it cannot account for all of the error. Considering all of the above, based on engineering judgement, a $\Delta LSN$ of 5 is considered the minimum practical value for the assessment of ILV to enable confidence that, on the balance of probabilities, a change has occurred.

### 7.4.2 Materiality

As a result of the ground surface subsidence caused by the CES, both the land and the residential buildings on properties which have ILV now have a greater likelihood of more severe liquefaction related damage. That is, a greater likelihood in future M6 0.3g levels of earthquake shaking of:

- Moderate-to-severe liquefaction related land damage; and
- Building Damage Ratio (BDR) greater than 0.5.

A 5 to 10% increase in the likelihood of moderate-to-severe land damage or high BDR, is the minimum change that would be taken into account, as a matter of engineering judgement, for re-evaluating design assumptions and land use decisions.

Figure 7.6 and Figure 7.7 show how the likelihood of the liquefaction related damage increases with LSN in $\Delta LSN$ increments of 5 for the land damage and BDR correlations that were shown in Figures 7.1 and 7.2 respectively.

![Figure 7.6: 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 data from the TC3 and residential Red Zone properties.](image)
Figure 7.7: Frequency bar chart showing the likelihood of BDR values of less than 0.2, 0.2 to 0.5 and greater than 0.5 for different LSN bands based on data from the TC3 and residential Red Zone properties.

Figure 7.6 shows that a ΔLSN value of 5 results in approximately a 10 percentage point increase in the likelihood of moderate-to-severe liquefaction land damage in the range of LSN values from 15 to 40 which is the typical range of estimated post-CES LSN values for most of the ILV properties. Similarly Figure 7.7 shows that a ΔLSN value of 5 results in approximately an 8 percentage point increase in the likelihood of BDR exceeding 0.5 in the range of LSN values from 15 to 40.

Based on engineering judgement a 5 to 10% increase of vulnerability is considered to be a material value for re-evaluating design assumptions and decisions, and hence a corresponding ΔLSN value of 5 is a good indicator of a material change in Liquefaction Vulnerability.

7.4.3 Summary of Uncertainty and Materiality

Having regard to uncertainties in the assessment methodology, a ΔLSN of 5 is considered the minimum practical value for the assessment of ILV to enable confidence that, on the balance of probabilities, a change has occurred.

There are also good engineering reasons to justify a ΔLSN indicator of 5 points as the minimum level of estimated change in liquefaction vulnerability that could be regarded as material. A ΔLSN indicator of 5 points indicates a 5 to 10% increase in the likelihood of moderate-to-severe land damage or high BDR. This is the minimum change that would be taken into account, as a matter of engineering judgement, for re-evaluating design assumptions and land use decisions.

Based on this engineering advice, EQC has asked T+T to adopt a ΔLSN of 5 as an indicator of material change in liquefaction vulnerability. Whether or not this is significant to a property owner will be assessed by EQC’s valuers in Criterion 3 (refer to Section 2.4).
It is important to reiterate that, the ΔLSN value of 5 is one of several tools, rather than a strict threshold value, in the determination of whether or not a property has had a material change in liquefaction vulnerability (*Criterion 2*).

### 7.5 Median and Mean LSN

Finally, in order to use the LSN parameter for assessment of liquefaction vulnerability, it is necessary to determine what account to take of seasonal variation in vulnerability, and therefore LSN.

The groundwater level in the Christchurch area varies naturally from season to season, and from year to year. For the parts of the Christchurch area where ground surface subsidence has occurred (i.e. areas that may potentially meet the *engineering criteria*), this range of variation in the depth to the groundwater is typically 0.5m above and below the median groundwater surface.

Because the groundwater levels are continually fluctuating, the liquefaction vulnerability of the land (as represented by the LSN parameter) also fluctuates above and below the median value. However, because the LSN parameter has a depth-weighting factor, the LSN value increases more due to a rise in the groundwater level than it decreases due to a lowering in the groundwater level. This means that the variability in the LSN value over time is not equally distributed about the median. Furthermore, the presence of layers of non-liquefying soils within the range of groundwater fluctuation will affect the distribution of LSN over time.

A significant amount of work was undertaken to investigate the difference between the median and mean liquefaction vulnerability as a result of fluctuating groundwater levels and the influence this would have on the assessment of liquefaction vulnerability in the Christchurch area. Further detail about this work is provided in Appendix H.

The key findings from this work were:

- When considering the mean and median post-CES LSN values and their respective ΔLSN values, from a technical perspective, neither is more correct than the other. In general it could be considered that:
  - Because engineering design is likely to be based on the median groundwater level, the median post-CES LSN is more representative of the level of engineering effort (e.g. ground improvement or enhanced foundations) that would be specified in practice;
  - Because the liquefaction vulnerability has a non-uniform variation with time, the mean post-CES LSN is more representative of the average exposure to the liquefaction hazard over time; and
  - There is not a significant difference between the spatial distribution of the median post-CES LSN and the mean post-CES LSN at M6 0.3g levels of earthquake shaking. Similarly, there is also not a significant difference between the spatial distributions of the ΔLSN values (as a result of ground surface subsidence caused by the CES) for the median and mean estimated LSN cases.

Based on the above information, EQC asked T+T to apply the *engineering criteria* using the median and/or the mean liquefaction vulnerability approaches. This approach is more inclusive than adopting either the median or the mean LSN parameter exclusively, and accounts for both measures having strengths in predicting different vulnerability impacts (engineering levels and average hazard exposure over time, respectively).
8 Automated ILV Modelling and Manual Assessment Process – Common Features

8.1 Purpose and Outline
The objective of this section of the report is to describe key details of the ILV assessment process which are common to both the Stage 1 and Stage 2 assessments outlined in the process in Figure 4.2. The Stage 1 and Stage 2 assessment processes are described in Sections 9 and 10 respectively.

This section describes:

- The development and purpose of the automated ILV model, including a discussion of some of its inherent limitations;
- The manual ILV assessment process which is consistent across both the Stage 1 and Stage 2 methodologies; and
- The differences between the automated ILV model and the manual ILV assessment results.

8.2 Automated ILV Model
In order to assist with the manual ILV assessment process for the 140,000 urban residential properties in Christchurch (discussed in Sections 8.3, 9 and 10), an automated ILV model was developed using the CPT-based LSN parameter. This model was used to indicate on a regional basis the areas where the residential properties are likely to qualify for ILV. It was also used as a tool to help the assessing engineers with spatial interpolation in the manual assessment process (refer to Section 8.3.2.5). It was particularly useful for determining ΔLSN values for properties which had subsided more than the adjacent roads (ΔLSN values can be extracted from the automated ILV model).

8.2.1 Automated ILV Model Outline
An outline of the automated ILV assessment model is shown in Figure 8.1.

The first part of the automated ILV assessment model assesses the M6 0.3g mean and median LSN at each CPT location based on the post-CES DEM and the various groundwater surfaces (discussed in Section 5.4).

Approximately 18,000 CPT investigations have been undertaken across Christchurch. Of these 18,000 CPTs approximately 15,000 have been undertaken in a manner that provides a sufficient length of soil profile for the purposes of estimating LSN at the test location. The remaining 3,000 CPTs 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.

A process of LSN slicing has been developed to make use of the estimated vertical LSN slice increments available in the 3,000 CPTs that do not contain a sufficient length of soil profile. A number of constraints are applied to the slicing process to ensure only relevant vertical LSN slice increments from nearby CPT investigations are used to fill in incomplete portions of the CPT profiles. These limitations are discussed in Appendix I.

The purpose of applying the slicing method was that the mean and median LSN values at each CPT could be standardised to the same depth of soil profile (i.e. the top 10m) and relatively compared. Following the slice interpolation process the mean and median LSN values at each CPT location were then interpolated to produce LSN contour maps. The geospatial LSN values for both the mean and median LSN cases were then estimated for the ILV assessed land area using both the median and mean LSN contour maps.
Figure 8.1: Automated model process for assessing the likely ILV status.
8.2.2 Automated ILV Model Results

The maximum estimated representative post-CES LSN value for each property (i.e. the maximum estimated LSN value from the median or mean cases) is shown in Figure 8.2. Estimated LSN values less than 16 (i.e. the blue areas in Figure 8.2) generally indicate the land areas which are not expected to be vulnerable to a material level of liquefaction damage at M6 0.3g levels of earthquake shaking based on the post-CES ground surface levels.

Estimated LSN values greater than 16 (i.e. the green, orange and red areas in Figure 8.2) generally indicate the land areas which are expected to be vulnerable to a material level of liquefaction damage at the post-CES ground surface levels. These areas indicate the residential properties throughout the region that would be likely to satisfy Criterion 1 in the ILV eligibility assessment process (refer to Section 2.4). The white areas on the map indicate either non-residential properties or urban residential properties where there was an insufficient density of CPT information to sensibly calculate LSN values.

While the automated assessment of the LSN value on each property is based on a 25m x 25m DEM (to determine the depth to groundwater), the assessment of the ΔLSN for each property is based on a 5m x 5m difference DEM used to estimate the ground surface subsidence as a result of the CES. This resolution of the difference DEM was used to appropriately estimate the ΔLSN as a result of the more localised ground surface subsidence of each property.

Initially a 5m x 5m grid post-CES DEM was used for the assessment of the LSN value for each property for the automated ILV model. However, this resolution was too high and resulted in significant noise in the model. This is because of DEM interpolation error resulting in areas where the LiDAR survey points could not capture the ground surface due to structures and vegetation
(refer to Appendix G). This commonly occurs in areas where the properties are built up higher than the roads. In flat areas the depth to groundwater is a lot more consistent.

In areas of topographic variability the estimation of LSN values would result in a large amount of variability between the CPTs pushed in the roadways (which typically had higher estimated LSN values due to the shallower depth to the groundwater surface) and the CPTs undertaken on residential properties (which typically had lower estimated LSN values due to the deeper groundwater surface because the property was elevated).

The resulting LSN maps were also inconsistent with the land damage observations from the main CES earthquakes and they did not represent the same patterns of observed land damage performance. Therefore, for the automated ILV model, a 25m x 25m post-CES DEM was used to spatially smooth out the variability in the depth to groundwater. This resulted in significantly smoother LSN maps which were more consistent with the observed land damage performance.

