Liquefaction susceptibility verification for Wellington City Council

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EXECUTIVE SUMMARY

The greater Wellington region has a higher annual probability of damaging ground shaking than most of the rest of New Zealand. This means that the region is exposed to all effects of strong earthquake shaking, including fault rupture, ground movement, liquefaction and landslides. This report deals specifically with the liquefaction hazard in the Wellington City Council jurisdiction.

Liquefaction is a process that leads to a soil suddenly losing much of its strength, most commonly as a result of strong ground shaking during a large earthquake. However, not all soils can liquefy in an earthquake. For liquefaction to occur, sediments must be relatively young (less than ~10,000 years old) and deposited in a low energy environment (e.g. settle out of suspension) and the groundwater table must be high enough to saturate the sediments. Thus, the places most likely to accumulate sediments prone to liquefaction are lagoons and estuaries near the coastline where sand and silt suspended in floodwaters can settle out of suspension. Other locations are overbank silt deposits (again, silt settling out of suspension from floodwaters) and point bar and channel deposits in meandering river systems. Engineered (and un-engineered) fills constructed on these deposits are also susceptible to the effects of liquefaction, such as around the Wellington waterfront and the Port of Wellington (CentrePort). In Wellington, areas of sandy and gravelly sediments (e.g. Lyall Bay) and alluvium (e.g. Te Aro) also appear prone to liquefaction.

At least six historical earthquakes since 1840 have caused some liquefaction in Wellington Region (1848 Marlborough, 1855 Wairarapa, June 1942 Masterton, 2013 Cook Strait, 2013 Lake Grassmere and 2016 Kaikōura). The liquefaction damage was greater where the earthquake shaking was stronger. In addition, the construction of harbour reclamations (hydraulically filled sand, in general) increased the areas that were affected by those earthquakes, which occurred in the 19th, 20th and 21st centuries.

This report has been prepared for the Wellington City Council and tests the accuracy and/or reliability of the existing maps currently used in their district plan, which show areas where potentially damaging liquefaction may occur, by using publicly available cone penetration test (CPT) data. The report describes the liquefaction process and how it translates into different liquefaction hazards, including ways of quantifying the hazards. A summary of the historical occurrences of liquefaction in Wellington City is given. The datasets and analysis used in the report are given, along with the results and maps showing the probability of liquefaction occurring in the Wellington City Council area, and the expression of damage expected, based on the analyses used for this project.

The liquefaction maps in the current Wellington City Council corporate spatial database are taken from Dellow et al. (2018) and show areas of potentially damaging liquefaction around the city. However, these binary maps are based on geological mapping, historical data and limited subsurface boreholes and CPT data. They are limited in that they display areas that are or are not susceptible to liquefaction damage. This report is the result of one of the recommendations in Griffin and Dellow (2020) and tests the accuracy/reliability of the Dellow et al. (2018) maps using publicly available CPT data, groundwater, LiDAR and peak ground acceleration data for two likely earthquake scenarios that could affect Wellington City.

The resulting maps are more site-specific compared to the Dellow et al. (2018) maps and are based on the CPT locations, showing the probability of liquefaction occurring at specific CPT sites in Wellington City, along with the Liquefaction Severity Number (LSN) and the expression of liquefaction-induced damage that may occur. The fault source scenarios of magnitude 7.5

and 8.0 and the probabilistic seismic hazard scenario with mean peak ground acceleration values between 0.67 and 1.20 showed that there was little difference in the severity of liquefaction between these two scenarios. The LSN values derived from the CPTs mostly agree with the Dellow prediction maps, but the Dellow maps show underprediction of liquefaction in areas of reclaimed land.

While this study has not provided a liquefaction assessment for every CPT in the Wellington City Council jurisdiction, the resulting maps enhance those maps already used in the District Plan. The results show that the probability of liquefaction occurring is generally high to very high in all of the liquefaction susceptibility zones identified by Dellow et al. (2018), including the zones of low liquefaction susceptibility. Little to minor damage associated with liquefaction is expected around the Wellington waterfront, with more severe damage possible around the airport, Queens Wharf to Clyde Quay, and Seatoun and Island Bay areas.

Recommendations include analysing more CPTs within each geological/geomorphological unit to help better characterise the underlying materials and acquiring/analysing more CPTs in areas deemed susceptible to liquefaction by Dellow et al. (2018), such as along State Highway 2, Shelly Bay and other coastal areas.

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1.0 INTRODUCTION

GNS Science and its predecessors have undertaken regional earthquake hazard assessments in the greater Wellington region dating back to the early 1990s. This includes assessments of the liquefaction hazard in Wellington City to identify areas where liquefaction might occur and be damaging to infrastructure, what the extent and severity of the liquefaction might be in response to different levels of ground shaking and where further investigation of liquefaction hazards would be advisable.

The greater Wellington region has a higher annual probability of damaging levels of earthquake ground shaking than most of the rest of New Zealand (Stirling et al. 2012). As a consequence of this high seismic hazard, the region is exposed to all effects of strong earthquake shaking, including fault rupture, ground movement, liquefaction and landslides. This report deals specifically with the liquefaction hazard in the Wellington City Council jurisdiction.

Assessing liquefaction hazard entails estimating the susceptibility of the region’s soils to liquefaction and determining the frequency (return time) of the different levels of strong earthquake shaking that trigger liquefaction. Combining soil liquefaction susceptibility with the return times of levels of earthquake shaking that cause liquefaction allows an estimate of liquefaction hazard (or the probability of a stated level of liquefaction occurring in a given timeframe) to be made.

As demonstrated by the 2010–2011 earthquake sequence in Canterbury, liquefaction has a devastating impact on affected buildings and buried infrastructure. The loss of amenity from liquefaction in Christchurch is costing billions of dollars to rectify, with remediation ranging from retiring land now recognised as unsuited to development through to the replacement of damaged infrastructure with more resilient forms.

This report has been prepared at the request of Wellington City Council and fulfils the project objectives as described in GNS Proposal Q27863878, which forms part of the Contract of Service for ‘Natural Hazards: Ground Shaking Classification, Mapping; and Liquefaction Verification’ between Wellington City Council and GNS Science (signed and dated 11 May 2020).

The report describes the liquefaction process and how it translates into different liquefaction hazards, including water and sand ejection causing differential settlement (variations in vertical displacement) and lateral spreading causing variations in horizontal displacement. A summary of the historical occurrences of liquefaction in the Wellington City Council jurisdiction is then provided. This is followed by a summary on the quantification methods used in this study to determine the probability of liquefaction occurring in the Wellington City Council area, based on two possible earthquake scenarios affecting the Wellington region. The datasets used in this study are described, along with how the data was analysed. The resulting maps include the probability of liquefaction occurring in selected locations in Wellington City, along with the expected severity of liquefaction-induced damage. A brief high-level commentary on engineering design options is also included. This is followed by recommendations for further, longer-term work to improve existing knowledge of liquefaction hazards in the Wellington City Council area.

1.1 Objective of this Report

Wellington City Council hold maps of liquefaction zones for Wellington that describe the spatial variation in liquefaction that could be expected in a given earthquake. These variations in liquefaction arise from the differences in shallow soil properties.

The maps currently used were compiled from a variety of geotechnical and liquefaction data held by Wellington City Council, GNS Science (and its predecessors) and other parties. Due to the nature of the data, it is likely that the extent, robustness, quality and limitations of the data will be variable.

In late 2019, Wellington City Council commissioned GNS Science to summarise the liquefaction information currently used in the District Plan and advise on the soundness and limitations of this data. The resulting report, CR 2020/08 'Wellington City Council Liquefaction: review and recommendations' by Griffin and Dellow (2020) included short-term and longer-term recommendations to enhance and build on the existing information. One of the short-term recommendations from Griffin and Dellow (2020) was to confirm (or otherwise) the reliability of existing liquefaction susceptibility maps using currently available cone penetration test (CPT) data and is the purpose of this report.

The existing binary maps from Dellow et al. (2018) are currently included in the Wellington City Council's District Plan and show areas within the Wellington City Council jurisdiction where potentially damaging liquefaction might occur. These maps are not deemed robust enough, as they are to be included in formal documents such as a district plan and do not fulfil the requirements for a 'Level B – Calibrated Desktop Assessment' as specified in the 2017 Ministry of Innovation, Business & Employment (MBIE) document 'Planning and engineering guidance for potentially liquefaction-prone land Resource Management Act and Building Act aspects'. The aim of this project is to confirm (or otherwise) the reliability of the maps in Dellow et al. (2018). Outputs include a descriptive report, including the methodology used in the analyses and the results, maps showing the probability of liquefaction occurring and the expected severity of liquefaction-induced damage. The results are compared against the maps of Dellow et al. (2018). Further recommendations have been included in this report, based on the findings while undertaking this project.

Upon completion of this work, the maps will fulfil the requirements for 'Level B – Calibrated Desktop Assessment' as specified in the 2017 MBIE document 'Planning and engineering guidance for potentially liquefaction-prone land Resource Management Act and Building Act aspects'.

2.0 THE LIQUEFACTION PHENOMENA

Dellow et al. (2018) and Griffin and Dellow (2020) both provide good summaries of the liquefaction phenomena and liquefaction susceptibility, and the text below is taken from those reports.

Earthquakes pose hazards to the built environment through five main types of processes. These include strong ground shaking (the most pervasive hazard), primary breakage of the ground surface (fault rupture), deformation of the ground surface due to fault rupture (tectonic tilting, differential uplift and subsidence), seismically induced gravitational slope movements slope failures, and ground deformation resulting from soil liquefaction.

The section below briefly describes the liquefaction process, the conditions necessary for liquefaction to occur and some common consequences of liquefaction. This has mostly been adapted from the Institution of Professional Engineers of New Zealand Liquefaction fact sheet (IPENZ [date unknown]; Figure 2.1) and the GNS Science publication by Saunders and Berryman (2012) titled: ‘Just add water: when should liquefaction be considered in land use planning?’.

2.1 Background

In New Zealand, the most widespread observations of liquefaction since European settlement occurred in the 2010–2011 Canterbury Earthquakes Sequence (Cubrinovski et al. 2011a, Cubrinovski et al. 2012). However, earlier instances of significant liquefaction were documented after the 1848 Marlborough, 1855 Wairarapa, 1929 Murchison, 1931 Napier, June 1942 Masterton, 1968 Inangahua and 1987 Edgecumbe earthquakes. Liquefaction was also observed around Wellington during the 2013 Cook Strait and 2016 Kaikōura earthquakes (Hancox et al. 2013; Cubrinovski et al. 2017). Most of these events generated strong shaking in coastal regions, with extensive deposits of recent, cohesionless, fine-grained sedimentary deposits (Fairless and Berrill 1984; Hancox et al. 1997). The effects of soil liquefaction during these earthquakes have been the ejection of water and sand (sand boils or earthquake fountains) and lateral spreading. These phenomena resulted in vertical and horizontal displacement of the ground surface, which caused extensive damage to buildings, wharves, roads and bridges, embankments and buried services (e.g. Hancox et al. 1997).

The Modified Mercalli (MM) intensity scale (Appendix 1) threshold for liquefaction in New Zealand is generally MM7 for sand boils and MM8 for lateral spreading, but both may occur at one intensity level lower in highly susceptible materials (Hancox et al. 1997). Liquefaction-induced ground damage is most common at MM8–10 (Hancox et al. 1997). The minimum earthquake magnitude for liquefaction is magnitude 5, based on recent experience in Christchurch, but liquefaction is more common at magnitudes of 6 and greater. In terms of peak ground acceleration (PGA), a common instrumental measure of the strength of earthquake shaking at a site, the threshold for liquefaction in highly susceptible sediments is between 0.057 g (Quigley et al. 2013) and 0.09 g (Santucci de Magistris et al. 2013) (where 1 g is the acceleration due to the force of gravity at the Earth’s surface).

2.2 What is Liquefaction?

Liquefaction is the phenomenon where a soil suddenly decreases in strength, most commonly as a result of strong ground shaking during an earthquake. However, not all soils can liquefy in an earthquake. The following are particular features of soils that can liquefy:

- The soils need to be composed of loose sands and silts. Such soils do not stick together the way clay soils do.
- The soils need to be saturated (i.e. located below the water table) so that all of the space between the grains of sand and silt is filled with water. Dry soils above the water table will not liquefy.

When an earthquake occurs, strong shaking may cause the sand and silt grains to compress the spaces filled with water, but the water pushes back and pressure builds up until the grains ‘float’ in the water. When this happens, the soil loses strength and has liquefied. Soil that was once rigid now flows like a fluid.

Soils that cannot liquefy may be unsaturated or cohesive (clay is present and binds the soil together) or dense (for example, gravels deposited in a high-energy river or marine environment). If any of these features are present in a soil, it will not liquefy.

Liquefied soil, like water, cannot support the weight of whatever is lying above it – be it the surface layers of dry soil or the concrete floors (or piles) of buildings. The liquefied soil under that weight is forced into any cracks and crevasses it can find, including those in the dry soil above or in the cracks between concrete slabs. It flows out onto the ground surface as sand boils and rivers of silt and water. In some cases, the liquefied soil flowing up a crack erodes and widens the crack (even to a size big enough to accommodate a car). Some other consequences of the soil liquefying are:

- differential settlement of the ground surface due to the loss of soil from underground;
- loss of support to building foundations;
- floating of manholes, buried tanks and pipes in the liquefied soil – but only if the tanks and pipes are mostly empty; and
- near streams and rivers, unsaturated surface soil layers can slide sideways on the liquefied soil towards the streams. This is called lateral spreading and can severely damage buildings and buried infrastructure, such as buried water and wastewater pipes. It typically results in long tears and rips in the ground surface.

Not all of a building’s foundations, buried pipe networks, road networks or flood protection stop-banks need be affected by liquefaction. An affected part may subside (settle) or be pulled sideways by lateral spreading to severely damage the building. Buried services such as sewer pipes can be damaged when they are warped by lateral spreading, ground settlement or floatation.

2.3 Which Soils are Susceptible to Liquefaction?

Not all soils are susceptible to liquefaction. Generally, for liquefaction to occur there needs to be three soil preconditions (Tinsley et al. 1985; Youd et al. 1975; Ziony 1985):

- Geologically young (less than ~10,000 years old) loose sediments, which are
- fine-grained and non-cohesive (coarse silts and fine sands), and
- saturated (below the water table).

When all three of these preconditions are met, an assessment of the liquefaction hazard is required. Assessment of liquefaction hazard can be conducted on a regional or district scale or can be site-specific using, for example, cone-penetrometer tests (e.g. this report). Note that the level of the water table can fluctuate seasonally and can affect the depth to saturated materials, which should be taken into account when assessing the liquefaction hazard.

If one of these preconditions is not met, then soils are not susceptible to liquefaction. If soils are not susceptible to liquefaction, then liquefaction potential does not need to be assessed in an urban or rural planning context.

2.4 Are the Consequences of Liquefaction Significant?

Once it has been ascertained that soils are susceptible to liquefaction, it needs to be determined if the seismic hazard is large enough to trigger liquefaction. This is done by considering whether the likelihood of earthquakes is strong enough and frequent enough to warrant concern, and this depends on the type of facility or infrastructure being considered. For example: for domestic dwellings, the seismic hazard that can be expected to occur more frequently than once every 500 years should be considered, but, for a critical facility, liquefaction should not impact on continued functionality of the facility in a 1-in-2500-year event.

An assessment should be done for buildings and infrastructure where the seismic hazard is large enough to generate liquefaction. Liquefaction damage can occur to a range of assets and infrastructure: the built environment (e.g. buildings), infrastructure (i.e. underground pipes and services, roads), and also to socio-economic resilience, if people are not able to live in their homes and/or attend places of education and employment.

If the impacts of liquefaction are insignificant, it may be appropriate that no planning actions are required. If, however, the potential consequences are significant, and a cost-benefit assessment indicates that possible future losses can be mitigated, either by avoidance or by engineering solutions, then liquefaction should be a criteria assessed during land-use planning. Saunders and Beban (2012) provide an explanation for how the consequences of liquefaction can be assessed in a risk-based planning context.

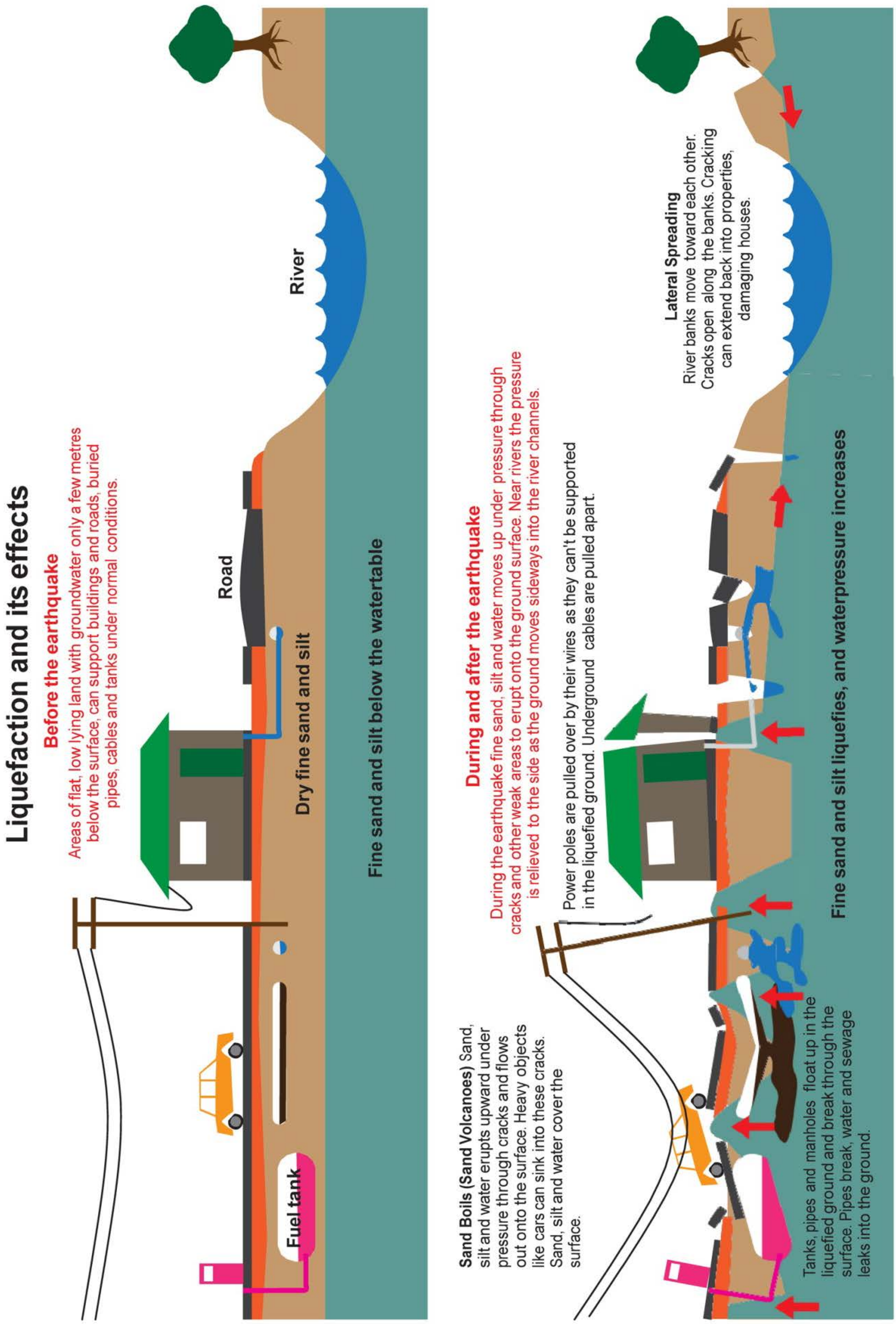

Figure 2.1 Diagramatic illustration of liquefaction and its effects (IPENZ [date unknown]).

3.0 HISTORICAL OCCURRENCES OF LIQUEFACTION IN THE WELLINGTON REGION

Strong earthquake shaking in the Wellington Region caused liquefaction on seven occasions since 1840. These were the 1848 M_w 7.1 Marlborough earthquake, the 1855 M_w 8.1 Wairarapa earthquake, the 1904 M_w 7.5 Cape Turnagain earthquake, the June 1942 M_w 7.0 Wairarapa earthquake, the 2013 M_w 6.6 Cook Strait and M_w 6.6 Lake Grassmere earthquakes and the 2016 M_w 7.8 Kaikōura earthquake. The sites where liquefaction was observed were a function of the location of the epicentre of the earthquake and the strength of the shaking at susceptible sites. The sites where liquefaction has occurred in the past provides information on where it is likely to occur in the future.

