REPORT

For a comprehensive understanding of this report, please also refer to the relevant s92 responses

Auckland Regional Landfill

Probabilistic Seismic Hazard Assessment

Prepared for

Waste Management NZ Ltd

Prepared by

Tonkin & Taylor Ltd

Date

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Executive summary

Waste Management NZ Ltd has engaged Tonkin & Taylor Ltd to undertake a probabilistic seismic hazard analysis (PSHA) to support the resource consent application for the Auckland Regional Landfill project.

A site specific PSHA incorporates recent developments in understanding of the seismic hazard in the region, recent advances in seismic knowledge and site specific ground conditions. In particular, for this project, the PSHA includes recent updates to the National Seismic Hazard Model since the publication of NZS1170.5 and the NZTA Bridge Manual.

The Auckland Regional Landfill project is in an area of relatively low seismicity compared to the rest of New Zealand. This is supported by the results of the PSHA, with lower levels of shaking than specified in the design standards calculated for the site. However, for regions of relatively low seismicity, NZS1170.5 and the Bridge Manual prescribe a minimum criteria that is to be considered in determining the ultimate limit state (ULS) seismic actions for design even if a PSHA indicates a low probability of this occurring. The minimum criteria is to provide a margin of safety against collapse if a major earthquake occurs on a currently unknown fault.

The key outcomes from the PSHA undertaken for the Auckland Regional Landfill project, including consideration of the minimum requirements in the relevant New Zealand Standards, are summarised below.

Key consideration	Background	Outcome
Site Subsoil Class	The Subsoil Class is related to the site's ground conditions. The Subsoil Class is used in the New Zealand Design Standards (such as NZS1170.5 and NZTA Bridge Manual) to determine the seismic accelerations for design.	The Auckland Regional Landfill project site has been assessed as **Subsoil Class C (shallow soil site)** in accordance with the criteria presented in NZS1170.5. The results from this PSHA support this Subsoil Class assessment. For futher details refer to **Section 4.4.1** and **6.3.2**.
Seismic accelerations for geotechnical design	A peak ground acceleration (PGA) and magnitude is required for geotechnical design. Where a site specific PSHA is not undertaken, PGA and magnitude are determined from the Bridge Manual.	The magnitude and PGA determined from this PSHA are **less** than the minimum values stated in the Bridge Manual for 500, 1000 and 2500 year return periods (i.e. the return periods considered for ULS design). Probabilistically derived values that are less than the minimum criteria provided in the New Zealand design standards cannot be used for design. As such, for 500, 1000 and 2500 year return periods the **minimum values** provided in the Bridge Manual for **Subsoil Class C** are recommended for geotechnical design. However, based on the PSHA results, for 2500 year return period, the minimum value for PGA (PGA=0.19 g) can be used rather than a higher value (PGA=0.24 g) that would otherwise be calculated using the Bridge Manual for the site. For 25 year return period, a **PGA of 0.01 g** and **magnitude of 5.7** is recommended for design. These are less than the respective values calculated using the Bridge Manual. For further details refer to **Section 6.3.1**.

Key consideration	Background	Outcome
Seismic acclerations for structural design	Spectral acceleration (SA) at the fundamental period of the structure is required for structural design. Where PSHA is not undertaken, NZS1170.5 provides a design spectra with spectral acceleration values at various vibrational periods.	The SA values determined from the PSHA for the proposed landfill site are consistently less than the SA values calculated using NZS1170.5 for all vibrational periods. However, the minimum hazard factor of 0.13 prescribed in NZS1170.5 is required to be considered for regions of relatively low seismicity. As such, for structural design it is recommended that the SA values are determined from NZS1170.5 for **Subsoil Class C** and a minimum **hazard factor of 0.13**. For further details refer to **Section 6.3.2.**

Glossary of terms

Term	
AEP	Annual exceedance probability. Probability of a particular rupture event occurring within any one year period; or Probability that a ground motion intensity measure (IM) will exceed a defined value at a given site within any one year period.
Deaggregation	PSHA combines the hazard from all potential earthquake sources. Deaggregation is the procedure to show the relative contribution to the overall hazard from different sources. For example, the relative contribution from a particular fault or the contribution from all sources at a particular distance from the site.
GMPE	Ground Motion Prediction Equation (or Ground Motion Model). Gives the probability that a particular ground motion intensity measure (e.g. peak ground acceleration or spectral acceleration) will exceed a specified value at a given distance from the source, for a given earthquake rupture.
IL	Importance level (for a building) as defined in the New Zealand loadings standard NZS1170.0:2004.
IM	Intensity Measure. A measure of the level of ground shaking. For example, Peak Ground Acceleration (PGA).
M_w	Moment magnitude.
NSHM	National Seismic Hazard Model for New Zealand. The 2002 version forms the basis for the New Zealand standard for earthquake loading NZS1170.5:2004. The latest published version is the 2010 NSHM (Stirling et al. 2012).
PGA	Peak Ground Acceleration (units of g).
PSHA	Probabilistic Seismic Hazard Analysis. A method to quantify the probability of exceeding various ground motion levels at a site given all possible earthquakes.
Return period	The inverse of the AEP. For example an AEP of 0.01 is equivalent to a return period of 100 years. This is the expected *average* time period between occurrences of a particular event (e.g. an earthquake rupture, or exceedance of an IM at a site).
R_{rup}	Shortest distance between the site and the fault rupture plane.
SA (T=x)	Spectral acceleration experienced by a single degree of freedom system with a period of T=x (seconds).
SLS	"*Serviceability Limit States* for earthquake loading are to avoid damage to … the structure and non-structural components that would prevent the structure from being used as originally intended without repair after the SLS earthquake…" NZS1170.5:2004 definition.
Spectral shape factor	Spectral shape factor, $C_h(T)$ as defined in NZS1170.5:2004 Clause 3.1.2.
SSC model	Seismic source characterisation model. Defines the earthquake rupture sources (e.g. location, geometry, faulting type, magnitude) and their probability of occurrence within a given time period.
UHS	Uniform Hazard Spectra. Represents a locus of spectral accelerations at various vibration periods which have the same annual frequency of exceedance (or equivalently, return period).
ULS	"*Ultimate Limit State* for earthquake loading shall provide for … avoidance of collapse of the structural system … or loss of support to parts … damage to non-structural systems necessary for emergency building evacuation that renders them inoperative." NZS1170.5:2004 definition.
V_s	Shear wave velocity.
$V_{s,30}$	The time-averaged shear wave velocity of the upper 30 m of soil and rock at a site.
Z	Hazard factor as defined in NZS1170.5. It is used to scale the NZS1170.5 design spectra for different areas across New Zealand with different levels of seismicity. The default value specified for Auckland (without a site-specific study) is Z=0.13.
$Z_{1.0}$	Depth to a soil or rock shear wave velocity of 1.0 km/s.
$Z_{2.5}$	Depth to a soil or rock shear wave velocity of 2.5 km/s.

1 Introduction

Waste Management NZ Ltd (WMNZ) has engaged Tonkin & Taylor Ltd (T+T) to undertake a probabilistic seismic hazard analysis (PSHA) to support resource consent application for the Auckland Regional Landfill project. The Auckland Regional Landfill project proposes to provide a new solid waste landfill to replace the Redvale landfill, which currently provides for waste disposal of a significant portion of Auckland's solid waste.

This site-specific PSHA incorporates recent developments in the understanding of the seismic hazard within the site region, as well as advances in seismic knowledge since the publication of the 2004 New Zealand Loadings Standard for earthquakes (NZS1170.5:2004).

A preliminary assessment of design seismic ground motions and site-specific seismic hazard for the Auckland Regional Landfill project has been previously undertaken by T+T and is provided in Appendix A. This report presents the PSHA undertaken to determine the level of earthquake hazard for 25, 500, 1000 and 2500 year return periods.

In particular this report provides:

1 Hazard exceedance curves for periods of 0 s (PGA), 0.4 s and 1 s;
2 Serviceability Limit State (SLS) uniform hazard spectra (UHS) for the return period of 25 years;
3 Ultimate Limit State (ULS) uniform hazard spectra for return periods of 500, 1000 and 2500 years;
4 Deaggregation of the seismic hazard to characterise the relative contribution from each of the earthquake sources to the seismic hazard at period of 0 s (PGA) for return periods of 25, 500, 1000 and 2500 years.

Notwithstanding the PSHA presented in this report, due to the low level of seismicity in the Auckland region, the minimum provisions for ULS provided in the Bridge Manual (NZTA 2016) and NZS1170.5 will also need to be considered. The Bridge Manual has been considered for this project as NZS1170.5 was developed for structural design actions only and excludes earthquake actions for geotechnical design (such as liquefaction, slope instability and soil retaining structures). As such, the Earthquake Geotechnical Engineering Practice guidelines (NZGS/MBIE 2016) recommend determining the earthquake actions for geotechnical design using the Bridge Manual.

This SHA is undertaken as a special study as defined by the *New Zealand Structural Design Actions Part 5: Earthquake actions* (NZS1170.5, Standards New Zealand 2004) Clause 1.4, as a departure from Clause 3.1.1 Elastic site spectra. As such, the minimum requirements elsewhere in the standard (i.e. not addressed by this special study) will also still apply unless they are subject to a special study themselves.

For the assessment of earthquake actions for geotechnical design, this SHA is undertaken in accordance with '*Method 2: Site-specific probabilistic seismic hazard analysis'* outlined in the NZGS/MBIE (2016) guidelines for estimating ground motion parameters.

The PSHA presented in this report has been undertaken for the assumptions stated in this report (i.e. ground conditions, structure types and structural period of interest, earthquake return periods etc.). Where actual conditions or proposed structures deviate from these assumptions, a review of the PSHA presented in this report will be required.