The next part in the automated ILV assessment model repeats the process outlined above with groundwater offsets of -0.2, -0.1, 0, 0.1, 0.2, 0.3, 0.4, 0.5, 0.6, 0.8 and 1.0m from the various groundwater surfaces (where positive offset values represent a greater depth to groundwater). This is to determine the difference in liquefaction vulnerability (ΔLSN) between the post-CES and the pre-CES estimated LSN values.

Based on the geospatial median ground surface subsidence from the LiDAR derived difference 5m x 5m DEMs, for the approximate land area covered under the EQC Act, the corresponding median and mean estimated ΔLSN geospatial median ΔLSN value for the median and mean cases was determined for each property. The maximum representative estimated ΔLSN value for each property (i.e. the maximum from the estimated median or mean cases) is shown in Figure 8.3.

![Figure 8.3: Estimated maximum ΔLSN for M6 0.3g levels of earthquake shaking for the ground surface subsidence over the CES.](image)
The properties that potentially satisfy the two engineering criteria (Section 2.4) based on the automated assessment of ILV (i.e. where the post-CES LSN value is greater than 16 and ΔLSN value is greater than 5) are identified in red in Figure 8.4.

Figure 8.4: Spatial distribution of urban residential properties likely to be eligible for ILV using the ILV automated model.

The marginal ILV cases based on the automated ILV model (i.e. where the post-CES LSN value is between 14 and 16 and ΔLSN value is greater than 5 or the post-CES LSN value is greater than 16 and ΔLSN value is between 4 and 5) are identified in yellow in Figure 8.4.

At a regional scale, it is apparent that the majority of properties likely to qualify for ILV on an automated basis are clustered together. These clusters are mainly in the eastern parts of Christchurch along the banks of the Avon River and to the north of the CBD. These areas generally coincide with the areas where the observed land damage and ground surface subsidence were most severe (refer to Section 3.7.3 and 3.7.4).

It is noted that the marginal properties (i.e. the yellow areas in Figure 8.4) are generally occurring as thin transition zones between the areas likely to qualify for ILV and the areas unlikely to qualify for ILV.

8.2.3 Automated ILV Model Limitations

While the automated ILV model is a very useful tool for the manual assessment process, it has four overarching limitations. These four limitations are:

- **Interpolation across geological boundaries** - The automated ILV model involves linear interpolation between discrete CPT locations. The model therefore does not account for non-
linear geological or topographic transitions (e.g. old infill river channels or open watercourses) which may occur between the CPT locations.

Theoretically, this limitation could be overcome by increasing the density of the CPTs to a 10m x 10m grid pattern⁶, however this is not technically feasible. Practically, this limitation is overcome by the manual assessment process which uses engineering judgement to manually interpolate and extrapolate between the geotechnical data point locations. This allows the ILV assessment to include more geotechnical information and ensures that the methodology is not applied mechanically (i.e. it adheres with the objectives consistent with the Land Declaratory Judgment discussed in Section 2.6).

The Stage 2 ILV assessment of worked example 1 in Section 10.3 provides a good demonstration of how the manual assessment process has been used to overcome this limitation.

- **Variable ground conditions** – The two ground conditions that affect the estimation of liquefaction vulnerability with the LSN parameter the most are relative soil density directly affecting the measurement of q_c and FC. In parts of Christchurch, both of these ground conditions can vary significantly over short distances within the same geological unit.

  Given that the automated ILV model involves linear interpolation between discrete CPT locations, the model is unable to capture this ground variability and can result in the prediction of significantly different liquefaction vulnerability for areas with consistent ground damage across the CES.

  This limitation is overcome by the application of engineering judgement in the manual assessment process to determine on the balance of probabilities which ground conditions are the most representative of the observed land damage across the CES. This is also important to ensure that the methodology is not applied mechanically.

  The Stage 2 ILV assessment of worked example 5 in Appendix L provides a good demonstration of how the manual assessment process has been used to overcome this limitation.

- **Slicing of adjacent CPT investigations** – As discussed previously in this section, LSN slicing (as described in Appendix I) is an important part of the automated ILV model to improve the automated results at a regional level. However, as the slicing process estimates ground conditions from nearby CPT traces and in some areas ground conditions are spatially variable and the sliced values are not necessarily representative of the actual ground conditions at that CPT location.

  This limitation is overcome in the manual assessment process through the use of the CPT classification process described in Section 8.3.2.3.

- **LSN estimation limits** - The estimation of LSN and hence the automated ILV model is subject to uncertainties which cannot be appropriately captured with the use of the current investigation and liquefaction triggering assessment tools. The only way to overcome this is to use engineering judgement to manually review the results of the automated model 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.

  The nature of the uncertainties and natural complexities and its effect on the calculation of LSN is discussed in detail in Section 7.2.2.

- **Cross lease and unit titles** – The representative pre and post CES LSN values on cross lease and unit titles are the average values across the entire property (i.e. if there are three

---

⁶ The current density of geotechnical investigations is approximately 50m apart with higher density of geotechnical data points in areas where there is significant liquefaction related land damage and dwellings require rebuilding, and lower density in areas where there has been less liquefaction related land damage.
dwellings on the cross lease, the LSN values are a spatial average over the three dwellings). However, in a very small number of cases, the available information has indicated that different ILV qualifications are appropriate for properties under the same cross lease or unit title. In these cases engineering judgement has been used to overcome this limitation and the automated model has been overruled with the appropriate ILV qualification.

### 8.3 Manual ILV Assessment Process

The process for determining ILV qualification was divided into the seven tasks presented in Figure 8.5. This process was the same for both the Stage 1 and Stage 2 assessments. The key difference between the two stages was the level of detail applied, with the more complex Stage 2 assessments requiring more detailed analysis.

<table>
<thead>
<tr>
<th>Task</th>
<th>Detailed Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Regional data analysis. Review regional maps for local visual correlations and trends between topography, geomorphology, observed land performance relative to the estimated levels of earthquake shaking, ground surface subsidence maps based on LiDAR surveys, and raw and interpretive geotechnical data (including CPT and laboratory test results).</td>
</tr>
<tr>
<td>2</td>
<td>Local data analysis. Review local data packs and interpretive geotechnical data. Group properties according to local trends (e.g., group according to common land damage, topography, ground surface subsidence and soil profiles inferred from raw CPT data and borehole logs).</td>
</tr>
<tr>
<td>3</td>
<td>Specific data analysis. Using CPT-based liquefaction consequence analyses and the analysis toolpack, undertake site specific data analysis based on the grouped borehole and CPT data and laboratory test data as defined in Task 2.</td>
</tr>
<tr>
<td>4</td>
<td>Determine the ILV status. Determine the ILV status for each property in accordance with the ILV land damage assessment principles and assumptions as either yes or no using engineering judgement based on the interpretation of the information and analyses undertaken in Tasks 1 to 3. Write up the draft ILV qualification assessment report.</td>
</tr>
<tr>
<td>5</td>
<td>Technical internal peer review. Senior engineers review the draft qualification assessment for each pack and provide feedback at the appropriate task level, and also provide review to ensure consistency with decisions in other packs.</td>
</tr>
<tr>
<td>6</td>
<td>Data entry and report write up. Data entry of the ILV assessment outcome into a database and finalise the ILV qualification assessment report.</td>
</tr>
<tr>
<td>7</td>
<td>Final internal peer review and project director approval. Senior engineers and the project director undertake a final review of the decisions from each pack relative to the other packs to ensure that the decisions are both consistent and explainable.</td>
</tr>
</tbody>
</table>

**Figure 8.5: Overview of the ILV manual assessment process.**

The qualification process begins with consideration of regional level data followed by consideration of local data packs comprising a number of neighbouring properties (typically 20 for areas of reasonable complexity). The data packs are described in Sections 8.3.2.1, 9.2.2 and 10.2.2.

As appropriate, specific analysis was undertaken for each group of properties prior to the determination of the ILV status. Each qualification status for every property was then technically...
reviewed by senior engineers to ensure an appropriate and consistent outcome was reached for all 140,000 urban residential properties in Christchurch. The results were then entered into a database before the final review by the senior technical review team and the project director.

Each of the tasks in Figure 8.5 is described in turn below:

- Section 8.3.1 discusses the regional data analysis (Task 1 in Figure 8.5);
- Section 8.3.2 discusses the local and specific data analysis (Tasks 2 and 3 in Figure 8.5);
- Section 8.3.3 discusses the determination of the ILV status (Task 4 in Figure 8.5); and
- Section 8.3.4 discusses the review and quality assurance and control processes that have been applied (Tasks 5, 6 and 7 in Figure 8.5).

8.3.1 Regional Data Analysis (Task 1 in Figure 8.5)

In order to understand regional visual patterns and potential sources of error relevant to the assessment of liquefaction vulnerability and change in vulnerability for a cluster of properties in each information pack the information sources listed below were reviewed at a regional scale.

For each information source, the engineer undertaking the assessment would note relevant information that was not always evident in the zoomed in views provided in the ILV assessment packs.

The information sources reviewed in the regional data analysis included:

- **Land damage observations and estimated PGA for the main CES earthquakes** – These maps (shown in Figure 3.6) were used to investigate the correlation between the event specific land damage observations relative to the estimated event specific groundwater levels and estimated PGA for each event. Typical questions that would be considered when reviewing these maps were:
  - How do the land damage observations relate to the estimated PGA values experienced during the earthquake sequence within the area of the data pack?
  - Given the distribution of the seismic monitoring stations, what level of certainty is associated with the estimated PGA? That is, are the PGA contours rapidly changing or gradually changing in the area of the data pack? In areas of higher uncertainty, does this uncertainty help to explain the observed land damage with reference to the estimated LSN values at the CPT locations within the area being assessed in the data pack?
  - Does the area regionally have sufficient ground surface subsidence such that the liquefaction vulnerability may have materially increased as a result of the CES? Conversely, does the area generally have less than 0.1m of ground surface subsidence so that the liquefaction vulnerability of the area is unlikely to have materially increased as a result of the CES?