3.1 Summary of Historical Liquefaction in Wellington City

Liquefaction has occurred locally in the Wellington region during strong earthquake shaking. The more severe the shaking, the more severe and extensive the liquefaction effects. The locations where liquefaction occurred can be obtained from historical records of the 1848, 1855, 1904, 1942, 2013 and 2016 earthquakes. Dellow et al. (2018) provides more detail on most of these earthquakes and the liquefaction effects seen in the Wellington region.

The observed threshold for liquefaction in the Wellington region is MM7. At this level of shaking, in Wellington City, the reclaimed land in Lambton Harbour experienced liquefaction damage (June 1942, July 2013), as did the Port of Wellington (CentrePort) during the November 2016 Kaikōura earthquake (Cubrinovski et al. 2017; Dhakal et al. 2020).

The Wellington region has been subjected to earthquake shaking of MM8 at least once (southern and western parts of the region in 1848 and the Wairarapa in June 1942). At MM8, minor ground cracking was reported at sites along the original shoreline of Lambton Harbour in Wellington City and between Thorndon and Te Aro. As expected at MM8, the liquefaction damage was a little more widespread in Wellington Region and increased moderately in severity with the increased level of shaking.

Most of the Wellington region experienced MM9 shaking during the 1855 Wairarapa earthquake. Within 10–20 km of the fault rupture, the shaking intensities may have reached MM10. It is at MM9 shaking that the first reports of liquefaction ejecta in Wellington City and Lower Hutt were recorded. In Wellington, liquefaction ejecta was recorded at the corner of Willis and Manners Streets and below the low-water mark in Lambton Harbour (i.e. beneath what is today reclaimed land). Other fissures, without ejecta, were reported along the shoreline where small streams had their mouths between Hobson Street and Lambton Quay. Although not directly mentioned, the Te Aro swamp may be indirectly referred to in reports of ejecta being observed in places that were swampy.

4.0 LIQUEFACTION HAZARD

4.1 Introduction

Liquefaction hazard (the probability of liquefaction occurring) is a measure of the probability of the soils at a site liquefying when subjected to strong earthquake shaking. Certain soils are more susceptible to liquefaction than others. Generally, the assessment of liquefaction hazard involves two steps:

- First, evaluation of liquefaction susceptibility. This involves the identification of those layers at the site that have the physical characteristics of liquefiable soil.
- Second, assessing the probability of strong ground shaking. This involves identifying seismic sources that are capable of generating moderate to large magnitude earthquakes and estimating the likelihood of ground shaking strong enough to cause liquefaction in the materials present at the site.

Liquefaction hazard at a site is assessed by estimating the extent and severity of liquefaction in response to different levels of shaking based on historical records and geological and geotechnical similarities in materials and their behaviour.

For this project, the probability of liquefaction occurring at a particular location (based on a probabilistic and two fault source scenarios for Wellington City) and the severity and expression of liquefaction damage (if applicable) was determined using liquefaction-triggering assessment software (CLiq) on a number of locations from which a CPT was taken. The peak ground motion accelerations for each site were calculated based on two earthquake scenarios, one for a M7.5 earthquake occurring on the Wellington Fault and the other for a M8.1 earthquake situated on the Hikurangi Margin, which lies offshore northeast of Wellington.

4.2 Liquefaction Susceptibility

The existing liquefaction hazard maps of Dellow et al. (2018) in the current Wellington City Council District Plan use liquefaction susceptibility zones to determine the likelihood of liquefaction occurring.

Dellow et al. (2018) used the geological units of Begg and Mazengarb (1996) to identify areas of potential liquefaction susceptibility in Wellington City. The 1:50,000-scale late Quaternary and Holocene geology were simplified to aid in the liquefaction assessment, and four late Quaternary and Holocene units that could potentially liquefy were recognised. Figure 4.1 shows the geological units that underpin the existing liquefaction susceptibility maps in the current Wellington City Council District Plan.

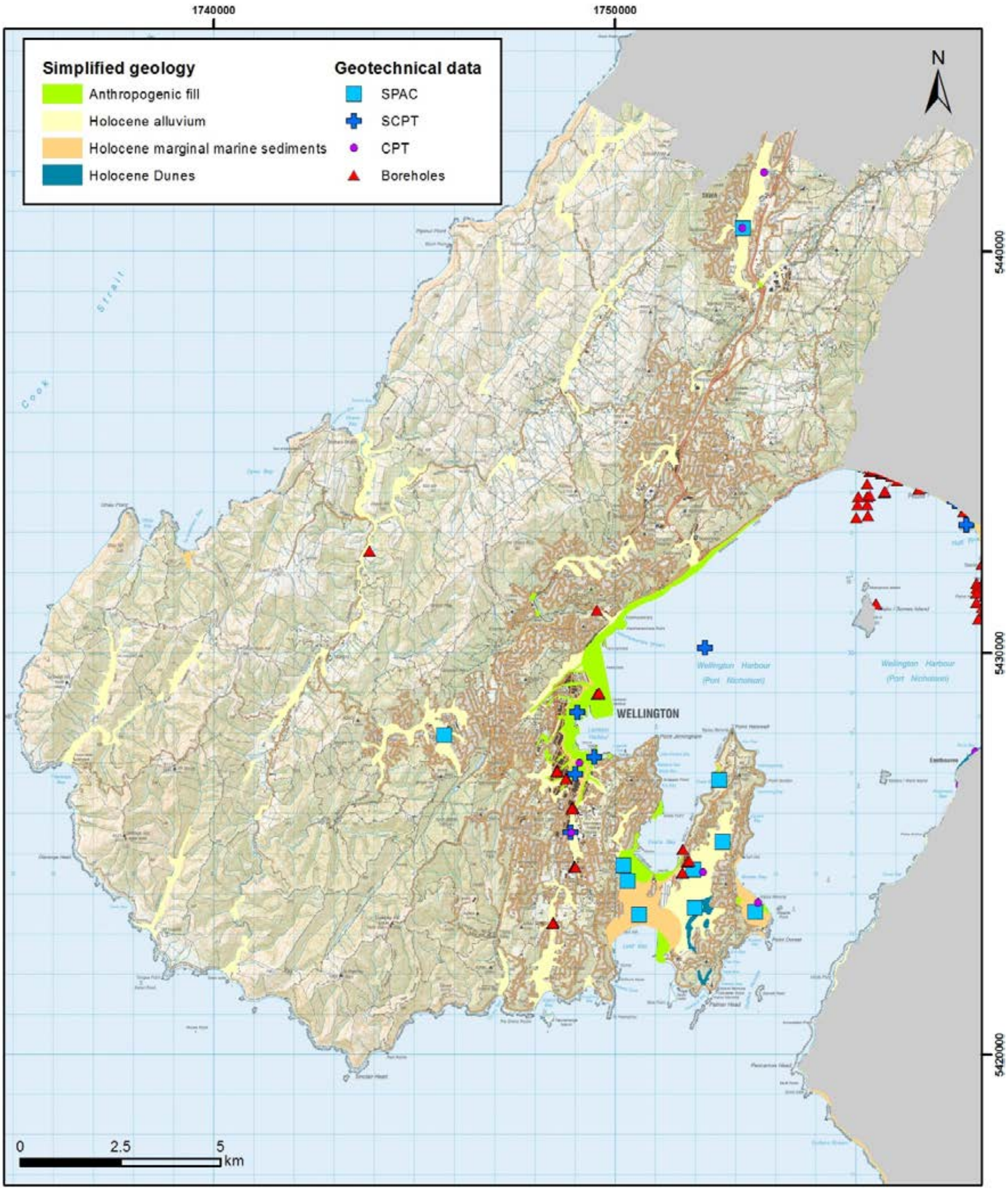

Figure 4.1 The simplified geology map of Wellington City in Dellow et al. (2018), showing locations of sediments potentially susceptible to liquefaction. The locations and types of subsurface data used in their liquefaction assessment are also shown.

These geological units are:

Beach deposits: dominantly medium-dense to dense-fine to coarse sand (marginal marine sediments) around the original shoreline of Lambton Harbour, the Kilbirnie-Lyall Bay isthmus and the Worser Bay, Seatoun and Breaker Bay areas. This also includes two areas of mixed marginal marine sands and fan alluvium in southern Miramar – Strathmore and Island Bay.

Recent alluvium: loose to dense-fine sands to gravel in the small streams and valleys of the city, e.g. Karori, Makara, Tawa and Khandallah-Ngaio. An area of mixed Holocene and older gravels along the south-eastern side of Tinakori Hill is also included in this unit.

Dunes: loose to medium-dense fine sand – a small area of dunes is present in Miramar.

Anthropogenic fills: divided into five separate units ranging from:

- Rock-fill – medium-dense to dense-coarse angular gravels (airport, stream and some harbour fills in the Lambton area; Begg and Mazengarb 1996).
- Hydraulic fill – very loose to loose silt and fine to medium sand (Aotea Quay area; Bastings 1936) .
- Old refuse dumps – medium-dense to dense mixed weathered gravel and human refuse (Wilton; Begg and Mazengarb 1996).
- Engineered fills dense angular gravel (motorway interchange at Tawa; Begg and Mazengarb 1996).
- Unrestrained fill at the southern end of the CentrePort container terminal (Van Dissen et al. 2013).

The fills are subdivided into four separate units because it can be shown that they have responded very differently, from a liquefaction perspective, during historical earthquake shaking (Table 4.1).

Table 4.1 Liquefaction and lateral spreading damage ratings assessed using historical records and geological precedent for the Holocene sediments of Wellington City, used in Dellow et al. (2018). Note: tables referenced are those in Dellow et al. (2018).

Geological Unit[1]	**Modified Mercalli Intensity**					**Liquefaction Susceptibility (see Table 5.4)**
	6[2]	7[3]	8	9[4]	10	
	Liquefaction Damage Rating (see Table 5.1)					
Q1n (hydraulic fill)	**None**	**Minor**	*Moderate*	*Major*	*Severe*	Very High
Q1n[5] (unrestrained fill)	**Moderate**	*Major*	*Severe*	*Severe*	*Severe*	Very High
Q1n (un-engineered fill)	**None**	**None**	*Minor*	*Moderate*	*Major*	High
Q1b (beach deposits)	**None**	**None**	*None*	**Minor**	Moderate	Moderate
Q1a (recent alluvium)	**None**	**None**	*None*	**None**	*Minor*	Low
Q1d (dunes)	**None**	**None**	*None*	*None*	*None*	Low
Q1n (refuse fill)	**None**	**None**	*None*	*None*	*None*	None
Q1n (engineered fill)	**None**	*None*	*None*	*None*	*None*	None

Bold numbers are for historical observations, while the italic numbers are assessments made where no historical data exists.

[1] Geological unit codes (e.g. 'Q1n') from Begg and Mazengarb (1996). 'Q1' refers to oxygen isotope stage one and denotes that the unit has an age between 0 and 14,000 years old. Fills differentiated based on age and construction type (from Bastings 1936; Begg and Mazengarb 1996; Van Dissen et al. 2013).

[2] MM6 data based on the 2013 Cook Strait and Lake Grassmere earthquakes.

[3] MM7 data based on the June 1942 Masterton earthquake.

[4] MM9 data based on January 1855 Wairarapa earthquake

[5] Unrestrained fill is limited to the southern end of the container terminal facilities of CentrePort Wellington.

The variation in liquefaction susceptibility was identified in the first instance using the historical response of the geological units present to strong ground shaking (Beetham et al. 1998; Dellow et al. 2003). For a given earthquake with a known shaking intensity (derived from the MM intensity scale), a liquefaction damage rating can be applied using the scale in Table 4.2. Dellow et al. (2018) applied the observations of liquefaction damage made after the 2010

Darfield and the 2011 Christchurch (Cubrinovski et al. 2011b) earthquakes to the ratings tables and found that the observations were in general agreement with these older tables.

Table 4.2 Descriptions of expected liquefaction-induced ground damage for liquefaction damage ratings (after Dellow et al. 2003).

Liquefaction Damage Rating	Description of Expected Liquefaction-Induced Ground Damage
NONE	• No liquefaction damage is seen.
MINOR	• A few sand boils and minor fissures. • Estimate up to 10% of total area affected.
MODERATE	• Sand boils and moderate fissuring – more extensive near basin edges and in waterlogged areas: banks of rivers broken up and embankments slumped. • Settlements of up to 0.2 m. • Estimate 10–20% of total area affected.
MAJOR	• Lateral spreading common, with many fissures in alluvium (some large), slumping and fissuring of stop-banks, common sand boils. • Settlements of up to 0.5 m. • Estimate 20–50% of total area affected.
SEVERE	• Lateral spreading widespread, with extensive fissures and horizontal (and some vertical) displacements of up to 10 m common, especially near channel edges. • Settlement of uncontrolled fills by up to 1.0 m. • Estimate >50% of total area affected.

Once the liquefaction damage rating has been assigned for known earthquakes, Table 4.3 is used to assign the liquefaction damage rating for intensities that are not represented in the historical record. If the liquefaction response of any geological units cannot be determined from historical data, then a liquefaction damage rating is assigned by firstly considering any geotechnical data available for the unit and then by comparing the unit with similar materials in other areas where a liquefaction damage rating is available that has been assigned based on historical liquefaction.

This method derives a liquefaction susceptibility class (Table 4.3) by assigning the highest liquefaction susceptibility class to the geological units where liquefaction is observed at the lowest shaking intensity (generally a MM intensity of MM7; Appendix 1). As the shaking intensity increases, the severity and extent of liquefaction damage may increase in the very high liquefaction susceptibility class. The onset of liquefaction damage in the high liquefaction susceptibility class occurs at MM8. By using this method, liquefaction susceptibility classes are assigned based on the level of shaking at which liquefaction damage first appears. This method is based on historical observations of liquefaction at the same sites that report increasing severity and extent of liquefaction with increasing shaking intensity. However, these observations are limited because of the short historical record in New Zealand.

Table 4.3 Liquefaction susceptibility classes and liquefaction damage ratings assigned at different Modified Mercalli shaking intensities (after Dellow et al. 2003).

Liquefaction Susceptibility Class	**MM Intensity**				
	MM6	MM7	MM8	MM9	MM10
	Liquefaction Damage Rating				
Very High	NONE	MINOR	MODERATE	MAJOR	SEVERE
High	NONE	NONE	MINOR	MODERATE	MAJOR
Moderate	NONE	NONE	NONE	MINOR	MODERATE
Low	NONE	NONE	NONE	NONE	MINOR
None	NONE	NONE	NONE	NONE	NONE

Using the tables compiled from historical accounts, the liquefaction damage is described and labelled, in many cases based on quite limited descriptions, with respect to the severity of the damage (in terms of the measured displacements) and the extent of the liquefaction in terms of the percentage of the area of a susceptible unit that will visibly manifest liquefaction and lateral spreading.

At low levels of ground shaking, liquefaction will occur in only the most susceptible deposits, namely saturated, relatively uniform fine sands or coarse silts in a loose state, at depths less than 10 m, where the groundwater level is within about 2 m of the ground surface. In Wellington City, the beach and alluvium deposits and anthropogenic fills identified by Dellow et al. (2018) fit these criteria. However, liquefaction may occur in other less-susceptible deposits during stronger ground shaking.

The other variable to consider besides the intensity of ground shaking is the frequency with which shaking of a given intensity occurs. The stronger the earthquake ground shaking, the less frequently it will occur at a site. Dellow et al. (2018) show that the frequency with which MM intensity shaking from MM6 to MM10 occurs in Wellington City (Parliament area) ranges from ~7.6 (MM6) to 1500 (MM10) years, with the frequency for MM7 intensity shaking of approximately 29.3 years, based on long-term average recurrence intervals for known active fault sources.

Susceptibility to liquefaction may reduce if one or more of the following conditions apply: increasing depth to groundwater, increasing the fines (e.g. clay) content in the sediments, increasing coarseness of the sediments (greater gravel content) or increasing variability in the grainsize of the sediments. Some of these conditions can be better understood by inputting geotechnical data (e.g. CPT) in liquefaction assessment software to determine soil behaviour types (e.g. cohesive or non-cohesive soil behaviour).

4.3 Probability of Liquefaction Occurring

The probability of liquefaction occurring at a selected CPT location is based on a number of factors, such as the age of the sediments, how saturated they are and the type of sediment (e.g. fine-grained and cohesive). For this study, the probability of liquefaction occurring (Table 4.4) was calculated by the CLiq software (provided by GeoLogismiki) based on the data inputs for each individual CPT location.

Table 4.4 Overall probability of liquefaction and the associated percentage, according to the Boulanger and Idriss (2014) liquefaction-triggering assessment method in the CLiq software provided by GeoLogismiki. These values have been applied to this study.

Probability of Liquefaction Occurring	Equivalent Percentage
Low	0–12%
High	12–55%
Very High	55–100%

4.4 Liquefaction Severity Number

The Liquefaction Severity Number (LSN) is a new calculated parameter developed by Tonkin & Taylor in 2013, following the 2010–2011 Canterbury Earthquake Sequence (Tonkin & Taylor 2013; van Ballegooy et al. 2014). The unprecedented liquefaction-related land and dwelling damage that occurred as a result of the earthquakes highlighted the need to better understand the vulnerability of the land to liquefaction damage caused by future earthquakes. Tonkin & Taylor used geotechnical investigations (from thousands of CPT and boreholes) and laboratory testing, along with groundwater data from hundreds of monitoring wells, to characterise land vulnerability to the liquefaction hazard. They reviewed the existing published liquefaction vulnerability assessment tools (i.e. the Liquefaction Potential Index [LPI], calculated settlement indicator from predictive correlations [S] and the LSN) and assessed them against the observed land damage (Tonkin & Taylor 2013; van Ballegooy et al. 2014). The study concluded that, of the three calculated parameters considered, the LSN is the most suitable tool for predicting land performance in Canterbury and provided the best correlations with the observations made in Canterbury. The LSN is also considered to be a good indicator of liquefaction vulnerability for residential land that is flat and confined; however, the LSN is not considered as an indicator of vulnerability to lateral spreading hazard (Tonkin & Taylor 2013; van Ballegooy et al. 2014).

Tonkin & Taylor (2013) showed that there was a correlation between the calculated settlement indicator and the earthquake damage datasets and that the calculated settlement could be considered a proxy for predicting the likelihood of liquefaction-related damage (Table 4.5). However, the LSN parameter refined the calculated settlement parameter by including a depth weighting function, whereby ground surface damage from shallow liquefied layers is more likely than from deeper layers (Tonkin & Taylor 2013). It is important to remember that liquefaction may still occur at greater depths below the ground surface but may not manifest itself at the surface, thus going undetected (Idriss and Boulanger 2008). Liquefaction at greater depths can be a concern in many situations, such as earth embankments constructed of looser material (e.g. hydraulic fill) or constructed over younger sediments (Idriss and Boulanger 2008).

Table 4.5 Liquefaction Severity Number (LSN) and associated observed land effects (from Tonkin & Taylor 2013). The expressions of damage are also from Tonkin & Taylor (2013) and have been used in the Boulanger and Idriss (2014) liquefaction-triggering assessment method in the CLiq software provided by GeoLogismiki. These LSN ranges and expressions of damage have been used in this study.

LSN Range	Predominant Performance
0–10	Little to no expression of liquefaction, minor effects.
10–20	Minor expression of liquefaction, some sand boils.
20–30	Moderate expression of liquefaction, with sand boils and some structural damage.
30–40	Moderate to major expression of liquefaction; settlement can cause structural damage.
40–50	Major expression of liquefaction, undulations and damage to ground surface; severe total and differential settlement of structures.
>50	Severe damage, extensive evidence of liquefaction at surface, severe total and differential settlements affecting structures, damage to services.

However, there are many uncertainties associated with using the LSN as a tool for liquefaction vulnerability assessments, including earthquake motion characteristics, geological spatial variability, soil profile complexities, groundwater pressure and saturation complexities and soil behaviour characteristics (Tonkin & Taylor 2016). In addition, there are significant variables that contribute to the uncertainty of the LSN, including earthquake magnitude (M_w) and ground motions (PGA), liquefaction triggering to calculate LSN, depth to groundwater estimates, CPT measurement accuracy and spatial variation and interpolation of LSN values at CPT locations (Tonkin & Taylor 2016). Not all of these uncertainties will be applicable to a particular site; however, it is reasonable to assume that a given site may be affected by some of them, thus engineering judgement is important to address the uncertainties of the inputs into LSN. Such judgement includes reviewing the datasets, including land damage observations, geological and topographical assessments and detailed specific analysis of the geotechnical information (Tonkin & Taylor 2016).