2 Project Description

The Auckland Regional Landfill project proposes to provide a new landfill that will serve the wider Auckland region. The proposed landfill site is located in the Wayby Valley area, which is approximately 6 km southeast of Wellsford and approximately 13 km northwest of Warkworth. The key features of the proposed works include:

- Earthworks (cut and fill) to modify the existing valley landform to meet the required storage volume (air space);
- Construction of clay and HDPE lining system along the base of the landfill;
- Construction of an access road from the existing State Highway 1 up the Springhill farm valley and into the proposed landfill, involving multiple cut slopes and earth fills along the alignment;
- Construction of a bin exchange area on the eastern side of Waiteraire stream;
- Construction of a bridge over the Waiteraire stream.

2.1 Location

The PSHA was undertaken specifically at Latitude 36.332°S and Longitude 174.579°E, which is within the site area shown in Figure 2.1.

Figure 2.1: Site location within the Auckland Regional Landfill project shown by the red marker. Analysis undertaken for 36.332°S and Longitude 174.579°E.

2.2 Structural period

Based on the information currently available, the structures proposed are understood to consist of cut and fill slopes to modify the existing landform to meet the required storage volume and to construct an access road from the existing Stage Highway 1, small office buildings, a weighbridge and a single span bridge to cross the Waiteraire stream.

As such hazard exceedance curves at period of 0 s (PGA) and average magnitude (M_w) based on the deaggregation of the seismic hazard for return periods of 25, 500, 1000 and 2500 years are presented for geotechnical design.

Hazard exceedance curves at period of 0.4 s and 1 s and uniform hazard spectra (UHS) for return period of 25 years (SLS) and return periods of 500, 1000 and 2500 years (ULS) are presented for structural design.

It should be noted that the assumptions for the PSHA presented in this report will require review by T+T where the periods of interest for structures vary from those presented in this report.

2.3 Return periods

The PSHA was undertaken to determine the level of earthquake hazard for 25, 500, 1000 and 2500 year return periods. A 25 year return period has been considered to determine the level of earthquake hazard for a SLS event. 500, 1000 and 2500 year return periods have been considered for an ULS event to allow for possible differences in design life (50 and 100 years) and possible differences in Importance Level (IL2, IL3 and IL4) for various structures proposed on site.

3 Geological and tectonic setting

3.1 Local geology

An extract of the 1:250,000 geological map for the Auckland region (Edbrooke 2001) published by the Institute of Geological & Nuclear Sciences is provided in Figure 3.1 below. The Auckland Regional Landfill project site is inferred to be predominantly underlain by Pakiri Formation sediments (Mwp) consisting of Pakiri Formation residual soil overlying Pakiri Formation rock. The Pakiri Formation residual soil typically consists of firm to very stiff sandy silt to silty clay and medium dense silty sand and the Pakiri Formation rock typically consists of alternating beds of weak to moderately strong sandstone and very weak to weak siltstone.

The gently sloping farmland to the west of the proposed landfill site is inferred to be underlain by alluvium and colluvium deposits (Q1a), in the immediate vicinity of the meandering Hōteo River, and Northland Allochthon deposits (Kk). While the geological maps indicate Northland Allochthon deposits to be present immediately to the west of the proposed landfill site, Northland Allochthon has not been identified in any of the geotechnical investigations undertaken to date.

The interpretation of the geotechnical investigations and the geotechnical considerations for design of the Auckland Regional Landfill project are discussed in the Geotechnical Interpretive Report (GIR) (Technical Report B, Volume 2).

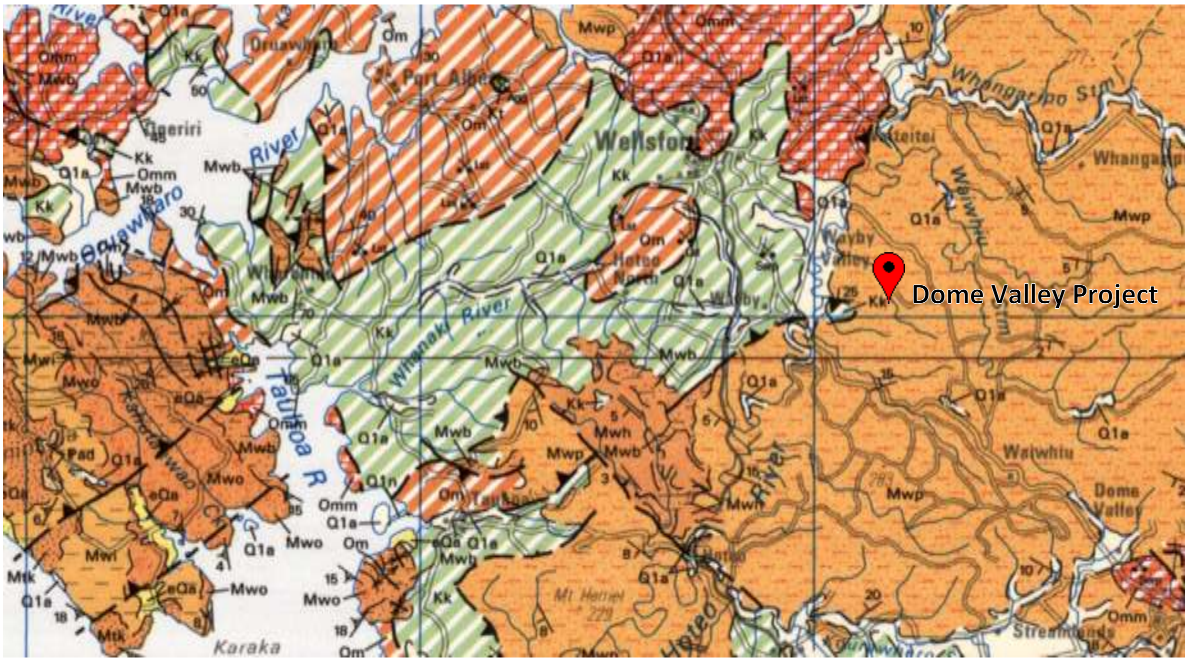

Figure 3.1: Extract of 1:250,000 Geological Map for the Auckland region. Project site shown by the red marker.

3.2 Regional tectonic setting

New Zealand lies along the boundary between the Australian and Pacific tectonic plates. In the North Island the relative displacement between these plates is taken up along the Hikurangi subduction zone off the east coast, where the Pacific plate is moving under the Australian plate at an estimated rate of about 40 mm per year (Stirling et al. 2012). This relative plate motion is expressed by a large number of active faults and a high rate of earthquakes, particularly in the forearc region to the east and south of the Taupo Volcanic Zone.

The Auckland Regional Landfill project site is located in a region that is subject to a relatively low seismic hazard in New Zealand terms. The main nearby active fault zones are the Kerepehi fault zones (characteristic magnitudes M_w6.6 to M_w7.2) located about 70 km to the south east and Wairoa North fault zone (characteristic magnitude M_w6.7) about 80 km to the south.

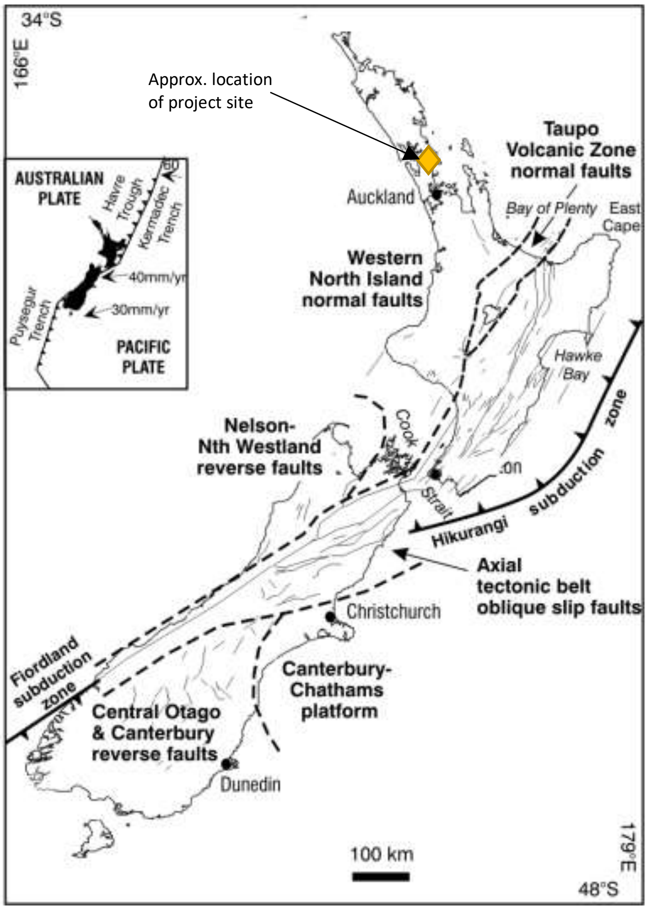

Figure 3.2: Active fault zone courtesy of GNS Science. Relatively low level of seismic activity within the Auckland region.

4 Site specific input for seismic hazard analysis

4.1 General

Probabilistic seismic hazard analysis (PSHA) generally consists of two main components:

1. A seismic source characterisation (SSC) model. This defines the known earthquake rupture sources (e.g. geographic location, fault geometry, faulting type, magnitude) and their associated probabilities of occurrence with time in a region.
2. One or more predictive relationships to estimate the level of ground motion produced at a particular site – commonly referred to as ground motion prediction equations (GMPEs) or more recently referred to as ground motion models (GMMs). These estimate the ground motion intensity (e.g. peak ground acceleration or spectral acceleration) at the site resulting from any possible earthquake rupture (with respect to size and location), for a given probability of exceedance.

The SSC model and GMPEs that have been adopted for this analysis are discussed in the following sections.