- **Observed ground cracking and horizontal ground surface movement** – These maps were used to understand how the magnitude and direction of horizontal land movements correlate with observed land damage and topography. Typical questions that would be considered when reviewing these maps were:
  - Was the observed ground surface subsidence within the pack area influenced by lateral spread?
  - Was the observed land damage within the pack area predominantly caused by lateral spreading?
  - Would there have been land damage in the main CES events if lateral spreading had not occurred, and if so, does this help explain the calculation of low LSN values even though
there has been observed land damage (refer to Section 8.4 for further discussion on understanding the mechanisms causing the observed land damage)?

- **Ground surface elevation** - These maps were used to look for topographic transitions that could indicate changes in the underlying geology and geomorphology over the area being assessed. Typical questions that would be considered when reviewing these maps were:
  - Does the data pack cover an area which transitions over an old terrace feature?
  - Is the data pack within a low-lying part of Christchurch which could be affected by elevated groundwater levels?
  - Are the CPTs located at an elevation that is representative of the elevation of property?

- **Geological and soil maps** – These maps help identify if there are different geological units mapped in the surrounding area that are susceptible to liquefaction. Typical questions that would be considered when reviewing these maps were:
  - What is the liquefaction susceptibility of the geological units which are mapped in the area covered by the pack?
  - Does the pack cover an area which transitions from one mapped geological unit to another?

- **Groundwater surface elevation maps** – These maps were used to understand the regional variation in the groundwater surface and the effect this may have on liquefaction vulnerability. Typical questions that would be considered when reviewing these maps were:
  - Is the groundwater surface shallow enough for the area to be potentially materially vulnerable to liquefaction damage? Conversely, is the groundwater surface too deep for the area to be materially vulnerable to liquefaction damage?
  - Is the groundwater surface very shallow such that the LSN parameter is potentially hypersensitive to very small changes in the depth of the groundwater surface?

- **Total change in elevation and liquefaction related ground surface subsidence** – These maps (shown in Figure 3.8) were used to look for patterns in the change in ground surface elevations that may not be obvious when zoomed in. Typical questions that would be considered when reviewing these maps were:
  - Are there areas of higher localised subsidence within the area covered by the pack which may indicate the location of infilled stream channels?
  - Are there error bands in the LiDAR which could influence the estimated ΔLSN values for each property within the data pack (as discussed in Appendix G)?

Note that for Stage 1 the regional maps were accessed from the CGD electronically. This enabled the assessing engineers to zoom in and out of the maps and therefore they could observe both regional and local trends. For Stage 2 the maps were available both electronically on the CGD and in large (A0) hardcopy format.

### 8.3.2 Local and Specific Data Analyses (Tasks 2 and 3 in Figure 8.5)

As part of the ILV assessment process for both Stage 1 and Stage 2 assessments:

- The CPT and borehole data within a pack was reviewed;
- Liquefaction triggering and liquefaction vulnerability sensitivity assessments were undertaken for each CPT;
- Each CPT and borehole and the associated liquefaction triggering and vulnerability assessments were manually assessed and classified for ILV.

These classifications were then reviewed and correlated with the local maps presented in the ILV assessment packs with reference to observations made during the regional assessment. Whenever
there was doubt about the quality of the recorded land damage the observations were verified with reference to the aerial imagery that was collected following each of the four main earthquake events.

This process was followed to check that the classification of the CPT data points reconciled with the information presented in the local maps. When the classification of the CPT data points reconciled with the information in the local maps and the regional assessment observations, then the properties in the pack would have an ILV decision made in accordance with the engineering criteria listed in Section 2.4.

If the classification of the CPT data points did not reconcile with the information in the local maps and the regional assessment observations, then the CPTs liquefaction triggering and vulnerability analyses would be further examined to understand the reasons for the inconsistencies between the observed and predicted land performance before undertaking the ILV decisions.

Figure 8.6 is a flowchart which summarises the steps involved in this process. Further detail about each step in Figure 8.6 is provided as follows:

- **Step 1** – The collation of CPT and borehole data information is discussed in Section 8.3.2.1;
- **Step 2** – The liquefaction triggering and vulnerability sensitivity analyses undertaken for each CPT are discussed in Section 8.3.2.2;
- **Step 3** – The classification of CPTs and borehole logs for ILV is discussed in Section 8.3.2.3;
- **Steps 4 to 7** – The spatial collation of the CPTs and borehole classification results and correlation with the other mapped data are discussed in Section 8.3.2.4; and
- **Step 8** – The manual interpolation of ILV qualification decisions is discussed in Section 8.3.2.5.

As part of the local and specific data analyses for each pack prior to undertaking steps 1 - 8, pre-CES aerial photography was reviewed to identify areas where the pre-CES LiDAR elevations are not representative of the actual pre-CES elevation of the property at the time of the September 2010 earthquake event. This was generally in areas of residential development where earthworks had occurred between 2003 (when the pre-CES LiDAR was flown) and September 2010.

These anthropogenic changes to the land mean that the 2003 based LiDAR data are not necessarily an appropriate representation of the pre-CES ground surface. This may result in negative subsidence in the difference DEMs (i.e. apparent uplift). To remove this error, the post-September LiDAR was used as a proxy to represent the pre-CES ground surface elevation. This removed the apparent uplift and captured the subsidence from the other main CES events (i.e. February 2011, June 2011 and December 2011).

An allowance based on engineering judgement was made during the ILV qualification to account for the likely ground surface subsidence that would have occurred in such areas during the September 2010 earthquake event. This allowance was based on a review of the subsidence in the surrounding area.

The majority of properties without an appropriate pre-CES DEM were not assessed as vulnerable (i.e. did not satisfy Criterion 1). Therefore, the limitations of not knowing the estimated amount of ground surface subsidence as a result of the CES for the majority of these properties was not important for ILV assessment purposes.
Figure 8.6: Local and specific ILV manual assessment process (refer to tasks 2, 3 and 4 in Figure 8.5).
Also while undertaking the local and specific analysis tasks, approximately 2,000 properties were identified as requiring additional geotechnical investigation data in order to proceed with the analysis and complete the ILV assessment. The decisions process for the collection of additional data is outlined in Figure 4.2. The majority of these properties were in areas where land is categorised as TC2 and there was observed land damage as a result of the CES events. As this additional geotechnical information was obtained it was analysed and incorporated into ILV assessment packs.

8.3.2.1 CPT and Borehole Data Collation (Step 1 in Figure 8.6)

Prior to being able to undertake the manual assessment of ILV the publicly available CPTs and borehole data needed to be collated. This geotechnical information was compiled by teams of science and engineering graduates into ILV assessment packs. Each ILV assessment pack includes all the publicly available maps listed in Section 4.4 and the CPTs and borehole logs on the CGD at the time the pack was created.

As a result of the update to the liquefaction triggering assessment methodology (Boulanger & Idriss, 2014), the Idriss and Boulanger (2014) method was applied and the ILV assessment process was started again in July 2014 (refer to the evolution of the ILV Assessment Methodology in Appendix C). The final Stage 1 packs were made from July 2014 onwards and the Stage 2 packs were made from January 2015 onwards. Therefore, most of the available geotechnical information has been incorporated into the ILV assessments.

The specific detail about the creation of ILV assessment packs for Stage 1 and Stage 2 are covered in Sections 9.2.2 and 10.2.2 respectively.

8.3.2.2 Liquefaction Vulnerability Assessment of CPT Data (Step 2 in Figure 8.6)

Liquefaction triggering assessment analyses were undertaken at each CPT location. The results were examined to develop an understanding of the ground profile at each CPT location, the soil layers which are likely to liquefy at M6 0.3g levels of earthquake shaking and the whether or not the land (based on the interpreted soil profile) is likely to be vulnerable to liquefaction related damage.

Liquefaction vulnerability parameters were estimated for the Cumulative Thickness of Liquefaction (CTL), $S_{V10}$ and LSN using the median post-CES depth to groundwater. Sensitivity analyses of these parameters for PGA and depth to groundwater were also undertaken for the CTL, $S_{V10}$ and LSN liquefaction vulnerability parameters to further inform whether or not the land at the CPT location is likely to be vulnerable to liquefaction. The use of these sensitivity analyses in the ILV Assessment Methodology is discussed in detail in the latter parts of this section.

A standard CPT liquefaction triggering and vulnerability analysis comprising two output pages was created. It summarised the most pertinent information and was included for each CPT in the information packs. Copies of these summary pages are given in the Stage 1 and Stage 2 worked examples included in the Worked Example Material.

Screenshots showing examples of these summary CPT-based liquefaction triggering and vulnerability analyses are provided below to highlight pertinent features. The bottom of the first summary page (refer to Figure 8.7) and a screenshot of the top of the second summary page (refer to Figure 8.8) provide the general information.

- The general information section includes:
  - 1 – CGD ID number;
  - 2 – The date on which the CPT was undertaken;
  - 3 – The drilling company;
  - 4 – The make and model of the rig used to push the CPT;
5 – The supervising company (i.e. the engineering firm that supervised the investigation);
6 – The total depth of the CPT;
7 – The depth to which the hole was predrilled (if any);
8 – The assumed predrilled soil parameters (if required);
9a and 9b – Pre-CES and post-CES depth to the median groundwater surface respectively;
10 – The assumed unit weight of the soil;
11 – The value of site specific Fines Content Correlation Coefficient (C_FC) (discussed in Section 3.4);
12 – The I_c cutoff limit applied in the liquefaction triggering assessment;
13a and 13b – LSN values estimated for both pre-CES and post-CES ground surface levels respectively. These are estimated based on the median groundwater condition assuming a P_L value of 15% (P_L is discussed in Section 3.4 and Appendix A);
14 – The ΔLSN values as a result of the subsidence caused by the CES based on the change to the depth to the median groundwater surface.

Figure 8.7: Example of the first summary page of the CPT-based liquefaction triggering and vulnerability sensitivity analysis for M6 0.3g levels of earthquake shaking at the median depth to groundwater levels.
Figure 8.8: Example of information section from the top of the second page of the CPT-based liquefaction vulnerability sensitivity assessment analysis.