Following the Tonkin & Taylor (2013) study, in 2017, the Ministry of Business, Innovation & Employment (MBIE) included the LSN method as a quantitative approach to estimate the degree of liquefaction-induced ground settlement damage in their Planning and Engineering guidance document (MBIE 2017). Based on the findings in van Ballegooy et al. (2014), this project has used the LSN method to determine the severity and expression of liquefaction damage that may occur to land in the Wellington City Council jurisdiction.

5.0 DATASETS USED TO EVALUATE LIQUEFACTION SUSCEPTIBILITY IN THE WELLINGTON CITY COUNCIL JURISDICTION

5.1 Introduction

This report has used existing datasets, where possible, to identify locations that are susceptible to liquefaction occurring in the Wellington City Council territorial area and to determine the probability and severity of liquefaction occurring, if applicable. The data from these datasets are input into the CPT processing software to provide an overall output LSN number for each CPT site.

5.2 Cone Penetrometer Tests

Following the Canterbury Earthquake sequence of 2010–2011, the New Zealand Geotechnical Database (NZGD) was established to allow geotechnical data to be uploaded and shared amongst professional organisations. This database contains thousands of CPTs, which are the most common method of investigation to assess the material properties of the subsurface soils in areas prone to liquefaction. MBIE (2017) state that CPTs are one of the most useful deep investigation methods for assessing liquefaction and that investigations should be deep enough to characterise the ground to at least 10–15 m depth below ground level for residential or light commercial development.

The CPT is a ground investigation method for soils, whereby a conically shaped tip is pushed into the ground (either mechanically or electronically) and, while doing so, measurements (e.g. the cone resistance, sleeve friction and pore pressure) are taken. The CPT data provides insight about strength and behaviour characteristics of the soil. The penetration of the CPT will be limited if very hard soils, gravel layers or rock are encountered (Robertson and Cabal 2014). For this study, the total depth of the CPT is assumed to be the depth at which the tip of the cone refuses to penetrate the ground any further, most likely due to hard ground, as little or no or information was given about the reason for refusal in the raw data. Liquefiable layers may exist below the extent of the CPT, underestimating the potential for liquefaction to occur or the severity of the liquefaction; however, without this data, there is no way of knowing.

For this project, all available CPTs in the Wellington urban area were downloaded from the NZGD on 30 March 2020. Any CPTs loaded into the NZGD after this date have not been included in this project. A total of 376 records were downloaded, five of which are SCPT (seismic CPTs), two are SDMTs (seismic dilatometer tests) and the remainder (369) being CPTs. The SDMTs have not been included in this project. Digital data were available for 330 CPTs, with the remaining CPTs having analogue data in the database. Based on the data availability to perform liquefaction analysis and the distribution of CPTs across the city, the CPTs were mapped and ranked to determine which CPTs to analyse. For this project, representative CPTs were selected for most CPT clusters in the Wellington City Council jurisdiction, with five representative CPTs selected from the large cluster located at the Wellington Airport carpark building (Figure 5.1; Appendix 2). A total of 80 CPTs acquired between December 2011 and November 2019, and with total depths ranging between 2.2–18.7 m, have been analysed for this project (Appendix 2), satisfying the ‘Level B’ requirements of MBIE (2017).

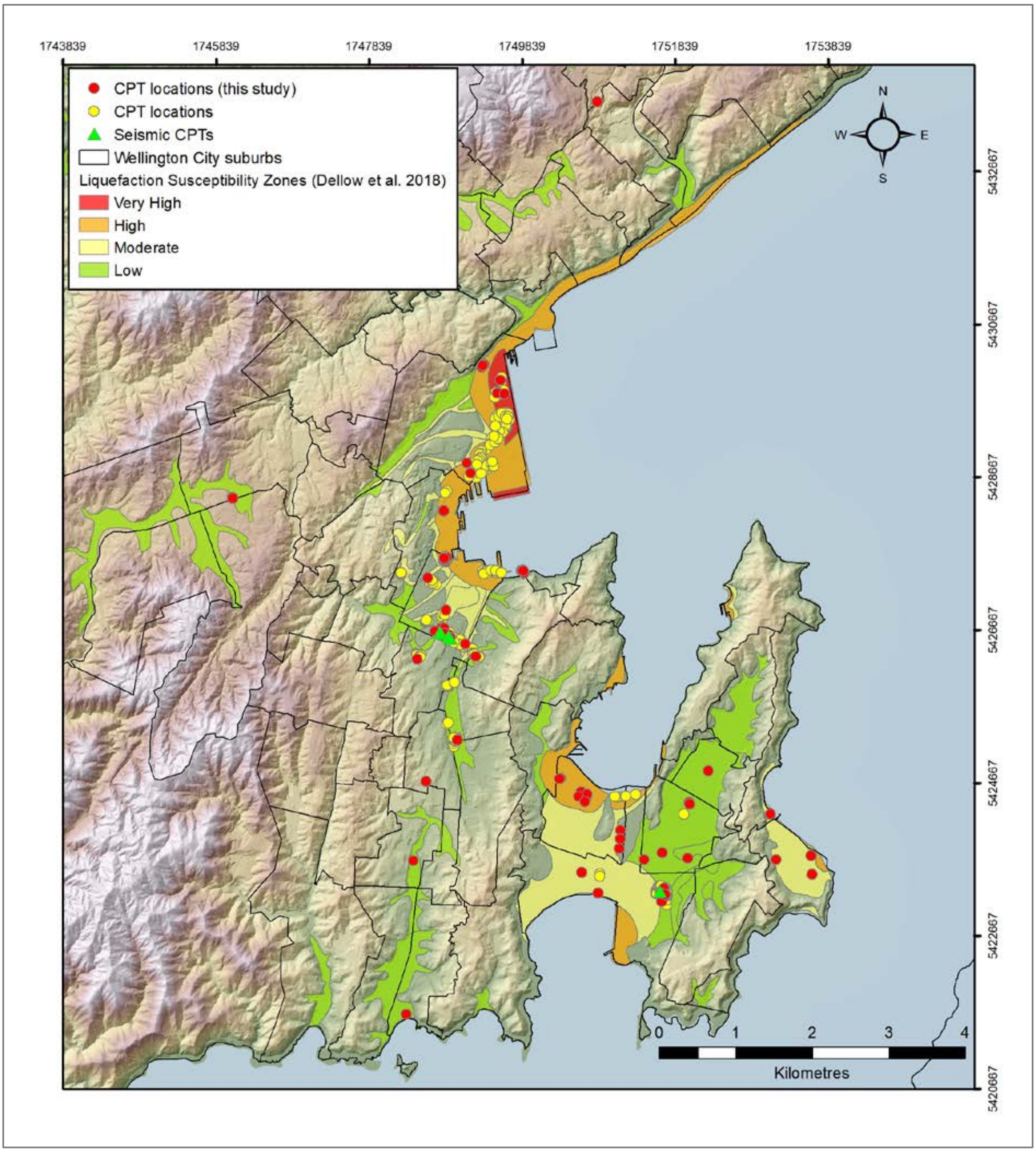

Figure 5.1 Location map of the Wellington City Council jurisdication, showing the CPTs (yellow circles) and SCPTs (green triangles) used in this study. All available CPTs with data lodged in the New Zealand Geotechnical Database (NZGD; as of 30 March 2020) are shown by the dark red circles. Wellington City suburb boundaries are delineated by the black lines. The liquefaction susceptibility zones of Dellow et al. (2018) are also shown.

5.3 Setting and Geomorphology

At the request of Wellington City Council, the area of interest is the Wellington City Council jurisdiction (Figure 5.1). The distribution of CPTs in the jurisdiction is mixed, with some areas (e.g. around the waterfront and coastal areas) having a high density of CPTs, whereas other areas have none or few (e.g. Raroa, Karori). The liquefaction susceptibility zones of Dellow et al. (2018) are also shown in Figure 5.1 to show the spatial distribution of CPTs within each zone.

This study has used the detailed Wellington regional geomorphological study of Townsend et al. (2020) to provide information on the sediments and materials present at/near the surface (Figure 5.2), which are more applicable in liquefaction studies than those found deeper underground. This sub-surface data differs from that used in Dellow et al. (2018), who used the sub-surface geology mapped by Begg and Mazengarb (1996) to define their liquefaction susceptibility zones. Townsend et al. (2020) used aerial photographs; Light Detecting and Ranging (LiDAR) digital elevation models (DEMs) without vegetation; digital surface models (DSMs) with vegetation, derived from historical aerial photos; and targeted field verification to produce their 1:5000-scale geomorphological maps.

Most of the CPTs analysed for this study overlie alluvium, fill, sand and gravel, or a mix (Figure 5.2). These sediments roughly coincide with the geological units and liquefaction susceptibility zones of Dellow et al. (2018), with alluvium overlying Holocene alluvium (generally low liquefaction susceptibility), sand and gravel overlying Holocene marginal marine sediments (moderate liquefaction susceptibility) and fill overlying anthropogenic fill (high to very high liquefaction susceptibility).

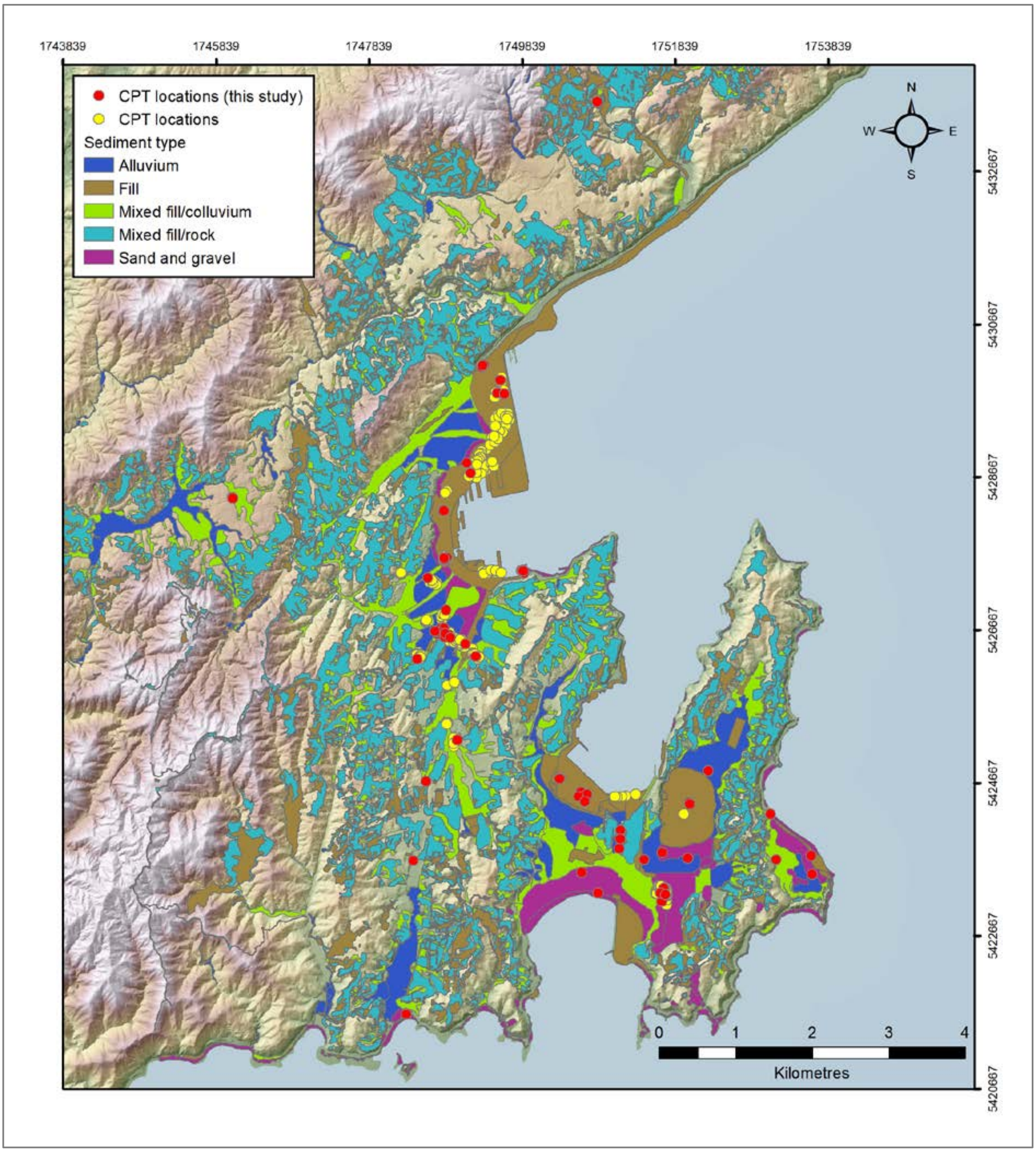

Figure 5.2 Geomorphological map of the Wellington urban area, showing all CPT locations (green circles) that have been lodged in the New Zealand Geodetic Database (light-green circles) for the Wellington urban area as of 30 March 2020. The CPTs analysed for this study are represented by the yellow circles. Geomorphology data are taken from Townsend et al. (2020).

5.4 Groundwater Levels

Groundwater is an important factor in liquefaction analyses, as the soil needs to be fully saturated (among other factors) for liquefaction to potentially occur. Most of the selected CPTs with a groundwater level (GWL) recorded at the time that the CPT was taken and greater than 0 metres below ground level (m bgl) were used in the analyses. For those CPTs with no or a ‘0 m’ GWL recorded in the CPT header information, it was initially proposed that the GNS Science Shallow Groundwater model be used to estimate groundwater levels. However, after comparing the modelled GWLs to GWLs recorded in the CPTs (‘ground truthing’ the data), it was decided that the groundwater model was too coarse for use in this project. Instead, for the CPTs with an inaccurate GWL, the GWL in nearby borehole logs (also lodged in the NZGD) were used as an approximation. In cases where there was more than one nearby borehole, the lowest GWL was used in the liquefaction analysis, as it is likely that the ground would not be fully saturated at higher ‘winter’ values.

It should be noted that the selected CPTs were acquired throughout the year, from January to December, and thus seasonality will also affect the GWL recorded at the time. No attempt has been made to adjust the liquefaction analyses for this, and therefore brings an uncertainty to the analyses.

5.5 Peak Ground Acceleration and Magnitude

Peak ground acceleration (PGA) and earthquake magnitude values are used in liquefaction analyses to provide the scenario to determine whether liquefaction will occur or not and the severity of the liquefaction for a particular seismic event. The mean PGA and magnitude estimates required as input for liquefaction analyses in this study are derived in accordance with the third edition of the NZTA Bridge Manual (NZTA 2018). In an email to GNS Science on 11 September 2020 from a senior geotechnical engineer at MBIE (pers. comm. K. Saligame), it is indicated that the 2014 NZTA Bridge Manual is one of the possible guidance documents for the calculation of ground motion for liquefaction triggering/assessment. As such, we opt to use the 2018 NZTA Bridge Manual, as it supersedes both the 2014 and 2016 editions of the document, with the exception that we are using a 50-year design life (instead of 100 years, which is common for Bridge Manual structures) as indicated in the contract with the client. For this study, the seismic hazard and magnitude combinations developed to assess liquefaction triggering are:

- 10% in 50 years probabilistic seismic hazard for PGA with the associated average magnitude (7.3)
- The 50th- and 84th-percentile PGA scenario hazard for the 2010 National Seismic Hazard Model Wellington-Hutt Valley fault source (magnitude 7.5), and
- The 50th- and 84th-percentile PGA scenario hazard for the 2010 National Seismic Hazard Model Hikurangi-Wellington minimum source (magnitude 8.1).

Mean probabilistic seismic hazard for PGA for 10% probability of exceedance in 50 years is produced using the corrected version of the 2010 National Seismic Hazard Model (NSHM; Stirling et al. 2012; pers. comm. M Gerstenberger, G McVerry, 2018) and a logic tree of ground motion characterisation models (GMMs; Section 5.4.2; Appendix 3) using specific site conditions (Section 5.4.1). The most important modifications of the 2018 corrected version of the 2010 NSHM for Wellington are the recurrence intervals of the Hikurangi subduction interface sources shown in Table A3.1. The results are calculated at each of the CPT sites listed in Table A2.1 using version 3.10 of the OpenQuake engine (Pagani et al. 2020). The results used in the liquefaction assessment are listed in Table A3.2.

The mean 10% in 50 years probabilistic hazard results were disaggregated to determine the average magnitude associated with the 10% in 50 years PGA hazard. The average magnitude value is 7.3. The top contributors to the hazard are the Hikurangi-Wellington minimum source (M8.1, 23% contribution, Wellington-Hutt Valley [M7.5, 16%] and Hikurangi-Wellington maximum [M8.4, 13%]). No other sources contribute higher than 4% individually.

The mean PGA hazard for the Wellington-Hutt Valley fault source and the Hikurangi-Wellington minimum source were also calculated using those specific sources from the corrected 2010 NSHM (Stirling et al. 2012) and the same site conditions (Section 5.4.1) and GMMs as the probabilistic hazard (Section 5.4.2; Appendix 3). The 50^{th}- and 84^{th}-percentile PGA hazard values were produced for each scenario; the 84^{th}-percentile values were used in the liquefaction analysis. These results are listed in Table A3.2.

The 84^{th}-percentile scenario PGA values for both sources exceed the 10% in 50 years probabilistic hazard values, which in turn exceed the 50^{th}-percentile estimates. The magnitude of 7.3 associated with the probabilistic hazard estimates is lower than the magnitudes of 7.5 and 8.1 associated with the two scenario events.

5.5.1 Site Conditions

Site conditions, including average shear-wave velocity to 30 m depth (Vs30 in m/s) and depth to 1 km/s (Z1k in m), were estimated for use in this study. These data were calculated from a 3D geological model of Wellington Region (see Hill et al., forthcoming 2020); Vs30 values are calculated from shear wave velocity and thickness of units in the top 30 m and Z1k from the depth to basement where basement is approximately 1000 m/s and considered in the Z1k horizon. The majority of the sites have Vs30 values of less than 400 m/s, and Z1 estimates range from a few metres to around 200 m depth. The parameter representing depth to 2.5 km/s (Z2.5 in km) is estimated from the Vs30 value using an equation provided by Chiou and Youngs (2008); it should be noted that this parameter is only used by one crustal ground motion model (Campbell and Bozorgnia 2014, with a total weight of 0.25). In the hazard calculation, the site condition closest to the CPT site location of interest is used to calculate the hazard.

5.5.2 Selection of Ground Motion Models

Ground motion models (GMMs) are used in seismic hazard studies to estimate ground shaking at a given site, given an earthquake source model and set of site conditions. Modern probabilistic seismic hazard analyses (PSHAs) generally incorporate multiple GMMs to cover modelling uncertainties, consistent with international best practice. This study implements this approach by using a logic tree to combine multiple GMMs for crustal and subduction zone earthquakes. The crustal GMMs have been selected from those of the NGA-West2 project (Gregor et al. 2014) plus the Bradley (2013) model developed for New Zealand earthquake motions, while the subduction zone expressions are from the BC Hydro project (Abrahamson et al. 2014, 2016). A logic tree of these results is shown in Figure 5.3. Each of these models is converted to provide results in the larger of two horizontal components, rather than the geometric mean or similar, using the models of Boore and Kishida (2017). Additional epistemic uncertainty is applied by recommendation of Van Houtte (2017) and uses the model of Al Atik and Youngs (2014). Further explanation and justification of the models selected is provided in Appendix 3.