4.2 Seismic source characterisation model

The probabilistic seismic hazard analysis reported herein uses the most recently published seismic source characterisation (SSC) model for New Zealand developed by GNS Science. This SSC model is the basis for the 2010 National Seismic Hazard Model (NSHM) for New Zealand (Stirling et al. 2012) and provides an estimate of the magnitude, frequency and location of earthquake ruptures. It has two parts;

1. A fault source model, representing only known active fault sources as rupture planes. The fault source model only considers the characteristic or maximum magnitude rupture on these faults. Smaller magnitude ruptures are represented by the background distributed source model.
2. A background distributed source model, representing the occurrence of earthquakes on known faults less than the characteristic magnitude and unknown faults. The background model comprises a multi-layered grid of point sources each with its own magnitude-frequency distribution as assessed from historical earthquake records.

Figure 4.1 shows a depiction of the fault source model and the background seismicity regions used for this PSHA.

The 2010 SSC model is an update of the earlier SSC model published in 2002 (Stirling et al. 2002) which was used as the hazard basis for NZS1170.5 and the NZTA Bridge Manual. Compared to the 2002 model the updated 2010 model includes the addition of over 200 newly identified onshore and offshore fault sources as well as an additional 11 years of seismicity data. In particular, around the Auckland region, the inclusion of new seismicity data has resulted in a lower estimate of large-magnitude background seismicity and a lower overall hazard for the region (Stirling 2012).

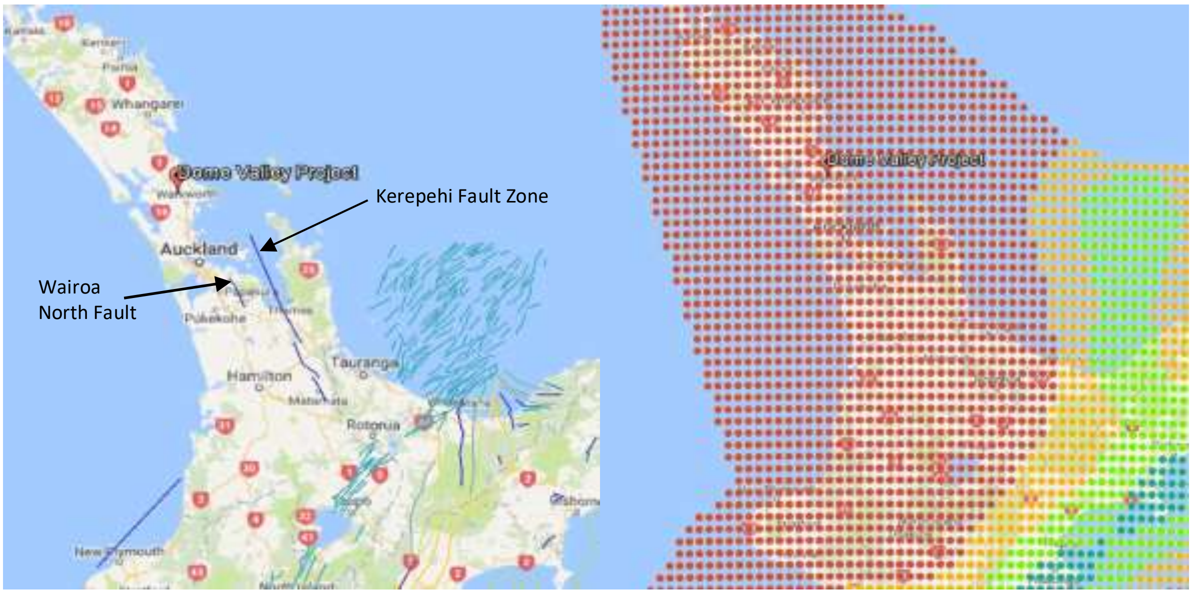

Figure 4.1: Left: Modelled characteristic fault sources coloured by fault type. Right: Seismotectonic background seismicity grid coloured by N-value (expected number of events > M_w5) at 30 km depth.

4.3 Site characterisation

4.3.1 Ground model

T+T has carried out a set of field investigations between 26 February 2018 and 7 June 2018, which consist of:

- 14 machine cored boreholes drilled between 25 to 50 m depth;
- 21 hand augured boreholes drilled to a maximum depth of 4.0 m;
- 10 test pits excavated by a hydraulic excavator to a maximum depth of 4.5 m;
- Geophysics consisting of downhole shear wave velocity and Multi-channel Analysis of Surface Waves (MASW) testing;
- Installation of groundwater monitoring wells in all boreholes; and
- Rock mass permeability (Packer Testing) testing.

The factual results from these investigations, including site plans showing the location of the investigations, are presented in the T+T Geotechnical Factual Report (GFR) (Technical Report A, Volume 2) for the Auckland Regional Landfill project. An interpretation of the investigations undertaken for preliminary design purposes are presented in the T+T Geotechnical interpretive report (GIR) (Techncial Report B, Volume 2).

The shear wave velocity profile for the PSHA was estimated using downhole shear wave velocity testing undertaken in BH01, BH02, BH03, BH04, BH06 and BH08. These boreholes were undertaken along the existing ridgeline and indicate similar ground conditions, which consists of Pakiri Formation residual soil, typically between 5 m and 11 m below ground surface, overlying Pakiri Formation rock.

4.4 Shear wave velocity profile

The time-average shear wave velocity of the upper 30 m of soil and rock ($V_{s,30}$) at the site is a key input into the ground motion prediction equations (GMPE) used for this PSHA. As noted above, the

V_{s30} for the Auckland Regional Landfill project PSHA has been estimated from site specific downhole shear wave velocity testing.

The downhole shear wave velocity testing for the project was carried out by Resource Development Consultants Ltd (RDCL) between 22 and 25 May 2018 within the six boreholes noted above. The measurements indicate $V_{s,30}$ to vary between 369 m/s and 542 m/s. The interpreted $V_{s,30}$ with depth and the shear wave travel times are presented in Figure 4.2 below.

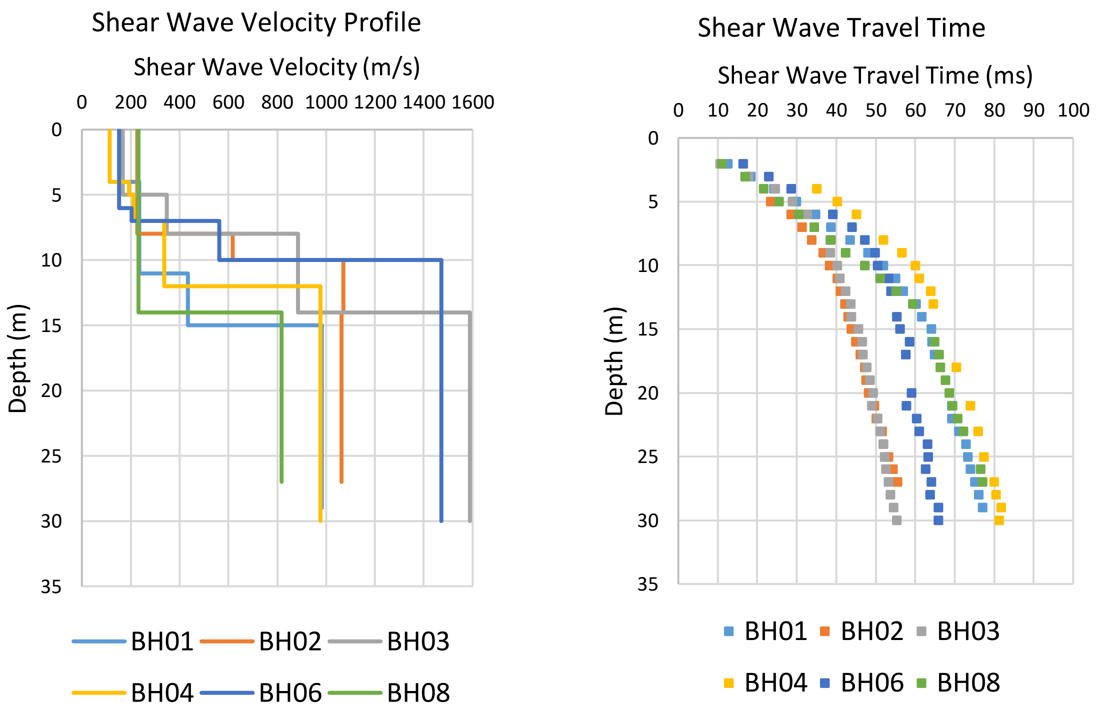

Figure 4.2: Left: Interpreted shear wave velocity profile based on downhole shear wave velocity testing and stratigraphy interpreted from boreholes. Right: Average vertical shear wave travel time with depth

The lowest time-average shear wave velocity of the upper 30 m of soil and rock (a V_{s30} of 369 m/s) that was interpreted from the downhole testing has been adopted for this PSHA. A sensitivity analysis considering a mean V_{s30} of 439 m/s and the highest interpreted V_{s30} of 542 m/s were undertaken and are discussed in Section 6.6.

Although the near surface ground stiffness (e.g. $V_{s,30}$) is generally the most important factor when estimating the site effects on ground shaking, the ground conditions at depth can also be important, particularly for long period motions. The ground motion prediction equations consider this by using $Z_{1.0}$ (depth to a V_s of 1000 m/s) and $Z_{2.5}$ (depth to a V_s of 2500 m/s) to characterise the deep ground stiffness at the site.

The average shear wave velocities from the downhole shear wave testing indicated that the depth to an average V_s of 1000 m/s was typically between 10 m to 15 m. As such a depth of 15 m has been adopted for $Z_{1.0}$. The cross section presented in the 1:250,000 geological map for the Auckland region indicates that the depth to marine basement rock (Waipapa Group) is approximately 1.2 km near the site location. It has been assumed that the basement rock has a V_s of greater than 2500 m/s. As such $Z_{2.5}$ has been assumed to be 1.2 km. Sensitivity analysis show that the PSHA is relatively insensitive to the $Z_{1.0}$ and $Z_{2.5}$ assumptions.

Table 4-1: Site shear wave velocity parameterisation values used for this assessment.