Plots A to E in Figure 8.7 present the results of the liquefaction analysis with depth over the top 10m of the soil profile. On all of these plots the blue line indicates the groundwater level assumed in the analysis. Each plot is described in turn as follows:

- **Plot A** – This plot shows the raw data collected from the CPT. The red line shows the $q_c$ in MPa, and the yellow line shows the sleeve friction ($f_s$) as a ratio of the tip resistance ($R_t$). The CPT test data are typically collected at 10 to 20mm intervals, giving an effectively continuous measurement of the soil profile with depth.

- **Plot B** – This plot shows the $I_c$ which is estimated based on the normalised $q_c$ and $f_s$ based on the iterated results presented by Robertson & Wride (1998). The $I_c$ parameter can be used to assess the behaviour of the soil, and whether it responds as a fine grained or coarse grained material. The higher the $I_c$ parameter, the more fine grained the material behaviour is. The dashed black vertical lines show various ranges of $I_c$ value that are typically associated with different soil types. The $I_c$ value is plotted with a different colour for each typical soil type, as explained in the legend below the plot. For the typical liquefaction analyses in this report, where the estimated $I_c$ exceeds 2.6 the soil is assessed to be too fine grained to liquefy (refer to Section 3.3). In some cases a sensitivity analysis is undertaken to assess the effects of changing this $I_c$ cut off value.

- **Plot C** – The Cyclic Stress Ratio (CSR) is a normalised parameter that represents the cyclic stress predicted to occur in the soil for a given earthquake scenario to the vertical effective stress, as a ratio of the vertical effective stress in the soil. The CSR parameter represents the “demand” side of the liquefaction triggering analysis. The CRR is an estimate of the cyclic stress above which liquefaction triggering is predicted to occur. The CRR parameter represents the “capacity” side of the triggering analysis. The CRR is estimated using the Boulanger and Idriss (2014) empirical liquefaction triggering from the case history database, which includes significant uncertainties and natural variability (discussed further in Appendix A). To help understand this uncertainty, three different CRR profiles are shown on the plot, representing 15%, 50% and 85% certainty of the cyclic stress ratio above which liquefaction triggering is expected from the case history database. As discussed in Section 3.4, this is referred to as the $P_L$ parameter in the Boulanger and Idriss (2014) liquefaction triggering method.

- **Plot D** – The Factor of Safety (FS) against liquefaction triggering is based on the ratio of the “capacity” to the “demand” (i.e. the CRR to the CSR). Three different FS profiles are plotted, corresponding to different levels of certainty that liquefaction triggering will occur at a specified level of earthquake shaking and depth to groundwater based on the empirical case
history database. If soils are assessed as being too fine-grained to liquefy (i.e. $I_c$ value greater than the cutoff value, which is typically taken as $I_c=2.6$), then they are plotted with $FS > 2$.

- **Plot E** - This plot shows a simplified profile of results of the liquefaction analysis, with the different colours representing different levels of probability that liquefaction triggering will occur.

Sensitivity of the $CTL$, $SV_{1D}$ and LSN liquefaction vulnerability parameters to PGA and depth to groundwater for the second page of the CPT liquefaction vulnerability assessment analyses are shown in Figure 8.9 and Figure 8.10.

An example of the sensitivity to varying PGA section of the summary CPT analysis page is shown in Figure 8.9. As annotated, the bold blue and red curves on each of the graphs represent the sensitivity to PGA based on the pre-CES and post-CES median depth to groundwater surfaces respectively. The light blue and light orange curves represent the same groundwater conditions but estimated using the 50th and 85th $P_L$ values. The horizontal dashed line is the LSN = 16 indicator value for material vulnerability (discussed in Section 7.3).

The $CTL$ and $SV_{1D}$ sensitivity to PGA curves represent the same variables as annotated on the LSN sensitivity to PGA curve.

![Figure 8.9: Example screenshot of the second page of the CPT-based liquefaction vulnerability analyses for a CPT showing the sensitivity of the estimated liquefaction vulnerability parameters to PGA for a $M_w 6.0$ earthquake.](image)

The LSN sensitivity to PGA curve can be used in the following ways:

- **Assess the LSN and $\Delta$LSN values at levels of ground shaking up to $M_6 0.3g$** – The maximum LSN value occurs at the highest considered levels of earthquake shaking (i.e. $M_6 0.3g$) and at lower levels of PGA, the LSN values decrease. However, the maximum $\Delta$LSN does not always occur at the highest considered levels of earthquake shaking (i.e. $M_6 0.3g$) and often can
occur at lower levels of PGA. In accordance with Criterion 2 (Section 2.4) for the assessment of ILV, the change in liquefaction vulnerability is considered at up to M6 0.3g levels of earthquake shaking.

The LSN sensitivity to PGA curves make this assessment practically possible based on manual assessment of the PGA sensitivity analysis. This is important in instances where the ΔLSN at 0.3g is less than 5 but at levels of earthquake shaking lower than 0.3g the ΔLSN is greater than 5. This phenomenon has been referred to as “stress change” in this report. An example of a CPT with stress change is shown in Example J8 in Appendix J and is described in further detail in Section 8.3.2.3.

- **Reconcile the observed land performance through the CES relative to the estimated earthquake shaking with the estimated liquefaction vulnerability** - By correlating the land performance observations through the CES relative to the estimated earthquake shaking for the main CES events, with the LSN curves can be used to assist in evaluating whether the LSN parameter appropriately estimates the liquefaction vulnerability for the site.

For example, with reference to Figure 8.9 if the estimated ground shaking during the September 2010 earthquake was 0.2g in the location where the CPT was undertaken, the assessing engineer would consider whether or not the pre-CES LSN value of 14 reconciles with the land damage observations. For an estimated pre-CES LSN value of 14 none-to-minor land damage would be expected.

If the recorded land damage observations in the September 2010 earthquake were none-to-minor, the assessing engineer would conclude that the LSN value of 14 is appropriately predicting the observed performance for this particular event. If a similar assessment demonstrated a good fit for each of the other CES earthquakes, the assessing engineer would conclude that the LSN tool as a whole was providing a good fit for the assessment of ILV. Therefore they would have confidence in using the predicted M6 0.3g LSN value at that CPT location as an indicator for material liquefaction vulnerability.

However, if the recorded land damage observations in the September 2010 earthquake were moderate-to-severe, the assessing engineer would conclude that the LSN value of 14 was probably not appropriately predicting the observed performance for this particular event. If a similar assessment demonstrated a poor fit for each of the other CES earthquakes, the assessing engineering would conclude that the LSN tool as a whole was probably not appropriately predicting the observed performance for that site. Unless laboratory specific data to modify the liquefaction triggering inputs (i.e. I_c-cutoff and C_{Ec}) would improve the calibration, then the LSN tool could not be used for the assessment of ILV at that location.

When the LSN tool could not be used the other liquefaction vulnerability indicators listed in Section 7.2 would be considered. Generally, when these situations occurred in the Stage 1 process, the property was assigned Stage 2 status and reassessed using the detailed processes outlined in Section 10.

These assessments at each CPT location were undertaken with consideration of the uncertainty there may be associated with the information available. This is particularly important with respect to the estimated levels of earthquake shaking.

- **Understand the uncertainty in the LSN calculation at M6 0.3g levels of earthquake shaking** - By comparing the PGA sensitivity curves for the 15th, 50th and 85th Pi parameters at M6 0.3g an assessment of the uncertainty with respect to liquefaction triggering and the LSN parameter can be made.

In the example in Figure 8.9, at M6 0.3g levels of earthquake shaking the LSN curves for the 15th and 50th Pi input parameters are indicating similar values which provides a higher level of certainty in the assessment of liquefaction vulnerability at this location at these earthquake shaking. However, if there was a significant difference between the values of LSN represented by the 15th and 50th Pi curves at M6 0.3g then there would be a lower level of certainty in the
assessment of liquefaction vulnerability at this location. This could help to explain areas where no land damage was observed during the CES when higher LSN values were estimated at a $P_L$ of 15% but significantly lower values were estimated at a $P_L$ of 50%.

The CTL and $S_{v10}$ sensitivity to PGA curves were typically used to further understand the proportion of the upper 10m of the 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.

Another example is that there would be a higher level of confidence in the $\Delta$LSN representing a real increase in liquefaction vulnerability as a result of the ground surface subsidence caused by the CES if the $\Delta$CTL value is high and the $\Delta$S$_{v10}$ value was also high. This indicates the contribution to $\Delta$LSN is coming from a significant change in crust thickness.

Conversely, if the $\Delta$LSN value is high at 0.3g but the $\Delta$CTL and $\Delta$S$_{v10}$ values are low, then this indicates that the contribution to the $\Delta$LSN parameter is occurring in a relatively thin layer of material which may be close to the ground surface and the LSN parameter may be over-predicting the change in vulnerability to liquefaction due to ground surface subsidence. This phenomenon has been referred to as “hypersensitivity to $\Delta$LSN” in this report. An example of a CPT with hypersensitivity of $\Delta$LSN is shown in Example J10 in Appendix J and is discussed further in Section 8.3.2.3.

An example of the sensitivity to varying the depth to the groundwater surface ($GWD$) is shown in Figure 8.10. As annotated, the bold blue and red lines represent the 50th percentile GWD for the post-CES and pre-CES ground surface levels respectively. The light blue and light orange lines represent the 15th and 85th percentile GWD for post-CES and pre-CES ground surface levels respectively. It is important to note that the percentile GWD lines differ from the $P_L$ curves which are discussed with reference to the PGA sensitivity curves in Figure 8.9.

It should be noted that these analyses are for a different CPT than the one presented in Figure 8.7, Figure 8.8, and Figure 8.9. This CPT has been chosen because it clearly shows the separation between the different GWD lines as a result of the ground surface subsidence caused by the CES.
The LSN sensitivity to GWD curve was used in the following ways:

- **Calculating the mean ΔLSN** - A quick assessment of the mean ΔLSN can be made by averaging the associated ΔLSN values for the difference between the pre-CES and post-CES LSN values for the 15th, 50th and 85th percentile GWD.