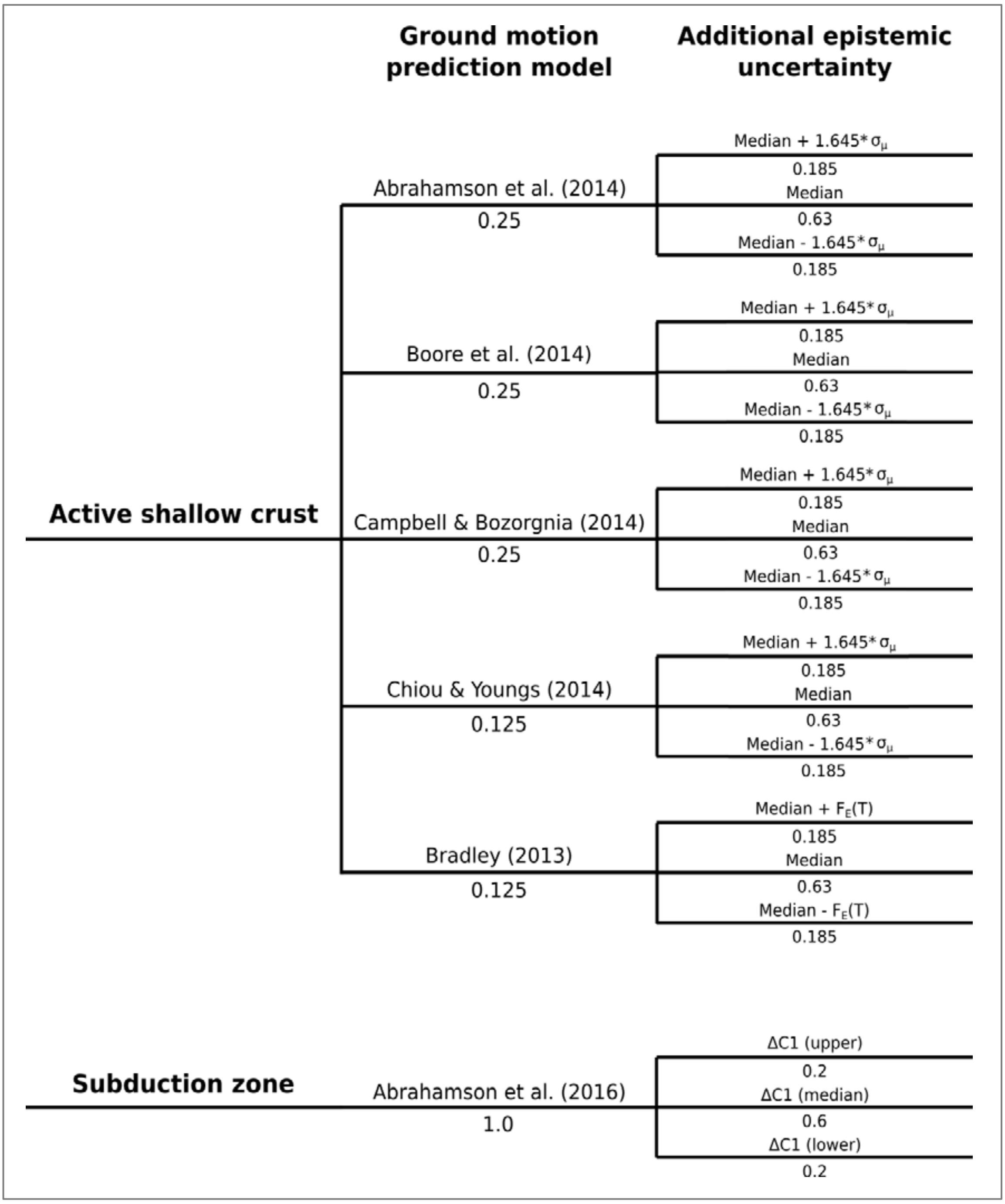

Figure 5.3 Ground motion characterisation model used for the seismic hazard analysis. The volcanic branch of this source model is not shown as it only contains the Bradley (2013) ground motion prediction equation. All models are converted from 'average' horizontal motions to 'larger horizontal component' using the Boore and Kishida (2017) models.

5.6 LiDAR / Ground Level Elevations

To obtain the ground level (GL), LiDAR data were used to precisely obtain an elevation for each CPT site. Data were downloaded from the Open Topography data repository (opentopography.org) for the 'Wellington, New Zealand 2013' airborne LiDAR dataset that was captured in January 2013. Data were clipped to the study area, processed into a digital terrain model (DTM) for ground returns only and then filtered and re-processed after manual removal of erroneous points. Each CPT site was assigned a GL from the final DTM. A final grid was created with a 1 m cell size using a triangulated irregular network from the point cloud that had a point density of approximately 3 points per square metre.

6.0 DATA ANALYSIS

6.1 Analyses of Cone Penetration Test Data

CPT data allow the application of liquefaction assessment methods used in Christchurch following the 2010–2011 Canterbury Earthquake Sequence, and further developed following the 2013 Cook Strait and 2016 Kaikōura earthquakes, to be applied to the Wellington City Council area. The purpose of this project was to determine whether the existing liquefaction maps in the District Plan are accurate/reliable. Therefore, lateral spreading and other physical mechanisms of settlement (e.g. including lateral discontinuity of strata, proximity of free faces, loss of soil ejected to the surface, etc.) have not been taken into account when performing the analyses.

The raw CPT data was loaded into CPeT-IT software, provided by GeoLogismiki, for initial quality control of the data. During the data loading, the raw CPT data was corrected by converting negative and zero values of cone resistance (q_c) and local friction (f_s) to 0.01, and local friction and pore pressure were converted to kPa where necessary. For some CPTs, the further corrections (e.g. removing negative spikes where CPT rods were changed) were applied to the raw data using the software defaults. Normalised interpretative derivative data (e.g. soil behaviour type index [I_c]) was generated in CPeT by using the calculation method of Robertson (2016) (conforming to Youd et al. 2001 [NCEER], Robertson and Wride 1998 and Robertson 2009), and the standard criterion of I_c <2.6 was used to identify soils susceptible to liquefaction. Location data, groundwater levels, earthquake magnitude and PGA values were added to the CPT files in CPeT-IT. Once the q_c was completed, the corrected files were imported into CLiq software, also provided by GeoLogismiki, for liquefaction analysis.

For this study, the Boulanger and Idriss (2014) liquefaction-triggering method, with amendments as per the Canterbury MBIE guidance (October 2014), has been used for the liquefaction analysis. The Fines Content (FC) has been calculated in accordance with a Boulanger and Idriss (2014) method-specific FC-I_c correlation assuming a default C_{FC} value of 0.0, as per MBIE (2014) guidelines, and settlements according to Zhang et al. (2002). The software default of 16% was used for the cyclic resistance ratio probability of liquefaction triggering (CRR P_L). The auto transition layer detection was turned off, as per MBIE guidance. The stress exponent calculation and magnitude scaling factor (MSF) were left as defaults. Each raw data record was averaged with the measurement above and below, creating a simple running average of 3, so there was no reduction in number points from the original data. Level ground conditions were assumed, meaning lateral displacements were not calculated as part of this project.

The Boulanger and Idriss (2014) method is a revised version of the Idriss and Boulanger (2008) CPT-based liquefaction-triggering procedure, in that it includes new relationships for the MSF and FC adjustment factors. These revisions were based on findings from experimental (e.g. cyclic laboratory tests), analytical (e.g. analyses of ground motion records) and history case studies from Christchurch (van Ballegooy et al. 2015).

The liquefaction-triggering analyses in the software have been applied to the entire depth (up to 18 m) of the CPTs for this study. Idriss and Boulanger (2008) mention that surface evidence of liquefaction has most commonly been associated with liquefaction occurring at depths of less than ~15 m. This is related to the fact that shallower deposits are typically the youngest and therefore most susceptible to liquefaction. Van Ballegooy et al. (2015) state that a 10 m cut-off depth had a negligible effect on the computed vulnerability parameters they used because the liquefying sediments with low resistance to liquefaction triggering are generally at shallower depths and that depth weighting function in the LSN parameter reduces the impacts of any liquefying soil layers at larger depths. Twenty two (22) of the CPTs analysed in this study are >10 m, with nine of these >15 m deep (Appendix 2); however, their results are not thought to be of lower confidence and have not been scaled.

The overall probability of liquefaction, LSN and expression of liquefaction damage have been calculated for all of the selected CPTs using the parameters specified by MBIE guidance and the software.

7.0 RESULTS

The aim of this project is to test the accuracy/reliability of the existing maps showing areas where potentially damaging liquefaction may occur. While the results are CPT-location-specific, caution is advised against inferring between CPT locations, as the soil behaviour type index obtained from the software analyses may differ between them, thus resulting in a different probability of liquefaction occurring, LSN and/or liquefaction damage.

For all scenarios, the overall probability of liquefaction, the LSN and the expression of liquefaction damage have been determined for all selected 80 CPT locations around Wellington City. For the two fault source scenarios (Wellington-Hutt Valley magnitude 7.5 and Hikurangi-Wellington minimum magnitude 8.1), the 84th-percentile PGA values have been used in the calculations.

7.1 Probability of Liquefaction Occurring

The overall probability of liquefaction occurring in the Wellington City Council jurisdiction, for selected CPTs, is shown in Tables 7.1 and 7.2 and in Figures 7.1–7.3. Approximately three quarters of the CPTs analysed have a high to very high probability of liquefaction occurring at the CPT locations in all three scenarios (Table 7.1); however, the software used for the analyses equates low probability of liquefaction occurring as being <12% and very high >55% (Table 4.4). It is important to note that the absence of liquefaction does not imply that the area is safe from deformation and that these results are site-specific, relating only to the CPT location and not the surrounding area.

Table 7.1 Number and percentage of CPT locations used in this study and the probability of liquefaction occurring at each location for each of the three seismic hazard scenarios in the Wellington City Council area. The probabilities were calculated using the Boulanger and Idriss (2014) liquefaction-triggering assessment method in CLiq software provided by GeoLogismiki.

	Wellington Fault M7.5 Fault Source Scenario		**Hikurangi Margin M8.1 Fault Source Scenario**		**10% in 50 Years Probabilistic Seismic Hazard (Based on an Average M7.3 Earthquake)**	
Probability of Liquefaction Occurring	**Number of CPTs**	**Percentage of CPTs**	**Number of CPTs**	**Percentage of CPTs**	**Number of CPTs**	**Percentage of CPTs**
Low	16	20	15	19	18	23
High	21	26	19	24	27	34
Very High	43	54	46	58	35	44
Total	80	100	80	100	80	100

There is little difference in the probability of liquefaction occurring between the two fault source scenarios, even though the investigation dates of the CPTs vary over the calendar year, which in turn may affect the groundwater levels recorded, which can affect the amount of soil saturation at the CPT location. In the probabilistic scenario, the number of CPT locations with a very high probability of liquefaction occurring decreases, although the number of CPTs with a low probability of liquefaction occurring remains relatively constant.

Table 7.2 Overall probability of liquefaction for selected CPT locations in the Wellington City Council area for specific seismic hazard scenarios. The probabilities were calculated using the Boulanger and Idriss (2014) liquefaction-triggering assessment method in CLiq software provided by GeoLogismiki. The table is ordered by the total depth of the CPT, from shallowest to deepest depth.

			Overall Probability of Liquefaction (Boulanger and Idriss 2014)		
CPT ID	**Investigation Date**	**Total Depth (m bgl)**	**Wellington Fault M7.5 Fault Source Scenario**	**Hikurangi Margin M8.1 Fault Source Scenario**	**10% in 50 Years Probabilistic Seismic Hazard (Based on an Average M7.3 Earthquake)**
CPT_72616	30/08/2015	2.18	Low	Low	Low
CPT_72619	30/08/2015	2.31	Low	Low	Low
CPT_72611	30/08/2015	2.58	Low	Low	Low
CPT_130478	30/06/2017	2.88	Low	Low	Low
CPT_72629	30/08/2015	2.91	Low	Low	Low
CPT_72614	30/08/2015	3.36	Low	Low	Low
CPT_88796	1/09/2016	4.19	Very high	Very high	High
CPT_88795	1/09/2016	4.28	High	Very high	High
CPT_93537	21/03/2017	4.57	High	High	High
CPT_72621	30/08/2015	4.84	High	High	High
CPT_130479	29/06/2017	4.98	High	High	High
CPT_112572	4/09/2018	5.04	High	High	High
CPT_72065	29/01/2013	5.14	Low	Low	Low
CPT_72623	30/08/2015	5.25	Very high	Very high	High
CPT_72630	30/08/2015	5.32	Low	Low	Low
CPT_72034	11/09/2012	5.96	Low	Low	Low
CPT_123512	6/03/2018	6.00	Very high	Very high	Very high
CPT_72620	30/08/2015	6.62	High	High	High
CPT_93540	23/03/2017	6.82	Very high	Very high	Very high
CPT_72613	30/08/2015	6.94	Very high	Very high	Very high
CPT_72061	14/12/2011	7.16	Very high	Very high	Very high
CPT_112575	4/09/2018	7.29	High	High	High
CPT_112603	28/03/2018	7.36	High	High	High
CPT_72622	30/08/2015	7.81	High	High	High
CPT_93536	21/03/2017	7.84	Very high	Very high	Very high
CPT_88792	1/09/2016	7.89	High	High	High
CPT_93583	7/06/2017	7.90	High	High	Low
CPT_93379	20/03/2017	7.92	Very high	Very high	Very high
CPT_90436	9/03/2017	8.00	Very high	Very high	Very high
CPT_72085	30/01/2013	8.01	Very high	Very high	High

			Overall Probability of Liquefaction (Boulanger and Idriss 2014)		
CPT ID	**Investigation Date**	**Total Depth (m bgl)**	**Wellington Fault M7.5 Fault Source Scenario**	**Hikurangi Margin M8.1 Fault Source Scenario**	**10% in 50 Years Probabilistic Seismic Hazard (Based on an Average M7.3 Earthquake)**
CPT_71948	30/09/2014	8.19	Low	Low	Low
CPT_113550	4/02/2016	8.26	High	High	High
CPT_104552	9/02/2018	8.29	High	Very high	High
CPT_112612	4/04/2018	8.30	Very high	Very high	High
CPT_72055	12/09/2012	8.45	High	High	High
CPT_93534	20/03/2017	8.62	Very high	Very high	Very high
CPT_72628	30/08/2015	8.68	Very high	Very high	High
CPT_93542	4/04/2017	8.69	Very high	Very high	Very high
CPT_72680	8/02/2013	8.80	High	High	High
CPT_112573	4/09/2018	8.91	Very high	Very high	High
CPT_93538	21/03/2017	8.93	Very high	Very high	Very high
CPT_71966	23/06/2015	8.95	Low	Low	Low
CPT_72644	7/05/2015	9.00	Very high	Very high	Very high
CPT_72647	5/05/2015	9.00	Low	Low	Low
CPT_137311	20/08/2018	9.01	Very high	Very high	Very high
CPT_72612	30/08/2015	9.03	Very high	Very high	Very high
CPT_93544	5/04/2017	9.10	Very high	Very high	Very high
CPT_72056	11/09/2012	9.15	Low	High	Low
CPT_72659	22/11/2015	9.20	Very high	Very high	High
CPT_72106	24/06/2015	9.31	High	High	Low
CPT_93535	21/03/2017	9.34	Very high	Very high	Very high
CPT_71973	23/06/2015	9.58	Low	Low	Low
CPT_72067	30/01/2013	9.69	Low	Low	Low
CPT_72625	30/08/2015	9.88	Very high	Very high	Very high
CPT_72627	30/08/2015	9.93	Very high	Very high	Very high
CPT_93539	21/03/2017	9.93	Very high	Very high	Very high
CPT_71958	20/06/2015	9.97	Low	Low	Low
CPT_72618	30/08/2015	9.98	High	High	High
CPT_93545	4/04/2017	10.44	Very high	Very high	Very high
CPT_93543	5/04/2017	10.51	Very high	Very high	Very high
CPT_137315	20/08/2018	10.79	Very high	Very high	Very high
CPT_72050	10/09/2012	10.90	Very high	Very high	Very high

CPT ID	Investigation Date	Total Depth (m bgl)	Overall Probability of Liquefaction (Boulanger and Idriss 2014)		
			Wellington Fault M7.5 Fault Source Scenario	Hikurangi Margin M8.1 Fault Source Scenario	10% in 50 Years Probabilistic Seismic Hazard (Based on an Average M7.3 Earthquake)
CPT_72626	30/08/2015	11.91	Very high	Very high	Very high
CPT_72037	10/09/2012	12.04	Very high	Very high	Very high
CPT_72615	30/08/2015	12.12	High	Very high	High
CPT_88794	1/09/2016	12.55	Very high	Very high	Very high
CPT_72617	30/08/2015	12.78	Very high	Very high	Very high
CPT_72053	10/09/2012	12.99	Very high	Very high	Very high
CPT_72057	12/09/2012	13.30	Very high	Very high	High
CPT_72655	23/11/2015	14.40	High	High	High
CPT_72035	11/09/2012	14.60	Very high	Very high	Very high
CPT_72624	30/08/2015	15.00	Very high	Very high	Very high
CPT_118999	27/08/2018	15.40	High	High	High
CPT_72052	10/09/2012	15.50	High	High	High
CPT_136860	5/11/2019	16.14	Very high	Very high	Very high
CPT_136866	5/11/2019	18.15	Very high	Very high	Very high
CPT_136864	5/11/2019	18.23	Very high	Very high	Very high
CPT_136865	5/11/2019	18.65	Very high	Very high	Very high
CPT_136861	5/11/2019	18.74	Very high	Very high	Very high
CPT_136868	5/11/2019	18.77	Very high	Very high	Very high

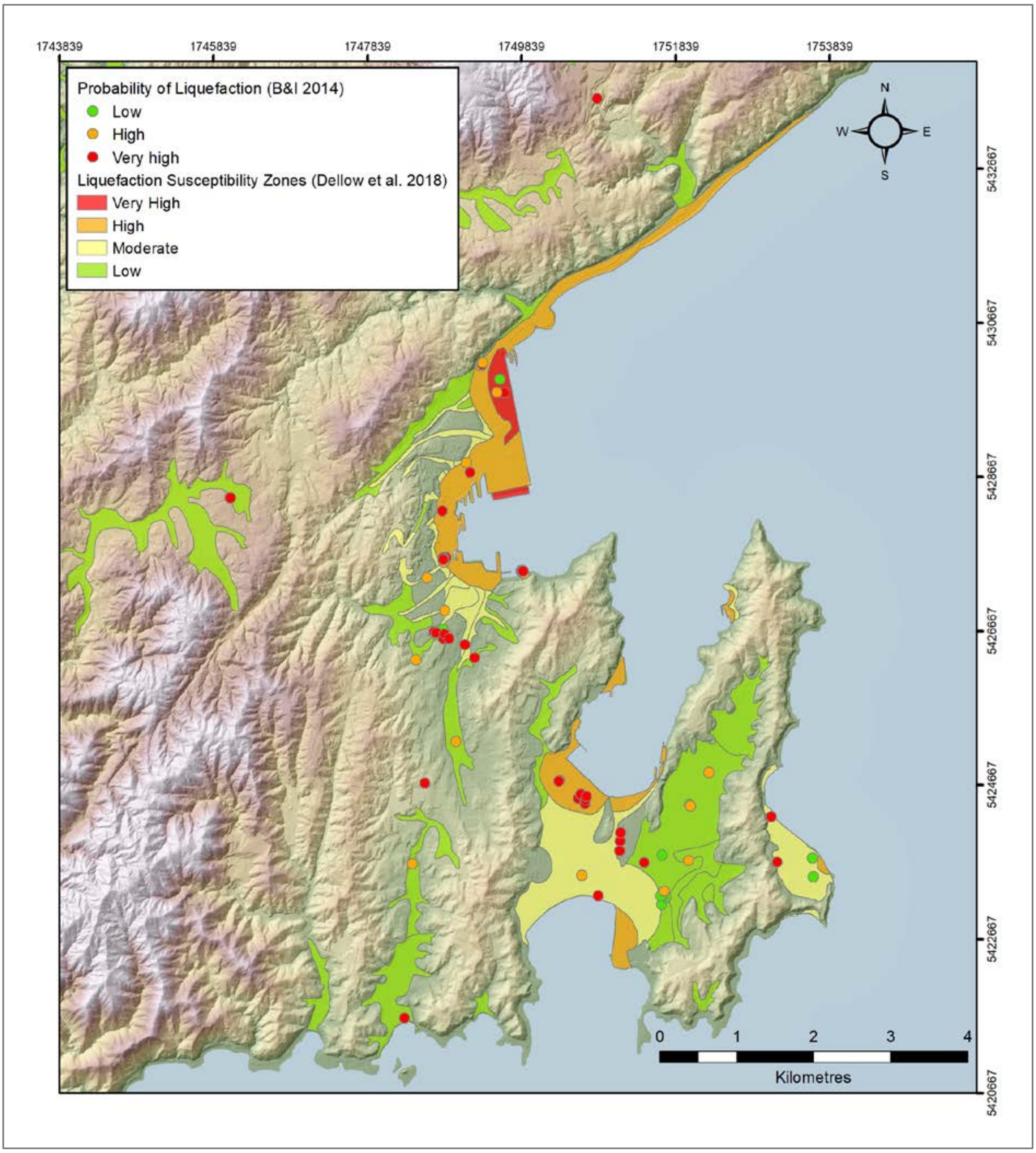

Figure 7.1 Overall probability of liquefaction occuring in selected CPTs in the Wellington City Council jurisdiction, based on an 84th-percentile mean peak ground acceleration scenario hazard for the 2010 NSHM Wellington-Hutt Valley fault source (magnitude 7.5). The probability is calculated using the Boulanger and Idriss (2014) liquefaction-triggering assessment method. It is important to note that the absence of liquefaction does not imply that the area is safe from deformation.