Site parameter	Adopted value for this study
$V_{s,30,}$ Time-averaged shear wave velocity over the top 30 m	369 m/s
$Z_{1.0}$, Depth to a shear wave velocity of 1000 m/s	15 m
$Z_{2.5}$, Depth to a shear wave velocity of 2500 m/s	1.2 km

4.4.1 NZS1170.5 Subsoil class

An assessment of the site subsoil class as defined by NZS1170.5 has been carried out and is summarised below. This assessment has been carried out to enable comparison of the site specific uniform hazard spectra (UHS) with the elastic spectra based on NZS1170.5.

As per NZS1170.5, the preferred method for assessment of site subsoil class is through using shear wave travel times or shear wave velocities.

1. The site is underlain by material with shear wave velocity less than 300 m/s and consists of a surface soil layer that is more than 3 m thick. As such the site is not Subsoil Class A or Class B.
2. The site is not underlain by more than 10 m of very soft material (defined as either having a shear wave velocity of less than 150 m/s, SPT N-value less than 6 or undrained shear strength less than 6). As such the site is not Subsoil Class E.
3. A low amplitude natural period for the site was estimated to vary between 0.12 and 0.26 s. The low amplitude natural period was determined as four times the shear wave travel time from top of rock to surface. The top of rock was defined as the top of highly weathered to unweathered Pakiri Formation rock. As the low amplitude natural period is less than 0.6 s, this indicates that the site is Subsoil Class C.

4.5 Ground motion prediction equations (GMPEs)

4.5.1 Consideration of epistemic uncertainty

Epistemic uncertainty refers to uncertainty which arises due to lack of knowledge about a particular model or parameter. An example is the uncertainty in the assessment of which GMPE is most appropriate to be used for a particular scenario under consideration.

There are a large number of available ground motion prediction equations that have been independently developed (e.g. Douglas 2016). Each of these GMPEs provide different estimates of ground motion intensity level depending on the specific source and site conditions, and some are more appropriate for shallow (e.g. crustal) earthquakes while others are developed more specifically for deeper (e.g. subduction) events.

There is uncertainty as to which is the best equation for any given condition and it is therefore standard practise to consider a suite of GMPEs for use in any seismic hazard analysis. Each of the considered GMPEs should then be assigned a relative weighting based on their estimated reliability given the site-specific conditions and whether they incorporate region-specific (in this case New Zealand) data. In selecting the weightings of GMPEs used in this study, we considered the degree to which each GMPE judged to provide an adequate representation of the ground motion effects expected at the site.

4.5.2 GMPEs for crustal earthquakes

The following ground motion prediction equations have been considered for shallow crustal earthquakes which dominate the hazard in the Auckland region.

1 Abrahamson et al. (2013),
2 Boore et al. (2013),
3 Campbell and Bozorgnia (2013),
4 Chiou and Youngs (2013), and
5 Bradley (2013).

The first four GMPEs were developed as part of the NGA-West2 programme (Bozorgnia et al. 2014). These are revisions of the GMPE models published in 2008 for the first NGA-West programme and incorporate additional strong motion data, especially for smaller magnitude earthquakes. The NGA-West models are currently used to develop the U.S. National Seismic Hazard Maps for the western United States.

Bradley (2013) compared the NGA-West GMPEs and found that the Chiou and Youngs (2008) model provided the best fit to the New Zealand strong motion dataset. The functional form of the Chiou and Youngs (2008) model was then adopted and the model coefficients modified to develop a GMPE that incorporates New Zealand data. This approach of adopting a model based on a large global dataset, and calibrating it to optimally fit the New Zealand data has the advantage that the global dataset provides constraints where the New Zealand dataset alone would be insufficient (generally larger magnitude, short distance records).

In line with international best practise, the GMPEs were weighted in the form of a logic tree (refer to the diagram in Section 5.2). More weight (50%) was given to the Bradley (2013) model as it is the only model explicitly calibrated with the New Zealand dataset. The remaining 50% weighting was evenly split between three of the NGA-West2 GMPEs. As the Chiou and Youngs (2013) model shares a similar functional form to the Bradley (2013) model, it was not used. Table 4-2 summarises the weightings applied to each GMPE for crustal earthquakes.

Table 4-2: GMPE logic tree weightings for crustal earthquakes.

Ground motion prediction equation for crustal earthquakes	Weighting applied for this study
Abrahamson et al. (2013)	16.7%
Boore et al. (2013)	16.7%
Campbell and Bozorgnia (2013)	16.7%
Bradley (2013)	50.0%

4.5.3 GMPEs for subduction earthquakes

Subduction interface and intraslab earthquake sources are located at significant distances from the Auckland Regional Landfill project site and therefore only have a minor contribution to the hazard at the project site. For the purpose of this analysis, the Bradley (2010) and Abrahamson et al. (2016) GMPEs (also referred to as the BC Hydro GMPE) have been used with equal weightings. However, a sensitivity analysis showed the results to be insensitive to the adopted GMPEs and therefore these are not discussed here further.

4.5.4 Near-fault directivity effects

The nearest known faults to the Auckland Regional Landfill project site are Kerepehi fault zone, with a characteristic magnitude of M_w7.2 and an average recurrence interval of 20,000 years and Wairoa North fault zone, with a characteristic magnitude of M_w6.7 and an average recurrence interval of about 13,000 years. Both of these fault zones are approximately 70 km to 80 km south-southeast of the site.

Given the distance to the nearest active faults, and the relatively long recurrence intervals associated with these faults, near-fault directivity effects are unlikely to contribute significantly to the hazard at the site, and are therefore not required for this study.

5 Probabilistic seismic hazard analysis

5.1 Methodology

Current practise in PSHA is based on the methodology originally proposed by Cornell (1968).

There is a large degree of uncertainty in the location, size and resulting shaking intensity of future earthquakes. PSHA is fundamentally a method that combines these uncertainties to calculate the probability that a ground motion intensity measure will exceed a given value during a particular time period.

The annual probability of exceedance of a particular intensity measure (IM), e.g. PGA or spectral acceleration, is calculated by integrating the contribution from all seismic sources as follows.

$$\lambda(IM > x) = \sum_{i=1}^{n_{sources}} \lambda(M_i > m_{min}) \int_{m_{min}}^{m_{max}} \int_{0}^{r_{max}} P(IM > x \mid m, r)\, f(m) f(r|m)\, dr\, dm$$

Where,

λ = annual probability of exceedance

$P(IM > x \mid m, r)$ = Probability that IM exceeds x, given magnitude, m and distance, r.

$f(\,)$ = Represents a probability density function.

The PSHA presented here has been undertaken using the OpenSHA software (Field et al. 2003). This program has been developed by the Southern California Earthquake Centre (SCEC) and the United States Geological Survey (USGS), and is widely used in the international earthquake engineering profession.

5.2 Logic tree model

For a PSHA to be rigorous it is required to account for all significant sources of uncertainty, both aleatory and epistemic. Aleatory variability refers to uncertainties that, for the given models adopted, are deemed to be purely random and unpredictable. For example the ground motion at a site given the predicted median value of motion. Aleatory uncertainty is considered within the hazard integral above. Epistemic uncertainties within the GMPEs (as discussed in Sections 4.5.1) have been accounted for outside the hazard integral using a logic tree. The logic tree model adopted is depicted in Figure 5.1.

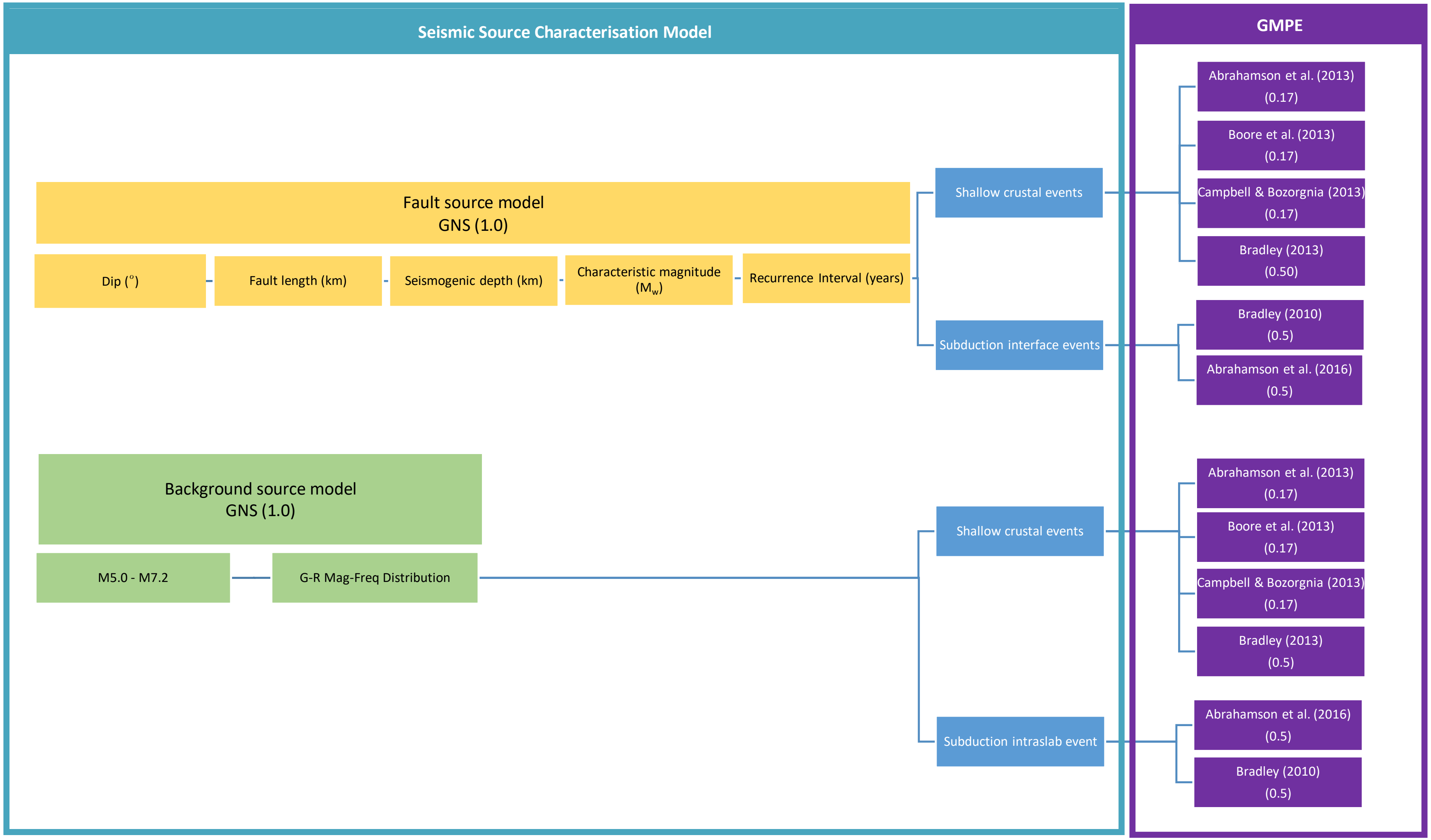

Figure 5.1: Logic tree model adopted for this PSHA.