  The method of assessment of the mean ΔLSN is demonstrated in Example J3 in Appendix J and is discussed further in Section 8.3.2.3. The concept of median and mean LSN and ΔLSN is discussed in detail in Section 7.5 and Appendix H.

- **Identifying the predrill component** - In some locations, predrilling was undertaken at investigation locations, to safely get past underground services. This means that the CPT data are missing for the important shallow soil layers, typically down to a depth of between 0.8 to 1.5m.

  Example J6 in Appendix J shows a predrilled CPT analysis which is affecting the LSN calculation and how this is identified in the LSN sensitivity to GWD curve. The effect of predrill on the calculation of LSN and how this was managed is discussed further in Section 8.3.2.3.

- **Understanding the liquefaction vulnerability of the ground profile** - The sensitivity to GWD curves were useful to build understanding of which components of the ground profile are contributing to liquefaction vulnerability at M6 0.3g.

  Flat portions of the curves indicate either denser or siltier material with a higher resistance to liquefaction whereas steeper portions of the curves indicate material which is either sandier or looser and therefore has a lower resistance to liquefaction. Prior to considering the location of the groundwater in the CPT it was useful to use these curves to develop understanding of the predicted liquefaction resistance of the sub-surface ground profile.
Sensitivity to GWD - Building on the interpretation discussed above, by overlaying the estimated GWD onto the LSN curve the sensitivity of the estimated LSN value to uncertainty in the GWD could be quickly assessed.

If there was uncertainty in the estimated GWD but the slope of LSN sensitivity to GWD curve was relatively flat then it showed that the estimated LSN was not sensitive to uncertainty of the estimated GWD.

However, if the LSN sensitivity to GWD curve is sloping then the estimated LSN is sensitive to uncertainty in the estimated GWD and further consideration needs to be given considering whether the soil profile at that location is classified as not vulnerable or materially vulnerable to liquefaction damage. This is particularly important if the estimated median and mean LSN values are near the indicator value of 16 (refer to Section 7.3).

Similar to the PGA sensitivity curves, CTL and $S_{\text{V1D}}$ sensitivity to GWD curves represent the same variables as annotated on the LSN sensitivity to GWD curve. These additional parameters assist the assessing engineer to better understand potential reasons for low or high estimated LSN values. As discussed above, they are particularly important if the estimated LSN values are not consistent with the observed land performance throughout the CES.

8.3.2.3 Classification of CPTs and Borehole Logs for ILV (Step 3 in Figure 8.6)

The CPTs were classified using the categories provided in Table 8.1 in order to assess each CPT location for ILV. When there was insufficient density of CPT data the borehole logs were assessed using engineering judgement to provide a qualitative equivalent to the quantitative CPT classifications provided in Table 8.1.

Table 8.1: 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 $\Delta$LSN 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 $\Delta$LSN 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 $\Delta$LSN 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 $\Delta$LSN 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 $\Delta$LSN between 4 and 5; or Post-CES median and mean LSN at M6 0.3g between 14 and 16 and $\Delta$LSN greater than or equal to 5</td>
<td>J5.1 and 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 – $\Delta$LSN at M6 0.3g is less than 5; but $\Delta$LSN 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 $\Delta$LSN &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 issue</td>
<td>Significant difference between the mean and median groundwater levels noted</td>
<td>J11</td>
</tr>
</tbody>
</table>
In addition to assessing the liquefaction vulnerability of soil layers, the borehole logs were particularly useful for identifying the depth and thickness of any gravel layers that were too dense for adjacent CPT to penetrate, and therefore justify using shorter CPT for assessing liquefaction vulnerability.

Table 8.1 shows the symbology that was used to classify the CPTs and borehole logs. The CPTs were classified with reference to the summary CPT analysis pages described in Section 8.3.2.2. Where applicable, the borehole logs were qualitatively assessed using the same classification system. Annotated examples of each of the classifications used in the CPT review are provided in Appendix J.

Each of the classifications in Table 8.1 are discussed in turn as follows:

- **CPT classified as ✓** - These CPTs have median LSN and ΔLSN values of greater than or equal to 16 and 5 respectively at levels of earthquake shaking of M6 0.3g. That is, based on the CPT-based liquefaction vulnerability assessment, the point location it represents would potentially qualify for ILV.

- **CPT classified as ✗** - These CPTs have either median or mean LSN values of less than 14 at levels of earthquake shaking of M6 0.3g or median and mean LSN values of between 14 and 16 and ΔLSN less than 5. That is, based on the CPT-based liquefaction vulnerability assessment, the point location they represent would be unlikely to qualify for ILV as it is unlikely that it would satisfy Criterion 1.

- **CPT classified as mean ✓** - These CPTs have mean LSN and mean ΔLSN values of greater than or equal to 16 and 5 respectively at levels of earthquake shaking of M6 0.3g. That is, based on the CPT-based liquefaction vulnerability assessment, the point location they represent would qualify for ILV.

  The mean ΔLSN can be estimated by averaging the difference between pre-CES and post-CES for the 15th, 50th and 85th percentile GWD. This process is demonstrated in CPT example J3. The concepts of mean and median LSN are introduced is discussed in Section 7.5 and in further in detail in Appendix H.

- **CPT classified as NC (i.e. No Change)** – These CPTs have estimated median or mean LSN values of greater than or equal to 16 and estimated median or mean ΔLSN values of less than 4 at levels of earthquake shaking of M6 0.3g. That is, based on the CPT-based liquefaction vulnerability assessment, the point location they represent would be unlikely to qualify for ILV because it is unlikely that it would satisfy Criterion 2.

- **CPT classified as M (i.e. Marginal)** – These CPTs have estimated median or mean LSN values of greater than or equal to 16 and ΔLSN between 4 and 5 or estimated median or mean LSN values of between 14 and 16 and ΔLSN greater than 5. That is, based on the CPT-based liquefaction vulnerability assessment, the point location the CPT represents would not qualify for ILV however it does qualify for one of the two engineering criteria and is close to qualifying on the other.

- **CPT classified as PD (i.e. Pre-Drill)** – These CPTs are those where prior to undertaking the CPT, the ground was pre-drilled in order to safely clear any buried services. A CPT was only classified as PD if the predrilled component was affecting the zone where the change in the depth to groundwater was occurring.

  If the predrilled component has no effect on the calculation of LSN at the pre- and post-CES estimated GWD because the GWD is below the predrill depth, then the CPT is not classified as PD and one of the other applicable CPT classifications is used.

  As a default, the estimated LSN value assumed a low strength (readily liquefiable) material is present over the predrill depth. This may over-predict the consequences of liquefaction at the site, which may make the results less realistic.
Assuming readily liquefiable material over the predrilled portion of the CPT can be a useful assumption because it provides an appropriate upper bound for the assessment of ILV. That is, if the change in GWD is occurring in part or all of the predrilled portion of the CPT and the estimated LSN is less than 16, then the CPT can be classified as \( \times \). Alternatively, if the estimated LSN is greater than 16 but the \( \Delta \text{LSN} \) value is less than 5 then the CPT can be classified as NC.

As shown in Example J6 in Appendix J, to indicate that a CPT has been predrilled, the portion of the LSN sensitivity to GWD which has been predrilled is highlighted in red.

- **CPT classified as \( ? \) (i.e. questionable result)** – These CPTs are those where the CPT data are of questionable quality. This was generally identified when a single point location the estimated LSN values (and the other CTL and \( S_{\text{VD}} \) parameters) were significantly different from the surrounding estimated LSN values.

  In order to understand the reasons for the questionable result, the raw CPT data was investigated. Common reasons for questionable CPTs were data entry errors such as the use of incorrect units for \( f_s \), negative \( f_s \), or incorrect coordinates being supplied for the CPT when it was uploaded to the CGD (i.e. the location address on the PDF of the CPT trace was different from the supplied coordinates). Some of the other common problems with CPT data are discussed in De Pascale et al. (2015).

- **CPT classified as SC (i.e. Stress Change)** – For these CPTs the \( \Delta \text{LSN} \) value is less than 5 at M6 0.3g levels of earthquake shaking but the \( \Delta \text{LSN} \) at 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.

- **CPT classified as S (i.e. Short)** – This classification was used when the CPT has been terminated at depths less than 5m\(^7\). A CPT may have been terminated at depths shorter than 10m because it was used for a specific purpose (e.g. 900 CPT in the CGD are less than 3m deep and come from Orion Group who undertook a large number of investigations prior to the CES for the installation of power poles). Alternatively some CPT are short because the CPT probe encountered soils that were too dense to penetrate (e.g. dense gravel or sand layers).

  While the short CPTs would not be able to give a reliable indication of the potential liquefaction vulnerability of the site, they could still provide valuable insight into the liquefaction vulnerability of the near surface soil layers where the change in groundwater was occurring. Therefore they could potentially give a good indication of the \( \Delta \text{LSN} \) value as a result of ground surface subsidence.

  Alternatively, if the CPT was terminated because it encountered a dense gravel layer, it could be reasonably assessed for ILV based on the shorter length of CPT data available. This was only possible if the thickness and competency of the gravel layer could be confirmed with adjacent borehole data.

- **CPT classified as \( \checkmark h \) (i.e. hypersensitivity to change in GWD)** – This classification was used when the LSN value is greater than 16 and the \( \Delta \text{LSN} \) value is greater than 5 at M6 0.3g levels of earthquake shaking but the calculation of LSN was very sensitive to small changes in the depth to groundwater. That is, based on the CPT-based liquefaction vulnerability assessment, the point location they represent would qualify for ILV but the LSN calculation may be over-estimating liquefaction vulnerability and the change in liquefaction vulnerability.