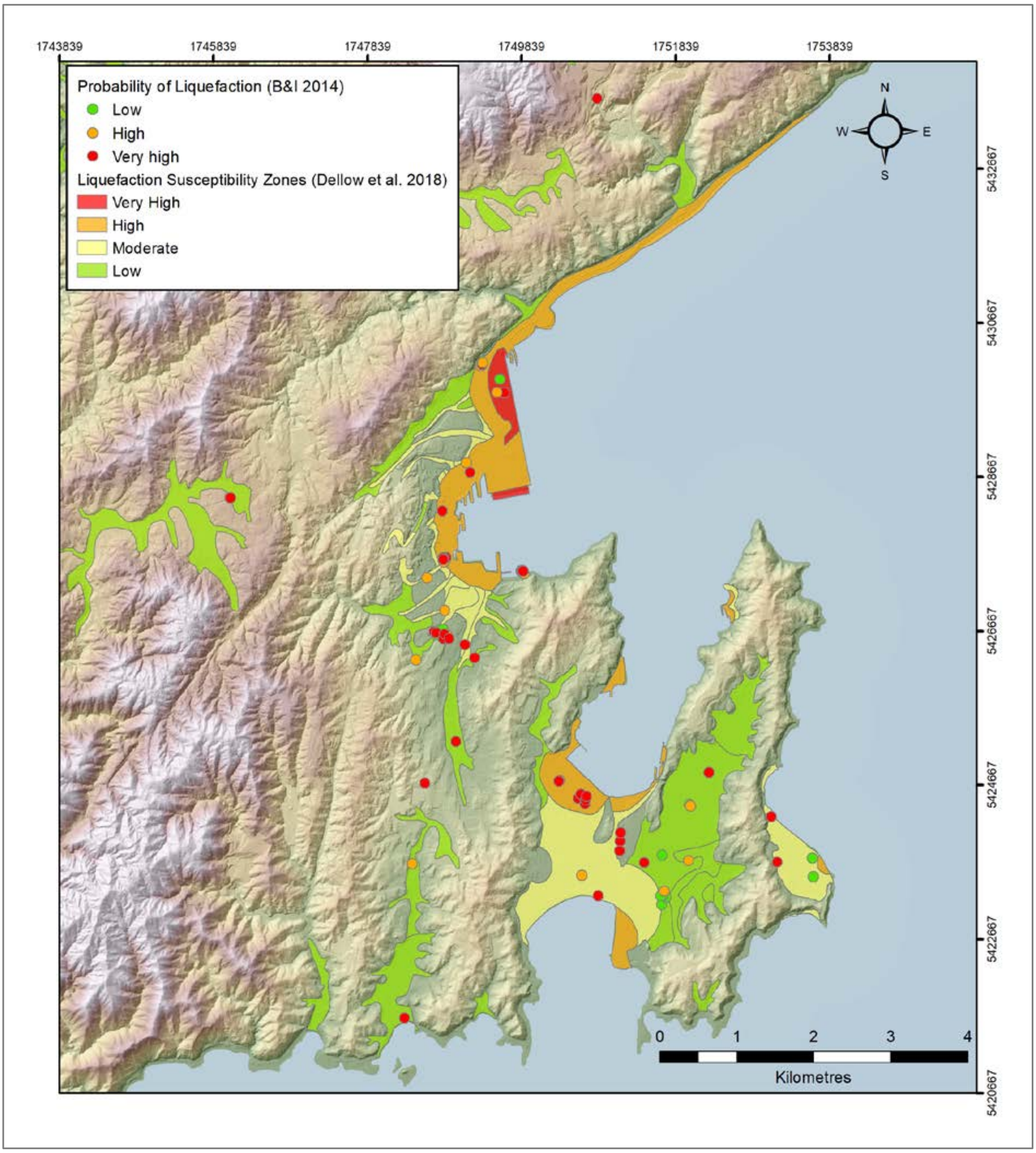

Figure 7.2 Overall probability of liquefaction occuring in selected CPTs in the Wellington City Council jurisdiction, based on an 84th-percentile mean peak ground acceleration scenario hazard for the 2010 NSHM Hikurangi-Wellington minimum source (magnitude 8.1). The probability is calculated using the Boulanger and Idriss (2014) liquefaction-triggering assessment method. It is important to note that the absence of liquefaction does not imply that the area is safe from deformation.

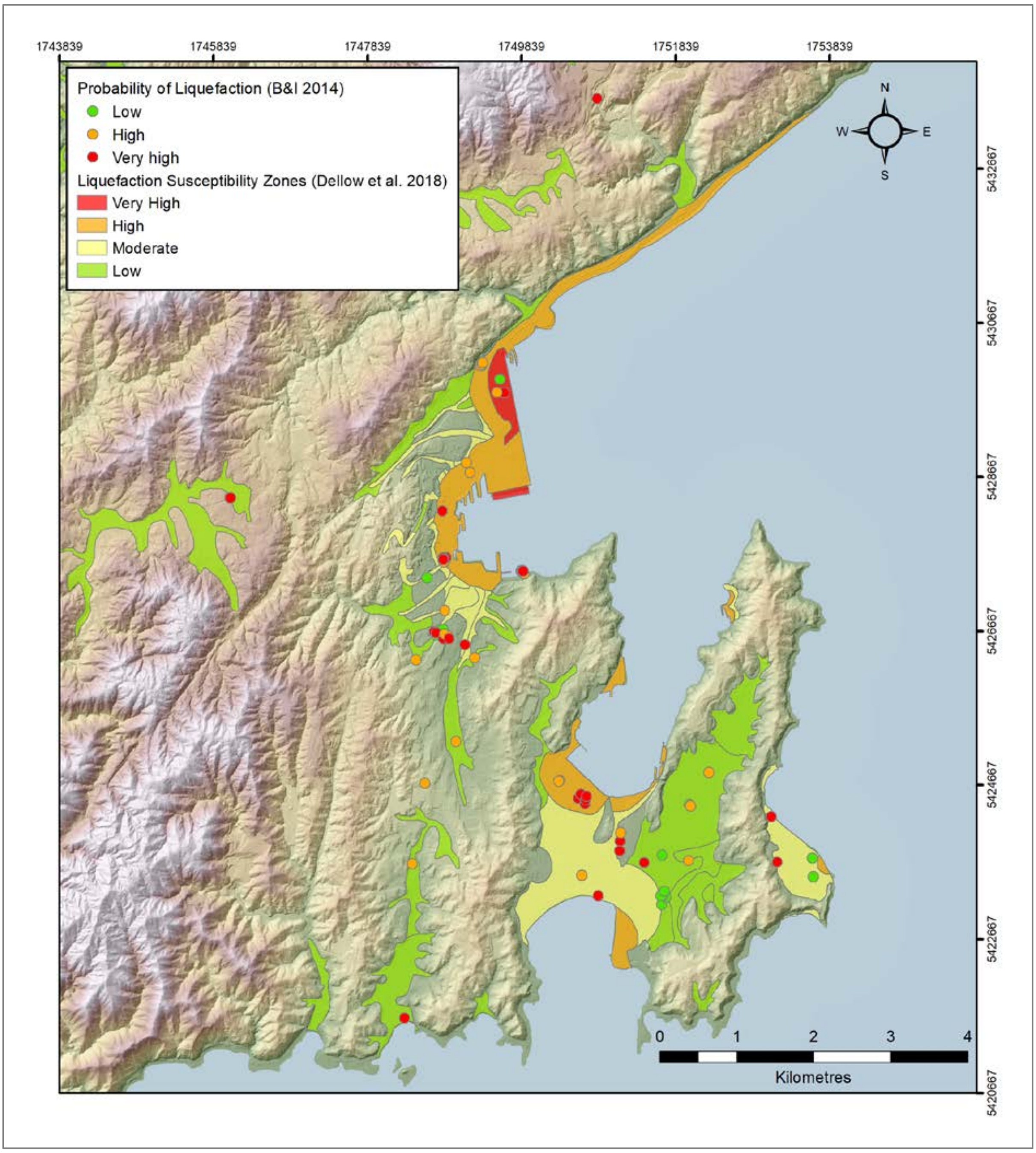

Figure 7.3 Overall probability of liquefaction occuring in selected CPTs in the Wellington City Council jurisdiction, based on a 10% in 50 years probabilistic seismic hazard for PGA with the associated average magnitude (7.3). The probability is calculated using the Boulanger and Idriss (2014) liquefaction-triggering assessment method. It is important to note that the absence of liquefaction does not imply that the area is safe from deformation.

The areas where the high to very high probability of liquefaction occurs mostly agree with the high to very high liquefaction susceptibility zones of Dellow et al. (2018), especially around the Wellington waterfront and airport areas. However, it is important to note that the Dellow et al. (2018) maps are more generalised, as they used the historical responses of underlying geological units to strong ground shaking to quantify the susceptibility zones (their Tables 5.1 to 5.4), as opposed to the more site-specific CPT soil behaviour type index used in the Boulanger and Idriss (2014) liquefaction-triggering assessment method used in this study.

The low-probability CPT (#72647) located on the northern end of the waterfront has a clay-like soil behaviour type index (I_c >2.60) over much of the CPT depth, making it more cohesive than looser sandier soils and less likely to liquefy. Dellow et al. (2018) assigned a very high liquefaction susceptibility zone to this site, as the area consists of anthropogenic fill that includes seabed sand and mud that was hydraulically pumped in behind retaining walls (Begg and Mazengarb 1996; Semmens et al. 2010; Townsend et al. 2020). CPT analyses show that parts of the area may have a greater percentage of non-liquefiable silts and clays. Dhakal et al. (2020) report that, in the CentrePort area, just south of CPT#72647, some of their CPTs were acquired in hydraulic fills and had a greater percentage of non-liquefiable silts and clays with I_c >2.6. A lack of ejecta material observed on the ground surface following the Kaikōura earthquake is consistent and expected due to the majority of non-liquefiable material (i.e. clay) within the fill deposits in this area (Dhakal et al. 2020). However, after the 2016 Kaikōura earthquake, the CentrePort area did sustain damage from ground shaking and suffer lateral spreading (Cubrinovski et al. 2017, 2018).

7.2 Liquefaction Severity Number and Expected Land Damage

The LSN was developed by Tonkin & Taylor (2013) and used to determine the level of liquefaction damage that may occur to flat land. The LSN integrates the volumetric strain values of saturated layers in the sediments up to 10 m deep in the CPT. The depth of the CPT is governed by how far it can be pushed into the ground. Depth refusal may be due to hard ground (e.g. rock). Liquefiable layers may exist below the extent of the CPT, underestimating the potential for liquefaction to occur or the severity of the liquefaction.

The LSN values and level of liquefaction damage (based on the LSN values) for the CPTs used in this study, for all three scenarios, are summarised in Table 7.3 and shown in more detail in Table 7.4 and in Figures 7.4–7.6. The LSN and damage values below are obtained from the CLiq software analyses (Table 4.5), following the calculation of the overall probability of liquefaction occurring using the Boulanger and Idriss (2014) liquefaction-triggering assessment methodology and MBIE guidance (October 2014).

Table 7.3 Number and percentage of CPT locations used in this study, and the Liquefaction Severity Number (LSN) occurring at each location for each of the three seismic hazard scenarios in the Wellington City Council area. The LSNs were calculated using the Boulanger and Idriss (2014) liquefaction-triggering assessment method in CLiq software provided by GeoLogismiki.

	Wellington Fault M7.5 Fault Source Scenario		**Hikurangi Margin M8.1 Fault Source Scenario**		**10% in 50 Years Probabilistic Seismic Hazard (Based on an Average M7.3 Earthquake)**	
LSN Range	**Number of CPTs**	**Percentage of CPTs**	**Number of CPTs**	**Percentage of CPTs**	**Number of CPTs**	**Percentage of CPTs**
0–10	26	33	23	29	28	35
10–20	20	25	23	29	18	23
20–30	12	15	10	13	12	15
30–40	13	16	14	18	13	16
40–50	6	8	7	9	6	8
50+	3	4	3	4	3	4
Total	80	100	80	100	80	100

The LSN values are grouped together by the software to indicate the level of liquefaction damage expected, with the lower values (<10) equating to little or no damage ranging up to values of 50+, which indicate severe damage (Tables 4.5, 7.3 and 7.4). As mentioned in Section 4.2, the depth-weighted LSN recognises that ground surface damage from shallow liquefied layers is more likely than from deeper layers; however, liquefaction may still occur in deeper layers and not manifest itself at the surface.

Liquefaction severity and associated ground damage varies across the Wellington City Council jurisdiction; however, there is little difference (<3) in the LSN values between the two fault source scenarios (Table 7.4), with some CPTs having slightly higher LSN values in the Hikurangi Margin fault source scenario. As a result of this increase, the expression of liquefaction damage at these CPT locations also rises (Table 7.4 and Figures 7.4–7.6).

Table 7.4 Liquefaction Severity Numbers (LSN) and expression of liquefaction damage for this study's CPT locations in the Wellington City Council area for a probabilistic and two specific fault source scenarios. The 84th-percentile peak ground acceleration values were used in the calculations. The LSNs and associated expressions of liquefaction damage were obtained from the CLiq software, provided by GeoLogismiki, following the calculation of the probability of liquefaction occurring using the Boulanger and Idriss (2014) liquefaction-triggering assessment. The liquefaction expressions of damage relate directly to the LSN (see Table 4.5 for more detail). The table is ordered by the total depth (in metres) of the CPT, from shallowest to deepest depth.

			Overall Liquefaction Severity Number			**Overall Expression of Liquefaction**		
CPT ID	**Investigation Date**	**Total Depth (m bgl)**	**Wellington Fault M7.5 Fault Source Scenario**	**Hikurangi Margin M8.1 Fault Source Scenario**	**10% in 50 Years Probabilistic Seismic Hazard (Based on an Average M7.3 Earthquake)**	**Wellington Fault M7.5 Fault Source Scenario**	**Hikurangi Margin M8.1 Fault Source Scenario**	**10% in 50 Years Probabilistic Seismic Hazard (Based on an Average M7.3 Earthquake)**
CPT_72616	30/08/2015	2.18	4	5	4	Little	Little	Little
CPT_72619	30/08/2015	2.31	6	7	6	Little	Little	Little
CPT_72611	30/08/2015	2.58	9	10	9	Little	Little	Little
CPT_130478	30/06/2017	2.88	6	7	6	Little	Little	Little
CPT_72629	30/08/2015	2.91	10	11	10	Minor	Minor	Minor
CPT_72614	30/08/2015	3.36	0.0	0.2	0.0	Little	Little	Little
CPT_88796	1/09/2016	4.19	20	20	20	Moderate	Moderate	Minor
CPT_88795	1/09/2016	4.28	18	18	17	Minor	Minor	Minor
CPT_93537	21/03/2017	4.57	31	32	31	Moderate to major	Moderate to major	Moderate to major
CPT_72621	30/08/2015	4.84	24	24	23	Moderate	Moderate	Moderate
CPT_130479	29/06/2017	4.98	19	19	18	Minor	Minor	Minor
CPT_112572	4/09/2018	5.04	13	13	12	Minor	Minor	Minor
CPT_72065	29/01/2013	5.14	0.4	0.5	0.2	Little	Little	Little
CPT_72623	30/08/2015	5.25	24	24	24	Moderate	Moderate	Moderate
CPT_72630	30/08/2015	5.32	1	2	2	Little	Little	Little

			Overall Liquefaction Severity Number			Overall Expression of Liquefaction		
CPT ID	**Investigation Date**	**Total Depth (m bgl)**	**Wellington Fault M7.5 Fault Source Scenario**	**Hikurangi Margin M8.1 Fault Source Scenario**	**10% in 50 Years Probabilistic Seismic Hazard (Based on an Average M7.3 Earthquake)**	**Wellington Fault M7.5 Fault Source Scenario**	**Hikurangi Margin M8.1 Fault Source Scenario**	**10% in 50 Years Probabilistic Seismic Hazard (Based on an Average M7.3 Earthquake)**
CPT_72034	11/09/2012	5.96	2	2	2	Little	Little	Little
CPT_123512	6/03/2018	6.00	31	32	31	Moderate to major	Moderate to major	Moderate to major
CPT_72620	30/08/2015	6.62	22	23	22	Moderate	Moderate	Moderate
CPT_93540	23/03/2017	6.82	39	41	38	Moderate to major	Major	Moderate to major
CPT_72613	30/08/2015	6.94	29	32	29	Moderate	Moderate to major	Moderate
CPT_72061	14/12/2011	7.16	18	18	18	Minor	Minor	Minor
CPT_112575	4/09/2018	7.29	15	16	14	Minor	Minor	Minor
CPT_112603	28/03/2018	7.36	9	9	9	Little	Little	Little
CPT_72622	30/08/2015	7.81	22	22	22	Moderate	Moderate	Moderate
CPT_93536	21/03/2017	7.84	36	36	35	Moderate to major	Moderate to major	Moderate to major
CPT_88792	1/09/2016	7.89	10	11	9	Minor	Minor	Little
CPT_93583	7/06/2017	7.90	6	7	6	Little	Little	Little
CPT_93379	20/03/2017	7.92	57	57	57	Severe	Severe	Severe
CPT_90436	9/03/2017	8.00	18	18	18	minor	Minor	Minor
CPT_72085	30/01/2013	8.01	14	15	13	minor	Minor	Minor
CPT_71948	30/09/2014	8.19	4	4	4	Little	Little	Little
CPT_113550	4/02/2016	8.26	6	6	6	Little	Little	Little
CPT_104552	9/02/2018	8.29	12	13	10	Minor	Minor	Minor

			Overall Liquefaction Severity Number			Overall Expression of Liquefaction		
CPT ID	**Investigation Date**	**Total Depth (m bgl)**	**Wellington Fault M7.5 Fault Source Scenario**	**Hikurangi Margin M8.1 Fault Source Scenario**	**10% in 50 Years Probabilistic Seismic Hazard (Based on an Average M7.3 Earthquake)**	**Wellington Fault M7.5 Fault Source Scenario**	**Hikurangi Margin M8.1 Fault Source Scenario**	**10% in 50 Years Probabilistic Seismic Hazard (Based on an Average M7.3 Earthquake)**
CPT_112612	4/04/2018	8.30	14	14	14	Minor	Minor	Minor
CPT_72055	12/09/2012	8.45	5	6	5	Little	Little	Little
CPT_93534	20/03/2017	8.62	57	57	55	Severe	Severe	Severe
CPT_72628	30/08/2015	8.68	15	17	13	Minor	Minor	Minor
CPT_93542	4/04/2017	8.69	22	23	22	Moderate	Moderate	Moderate
CPT_72680	8/02/2013	8.80	9	9	8	Little	Little	Little
CPT_112573	4/09/2018	8.91	15	15	14	Minor	Minor	Minor
CPT_93538	21/03/2017	8.93	47	47	46	Major	Major	Major
CPT_71966	23/06/2015	8.95	2	2	2	Little	Little	Little
CPT_72644	7/05/2015	9.00	18	18	18	Little	Little	Minor
CPT_72647	5/05/2015	9.00	4	4	4	Minor	Minor	Little
CPT_137311	20/08/2018	9.01	26	26	26	Moderate	Moderate	Moderate
CPT_72612	30/08/2015	9.03	33	35	33	Moderate to major	Moderate to major	Moderate to major
CPT_93544	5/04/2017	9.10	36	36	36	Moderate to major	Moderate to major	Moderate to major
CPT_72056	11/09/2012	9.15	3	3	2	Little	Little	Little
CPT_72659	22/11/2015	9.20	19	19	18	Minor	Minor	Little
CPT_72106	24/06/2015	9.31	9	12	8	Little	Minor	Little
CPT_93535	21/03/2017	9.34	59	60	59	Severe	Severe	Severe