6 Results and discussion

6.1 Hazard curves

The principal output of a PSHA is the seismic hazard curve, which provides the annual rate of exceedance of a particular ground motion intensity measure, obtained from the integral presented in Section 5.1.

Figure 6.1, Figure 6.2 and Figure 6.3 present the seismic hazard curves for PGA, SA (T=0.4s) and SA (T=1.0s) respectively that were obtained from this site-specific PSHA. For comparison, the design values based on NZS1170.5 are also provided. The PGA and SA values presented are for the 'larger horizontal component' (refer to Section 6.4 for definition). A discussion of the comparison between the hazard derived from the PSHA and the hazard determined in accordance with the Bridge Manual and NZS1170.5 is presented in Section 6.3.

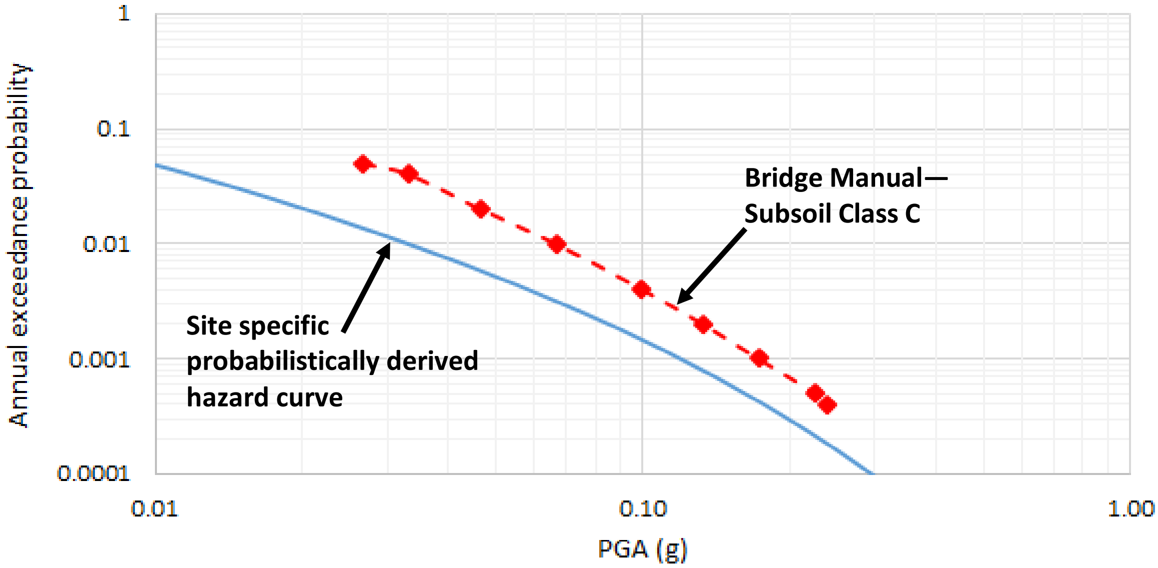

Figure 6.1: Site specific seismic hazard curve for PGA. The PGA hazard curve determined in accordance with Bridge Manual for Subsoil Class C site is provided for comparison.

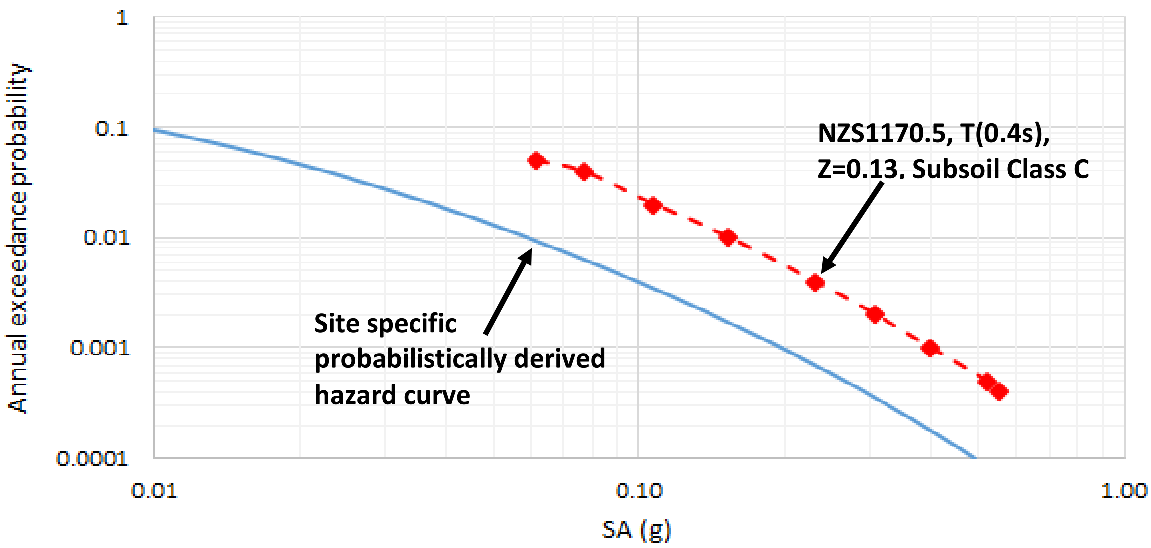

Figure 6.2: Site specific seismic hazard curve for SA (T=0.4s). The SA hazard curve (T=0.4s) determined in accordance with NZS1170.5 for Z=0.13 and Subsoil Class C is provided for comparison.

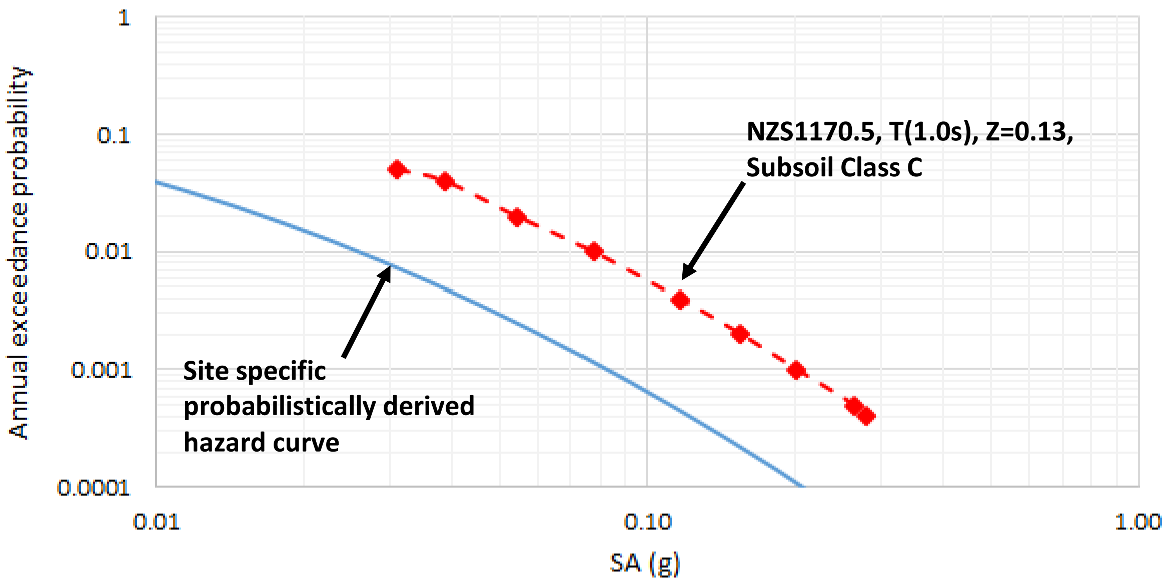

Figure 6.3: Site specific seismic hazard curve for SA (T=1.0s). The SA hazard curve (T=1.0s) determined in accordance with NZS1170.5 for Z=0.13 and Subsoil Class C is provided for comparison.

6.2 Uniform hazard response spectra

One way to present the seismic hazard at the site is with uniform hazard spectra (UHS). A UHS represents a locus of spectral accelerations at various vibration periods which have the same annual probability of exceedance (or equivalently, return period).

It is important to note that the different points on the UHS are obtained from separate PSHAs (i.e. seismic hazard curves) which are independent of each other. The rupture scenario that dominates the hazard for a particular spectral period is different from that at other spectral periods. Hence, the different points on the UHS have different dominant earthquakes and therefore no single earthquake rupture should be expected to produce a ground motion similar to the UHS over a wide range of vibration periods.