---

\(^7\) If a CPT was judged to be terminating into thick dense non-liquefying gravels and refusing, it was unlikely that there would be contribution to LSN from soil layers below the dense gravels. This judgement was applied with reference to nearby borehole data to confirm the thickness of any gravel layers. If the CPT appeared to have terminated without refusal on a dense gravel layer, then consideration would be given as to whether or not the LSN value could be higher (i.e. influence **Criterion 1**) but generally extra length of CPT would not influence \( \Delta \text{LSN} \) (i.e. influence **Criterion 2**). As necessary additional CPTs were undertaken adjacent to these sites (refer to Figure 4.2).
This hypersensitivity to shallow groundwater is attributable to the depth weighted factor in the LSN calculation from approximately 0 to 0.5m below the ground surface. Therefore, this occurs at groundwater depths of less than 0.5m and is characterised by steep spikes in the LSN sensitivity to groundwater depth curve. This results in estimated ΔLSN values of greater than 5 for relatively small estimated ground surface subsidence (e.g. approximately 0.1m) which may not necessarily be representative of material change in liquefaction vulnerability.

- **CPT classified as GW issue (i.e. Groundwater issue)** – This classification was used for those CPTs where there was a significant difference between the 15th and 85th percentile groundwater lines and the 50th percentile groundwater lines. This occurred in areas where the median groundwater model differed significantly from the median surrogate groundwater model. Further detailed information about the median and surrogate median groundwater models is summarised in Section 5.5 and discussed in detail in van Ballegoooy et al. (2014a).

As appropriate these classifications could be combined to provide further information. Common examples of this include:

- **PDx** - that is CPT is predrilled but would not qualify for ILV based on the CPT-based liquefaction vulnerability assessment; or
- **MVx** - that is CPT is marginal but is very close to qualifying for ILV based on the CPT-based liquefaction vulnerability assessment. An example of MVx and Mx are provided as examples J5.1 and J5.2 respectively in Appendix J.

### 8.3.2.4 Spatial Collation of the CPT and Borehole Classification Results and Correlation with the Other Mapped Data (Steps 5, 6 and 7 of Figure 8.6)

Following the classification of the CPT summary outputs for each pack, the CPT and borehole classifications were spatially collated on to an aerial image to understand the distribution of the available data. The classified CPT and borehole data were then cross-referenced with the other mapped information such as ground surface elevation, land damage observations for each of the main CES earthquakes, ground surface subsidence and depth to groundwater surfaces.

Due to space constraints on properties CPTs and boreholes were often undertaken within the road ways. In some cases the road ways did not subside as much as the adjacent residential properties. Where this occurred it was necessary to consider whether or not manual adjustments are required to ensure the classification reflects the ground surface subsidence and hence the increase in liquefaction vulnerability which has occurred on the adjacent residential properties (refer to Section 8.2). In these cases the interpolated ΔLSN values for the properties were extracted from the automated ILV model and used to inform appropriate allowances for ΔLSN in such cases.

If the cross-referenced CPT classifications reconciled with the land performance observations (i.e. observed land damage relative to the estimated levels of earthquake shaking, estimated ground surface subsidence and elevation) and the spatial distribution made sense then the property progressed to the next step. If it did not reconcile the property was held back for further detailed review and potential modifications to the input parameters for the liquefaction triggering and vulnerability analyses if supported by laboratory test data (i.e. the Stage 2 process).

### 8.3.2.5 Manual Interpolation for ILV Qualification Decisions (Step 8 of Figure 8.6)

Engineering judgement was then applied to manually interpolate and extrapolate the CPT and borehole classifications onto the adjacent properties. This task was undertaken with reference to the LSN interpolation model which is described in Section 8.2.

The results from the automated model were not accepted where the information under consideration indicated this was appropriate. For example, the automated model could not
interpolate across geological and topographic transitions which were apparent once reconciled with the land damage observations. In such cases, manual adjustments to the interpolation results would be made.

8.3.3 Determine the ILV status (Task 4 in Figure 8.5)

Following the completion of Tasks 1 to 3, Task 4 (in Figure 8.5) was to apply engineering judgement to determine whether or not a property would qualify for ILV in accordance with the engineering criteria set out in Section 2.4, by marking the property with either a ☑ or a ☑ as shown in the worked examples summarised in Sections 9.3 and 10.3 and included in the Worked Example Material. In order to qualify for ILV the engineer undertaking the assessment would need to have determined that, on the balance of probabilities, the property satisfied both engineering criteria.

8.3.4 Internal Review Processes (Task 5, 6 and 7 in Figure 8.5)

The internal review process covers Tasks 5, 6 and 7 in Figure 8.5. Each of these tasks are addressed in turn as follows:

- **Task 5** involved the technical review of each of the ILV decisions. This review was undertaken by senior geotechnical engineers with extensive experience working with the liquefiable soils in the Christchurch area. Each decision was reviewed and feedback was provided to the engineer who undertook the ILV assessment. This step was critical to ensure consistency of outcomes and that the assessors considered all relevant information.

- **Task 6** involved entering the ILV qualification status for each property into a geospatial database and finalising the ILV assessment. The data entry ILV decision for each property was also geospatially checked with the decisions shown on the datapacks. This was important to ensure that there were no data entry errors.

- **Task 7** involved a final review by senior geotechnical engineers and the project director. Each ILV assessment for each property was geospatially reviewed with reference to the surrounding ILV decisions. ILV decisions that appeared to be outliers (i.e. inconsistent with surrounding decisions) were held back for further analysis. This step was critical to ensure that the results were both consistent and explainable. The completion of this task was sign off and authorisation by the project director on behalf of T+T.

8.3.5 Additional Quality Control Processes

Further to the internal review processes outlined above, additional quality control processes were built into the Stage 1 and Stage 2 ILV Assessments.

First, the majority of the Stage 1 packs overlapped between adjacent packs. This intentional overlap resulted in different ILV assessors making ILV assessments on the same property on neighbouring packs. This duplication of ILV assessment on some properties was an inherent quality control aspect to ensure robustness and consistency.

In almost all cases, the duplicate ILV assessments would result in the same outcome, demonstrating that the decision-making was robust. However, where they differed such cases were reviewed, and any lessons of broader application arising from those cases incorporated back into the assessment process. Any different ILV decisions on the same property were identified during the uploading phase (i.e. Task 6 in Figure 8.5).

Second, for the Stage 2 process, while only the properties which were marked as “Stage 2” required ILV assessments to be undertaken, the Stage 2 marking also extended out to surrounding properties where an ILV decision had already been made in Stage 1. For almost all of the cases reassessed by the Stage 2 process the ILV decisions were the same.
8.4 Differentiating between Land Damage due to Lateral Spreading and Free Field Liquefaction

As discussed in Section 6.6 the assessment of Criterion 1 in the ILV Assessment Methodology excludes the assessment of liquefaction vulnerability due to lateral spreading because vulnerability to lateral spreading was judged to have not increased as a result of the changes to the land that occurred during the CES.

In some areas of Christchurch the land is vulnerable to both lateral spreading and free field liquefaction related land damage at 100 year return period levels of earthquake shaking. In other areas the land is vulnerable only to lateral spreading at 100 year return period levels of earthquake shaking (but not free field liquefaction related land damage).

An example of this would be a property in close proximity to a river or a stream, which has a thick non-liquefying crust relative to the thickness of the underlying liquefying soil layers. While in the free field situation the thick non-liquefying crust would protect the property from material liquefaction damage, the close proximity to the river or stream would result in the non-liquefying crust being compromised (i.e., stretched, cracked, distorted, or displaced) as a result of lateral spreading.

Therefore, to apply the ILV Assessment Methodology, it was necessary to develop a process to differentiate between these different forms of liquefaction vulnerability. This process is applicable in areas where the CPT-based assessment results indicate the land is not vulnerable to free-field material liquefaction damage (i.e. low LSN values) but the land damage observations relative to the estimated level of earthquake shaking indicate moderate to severe land damage occurred and that the property is potentially vulnerable to liquefaction.

Figure 8.11 summarises the process used to differentiate between the two different forms of liquefaction vulnerability.

The initial step is to estimate site specific LSN values for each of the four main earthquakes. This is achieved by reading the LSN values of the LSN sensitivity to PGA curve (refer to Figure 8.9) for a representative CPT using estimated PGA values for each of the four main earthquakes that have been scaled to an equivalent M6 earthquake. The LSN parameter does not take account of the potential for lateral spreading related land damage.

These site specific LSN values are then reconciled with the land damage observations at each site to see if the LSN model fits the observations.

If the LSN values are consistent with the land damage observations then the property is assessed for ILV in the normal manner. If the LSN values are over predicting liquefaction vulnerability then lateral spreading is unlikely to be an issue and reasons for this apparent over-prediction are investigated using the Stage 2 process described in Section 10. If the LSN values are under predicting liquefaction vulnerability relative to the land damage observations, then the possibility of lateral spreading is considered as a potential reason for the land damage sustained at the property.
Figure 8.11: Process to differentiate between observed land damage from CES events, with ground motions close to 100 year return period levels of earthquake shaking, due to free field liquefaction vulnerability and lateral spreading vulnerability for properties with low calculated LSN values.
In these cases, the question that was considered for Criterion 1 was would there have been land damage in the main CES events if lateral spreading had not occurred, and if so, did this help explain the calculation of low LSN values even though there has been land damage observed?

To check for lateral spreading the assessing engineer considers the following factors:

- Do the horizontal movement vectors indicate lateral spreading has occurred during the CES?
- Is the property in close proximity to a free face such as a river bank, road cutting or old river terrace?
- Do the geotechnical investigations indicate that the site has a competent crust overlying liquefiable soil layers such that the site is unlikely to be vulnerable to free field liquefaction damage?
- If all of these factors do not apply then it is likely that the land damage is attributable to free-field liquefaction damage and other reasons for the apparent under prediction of ILV need to be considered (refer to Section 10). If the assessing engineer determines that all of these factors do apply then it is likely that the land damage observations are attributable to lateral spreading only and the property can be assessed as not qualifying for ILV on the basis of not satisfying Criterion 1.

8.5 Comparison of Automated ILV Model and Manual ILV Assessment

Figure 8.12 shows a map which compares the automated ILV model results (shown in Figure 8.4) with the manually assessed ILV decisions at the completion of the Stage 1 and Stage 2 ILV processes (shown in Figure 10.17).

Properties which qualified for ILV and the automated outcome was “yes” are shown as pale red. Properties which qualified for ILV and the automated outcome was either “no” or “marginal” are shown as dark red.