			Overall Liquefaction Severity Number			Overall Expression of Liquefaction		
CPT ID	**Investigation Date**	**Total Depth (m bgl)**	**Wellington Fault M7.5 Fault Source Scenario**	**Hikurangi Margin M8.1 Fault Source Scenario**	**10% in 50 Years Probabilistic Seismic Hazard (Based on an Average M7.3 Earthquake)**	**Wellington Fault M7.5 Fault Source Scenario**	**Hikurangi Margin M8.1 Fault Source Scenario**	**10% in 50 Years Probabilistic Seismic Hazard (Based on an Average M7.3 Earthquake)**
CPT_71973	23/06/2015	9.58	0.0	0.3	0.0	Little	Little	Little
CPT_72067	30/01/2013	9.69	2	2	2	Little	Little	Little
CPT_72625	30/08/2015	9.88	36	37	36	Moderate to major	Moderate to major	Moderate to major
CPT_72627	30/08/2015	9.93	32	34	32	Moderate to major	Moderate to major	Moderate to major
CPT_93539	21/03/2017	9.93	47	47	47	Major	Major	Major
CPT_71958	20/06/2015	9.97	2	3	2	Little	Little	Little
CPT_72618	30/08/2015	9.98	4	6	4	Little	Little	Little
CPT_93545	4/04/2017	10.44	38	38	37	Moderate to major	Moderate to major	Moderate to major
CPT_93543	5/04/2017	10.51	43	43	42	Major	Major	Major
CPT_137315	20/08/2018	10.79	28	28	28	Moderate	Moderate	Moderate
CPT_72050	10/09/2012	10.90	12	12	12	Minor	Minor	Minor
CPT_72626	30/08/2015	11.91	33	35	33	Moderate to major	Moderate to major	Moderate to major
CPT_72037	10/09/2012	12.04	19	20	19	Minor	Minor	Minor
CPT_72615	30/08/2015	12.12	14	16	13	Minor	Minor	Minor
CPT_88794	1/09/2016	12.55	24	25	24	Moderate	Moderate	Moderate
CPT_72617	30/08/2015	12.78	47	47	47	Major	Major	Major
CPT_72053	10/09/2012	12.99	22	22	22	Moderate	Moderate	Moderate
CPT_72057	12/09/2012	13.30	15	15	14	Minor	Minor	Minor

CPT ID	Investigation Date	Total Depth (m bgl)	Overall Liquefaction Severity Number			Overall Expression of Liquefaction		
			Wellington Fault M7.5 Fault Source Scenario	Hikurangi Margin M8.1 Fault Source Scenario	10% in 50 Years Probabilistic Seismic Hazard (Based on an Average M7.3 Earthquake)	Wellington Fault M7.5 Fault Source Scenario	Hikurangi Margin M8.1 Fault Source Scenario	10% in 50 Years Probabilistic Seismic Hazard (Based on an Average M7.3 Earthquake)
CPT_72655	23/11/2015	14.40	4	4	4	Little	Little	Little
CPT_72035	11/09/2012	14.60	19	20	19	Minor	Minor	Minor
CPT_72624	30/08/2015	15.00	44	44	43	Major	Major	Major
CPT_118999	27/08/2018	15.40	11	12	10	Minor	Minor	Little
CPT_72052	10/09/2012	15.50	8	9	8	Little	Little	Little
CPT_136860	5/11/2019	16.14	48	48	46	Major	Major	Major
CPT_136866	5/11/2019	18.15	33	35	32	Moderate to major	Moderate to major	Moderate to major
CPT_136864	5/11/2019	18.23	40	40	40	Major	Major	Moderate to major
CPT_136865	5/11/2019	18.65	24	24	24	Moderate	Moderate	Moderate
CPT_136861	5/11/2019	18.74	30	31	29	Moderate	Moderate to major	Moderate
CPT_136868	5/11/2019	18.77	37	39	37	Moderate to major	Moderate to major	Moderate to major

The LSN and liquefaction damage maps are similar for all three scenarios (Figures 7.4–7.6), with only minor differences in some areas. The maps show little to minor liquefaction (LSN 0–20) is likely to occur in areas that are hydraulically filled (e.g. northern parts of the waterfront and around the stadium), inland (e.g. Te Aro / Mount Victoria / Mount Cook areas) and away from the shoreline (e.g. Miramar peninsula). Dellow et al. (2018) suggest that fills associated with harbour reclamation and infilling of stream valleys are potentially vulnerable to liquefaction-induced ground-damage at shaking intensities of MM6–7 or greater, with liquefaction confined to specific areas (e.g. reclamation areas created by pumping uncompacted harbour muds into confined areas) at lower levels of shaking (MM6 or MM7). The LSN values for the areas just north and west of CentrePort correlate well with those calculated by Dhakal et al. (2020). Parts of Seatoun, especially where there is reclaimed land, may also be vulnerable to minor liquefaction damage, although historical evidence shows that this may only occur at shaking intensities of MM9 or greater (Dellow et al. 2018). During the 2013 M_w6.6 Cook Strait and Lake Grassmere earthquakes and the 2016 M_w7.8 Kaikōura earthquakes, it was suggested that parts of Wellington reached shaking intensities of MM7, with liquefaction and associated lateral spreading occurring around CentrePort (Hancox et al. 2013; van Dissen et al. 2013; Cubrinovski et al. 2017; Dhakal et al. 2020) and around the waterfront (Orense et al. 2017). The maps presented here show little to minor liquefaction damage in the CPTs to the west of CentrePort, which correlates well with the minor liquefaction damage observed at CentrePort following the 2013 earthquakes (Cubrinovski et al. 2018; Dhakal et al. 2020). However, the CPTs used by Dhakal et al. (2020) were not available in the NZGD for use in this study, so a direct comparison between the two studies was unable to be carried out.

Moderate to major liquefaction damage (LSN 20–40) may occur in reclaimed land along the waterfront (e.g. Queens Wharf to Oriental Bay) and around the airport (e.g. Kilbirnie, Lyall Bay-Airport-Moa Point areas). Eastern areas (e.g. Worser Bay and inland areas of Seatoun) may also experience moderate to major liquefaction damage. Moderate liquefaction damage is also possible in the Island Bay-Berhampore areas. Alluvium, mixed fill and marginal marine (mostly beach sand and gravels) sediments underlie these areas (Begg and Mazengarb 1996; Townsend et al. 2020) and, in this study, the CPTs in these areas typically have a normalised soil behaviour type index I_c <2.6, suggesting that they are non-cohesive and likely to liquefy. Dellow et al. (2018) mention that none of these fills have been tested at stronger (i.e. MM8+) shaking levels; however, historical case studies from New Zealand and overseas suggest that reclamations on shallow marine sediments are vulnerable to liquefaction ground damage at moderate to strong levels of earthquake shaking (MM8–MM9).

Major to severe liquefaction damage (LSN 40+) is rare in the Wellington City area and may occur in low-lying areas near the coast (e.g. Te Aro, Island Bay, Kilbirnie and Miramar) or in close proximity to the Wellington Fault (e.g. Raroa). In Te Aro, the CPTs with expected major to severe damage are all located near the waterfront on pre-1938 fill, while harbour fill and reclamation sediments underlie the Kilbirnie CPT (Begg and Mazengarb 1996; Townsend et al. 2020). Sand and gravel underlie the Island Bay CPT on the south coast, while, at Miramar, the CPT overlies alluvium (Begg and Mazengarb 1996; Townsend et al. 2020). To the north, overlooking State Highway 2, the Raroa CPT is situated in mixed fill / rock (Townsend et al. 2020). While most liquefaction studies have been on non-cohesive sandy sediments, little is known about the effect of liquefaction in gravelly soils (e.g. Dhakal et al. 2019; Rhodes et al. 2019; Dhakal et al. 2020); however, studies are currently underway to rectify this.

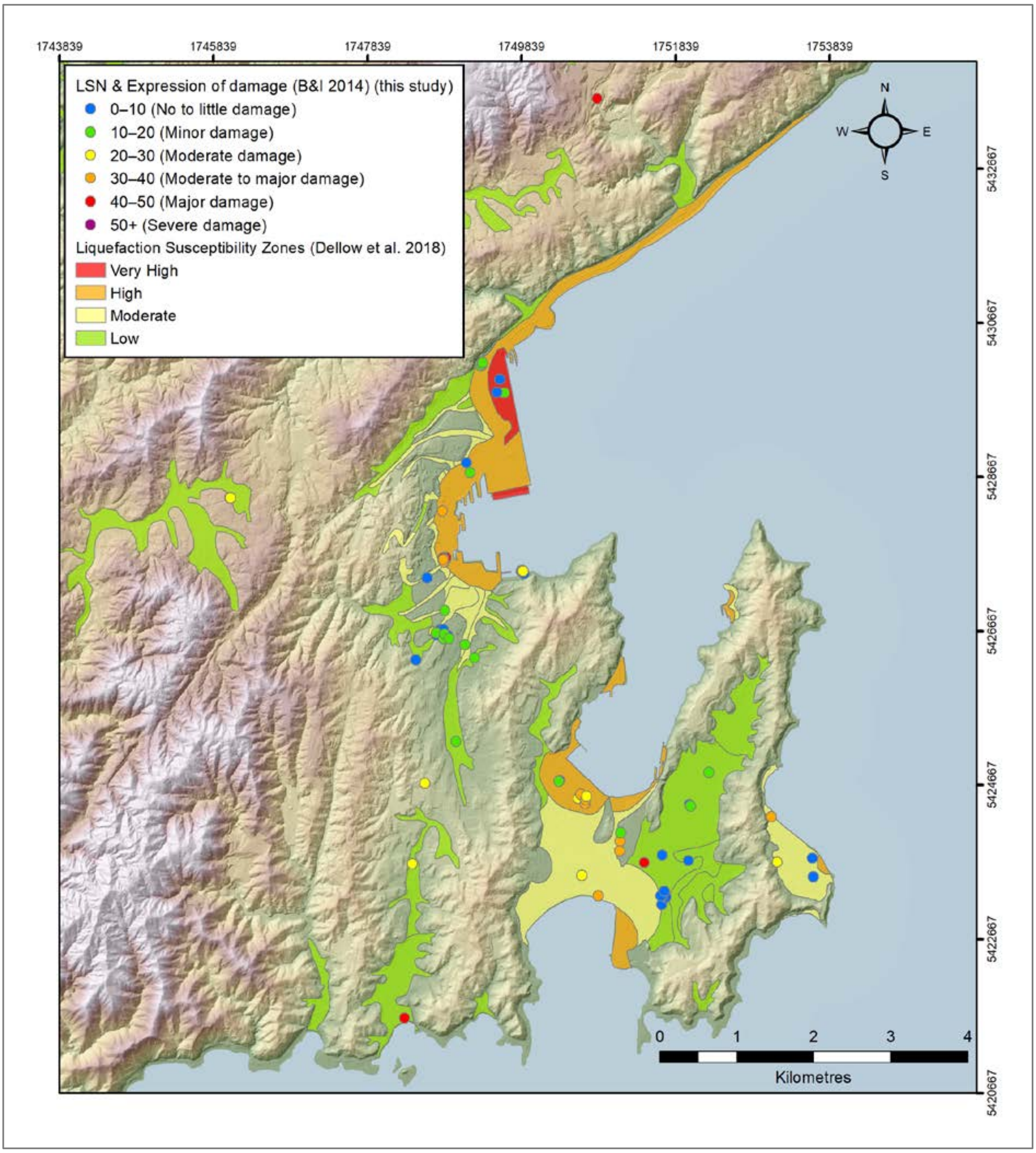

Figure 7.4 Overall Liquefaction Severity Numbers (LSN) and their related expression of liquefaction damage calculated for the CPTs in the Wellington City Council jurisdiction used in this study, based on an 84th-percentile mean peak ground acceleration scenario hazard for the 2010 NSHM Wellington-Hutt Valley fault source (magnitude 7.5). The LSN is derived from the Boulanger and Idriss (2014) liquefaction-triggering assessment method. It is important to note that the absence of liquefaction does not imply that the area is safe from deformation.

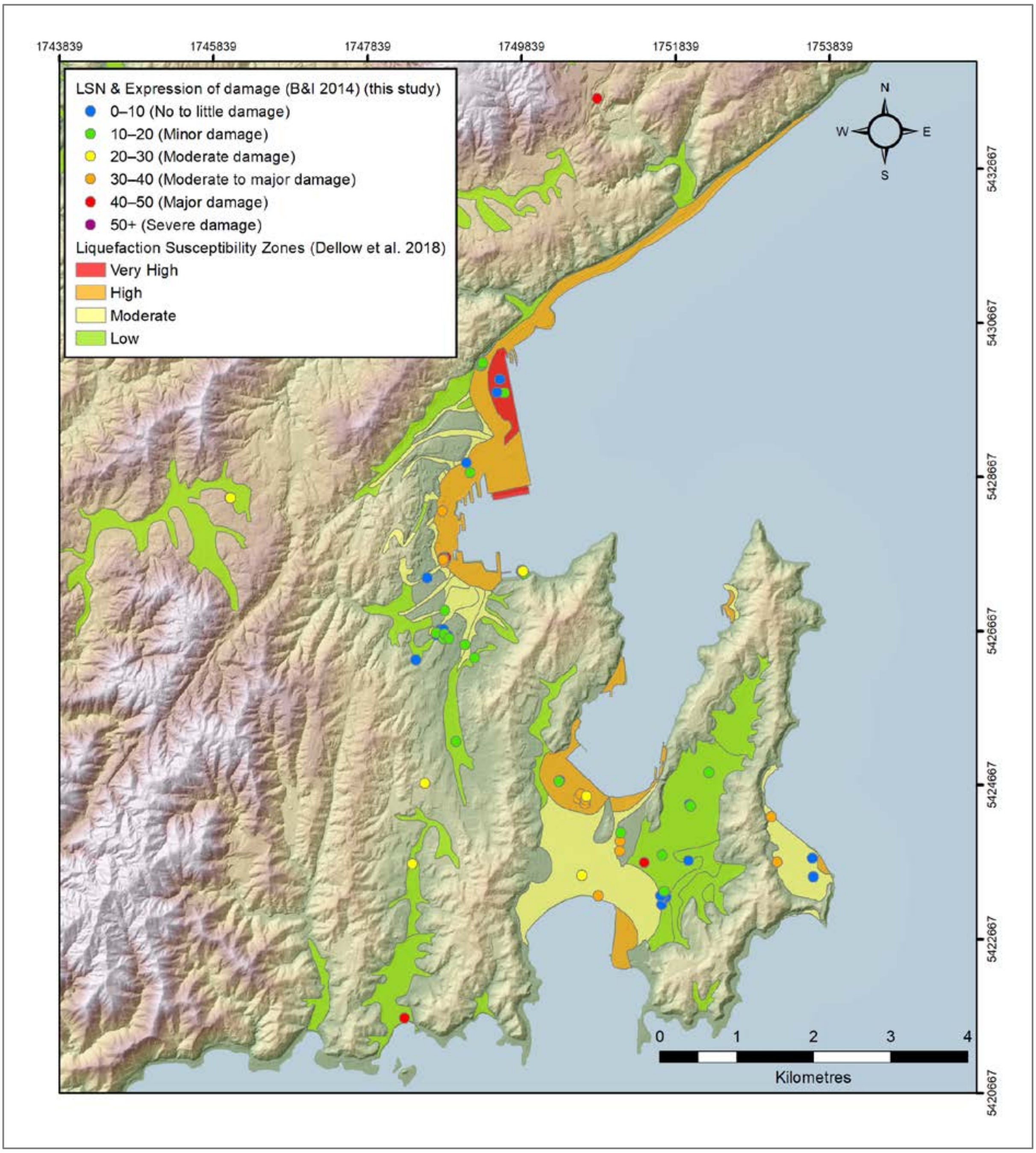

Figure 7.5 Overall Liquefaction Severity Numbers (LSN) and their related expression of liquefaction damage calculated for the CPTs in the Wellington City Council jurisdiction used in this study, based on an 84th-percentile mean peak ground acceleration scenario hazard for the 2010 NSHM Hikurangi-Wellington minimum source (magnitude 8.1). The LSN is derived from the Boulanger and Idriss (2014) liquefaction-triggering assessment method. It is important to note that the absence of liquefaction does not imply that the area is safe from deformation.

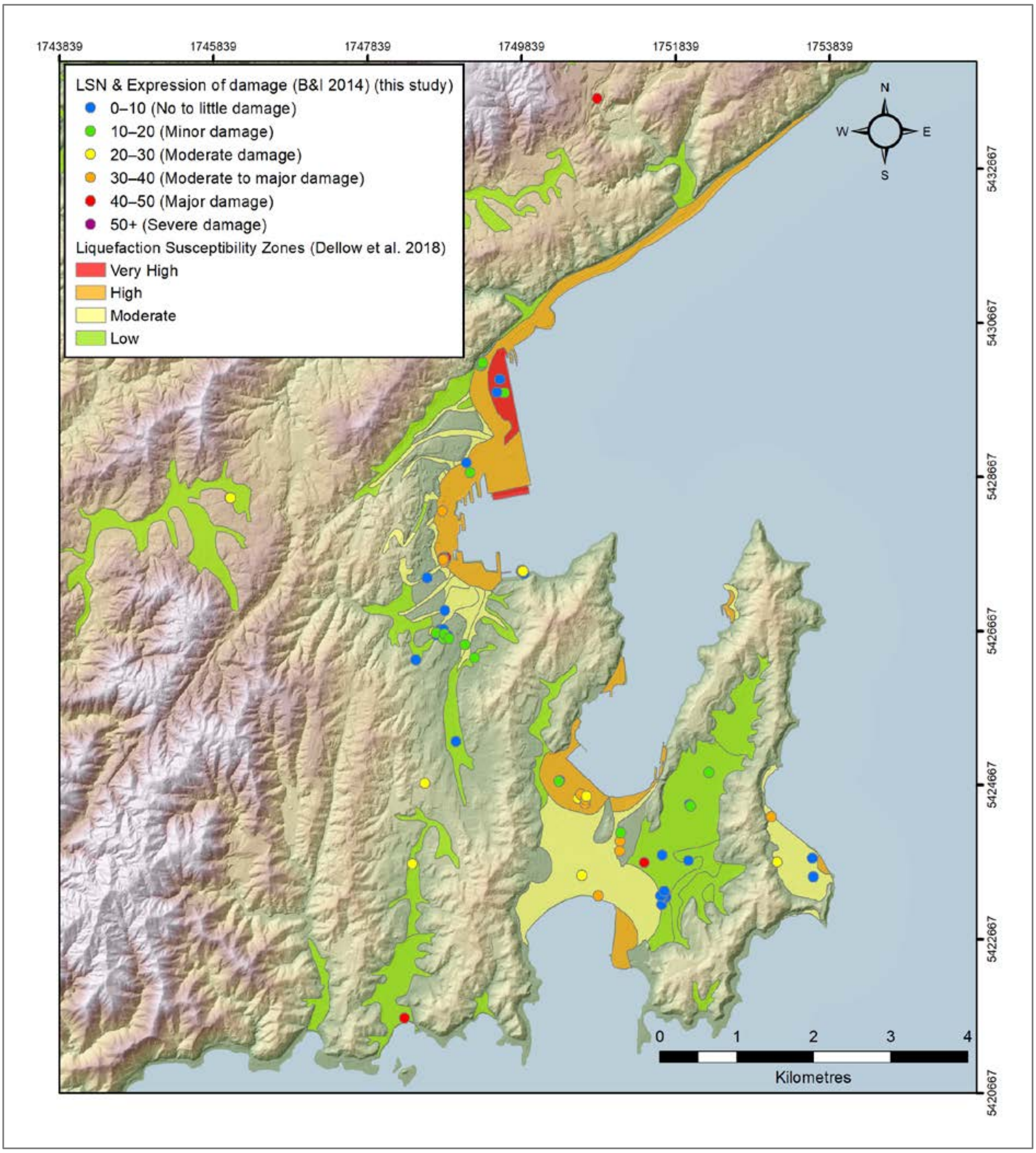

Figure 7.6 Overall Liquefaction Severity Numbers (LSN) and their related expression of liquefaction damage calculated for the CPTs in the Wellington City Council jurisdiction used in this study, based on a 10% in 50 years probabilistic seismic hazard for PGA with the associated average magnitude (7.3). The LSN is derived from the Boulanger and Idriss (2014) liquefaction-triggering assessment method. It is important to note that the absence of liquefaction does not imply that the area is safe from deformation.