The site-specific UHS for the 25, 500, 1000 and 2500 year return periods are presented in Figure 6.4 for 5% of critical damping. For comparison the NZS1170.5 design spectra for Z=0.13 and Subsoil Class C are also shown. Values are for the 'larger horizontal component' (refer to Section 6.4 for definition).

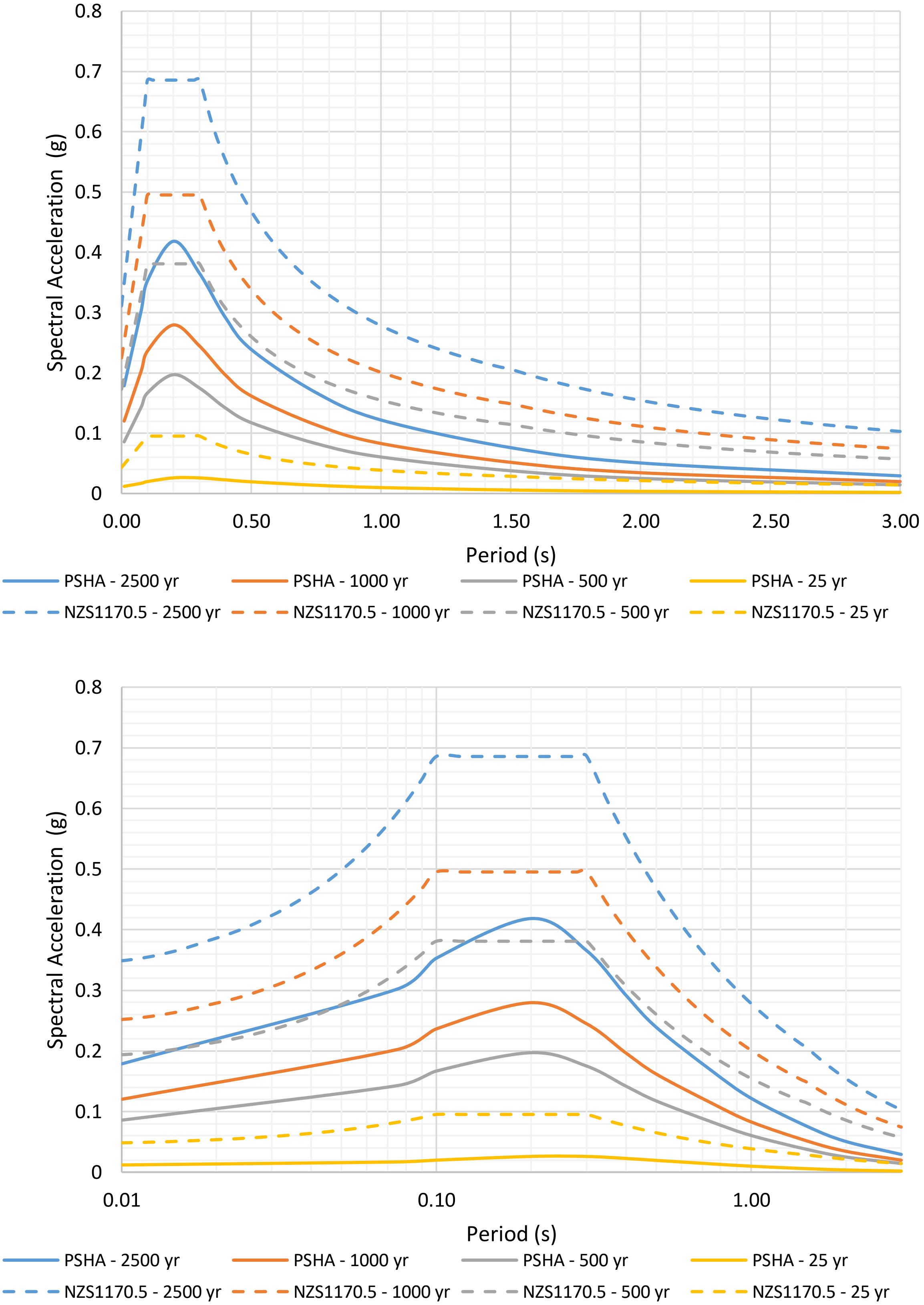

Figure 6.4: Site specific uniform hazard spectra (UHS) for the 25, 500, 1000 and 2500 year return periods. NZS1170.5 spectra for Z=0.13 and Subsoil Class C is presented for comparison. Top: Linear-Linear plot. Bottom: Linear-Log plot.

6.3 Comparison with the Bridge Manual and NZS1170.5

6.3.1 Comparison of site specific probabilistically derived PGA against Bridge Manual

As shown on Figure 6.1, for all return periods, the PGA derived from the site specific PSHA is consistently less than the PGA determined in accordance with the Bridge Manual for Subsoil Class C. The lower PGA derived from the PSHA reflects the relatively low level of seismic hazard at the Auckland Regional Landfill project site compared to the probabilistic value presented in the Bridge Manual for the Warkworth region.

The unweighted PGA and magnitude derived from the PSHA for liquefaction analysis purposes and the PGA and magnitude determined in accordance with the Bridge Manual for 25, 500, 1000 and 2500 year return periods are summarised in Table 6.1 below. The magnitude and distance deaggregation of the PGA hazard from the PSHA is discussed in Section 6.5.

Table 6.1: Summary of PGA and magnitude derived from PSHA. PGA and magnitude determined in accordance with the Bridge Manual for Subsoil Class C provided for comparison.

PSHA			Bridge Manual		
Return Period	PGA (g)	Magnitude	Return Period	PGA (g)	Magnitude
25	0.01	5.7	25	0.03	5.9
500	0.09	5.7	500	0.13	5.9
1000	0.12	5.8	1000	0.17	5.9
2500	0.18	5.8	2500	0.24	5.9

Although the probabilistic PGA and magnitude derived from the PSHA is lower than the respective PGA-magnitude pairs determined in accordance with the Bridge Manual, the Bridge Manual requires that when designing for ULS effects, the earthquake loading "shall not be taken to be less than those due to a 6.5 magnitude earthquake at 20 km distance". As such for a Subsoil Class C site, the lower bound PGA and magnitude presented in the Bridge Manual is summarised in Table 6.2 below.

Table 6.2: Lower bound PGA and magnitude that shall be considered when designing for ULS effects in accordance with the Bridge Manual

PGA	Magnitude
0.19	6.5

Where multiple pairs of magnitude and PGA exist, the Bridge Manual requires evaluation of each individual pair for liquefaction analysis (i.e. normally both the PGA-magnitude determined from the PSHA and the lower bound PGA-magnitude pair presented in the Bridge Manual will need to be evaluated). However, for the Auckland Regional Landfill project site as the probabilistically derived PGA and magnitude, at all return periods considered, are less than the lower bound values, the lower bound PGA of 0.19g and magnitude 6.5 will clearly govern for ULS liquefaction analysis. Similarly, as the probabilistically derived PGA is less than the lower bound PGA, at all return periods considered, the lower bound PGA of 0.19g will clearly govern for geotechnical design including stability assessment of cut and fill slopes.

6.3.2 Comparison of site specific hazard against NZS1170.5

The uniform hazard spectra (UHS) derived from the PSHA compared against the NZS1170.5 design spectra was presented in Section 6.2. The NZS1170.5 design spectra is calculated as the product of the spectral shape factor ($C_h(T)$), the hazard factor (Z), the return period factor (R) and the near-fault factor (N(T,D)). A brief description of these factors are provided in the Glossary section at the start of this report.

NZS1170.5 defines the hazard factor (Z) as 0.5 times the magnitude-weighted 5% damped response spectrum acceleration (SA) for 0.5 s period for Subsoil Class C (shallow soil site) that has a return period of 500 years. As such the hazard factor spatially varies across New Zealand and the codified values have been determined from the GNS probabilistic seismic hazard model (McVerry, 2003) with the exception of regions where there is a relatively low level of seismic hazard. In regions where there is a relatively low level of seismic hazard, a minimum deterministic hazard factor to determine ULS loading is specified in NZS1170.5.

For the Auckland region, NZS1170.5 prescribes a minimum level of hazard (Z=0.13) rather than a level of hazard that has been probabilistically-derived. As expected, this results in the hazard for the Auckland Regional Landfill project derived from the PSHA being significantly lower than the NZS1170.5-based hazard, as shown on Figure 6.2, Figure 6.3, and Figure 6.4.

The prescribed minimum level of hazard for regions of low seismicity allows for a margin against collapse in earthquakes that may occur without identification of pre-existing surface fault traces. More specifically the minimum level of hazard corresponds to two-thirds of the 84th percentile motions in a magnitude M_w 6.5 normal-faulting earthquake at a closest distance of 20 km from the site. This magnitude has been selected because it is about the largest that is likely to occur in the lower seismicity regions of New Zealand without identification of pre-existing surface traces. The 84th percentile level is generally taken as an upper-bound level for deterministic scenario spectra. The two-thirds scaling factor is based on the assumption that a margin against collapse in major earthquake shaking implied by the use of typical design procedures is 1.5, and that no margin against collapse is required for the most severe earthquake shaking.

As such, while the site specific hazard derived from the PSHA is less than the NZS1170.5-based hazard, the minimum level of hazard stipulated in NZS1170.5 is required to be applied in determining the ULS earthquake loading for the Auckland Regional Landfill project.

To apply the NZS1170.5 design spectra, the Subsoil Class for the site is required to be assessed. As discussed in Section 4.4.1, the site was classified as Subsoil Class C in accordance with NZS1170.5. In order to confirm if the probabilistically derived hazard is consistent with this classification, the UHS derived from the PSHA have been compared against the design spectra for Subsoil Class C and Subsoil Class D determined in accordance with NZS1170.5. To facilitate this comparison, the design spectra from NZS1170.5 have been revised using a probabilistic hazard factor determined for the Auckland Regional Landfill project site.