Properties which did not qualify for ILV and the automated outcome was either “no”, “unable to estimate LSN” or “marginal” are shown as pale blue. Properties which did not qualify for ILV and the automated decision was “yes” are shown as dark blue. Properties in the lighter shades of blue and red are properties where the manual assessment process confirmed the automated outcome.

The properties which are darker shades of blue or red have had automated outcome overturned by the manual process.

The purpose of Figure 8.12 is to demonstrate on a regional scale the difference between the automated ILV model and the outcome from the manual ILV assessment process.
Figure 8.1: Map and tabulated data for ILV decisions comparing the difference between the results from the automated ILV assessment model and the manual ILV assessment process

It is reasonable to group the automated “unable to estimate LSN” outcomes with the automated “no” outcomes (as pale blue) because the majority of these properties were in areas where there was no liquefaction related damage and low levels of ground surface subsidence and therefore they would not satisfy Criterion 1 and therefore would not qualify for ILV.

Inspection of Figure 8.12 shows that majority of properties in Christchurch are either pale blue or pale red indicating that the manual and automated models have produced very similar results. The data presented in the table in Figure 8.12 which shows that 78% of the manual “yes” decisions were automated “yes” and only 22% of the manual “yes” decisions were automated “no” or “marginal” (i.e. overturned from the automated result). Similarly, 97% of the manual “no” decisions were automated “no”, “marginal”, or “unable to estimate LSN” and only 3% of the manual “no” decisions were automated “yes” (i.e. overturned from the automated result).
This shows that the ILV automated model at a regional level was generally differentiating well between properties with and without ILV.

Figure 8.12 also shows that the dark red and blue properties (i.e. the overturned automated decisions) are primarily confined to transition areas between “yes” and “no”. Most importantly, it was never the case that entire suburbs were overturned. Instead, it was generally marginal cases on the fringes of large blocks of “yes” and “no” decisions. This further demonstrates that the ILV automated model was generally differentiating well between properties with and without ILV.

Finally, Figure 8.12 shows that the majority of dark blue properties (i.e. overturned automated “yes” outcomes) are located in western parts of Christchurch. The soil profile in these areas is generally characterised by highly complex interbedded silts and sands and regions of gravel and peat material. In these areas, the automated model struggled to predict liquefaction vulnerability because, generally, the liquefaction triggering method used in the estimation of LSN over-predicts either the liquefaction triggering or the liquefaction consequence relative to the land damage observations.
9 Stage 1 Assessments

9.1 Purpose and Outline

The purpose of Stage 1 of the ILV land damage qualification process was to classify urban residential properties in Christchurch as either qualifying or not qualifying for ILV in straightforward cases. Where a qualification decision for a property was not straightforward, it was recorded as Stage 2. These properties were identified as requiring further manual assessment because they are marginal cases or cases with complex ground conditions, where the predicted land performance is not consistent with the observed performance.

This section of the report describes the Stage 1 assessments in more detail:

- Section 9.2 describes the approach undertaken when assessing properties for Stage 1 ILV qualification;
- Section 9.3 works through an example by applying the approach to a small cluster of properties in the Richmond area to the northeast of the CBD; and
- Section 9.4 summarises the ILV assessment results following the completion of Stage 1 qualification.

The approach to Stage 2 assessments is described in Section 10 of this report.

9.2 Approach to Resolving Stage 1 Decisions

9.2.1 Overview

For the Stage 1 qualification process, packs were created and zoomed into areas with typically 10 to 20 CPTs. Depending on the density of the CPTs and the geological complexity, this could comprise anywhere between 5 to 100 properties. Typically, most packs focused on ILV decisions for 10 to 20 properties in areas of reasonable geological complexity.

A regional assessment of the data was undertaken initially. Any relevant notes (such as over/under-prediction in ground surface subsidence or geological transitions) were marked on the packs. This is described in Section 8.3.1 of this report.

Following the regional assessment, local and specific analysis was undertaken as described in Section 8.3.2 of this report. This included a liquefaction vulnerability sensitivity analysis of each CPT against PGA and depth to groundwater. The CPT data and sensitivity analysis was reviewed to classify the CPTs and boreholes within the area of interest. The CPT and borehole classifications were then spatially compared to the mapped data (i.e. observed land damage, ground surface elevation, etc.). If the datasets reconciled, then properties were qualified or not qualified for ILV accordingly.

When reviewing each pack, the following datasets were considered at both a regional (where applicable) and local level:

- Geological and soil maps;
- Post-CES ground surface elevations;
- Groundwater surface elevation maps;
- High resolution aerial imagery;
- Observed land damage mapped across the area for each of the main CES events relative to the estimated levels of earthquake shaking;
- Estimated vertical change in elevation over the CES;
- The results of automated modelling of ILV; and
Detailed assessment of the results of all available ground investigation data (CPTs, boreholes and laboratory testing) in the local geographic area.

Using the above information, the objective was to determine for each property within the pack if the available geotechnical data and corresponding liquefaction vulnerability assessments reconciled with the observed land damage relative to the estimated levels of earthquake shaking for the four main earthquake events in the CES. When this was the case, a property was either qualified or not qualified for ILV. Likewise, if it was clear from the data that one or both of the engineering criteria would not be satisfied, then that property was assessed as not qualifying for ILV.

Conversely, if it was not clear from the data whether both of the engineering criteria would be satisfied and the geotechnical data and observed land damage relative to the estimated levels of earthquake shaking did not reconcile, then a property was recorded as Stage 2 and requiring further review (discussed further in Section 10).

The four parts of the Stage 1 ILV qualification process is summarised in Figure 9.1.
Figure 9.1: Stage 1 ILV qualification approach.
9.2.2 Part 1 - Creating ILV Packs for Stage 1

The size of the area under consideration in each ILV assessment pack was determined largely by the density of the geotechnical investigation information available in the area as well as the geological complexity. In areas where there was a higher density of geotechnical information the packs covered a smaller area. In areas where there was a lower density of geotechnical information and the soil conditions were less geologically complex (in relation to the ILV assessment criterion), the packs covered a larger area. This meant the number of properties under consideration in each pack varied from as few as 5 to as many as 100. A typical pack would cover an area containing approximately 10 to 20 properties. In total more than 3,000 Stage 1 ILV assessment packs were created.

Basing the size of the area under consideration in each pack on the geological complexity and number of geotechnical investigations available was a reasonable approach. It provided the engineer undertaking the assessment with a practical level of resolution of information to apply engineering judgement to the ILV assessment without being overwhelmed with too much information.

In most of the residential Red Zone, where the land damage was consistently moderate-to-severe, the ground surface subsidence was generally greater than 0.5 m, and both the LSN and ΔLSN values were consistently high, the CPT density is lower and the corresponding pack size is larger. This is appropriate because based on observed land performance, the amount of ground surface subsidence and spatial consistency in predicted land performance, it is clear that large parts of the residential Red Zone satisfy both engineering criteria. The CPT density in the TC3 area is much higher than in residential Red Zone or TC2 areas. The land with the worst damage in Green Zone is within the TC3 area and hence, this is where the majority of residential dwellings are being rebuilt. In addition, EQC supplemented geotechnical investigations in TC3 areas with fewer rebuilds to improve the resolution at which the ILV assessments could be undertaken.

The CPT density in TC2 is generally similar to that in residential Red Zone although there are small isolated pockets in the TC2 area with higher CPT density because of poor land performance resulting in more residential buildings requiring rebuilding. As a result, fewer investigations have been undertaken in these areas. However, the majority of the land performed well in TC2 across the CES. As a result, the majority of these properties can be assessed as not qualifying for ILV land damage on the basis that the observed land performance demonstrated they did not satisfy Criterion 1. In the small isolated pockets of land within TC2 which did not perform well throughout the CES, EQC completed supplementary investigations to enable the ILV assessment in these areas to occur at a higher resolution.

It is important to note that the ILV Assessment Methodology included a process for undertaking further geotechnical investigations where there was an insufficient density of geotechnical data to determine the ILV qualification status for a given area. If the engineer undertaking the ILV assessment determined that a different level of detail was more appropriate for the ILV assessment pack, the pack was sent back to be reformatted accordingly.

9.2.3 Part 2 - Regional Assessment (Task 1 in Figure 8.5)

No additional regional considerations were made in the Stage 1 process over and above those listed in Section 8.3.1.

---

8 In TC3 areas, undertaking geotechnical investigations is a pre-requisite prior to designing new foundations for residential buildings as per the MBIE guidelines (refer to Section 3.8).
9.2.4 Part 3 - Local and Specific Assessment (Task 2 and 3 in Figure 8.5)

No additional local and specific considerations and steps were made in the Stage 1 process over and above those listed in Section 8.3.2.

9.2.5 Part 4 - ILV Qualification Assessment (Task 4 in Figure 8.5)

Properties were qualified or not qualified for ILV in Stage 1 if:

- **No land damage at levels of earthquake shaking equal to or above M6 0.3g** – Despite the geotechnical data indicating liquefaction vulnerability (i.e. LSN > 16), if there was no recorded land damage in areas where the estimated level of earthquake shaking in the CES was in excess of M6 0.3g and there was reasonable confidence in the shaking estimates, the property would be assessed as not having ILV. The absence of observed land damage in such an event provided a clear and reasonable basis not to qualify the properties for ILV because it would not satisfy Criterion 1. Secondary information such as small values of estimated liquefaction related subsidence were also used to support these decisions.

  In order for this conclusion to be reached, there needed to be a reasonable level of certainty in the accuracy of the estimated levels of earthquake shaking and that the land damage observations within the area of consideration were consistent (i.e. that there were no pockets of liquefaction ejecta on the properties under consideration and there was no liquefaction ejecta visible on the adjacent roads).

- **All available data reconciled** – The geotechnical data and the liquefaction vulnerability assessment analyses at each CPT location was in agreement with the observed land damage throughout the CES (i.e. the estimated LSN values are consistent with the observed land damage at respective estimated PGA values) and the estimated ground surface subsidence was such that that straightforward ILV decisions could be made. This was the case for approximately 90% of the ILV decisions.