8.0 ENGINEERING DESIGN OPTIONS

The results of this study, in particular the LSN and the expression of potential liquefaction damage for each of the CPTs, allows for the liquefaction susceptibility classes of Dellow et al. (2018) to be further refined in terms of the requirements of the Building Act 2004. MBIE societies such as the Structural Engineering Society of New Zealand (SESOC), the New Zealand Society for Earthquake Engineering (NZSEE) and the New Zealand Geotechnical Society (NZGS), as well as consultancy firms such as Tonkin & Taylor, all provide guidance and have a range of publications available to assist in earthquake and seismic engineering design options.

MBIE guidance publications (e.g. MBIE 2015) provide a set of principles to assist engineers in the interpretation and implementation of the proposed MBIE foundation solutions in accordance with the New Zealand Building Code. NZGS has published a series of earthquake geotechnical engineering guidelines, including liquefaction assessment evaluation (NZGS and MBIE 2016), while the New Zealand Transport Agency (NZTA) has published the Bridge Manual (NZTA 2018), which includes design and assessment criteria under both non-seismic and earthquake shaking for bridge building, including mitigation measures for liquefaction.

9.0 SUMMARY AND CONCLUSIONS

Liquefaction is a damaging effect related to strong earthquake shaking and was relatively unknown in New Zealand until the 2010–2011 Canterbury Earthquake Sequence. In general, the stronger the earthquake shaking, the more damaging the liquefaction in terms of both severity and extent. The effects of liquefaction result in vertical and horizontal ground displacements and lateral spreading, which in turn can affect infrastructure (e.g. roads, pipes) and building foundations. The Wellington region has experienced earthquake shaking strong enough to cause liquefaction on at least five occasions (1848, 1855, 1942, 2013 and 2016), whereby the extent and severity of the liquefaction was proportional to the level of shaking experienced at a site.

For liquefaction to occur at a specific site, three conditions need to be meet. The source material must be non-cohesive fine-grained sediment, the sediment must be loosely packed and ideally less than 10,000 years old (Holocene age) to negate the effects of consolidation that occurs over longer periods of time and the sediments must be fully saturated (i.e. lie below the water table). Such sites where all three criteria are met include areas of low-energy deposition where overbank flood deposits accumulate or where rivers and streams form lagoons and estuaries prior to discharge into the sea (or lakes). Engineered (and un-engineered) fills constructed on these deposits are also susceptible to liquefaction. This is consistent with the observations of liquefaction in the Wellington region. Geological maps indicate that the areas that fulfil these conditions are located around the Wellington waterfront, Wellington airport and other areas (e.g. Miramar Peninsula and Seatoun).

This report was commissioned by the Wellington City Council to test the accuracy/reliability of the existing maps of Dellow et al. (2018), which show areas where potentially damaging liquefaction might occur. Since the 2010–2011 Canterbury Earthquake Sequence, a lot of research has been focused on liquefaction and it's triggering conditions. The liquefaction susceptibility maps of Dellow et al. (2018) are qualitative and based on historical accounts of liquefaction and the most up-to-date geological maps and limited subsurface data, and are suitable for use at a regional or district level. In addition, they do not include the geotechnical and sub-surface data that has been made publicly available since 2016. This project is more quantitative, using publicly available CPT (obtained between 2012 and 2019) and LiDAR, and groundwater data and PGA estimates for two earthquake scenarios possible for the Wellington region. The Boulanger and Idriss (2014) liquefaction-triggering assessment method, in conjunction with MBIE guidance, has been applied to the data from selected CPTs in the Wellington City Council jurisdiction. As a result, the maps produced in this study are more site-specific and fulfil the requirements for 'Level B – Calibrated Desktop Assessment' as specified in the 2017 MBIE document 'Planning and engineering guidance for potentially liquefaction-prone land Resource Management Act and Building Act aspects'.

The maps produced for this study show the probability of liquefaction occurring; the LSN, which is commonly used in New Zealand to assess the vulnerability of land to liquefaction-induced damage; and the expression of liquefaction damage at selected CPT locations across Wellington City. Lateral spreading was not looked at in this study. Results indicate that the maps are in general agreement with those of Dellow et al. (2018), except at the CentrePort area where the CPTs suggest lower levels of susceptibility, albeit a difference in terminology (probability of liquefaction occurring versus liquefaction susceptibility). There is also little difference in the probability of liquefaction occurring between the 10% in 50 years probabilistic and the two fault source scenarios (magnitudes 7.5 and 8.1). However, the probability of liquefaction does not always translate into the expected level of liquefaction severity or damage (e.g. a very high probability does not always equate to moderate to severe damage) as liquefaction severity is directly related to the thickness of the liquefiable layers (e.g. a probability could be high but, if the layer is thin, the LSN will be small). It is important to remember that a very high probability of liquefaction occurring does not specify if the degree of liquefaction-induced damage will be mild, moderate or severe.

10.0 FURTHER RECOMMENDATIONS

The probability of liquefaction maps presented in this report are more site-specific than the liquefaction susceptibility maps by Dellow et al. (2018) currently in the Wellington City Council District Plan, and add quantitative data to the existing qualitative maps. The maps use selected CPT locations and other data analysed using standard geotechnical liquefaction software to determine the probability of liquefaction and the possible severity of damage occurring at specific locations around Wellington City.

These maps may be used as a guide to where potentially damaging liquefaction may occur in Wellington City and meet the requirements for a Level B liquefaction assessment as set out in MBIE liquefaction guidelines (MBIE 2017). The maps provide certainty required for inclusion in formal documents such as district plans; however, it is important to note that the input data and results are specific to that particular CPT and that the probability of liquefaction and severity of liquefaction-induced damage should not be interpolated or correlated between CPT locations. However, it may be possible to infer the probability of liquefaction and severity of damage for a particular geological or geomorphological unit.

While every attempt has been made to provide a more detailed liquefaction hazard assessment of the Wellington City Council jurisdiction, further recommendations are suggested.

1. Analysing more CPTs within each geological/geomorphological unit may help to characterise the probability of liquefaction occurring in that unit. Mapping the CPT locations highlights areas in Wellington City that may benefit from analyses of more CPTs to provide greater certainty for possible liquefaction-induced damage.
2. Compile/acquire more CPTs in areas deemed susceptible to liquefaction by Dellow et al. (2018), e.g. along State Highway 2, Shelly Bay, Greta Point, and Moa Point. Geotechnical consultancies may have existing data that can be utilised. Alternatively, CPT data can be acquired through a purpose-designed investigation. Any geotechnical (including CPT) data collected is encouraged to be lodged in the NZGD so that it is available to both the public and private sectors.
3. Compile a dataset of the unconfined shallow groundwater surface and it’s seasonal and tidal variations. It is important to understand the variation in the shallow groundwater surface as the extent and severity of liquefaction varies with the unsaturated sediments above the ground surface. This study used groundwater levels recorded in the CPTs or approximated from nearby borehole logs after comparing them to the levels estimated from the GNS Science Shallow Groundwater model for the Wellington Region, which was deemed too coarse for this project.
4. Check that the distribution of LSN values are consistent with the geomorphic map units (e.g. Townsend et al. 2020). Each geomorphic unit, or combination of geomorphic units, should have LSN’s distributed over a relatively small range (e.g. ±5 LSN units), indicating that the geomorphic unit will behave consistently with respect to liquefaction.

While this study has provided a quantitative evaluation of liquefaction hazard to selected sites around Wellington City, these recommendations will provide a more robust and defendable basis on which to include liquefaction hazard information in formal documents such as district plans.

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APPENDICES

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APPENDIX 1 MODIFIED MERCALLI INTENSITY SCALE (NEW ZEALAND)

The New Zealand Modified Mercalli Intensity Scale (includes felt effects and damage to buildings and structures), based on information in Downes (1995), Dowrick et al. (2008) and Hancox et al. (2002).

The Modified Mercalli intensity scale (MM)

The Modified Mercalli intensity scale (summarised from Downes (1995), Dowrick (1996) and Hancox et al. *(2002)) is a descriptive scale used to rank the intensity of an earthquake at a particular location. The intensity of any earthquake will vary from place to place, because of factors such as distance from the epicentre and localised differences in ground conditions (for example, shaking will be much greater on swampy ground than on solid rock).*

MM 2 *Felt by people at rest, on upper floors or favourably placed.*

MM 3 *Felt indoors; hanging objects may swing, vibration similar to passing of light trucks.*

MM 4 *Generally noticed indoors but not outside. Light sleepers may be awakened. Vibration like passing of heavy traffic. Doors and windows rattle. Walls and frames of buildings may be heard to creak.*

MM 5 *Generally felt outside, and by almost everyone indoors. Most sleepers awakened. A few people alarmed. Some glassware and crockery may be broken. Open doors may swing.*

MM 6 *Felt by all. People and animals alarmed. Many run outside. Furniture or objects may move on smooth surfaces. Objects fall from shelves. Glassware and crockery broken. Slight damage to some types of buildings. A few cases of chimney damage. Loose material may be dislodged from sloping ground. A few very small (e.g. <1000 m^3) shallow landslides and rockfalls occur.*

MM 7 *General alarm. Furniture and appliances may be shifted and unstable items overturned. Unreinforced stone and brick walls cracked. Some pre-earthquake code buildings damaged. Roof tiles may be dislodged. Many domestic chimneys broken. Small falls of sand and gravel banks. Some fine cracks appear in sloping ground and ridge crests. Rockfalls from steep slopes and cuttings are common. A few small to moderate landslides (e.g. 1 000 to 10 000 m^3) occur on steeper slopes. Some instances of liquefaction at susceptible sites.*

MM 8 *Alarm may approach panic. Steering of cars greatly affected. Some serious damage to pre-earthquake code masonry buildings. Most reinforced domestic chimneys damaged, many brought down. Monuments and elevated tanks twisted or brought down. Some post-1980 brick veneer dwellings damaged. Houses not secured to foundations may move. Cracks may appear on slopes and in wet ground. On slopes in steep or weak ground, numerous small to moderate landslides and some large landslides (e.g. 100 000 m^3). Collapse of roadside cuttings and unsupported excavations. Small sand fountains and other instances of liquefaction.*

MM 9 *Very poor quality unreinforced masonry destroyed. Pre-earthquake code masonry buildings heavily damaged or collapse. Damage or distortion to some pre-1980 buildings and bridges. Houses not secured to foundations shifted off. Brick veneers fall and expose framing. Conspicuous cracking of flat and sloping ground. On steep slopes, many small to large landslides and some very large (>1 000 000 m^3) landslides and rock avalanches that may block narrow valleys and form lakes. Liquefaction effects intensified, with large sand fountains and extensive cracking or settlement of weak ground.*

MM 10 *Most unreinforced masonry structures destroyed. Many pre-earthquake code buildings destroyed. Many pre-1980 buildings and bridges seriously damaged. Many post-1980 buildings and bridges moderately damaged or permanently distorted. Widespread cracking of flat and sloping ground. Widespread and severe landsliding on sloping ground. Very large landslides (>$10^6 m^3$) from steep mountain faces and coastal cliffs. Widespread and severe liquefaction.*

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APPENDIX 2 CONE PENETRATION TEST DATA USED IN THIS STUDY

Table A2.1 List of all 80 cone penetration tests (CPTs), their location and the parameters used in this study for liquefaction-triggering analysis. The CPT ID is the same as that in the New Zealand Geotechnical Database (NZGD). The investigation date is the date the CPT was taken. The ground surface level is taken from LiDAR data, and the groundwater level (GWL) is taken from the CPT header information in the NZGD or approximated from nearby borehole logs. The total depth of the CPT is depth at which the testing rod tip refused to penetrate any further and is in metres below ground level. The table is ordered by the total depth (in metres) of the CPT, from shallowest to deepest depth.

NZGD ID	Investigation Date	Total Depth (m bgl)	Latitude (WGS84)	Longitude (WGS84)	POINT_X (NZTM)	POINT_Y (NZTM)	Suburb	LiDAR_DTM	Ground Water Level Used (m bgl)
CPT_72616	30/08/2015	2.18	-41.32431	174.809417	1751430	5423661.26	Miramar South	3.04	1
CPT_72619	30/08/2015	2.31	-41.323379	174.835433	1753609	5423718.88	Seatoun	2.50	1
CPT_72611	30/08/2015	2.58	-41.313629	174.819134	1752268	5424830.09	Miramar	3.01	1
CPT_130478	30/06/2017	2.88	-41.31743202	174.81624	1752017	5424412.94	Miramar South	2.17	0.75
CPT_72629	30/08/2015	2.91	-41.323373	174.81221	1751666	5423760.41	Miramar South	3.44	1
CPT_72614	30/08/2015	3.36	-41.32431	174.809417	1751430	5423661.26	Miramar South	3.04	1
CPT_88796	1/09/2016	4.19	-41.290473	174.789531	1749843	5427452.46	Oriental Bay	1.83	1.6
CPT_88795	1/09/2016	4.28	-41.290516	174.789402	1749832	5427447.91	Oriental Bay	1.70	1.2
CPT_93537	21/03/2017	4.57	-41.28963833	174.777377	1748827	5427566.04	Te Aro	1.30	0.55
CPT_72621	30/08/2015	4.84	-41.325928	174.799719	1750614	5423498.52	Lyall Bay-Airport-Moa Point	4.62	1
CPT_130479	29/06/2017	4.98	-41.31758397	174.816365	1752027	5424395.85	Miramar South	2.50	0.7
CPT_112572	4/09/2018	5.04	-41.266625	174.78251	1749309	5430112.21	Thorndon	3.06	2.3
CPT_72065	29/01/2013	5.14	-41.29758442	174.776992	1748777	5426684.51	Te Aro	15.89	3.9
CPT_72623	30/08/2015	5.25	-41.315566	174.77516	1748582	5424691.29	Newtown West	60.21	1
CPT_72630	30/08/2015	5.32	-41.325548	174.835637	1753621	5423477.71	Seatoun	2.54	1
CPT_72034	11/09/2012	5.96	-41.29822309	174.776432	1748728	5426614.56	Mt Cook-Wallace Street	18.45	1.45
CPT_123512	6/03/2018	6.00	-41.28367055	174.777035	1748812	5428229.18	Lambton	2.79	0.6
CPT_72620	30/08/2015	6.62	-41.323379	174.835433	1753609	5423718.88	Seatoun	2.50	1
CPT_93540	23/03/2017	6.82	-41.2892072	174.777357	1748826	5427613.94	Te Aro	2.76	0.3
CPT_72613	30/08/2015	6.94	-41.32393	174.830029	1753156	5423667.26	Seatoun	9.04	1
CPT_72061	14/12/2011	7.16	-41.29930738	174.780856	1749096	5426486.59	Te Aro	5.60	1.9
CPT_112575	4/09/2018	7.29	-41.266318	174.782805	1749335	5430145.78	Thorndon	2.89	2.3
CPT_112603	28/03/2018	7.36	-41.314934	174.796087	1750336	5424725.4	Kilbirnie East	2.19	2.2
CPT_72622	30/08/2015	7.81	-41.324981	174.773447	1748418	5423648.94	Berhampore West	22.08	1
CPT_93536	21/03/2017	7.84	-41.28968342	174.777491	1748836	5427560.84	Te Aro	1.74	0.7
CPT_88792	1/09/2016	7.89	-41.290856	174.789814	1749866	5427409.45	Oriental Bay	2.96	2.1
CPT_93583	7/06/2017	7.90	-41.29158068	174.774783	1748605	5427354.84	Te Aro	9.39	2.45
CPT_93379	20/03/2017	7.92	-41.28932925	174.777591	1748846	5427599.99	Te Aro	2.84	0.5
CPT_90436	9/03/2017	8.00	-41.28970829	174.777459	1748834	5427558.13	Te Aro	1.53	1.7
CPT_72085	30/01/2013	8.01	-41.30077805	174.782439	1749225	5426320.59	Mt Victoria	11.55	1.9
CPT_71948	30/09/2014	8.19	-41.32918631	174.81227	1751657	5423114.9	Lyall Bay-Airport-Moa Point	5.18	1.5
CPT_113550	4/02/2016	8.26	-41.30117113	174.773378	1748466	5426292.49	Mt Cook-Wallace Street	31.58	0.5

NZGD ID	Investigation Date	Total Depth (m bgl)	Latitude (WGS84)	Longitude (WGS84)	POINT_X (NZTM)	POINT_Y (NZTM)	Suburb	LiDAR_DTM	Ground Water Level Used (m bgl)
CPT_104552	9/02/2018	8.29	-41.31062057	174.77986	1748987	5425232.28	Adelaide	17.91	2.2
CPT_112612	4/04/2018	8.30	-41.314978	174.795929	1750322	5424720.79	Kilbirnie East	2.21	2.2
CPT_72055	12/09/2012	8.45	-41.29828547	174.778215	1748877	5426604.58	Te Aro	17.75	4.2
CPT_93534	20/03/2017	8.62	-41.28943779	174.777534	1748841	5427588.03	Te Aro	2.59	0.5
CPT_72628	30/08/2015	8.68	-41.320862	174.805664	1751124	5424050.62	Kilbirnie East	4.68	1
CPT_93542	4/04/2017	8.69	-41.28932132	174.777206	1748813	5427601.53	Te Aro	2.62	2
CPT_72680	8/02/2013	8.80	-41.26972957	174.785038	1749514	5429763.18	Thorndon	2.25	2.5
CPT_112573	4/09/2018	8.91	-41.266473	174.782689	1749325	5430128.78	Thorndon	2.94	2.3
CPT_93538	21/03/2017	8.93	-41.28948978	174.777373	1748827	5427582.54	Te Aro	2.14	0.55
CPT_71966	23/06/2015	8.95	-41.32811247	174.812444	1751674	5423233.81	Lyall Bay-Airport-Moa Point	3.90	1.31
CPT_72644	7/05/2015	9.00	-41.26976101	174.786164	1749608	5429757.75	Thorndon	2.65	3.4
CPT_72647	5/05/2015	9.00	-41.26818568	174.78543	1749550	5429933.91	Thorndon	2.86	2.9
CPT_137311	20/08/2018	9.01	-41.28264038	174.744063	1746053	5428399.55	Karori North	176.67	2.6
CPT_72612	30/08/2015	9.03	-41.318608	174.828977	1753080	5424259.98	Karaka Bay-Worser Bay	2.56	1
CPT_93544	5/04/2017	9.10	-41.28941785	174.777219	1748814	5427590.79	Te Aro	2.17	1.3
CPT_72056	11/09/2012	9.15	-41.29811107	174.777658	1748831	5426624.89	Te Aro	19.43	4.2
CPT_72659	22/11/2015	9.20	-41.27916391	174.781151	1749167	5428722.45	Thorndon	2.08	1.6
CPT_72106	24/06/2015	9.31	-41.32760571	174.812596	1751688	5423289.81	Lyall Bay-Airport-Moa Point	3.84	1.19
CPT_93535	21/03/2017	9.34	-41.28952348	174.777349	1748825	5427578.84	Te Aro	1.97	0.5
CPT_71973	23/06/2015	9.58	-41.32814249	174.812089	1751644	5423231.1	Lyall Bay-Airport-Moa Point	3.94	1.52
CPT_72067	30/01/2013	9.69	-41.29752319	174.777509	1748820	5426690.42	Te Aro	17.57	6.8
CPT_72625	30/08/2015	9.88	-41.328282	174.802349	1750829	5423232.6	Lyall Bay-Airport-Moa Point	3.46	1
CPT_72627	30/08/2015	9.93	-41.321893	174.805624	1751118	5423936.22	Kilbirnie East	3.94	1
CPT_93539	21/03/2017	9.93	-41.28915827	174.777667	1748852	5427618.84	Te Aro	2.91	0.3
CPT_71958	20/06/2015	9.97	-41.3282924	174.812857	1751708	5423213.12	Lyall Bay-Airport-Moa Point	4.26	1.4
CPT_72618	30/08/2015	9.98	-41.323983	174.816224	1752000	5423685.66	Miramar South	3.35	1
CPT_93545	4/04/2017	10.44	-41.28937415	174.777329	1748824	5427595.45	Te Aro	2.39	1.3
CPT_93543	5/04/2017	10.51	-41.2892138	174.777265	1748819	5427613.36	Te Aro	2.83	1.3
CPT_137315	20/08/2018	10.79	-41.28268503	174.744088	1746055	5428394.55	Karori North	176.68	2.60
CPT_72050	10/09/2012	10.90	-41.29859937	174.777596	1748825	5426570.79	Mt Cook-Wallace Street	19.90	1.45
CPT_72626	30/08/2015	11.91	-41.323033	174.805509	1751106	5423809.86	Kilbirnie East	4.29	1
CPT_72037	10/09/2012	12.04	-41.29792505	174.776401	1748726	5426647.7	Te Aro	17.16	3.2
CPT_72615	30/08/2015	12.12	-41.313629	174.819134	1752268	5424830.09	Miramar	3.01	1
CPT_88794	1/09/2016	12.55	-41.290536	174.78966	1749853	5427445.24	Oriental Bay	1.70	1.6
CPT_72617	30/08/2015	12.78	-41.32431	174.809417	1751430	5423661.26	Miramar South	3.04	1
CPT_72053	10/09/2012	12.99	-41.29783482	174.776096	1748701	5426658.24	Te Aro	16.10	1.45