The probabilistic hazard factor for the Auckland Regional Landfill project site was determined as 0.5 of the unweighted 5% damped response spectrum acceleration (SA) for 0.5 s period for Subsoil Class C (shallow soil site) that has a return period of 500 years. It was assumed that the shear wave velocities presented in Table 6.3 below is representative of a shallow soil site.

Table 6.3: Shear velocities used to determine a probabilistic hazard factor for the site

Site parameter	Adopted value for this study
$V_{s,30}$, Time-averaged shear wave velocity over the top 30 m	360 m/s
$Z_{1.0}$, Depth to a shear wave velocity of 1000 m/s	Calculated from $V_{s,30}$ (Campbell & Bozorgnia 2013)
$Z_{2.5}$, Depth to a shear wave velocity of 2500 m/s	Calculated from $V_{s,30}$ (Campbell & Bozorgnia 2013)

A UHS for a return period of 500 years, using the shear wave velocities provided in Table 6.3, is shown on Figure 6.5 below. As shown on the UHS, the unweighted 5% damped response spectrum acceleration for 0.5 s period is approximately equal to 0.133 g. This results in a site specific hazard factor of approximately 0.066.

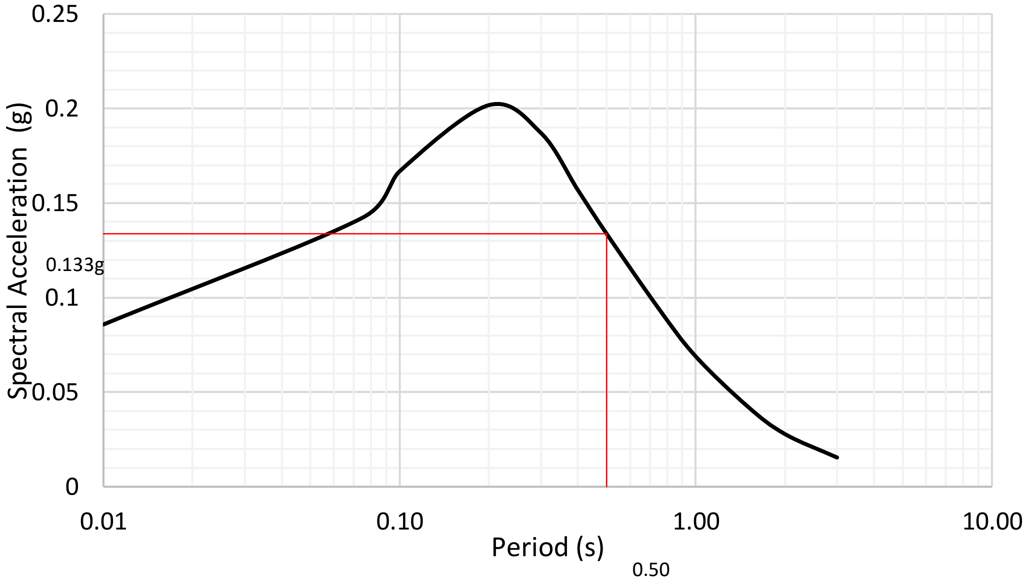

Figure 6.5: UHS for the Auckland Regional Landfill project site assuming a standard shear wave velocity (V_{s30} 360 m/s) for a Subsoil Class C site and 500 year return period.

The UHS derived from the PSHA and the revised NZS1170.5 design spectra for Subsoil Class C and Subsoil Class D using a Z=0.066 is presented in Figure 6.5 below. The plot shows that the UHS derived from the PSHA is reasonably comparable to the NZS1170.5 design spectra for Subsoil Class C.

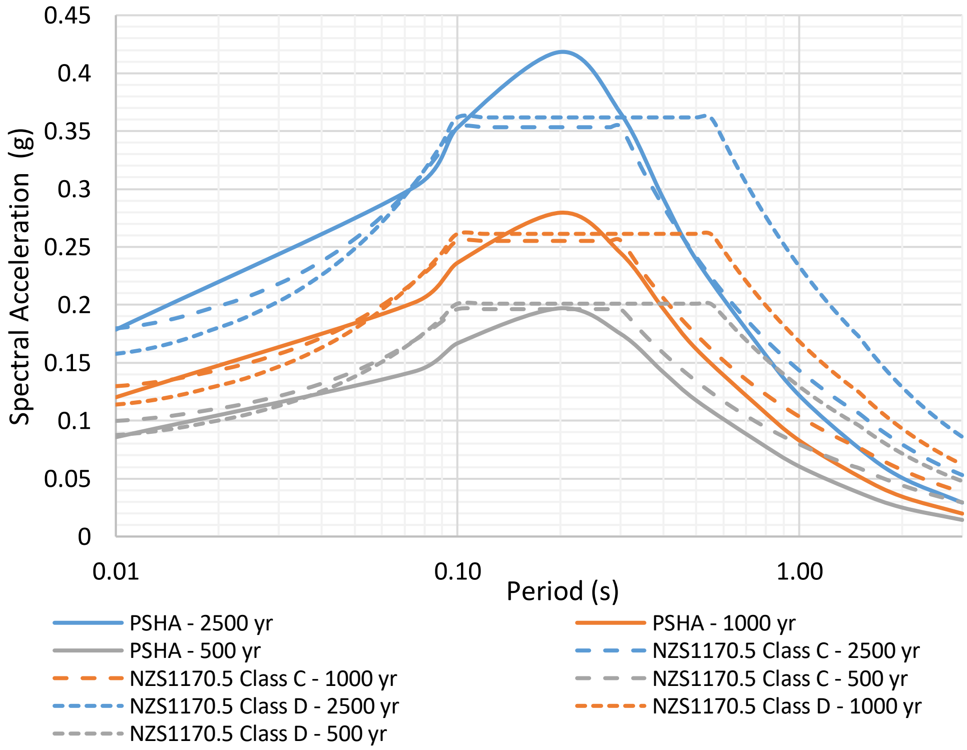

Figure 6.6: Comparison of UHS for 500, 1000 and 2500 year return periods and NZS1170.5 spectra for Z=0.066 and Subsoil Class C and D.

6.4 Orientation of ground motions

The acceleration response spectra presented in this report are for the '*larger of two horizontal components'*, which is consistent with the definition for spectral acceleration (SA) used in NZS1170.5. When using the response spectra, it is important to ensure that the definition of SA is consistent with the structural analysis so that design and performance expectations are not biased. This section provides an explanation of the different SA definitions.

The SA value of a single component of a ground motion is defined as the maximum response of a single degree freedom system with a specified period. For a ground motion with shaking in multiple horizontal directions, some method is needed to combine the directionally-varying single-component SA values into a single numerical value. This concept is depicted in Figure 6-7. Table 6-4 describes the common definitions.

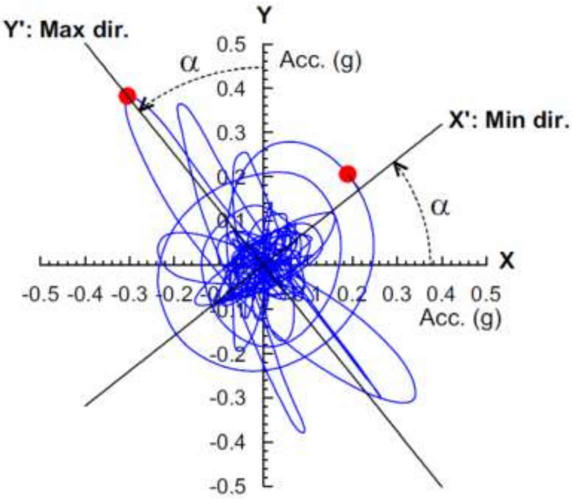

Figure 6-7: *Trace of acceleration orbit of a single lumped mass oscillator. The two axes (X and Y) refer to the directions in the horizontal plane in which ground motion is recorded. Angle α represents the rotation of those axes to the direction of minimum (X', SA_{RotD0}) and maximum (Y', $SA_{RotD100}$) ground motion. SA_{RotD50} is the median (50^{th} percentile) response when rotated over all directions. (Source: Stewart et al. 2011).*

Table 6-4: Ground motion orientation definitions.

Name	Description
SA_{RotD50}	50^{th} percentile (i.e. median) of the SA value obtained by rotating the horizontal components through all angles. The NGA-West2 GMPEs have been developed based on this definition.
$SA_{RotD100}$	The 100^{th} percentile (i.e. maximum) of the SA value obtained by rotating the horizontal components through all angles. This is the definition used in the U.S. NEHRP *Provisions* (BSSC 2015).
SA_{Larger}	Larger value of the two as-recorded components at each period. $SA_{RotD50} \leq SA_{Larger} \leq SA_{RotD100}$ This is the definition currently adopted in NZS1170.5, and is also the definition that has been adopted for the purpose of this SHA.

Generally, the SA_{RotD50} spectra may be considered appropriate for the design of structures that are *azimuth-dependent*, i.e. the dynamic properties of the structure, e.g. stiffness, are dependent on the orientation being considered (Stewart et al. 2011). However, in the case of an *azimuth-independent* structure, for example in-plan axisymmetric structures such as silos, the expected acceleration response will be larger than the SA_{RotD50} value (by definition, 50% of orientations for an *azimuth-*

independent structure will experience a response larger than the median value). In this case a different definition of horizontal spectra for design may be more appropriate (i.e. $SA_{RotD100}$).

The elastic response spectra in NZS1170.5 are based on PSHA results obtained using the SA_{Larger} definition for SA. However, this definition has the limitation that it is dependent on instrument orientation. As a results, although it is the 'larger' of the two components in their as-recorded orientation, it is not the 'largest' (i.e. $SA_{Larger} \leq SA_{RotD100}$).

Shahi & Baker (2014) and Bradley & Baker (2014) provide modification ratios that may be used to convert between the different definitions of SA. These are presented in Table 6-5.