  In some cases, while the available data did reconcile, the liquefaction vulnerability and change in vulnerability (LSN and ΔLSN values respectively) were close to the indicator values and it was not clear whether both of the engineering criteria would be satisfied (i.e. the cases were marginal). These properties were held back for further assessment using the Stage 2 process.

- **No estimated subsidence across the CES** – Despite the geotechnical data indicating liquefaction vulnerability (i.e. LSN > 16), if there were areas with consistently low levels of estimated ground surface subsidence, the property would be assessed as not having ILV. The absence of estimated ground surface subsidence across the CES provided a clear and reasonable basis to qualify the property as no for ILV because it would not satisfy Criterion 2.

  This was often applicable in south-western parts of Christchurch where there were only minor observations of liquefaction related land damage and in south eastern parts of Christchurch where tectonic movement has resulted in raising of the ground surface level resulting in no overall change in the liquefaction vulnerability.

Properties where the qualification was not straightforward and required further manual assessment were identified for the Stage 2 ILV qualification process.

9.2.6 Criteria for further review

The properties that were qualified as Stage 2 in Stage 1 can be categorised into one of three sub-categories. These categories are as follows:

1. Properties requiring further detailed review because the ILV qualification was marginal (i.e. borderline cases close to the indicators for the engineering criteria);
Properties requiring further detailed review because the ground conditions were complex; and

Properties where pre-CES LiDAR DEM is not representative of the ground surface elevations and post-September LiDAR DEM extents were insufficient to undertake an engineering assessment of ILV.

9.2.6.1 Marginal or Complex Cases

The first two categories of the Stage 2 properties identified in the Stage 1 qualification process were properties which were marginal or complex.

A property was considered to be marginal if it was close to the margins of qualifying for ILV on the basis of Criterion 1 and/or Criterion 2. Typical indicators of properties that were marginal with respect to Criterion 1 included minor-to-moderate land damage observations at levels of ground shaking of M6 0.3g or equivalent and estimated LSN values of between 14 and 16. Typical indicators of properties that were marginal with respect to Criterion 2 included moderate levels of estimated total ground surface subsidence and estimated ΔLSN values of between 4 and 5.

A property was considered to be complex if it had one or more of the following issues:

- Geological and topographical boundaries – The estimated LSN value for a given property was potentially unreliable due to geological and topographical boundaries such as river banks, infilled stream channels, terraces and other changes in elevation, but the mapped information available as part of the Stage 1 assessment process was not of sufficient accuracy to determine where the transition zones occurred;

- Variable prediction of LSN – The estimated LSN values for CPT across a localised area were highly variable with no apparent differentiation of land damage observations throughout the area;

- Potential under-prediction of LSN – The estimated LSN value for a given property was lower than would be anticipated for the level of ground shaking experienced during the CES and the land damage observations recorded;

- Potential over-prediction of LSN – The estimated LSN value for a given property was higher than would be anticipated for the level of ground shaking experienced during the CES and the land damage observations recorded; and

- Potential hypersensitivity of ΔLSN – The estimated ΔLSN value for a given property was higher than would be anticipated for the level of ground surface subsidence as a result of the CES where the groundwater surface is in close proximity to the ground surface (as discussed in Section 8.3.2.2 and 8.3.2.3).

9.2.6.2 Insufficient LiDAR Survey Coverage

The third category of the Stage 2 properties identified in the Stage 1 qualification process is properties with insufficient LiDAR survey coverage.

As discussed in Section 8.3.2, properties with 2003 LiDAR DEM elevations that were not representative of the pre-CES ground surface elevations were identified using aerial photography. The majority of these properties were assessed as not qualifying for ILV land damage as they did not satisfy Criterion 1 (refer to Section 2.4).

However, there is one large residential development in Northwood where earthworks occurred between 2003 (when the pre-CES LiDAR was flown) and the September 2010 event, which required more detailed assessment. In addition, the post-September LiDAR was not flown in this area despite the September 2010 event being the main CES event that caused land damage and potential liquefaction-related subsidence in that area.
Following a review of the available data, the size of the area with insufficient LiDAR was reduced to properties which had observed land damage following the September 2010 event and would most likely satisfy Criterion 1. These properties are identified in Figure 9.5. Without knowing the amount of ground surface subsidence which has occurred on a property, the ΔLSN could not be determined in this area.

Around 600 properties are affected by this lack of data. Section 10.2.4 outlines the process used to resolve the ILV assessments in this area.

9.3 Stage 1 Worked Example

To demonstrate the Stage 1 manual review process the following worked example is summarised below. The full ILV data pack for this case study is included in the Worked Example Material. The location of this worked example is shown in Figure 9.2.

This worked example shows a relatively simple assessment where engineering judgement generally supports the automated ILV model results. The assessment process is summarised as follows:

- **Regional Assessment (Task 1)** - The observed land damage maps indicate a transition from land which has performed well during the CES to the west of the assessed area to land which has performed poorly during the CES to the east of the assessed area. The elevation map shows a topographic transition from land to the west which sits higher than the land to the east. This lower lying area indicates shallower groundwater and potentially a thinner non-liquefying crust. There are no apparent LiDAR error bands over the assessment area. Most of these trends can be observed in Figure 9.3.
Local and Specific Assessment (Task 2 and 3) – A review of the aerial photography shows no residential development occurred between the 2003 pre-CES LIDAR and the September 2010 earthquake event. The automated model map indicates properties to the left do not qualify for ILV while those to the right qualify for ILV. All but one of the CPT is coded as either a ✓ or a ✗ (as set out in Section 8.3.2.3) based on whether or not the CPT location meets the ILV engineering criteria. The CPT classified with a ? is not considered because the raw file uploaded to the CGD contains qc and fs but no associated depth. The location of the CPTs and the associated classifications are shown in Figure 9.4.

ILV Qualification Assessment (Task 4) - Comparison of the CPT ✓ and ✗ classifications shown in Figure 9.4 with the CES ground surface elevation, depth to groundwater, September 2010 observed land damage, worst observed land damage across the CES and total ground surface subsidence maps shown in Figure 9.3 indicate good spatial agreement.

The CPTs identified with a ✗ are all located on land with a higher elevation and deeper depth to groundwater (i.e. a thicker non-liquefying crust). They are also on land which has performed well following the September 2010 levels of shaking (roughly equivalent to an M6 0.3g levels of earthquake shaking) and only subsided by around 0.2 to 0.3m.

Conversely, the CPTs identified with a ✓ are located on land with a lower elevation and shallower depth to groundwater (i.e. a thinner non-liquefying crust). They are also on land which has performed poorly following the September 2010 levels of shaking (roughly equivalent to 0.3g M6 levels of earthquake shaking) and subsided by more than 0.5m.

The automated ILV model map is supported by the CPT classifications which are plotted on Figure 9.3 and therefore most of the properties in the assessment area are qualified with either a ✓ or an ✗ as per the automated ILV model as also shown in Figure 9.4. One of the properties sits on a transition zone between qualifying and not qualifying for ILV. The ILV model result indicates the property does not qualify for ILV but the observed land damage and ground surface subsidence maps indicate significant levels of land damage. Therefore, this property has been categorised as Stage 2 and requiring further manual assessment. It is described in more detail in worked example 1 in Section 10.3.1.
Figure 9.3: A series of maps used when assessing a cluster of properties in Richmond, for the Stage 1 qualification process. (a) post-CES ground surface elevation; (b) post-CES depth to groundwater; (c) September 2010 observed land damage; (d) worst observed land damage across the CES; (e) total ground surface subsidence across the CES; (f) automated ILV model results.
Figure 9.4: The location and classification of the CPTs and boreholes and the ILV qualification of the properties for the Stage 1 manual assessment pack around 12 Flesher Ave, Richmond.

9.4 ILV Assessment Results following the Completion of the Stage 1 Process

In total more than 3,000 Stage 1 ILV packs were assessed. The spatial distribution of these Stage 1 ILV qualifications is shown in Figure 9.5. The majority of the properties qualifying for ILV are located in the eastern parts of Christchurch along the banks of the Avon River and to the north of the CBD. These are the areas where the observed land damage through the CES and ground surface subsidence over the CES was most severe.

The properties which passed to Stage 2 are located predominantly on the fringe of the areas of properties qualifying for ILV. The greatest number of Stage 2 properties is in the area to the north of the CBD where groundwater is generally shallow and the soils are often more silty and highly interbedded. The observed land damage in these areas was not as severe as indicated by the CPT-based LSN and $S_{vd}/D$ parameters which predict significant liquefaction triggering should have occurred in the soil profile.

As indicated on Figure 9.5, the properties with insufficient LiDAR discussed in Section 9.2.6.2 are located in the northern suburb of Northwood.
Figure 9.5: ILV assessment results after the completion of Stage 1 and worked example location. Note the white areas on the map represent the non-urban and non-residential land in Christchurch.

Table 9.1 below shows the categorisation of ILV land damage following Stage 1 qualifications divided into their respective TC1, TC2, TC3 and residential Red Zone areas. It is noted that 97% of the properties qualifying for ILV are in areas categorised as either TC3 or residential Red Zone (i.e. 3,403 TC3 properties plus 4,715 Red Zone properties out of a total 8,366 properties assessed as qualifying for ILV).

Table 9.1: ILV Land Damage Qualification following Stage 1 by Technical Category and Residential Red Zone

<table>
<thead>
<tr>
<th>Technical Category</th>
<th>Number of properties which meet engineering criteria for ILV</th>
<th>Number of properties which do not qualify for ILV</th>
<th>Number of properties which require a further Stage 2 assessment</th>
</tr>
</thead>
<tbody>
<tr>
<td>TC1</td>
<td>0</td>
<td>23,267</td>
<td>0</td>
</tr>
<tr>
<td>TC2</td>
<td>248</td>
<td>77,630</td>
<td>3,144</td>
</tr>
<tr>
<td>TC3</td>
<td>3,403</td>
<td>21,361</td>
<td>3,088</td>
</tr>
<tr>
<td>Red Zone</td>
<td>4,715</td>
<td>2,066</td>
<td>468</td>
</tr>
<tr>
<td>Total</td>
<td>8,366</td>
<td>124,324</td>
<td>6,700</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.