NZGD ID	Investigation Date	Total Depth (m bgl)	Latitude (WGS84)	Longitude (WGS84)	POINT_X (NZTM)	POINT_Y (NZTM)	Suburb	LiDAR_DTM	Ground Water Level Used (m bgl)
CPT_72057	12/09/2012	13.30	-41.29811512	174.777672	1748832	5426624.42	Te Aro	19.36	1.45
CPT_72655	23/11/2015	14.40	-41.27802557	174.780522	1749117	5428849.91	Thorndon	2.86	2.90
CPT_72035	11/09/2012	14.60	-41.29859422	174.778469	1748898	5426569.86	Te Aro	17.14	4.2
CPT_72624	30/08/2015	15.00	-41.343042	174.772762	1748319	5421644.93	Island Bay East	2.43	1
CPT_118999	27/08/2018	15.40	-41.29533086	174.777717	1748842	5426933.46	Te Aro	7.81	1
CPT_72052	10/09/2012	15.50	-41.29821984	174.777358	1748806	1748805.9	Te Aro	19.45	1.45
CPT_136860	5/11/2019	16.14	-41.235088	174.799647	1750817	5433583.89	Raroa	157.93	1.5
CPT_136866	5/11/2019	18.15	-41.317141	174.80013	1750669	5424473.36	Kilbirnie East	1.76	1.7
CPT_136864	5/11/2019	18.23	-41.316459	174.799356	1750606	5424550.42	Kilbirnie East	2.43	2.5
CPT_136865	5/11/2019	18.65	-41.316702	174.800288	1750683	5424521.82	Kilbirnie East	2.09	1.6
CPT_136861	5/11/2019	18.74	-41.316993	174.798999	1750575	5424491.75	Kilbirnie East	1.80	1.6
CPT_136868	5/11/2019	18.77	-41.317543	174.800026	1750659	5424428.91	Kilbirnie East	1.82	1.7

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APPENDIX 3 GROUND MOTION MODELS

A3.1 Ground Motion Models

International best practice has moved toward use of multiple ground motion prediction equations (GMPEs) in order to better consider epistemic uncertainties (i.e. those related to modelling uncertainty as opposed to random variability) in ground motion prediction. Therefore, a suite of GMPEs were considered for use in this project, consistent with the way general practice in seismic hazard is developing internationally. In seismic hazard analysis, epistemic uncertainties are often considered by applying multiple GMPEs in a logic-tree form (Kulkarni et al. 1984), which has been the focus of considerable research over the past decade (e.g. Abrahamson and Bommer 2005; McGuire et al. 2005; Musson 2005; Bommer and Scherbaum 2008; Scherbaum and Kuehn 2011; Bommer 2012; Musson 2012; Atkinson et al. 2014). In this study, a selection of GMPEs has been chosen for sources in the three categories of Active Shallow Crust, Subduction Interface and Subduction Intraslab to acknowledge that these types of earthquakes tend to produce very different expressions of ground motion.

A3.2 Crustal Ground Motion Prediction Equations

Crustal GMPEs selected for this study include both international and New Zealand GMPEs. There has typically been limited use of overseas models in New Zealand probabilistic seismic hazard assessment (PSHA), largely because it was unclear how overseas models perform with respect to New Zealand earthquakes. This issue has recently been addressed through the development of an open-access New Zealand strong motion database (Van Houtte et al. 2017; Kaiser et al. 2017) and testing of overseas models against New Zealand data (Van Houtte 2017). It was found that, in general, the overseas models perform better with respect to New Zealand data than those specifically designed for New Zealand application.

Based on these tests, Van Houtte (2017) recommended that four of the models from the NGA-West2 project (Gregor et al. 2014), namely, Abrahamson et al. (2014b), Boore et al. (2014), Campbell and Bozorgnia (2014) and Chiou and Youngs (2014), as well as a fifth model, Bradley (2013), be used for seismic hazard analyses in New Zealand, with weightings of 0.25 for each of the first three models and 0.125 for the Chiou and Youngs and Bradley models (Figure 5.3).

Inclusion of the Bradley (2013) model alongside the NGA-West2 models could be considered a controversial choice, given that the Bradley (2013) model is largely the same model as the Chiou et al. (2010) model, closely related to, but now superseded by, the Chiou and Youngs (2014) model. The logic-tree concept requires that each branch is independent, which is not the case if both Chiou and Youngs (2014) and Bradley (2013) are included. It is unclear from model testing which of the Chiou and Youngs (2014) and Bradley (2013) models is more appropriate for application in New Zealand. Therefore, the weight of the branch that would be applied to Chiou and Youngs (2014) and Bradley (2013) is split evenly and essentially combine to form a ‘Chiou and Youngs branch’ with a total weight (0.25) that is the same as the other included GMPEs.

The McVerry et al. (2006) GMPE, which underpins design codes in New Zealand (Standards New Zealand 2004, 2016), is only designed to spectral periods of up to 3 s, and therefore cannot be applied to this project, which requires spectral periods up to 4.5 s.

A3.2.1 Volcanic Sources

The Bradley (2013) GMPE is used to accommodate for volcanic sources as, again, the McVerry et al. (2006) GMPE often used for this type of source does not extend to 3 s.

A3.2.2 Subduction Zone Models

Subduction zone ground motions are expected to have an effect on hazard in Wellington and make a notable contribution to the estimated hazard at PGA for all probabilities of exceedance evaluated in this study.

Van Houtte (2017) recommended selection of the subduction zone models of Abrahamson et al. (2016), Atkinson and Boore (2003, 2008), McVerry et al. (2006) and Zhao et al. (2006) for New Zealand PSHA, applied with equal weights. However, there is international precedent for the sole use of Abrahamson et al. (2016) in global PSHA as part of the BC Hydro SSHAC Level 3 study (Abrahamson et al. 2014a). Abrahamson et al. (2016) provide 'high', 'median' and 'low' versions of their model to account for epistemic uncertainty. Additionally, as requirements for road/transportation structures dictate that hazard results for spectral periods out to 4.5 s must be estimated, the McVerry et al. (2006) and Atkinson and Boore (2003, 2008) GMPEs become insufficient, extending to 3 s and 4 s, respectively. As such, the logic tree includes only Abrahamson et al. (2016) with branch weights of 0.2, 0.6 and 0.2 for high, median and low, respectively (Figure 5.3).

This logic tree is used for both subduction interface and subduction intraslab earthquakes.

A3.2.3 Modification of GMPEs to Represent the Larger Horizontal Component

The New Zealand Loadings Standard, NZS1170.5, requires horizontal design motions to be defined in terms of the larger of two as-recorded horizontal components (SA_{larger}). The global NGA-West2 models are defined in terms of an orientation-independent metric for average horizontal motion known as RotD50 (SA_{RotD50}; Boore 2010), which is the 50th-percentile spectral acceleration out of all possible horizontal component orientations. It is therefore necessary to scale the NGA-West2 models to approximate the larger horizontal component.

Additionally, the Bradley (2013) model is defined in terms of a rotational, geometric-mean ground motion metric known as $SA_{GMRotI50}$ (Boore et al. 2006), and the Abrahamson et al. (2016) subduction zone model is defined in terms of the geometric mean ground motion, SA_{GM}. All of these models require scaling to approximate the larger horizontal component.

Boore and Kishida (2017) derived conversion equations for $SA_{larger}/SA_{GMRotI50}$, SA_{larger}/SA_{GM} and SA_{larger}/SA_{RotD50} using 21,000 pairs of horizontal-component response spectra from the NGA-West2 database (Ancheta et al. 2014). Given the vast quantity of data underpinning these equations, it is expected that they will provide the most reliable average conversion to SA_{larger}. These conversion factors only depend on oscillator period. These conversion factors are not negligible, with a more than 10% increase at peak ground acceleration (PGA) and up to 25% difference at long periods. The model standard deviations are also modified, using the values provided in Boore and Kishida (2017).

A3.2.4 Additional GMPE Epistemic Uncertainty

It was also recommended by Van Houtte (2017) that the logic tree should include a second branch level to include additional epistemic uncertainty. This level accounts for the GMPEs being derived from limited data. In typical statistical model developments, this type of uncertainty would be observable as standard errors of the model coefficients. However, these standard errors are not provided for any of the selected models.

While this type of uncertainty is expected to be model-specific, Al Atik and Youngs (2014) provide a model that is intended for general application to all of the NGA-West2 models (with the standard deviation of the epistemic uncertainty denoted as σ_μ in Figure 5.3). For convenient application, Al Atik and Youngs (2014) provide their model as a three-point discrete approximation to a normal distribution. σ_μ depends on magnitude, style of faulting and period and is generally less than 0.1 natural log units. The Al Atik and Youngs (2014) model cannot be applied to the Bradley (2013) model, given that the Bradley (2013) model was derived from a much smaller dataset. However, an alternative model has been previously calculated for the closely related Chiou and Youngs (2008) model as part of the BC Hydro SSHAC Level 3 study (Abrahamson et al. 2014a). This alternative model is only applied to the Bradley (2013) model and is denoted as FE(T) in Figure 5.3.

A3.3 Relevant Corrections to the 2010 National Seismic Hazard Model

Correction of parameters of the Hikurangi subduction interface sources from those published in Stirling et al. (2012; Appendix 1) will impact Wellington sites. Other corrections not listed here are generally minor corrections to typing errors in fault source traces around the country.

Table A3.1 Modification to parameters of the Hikurangi subduction interface sources from Stirling et al. (2012) to present.

Source	Length (km)	M_W	Recurrence Interval (Years)	
			Stirling et al. (2012)	Corrected (Present Model)
HikALL	620	9.0	7050	6700
Hik_Wgtn_Min	220	8.1	550	850
Hik_Wgtn_Max	220	8.4	1000	1600
Hik_HBay_Min	200	8.1	1100	1300
Hik_HBay_Max	200	8.3	1400	1700
Hik_Rauk_Min	200	8.1	900	1000
Hik_Rauk_Max	200	8.3	1150	1400

A3.4 Peak Ground Acceleration Hazard Results

The hazard results for both the probabilistic and scenario hazard results are given in the following tables.

Table A3.2 Mean larger horizontal component probabilistic peak ground acceleration (PGA) values for each CPT site for the 10% probability of exceedance in 50 years and mean larger horizontal component 50^{th}- and 84^{th}-percentile scenario PGA values for each CPT site. Site conditions at each CPT site vary. Disaggregation of the mean probabilistic hazard results in an associated average magnitude of 7.3; the largest contributor to the hazard is the Hikurangi-Wellington minimum source (magnitude 8.1). The scenarios are the Wellington-Hutt Valley (magnitude 7.5) and Hikurangi-Wellington minimum (magnitude 8.1) fault sources.

			Peak Ground Acceleration (g)				
CPT Site Information			Probabilistic	Wellington-Hutt Valley Scenario		Hikurangi-Wellington Minimum Scenario	
Site ID	Longitude (degrees)	Latitude (degrees)	10% in 50 Years	50^{th} Percentile	84^{th} Percentile	50^{th} Percentile	84^{th} Percentile
CPT_93537	174.777	-41.290	0.75	0.61	1.07	0.54	1.13
CPT_88795	174.789	-41.291	0.75	0.62	1.09	0.55	1.16
CPT_88796	174.790	-41.290	0.75	0.62	1.09	0.55	1.16
CPT_72621	174.800	-41.326	0.68	0.45	0.76	0.50	1.05
CPT_72616	174.809	-41.324	0.72	0.48	0.84	0.54	1.13
CPT_72629	174.812	-41.323	0.70	0.46	0.79	0.53	1.10
CPT_130478	174.816	-41.317	0.67	0.43	0.73	0.50	1.04
CPT_130479	174.816	-41.318	0.67	0.43	0.73	0.50	1.04
CPT_72611	174.819	-41.314	0.67	0.44	0.73	0.50	1.04
CPT_72619	174.835	-41.323	0.68	0.41	0.70	0.51	1.07
CPT_137311	174.744	-41.283	0.73	0.66	1.20	0.54	1.14
CPT_113550	174.773	-41.301	0.71	0.60	1.09	0.55	1.15
CPT_72622	174.773	-41.325	0.72	0.56	1.01	0.56	1.17
CPT_90521	174.775	-41.297	0.75	0.61	1.07	0.54	1.13
CPT_93583	174.775	-41.292	0.74	0.60	1.03	0.53	1.11
CPT_72623	174.775	-41.316	0.72	0.57	1.04	0.56	1.17
CPT_72034	174.776	-41.298	0.75	0.63	1.11	0.55	1.16
CPT_72065	174.777	-41.298	0.75	0.63	1.11	0.55	1.16
CPT_123512	174.777	-41.284	0.74	0.65	1.17	0.55	1.16
CPT_93542	174.777	-41.289	0.75	0.62	1.07	0.54	1.13
CPT_93544	174.777	-41.289	0.75	0.61	1.07	0.54	1.13
CPT_93535	174.777	-41.290	0.75	0.61	1.07	0.54	1.13
CPT_93540	174.777	-41.289	0.75	0.62	1.07	0.54	1.13
CPT_93538	174.777	-41.289	0.75	0.61	1.07	0.54	1.13
CPT_90436	174.777	-41.290	0.75	0.61	1.07	0.54	1.13
CPT_93536	174.777	-41.290	0.75	0.61	1.07	0.54	1.13

			Peak Ground Acceleration (g)				
CPT Site Information			Probabilistic	Wellington-Hutt Valley Scenario		Hikurangi-Wellington Minimum Scenario	
Site ID	Longitude (degrees)	Latitude (degrees)	10% in 50 Years	50th Percentile	84th Percentile	50th Percentile	84th Percentile
CPT_72067	174.778	-41.298	0.75	0.63	1.11	0.55	1.16
CPT_93534	174.778	-41.289	0.75	0.61	1.07	0.54	1.13
CPT_93379	174.778	-41.289	0.75	0.61	1.07	0.54	1.13
CPT_72056	174.778	-41.298	0.75	0.62	1.11	0.55	1.16
CPT_93539	174.778	-41.289	0.75	0.62	1.07	0.54	1.13
CPT_72055	174.778	-41.298	0.75	0.62	1.10	0.55	1.16
CPT_104552	174.780	-41.311	0.74	0.59	1.04	0.55	1.16
CPT_72061	174.781	-41.299	0.74	0.59	1.03	0.54	1.13
CPT_72659	174.781	-41.279	0.68	0.52	0.87	0.48	1.01
CPT_72085	174.782	-41.301	0.75	0.61	1.07	0.56	1.16
CPT_112572	174.783	-41.267	0.75	0.63	1.09	0.53	1.11
CPT_112573	174.783	-41.266	0.75	0.63	1.09	0.53	1.11
CPT_112575	174.783	-41.266	0.75	0.63	1.09	0.53	1.11
CPT_115280	174.783	-41.278	0.70	0.55	0.92	0.50	1.04
CPT_90512	174.784	-41.291	0.74	0.59	1.02	0.53	1.11
CPT_115291	174.785	-41.278	0.70	0.54	0.92	0.50	1.04
CPT_72680	174.785	-41.270	0.68	0.52	0.86	0.48	1.00
CPT_72647	174.785	-41.268	0.68	0.53	0.88	0.48	1.01
CPT_72644	174.786	-41.270	0.68	0.52	0.86	0.48	1.00
CPT_88792	174.790	-41.291	0.75	0.62	1.09	0.55	1.16
CPT_112612	174.796	-41.315	0.73	0.53	0.92	0.54	1.13
CPT_112603	174.796	-41.315	0.73	0.53	0.92	0.54	1.13
CPT_72625	174.802	-41.328	0.68	0.44	0.75	0.51	1.05
CPT_72627	174.806	-41.322	0.72	0.49	0.86	0.54	1.13
CPT_72628	174.806	-41.321	0.72	0.50	0.86	0.54	1.13
CPT_71973	174.812	-41.328	0.70	0.45	0.77	0.52	1.09
CPT_71948	174.812	-41.329	0.70	0.45	0.77	0.52	1.09
CPT_71966	174.812	-41.328	0.70	0.45	0.77	0.52	1.09
CPT_72106	174.813	-41.328	0.70	0.45	0.77	0.52	1.09
CPT_71958	174.813	-41.328	0.70	0.45	0.77	0.52	1.09
CPT_72618	174.816	-41.324	0.70	0.45	0.78	0.53	1.10
CPT_72612	174.829	-41.319	0.68	0.43	0.73	0.51	1.07
CPT_72613	174.830	-41.324	0.68	0.42	0.71	0.51	1.07
CPT_72620	174.835	-41.323	0.68	0.41	0.70	0.51	1.07

CPT Site Information			Peak Ground Acceleration (g)				
			Probabilistic	Wellington-Hutt Valley Scenario		Hikurangi-Wellington Minimum Scenario	
Site ID	Longitude (degrees)	Latitude (degrees)	10% in 50 Years	50th Percentile	84th Percentile	50th Percentile	84th Percentile
CPT_72630	174.836	-41.326	0.68	0.41	0.70	0.51	1.07
CPT_137317	174.744	-41.283	0.73	0.66	1.20	0.54	1.14
CPT_137315	174.744	-41.283	0.73	0.66	1.20	0.54	1.14
CPT_72624	174.773	-41.343	0.71	0.52	0.94	0.56	1.18
CPT_72053	174.776	-41.298	0.75	0.63	1.11	0.55	1.16
CPT_72037	174.776	-41.298	0.75	0.63	1.11	0.55	1.16
CPT_112050	174.777	-41.282	0.76	0.65	1.16	0.55	1.15
CPT_93543	174.777	-41.289	0.75	0.62	1.08	0.54	1.13
CPT_93545	174.777	-41.289	0.75	0.61	1.07	0.54	1.13
CPT_72052	174.777	-41.298	0.75	0.63	1.11	0.55	1.16
CPT_72050	174.778	-41.299	0.75	0.62	1.11	0.55	1.16
CPT_72057	174.778	-41.298	0.75	0.62	1.11	0.55	1.16
CPT_118999	174.778	-41.295	0.75	0.61	1.06	0.54	1.13
CPT_72035	174.778	-41.299	0.74	0.60	1.04	0.54	1.13
CPT_72655	174.781	-41.278	0.68	0.52	0.87	0.48	1.01
CPT_90528	174.785	-41.291	0.74	0.59	1.02	0.53	1.11
CPT_90530	174.785	-41.291	0.74	0.59	1.02	0.53	1.11
CPT_88794	174.790	-41.291	0.75	0.62	1.09	0.55	1.16
CPT_136861	174.799	-41.317	0.67	0.45	0.76	0.50	1.03
CPT_136864	174.799	-41.316	0.67	0.45	0.76	0.50	1.03
CPT_136860	174.800	-41.235	0.72	0.64	1.16	0.54	1.13
CPT_136868	174.800	-41.318	0.67	0.45	0.76	0.50	1.03
CPT_136866	174.800	-41.317	0.67	0.45	0.76	0.50	1.03
CPT_136865	174.800	-41.317	0.67	0.45	0.76	0.50	1.03
CPT_72626	174.806	-41.323	0.72	0.49	0.85	0.54	1.13
CPT_72617	174.809	-41.324	0.72	0.48	0.84	0.54	1.13
CPT_72615	174.819	-41.314	0.67	0.44	0.73	0.50	1.04

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