For the Auckland Regional Landfill project it has been assumed that there are no *azimuth-independent* structures proposed. Where no *azimuth-independent* structures are present using $SA_{RotD100}$ as the spectrum of a single ground motion component may lead to overly-conservative estimates of structural demand (e.g. Stewart et al. 2011). Therefore, SA_{larger} has been used for deriving the hazard curves and UHS for this PSHA (this is consistent with NZS1170.5).

Table 6-5: Average conversion ratios for different SA definitions that have been used for this study.

Period (s)	$SA_{RotD100}/SA_{RotD50}$ (Shahi & Baker 2014)	$SA_{RotD100}/SA_{Larger}$ (Bradley & Baker 2014)
0.01	1.19	1.08
0.02	1.19	1.08
0.03	1.19	1.08
0.05	1.19	1.08
0.07	1.19	1.08
0.1	1.19	1.08
0.15	1.20	1.08
0.2	1.20	1.08
0.25	1.21	1.08
0.3	1.22	1.08
0.4	1.23	1.08
0.5	1.23	1.08
0.75	1.24	1.08
1.0	1.24	1.08
1.5	1.24	1.08
2.0	1.24	1.08

6.5 Hazard deaggregation

The hazard curves presented in Section 6.1 represent the aggregate seismic hazard from all modelled earthquake sources. However, it is useful to understand the relative contribution from each of the sources. The magnitude and distance deaggregation of the PGA hazard is presented in Figure 6.8 for the 25, 500, 1000 and 2500 year return periods. The plots show that the PGA hazard is largely dominated by low-magnitude background seismicity (note that the nearest known active faults are approximately 70 km to 80 km away from the Auckland Regional Landfill project site). As the level of shaking intensity increases (i.e. increasing return period), the mean casual distance, R_{rup}

decreases as the more distance earthquakes of limited characteristic magnitude become incapable of causing those levels of shaking at the Auckland Regional Landfill project site.

The mean magnitude for 25 and 500 year return period PGA is approximately M_w5.7 while the mean magnitude for 1000 and 2500 year return period PGA is approximately 5.8. However, as noted in Section 6.3.1, the Bridge Manual requires the consideration of a lower bound magnitude and PGA combination of M_w 6.5 and 0.19 g for assessing ULS effects. As both the magnitude and PGA derived from the PSHA are lower than this lower bound criteria, a M_w of 6.5 will govern for design.

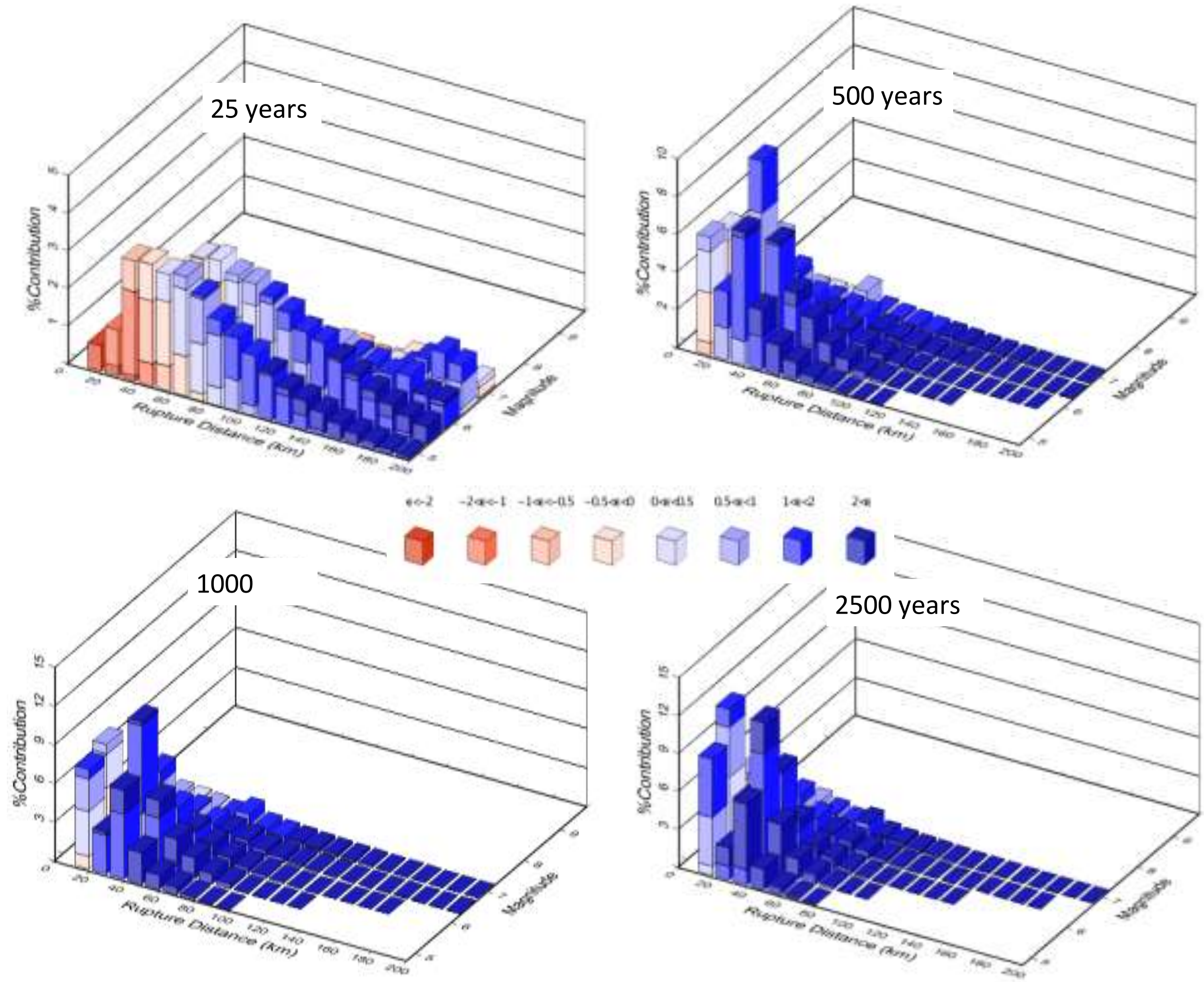

Figure 6.8: Deaggregation of the PGA hazard at the site for 25, 500, 1000 and 2500 year return periods. The colour of the bars (red and blue) indicate the deviation of the ground response from the mean value predicted using the GMPEs. i.e. red bars indicate the hazard contribution is from ground motions below the calculated mean value.

6.6 Sensitivity of the hazard results to V_{s30} values

As presented in Section 4.4, the $V_{s,30}$ interpreted from the downhole shear wave velocity testing varied between 369 m/s and 542 m/s, with an average $V_{s,30}$ of 439 m/s. The lowest interpreted $V_{s,30}$ of 369 m/s has been used for the PSHA. A sensitivity analysis has been undertaken using $V_{s,30}$ of 439 m/s and 542 m/s.

The seismic hazard curve for PGA and UHS for a return period of 1000 years, considering the range of $V_{s,30}$ noted above, are presented in Figure 6.9 and Figure 6.10 below. As shown in Figure 6.9, the hazard curve for PGA is relatively insensitive over $V_{s,30}$ range interpreted from the downhole shear wave velocity testing. There is a relatively small differences in the UHS for a return period of 1000 years over the $V_{s,30}$ range that was assessed. However, the UHS is still comparable to Subsoil Class C particularly at the lower periods (T < 0.2 s).

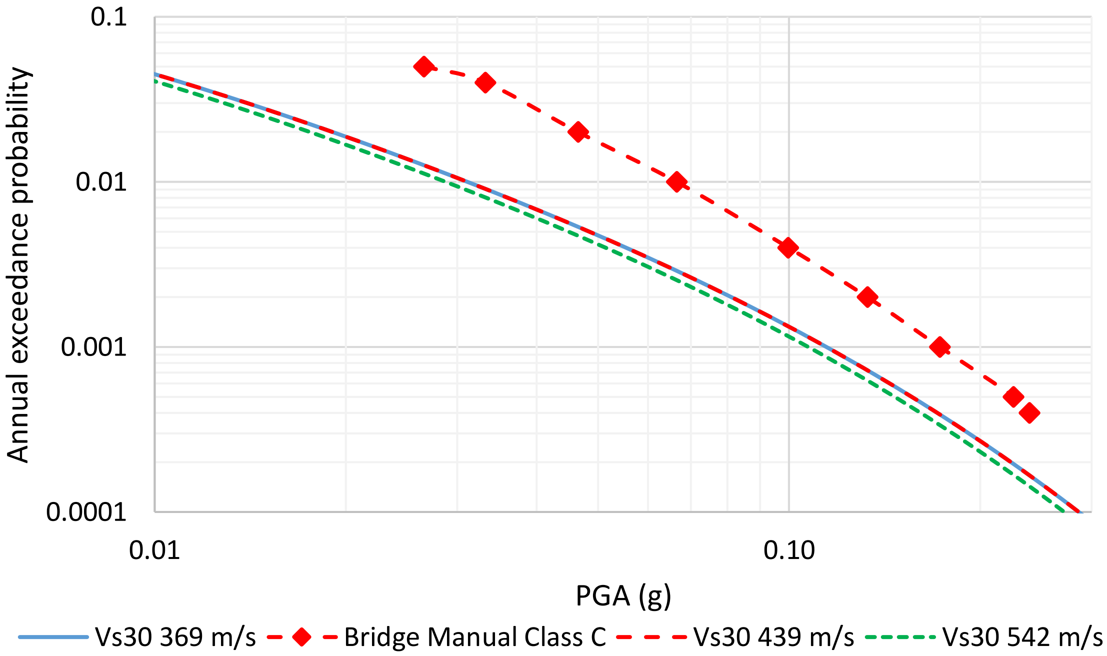

Figure 6.9: Sensitivity of the PGA hazard curve over the range of $V_{s,30}$ interpreted from downhole shear wave velocity testing.

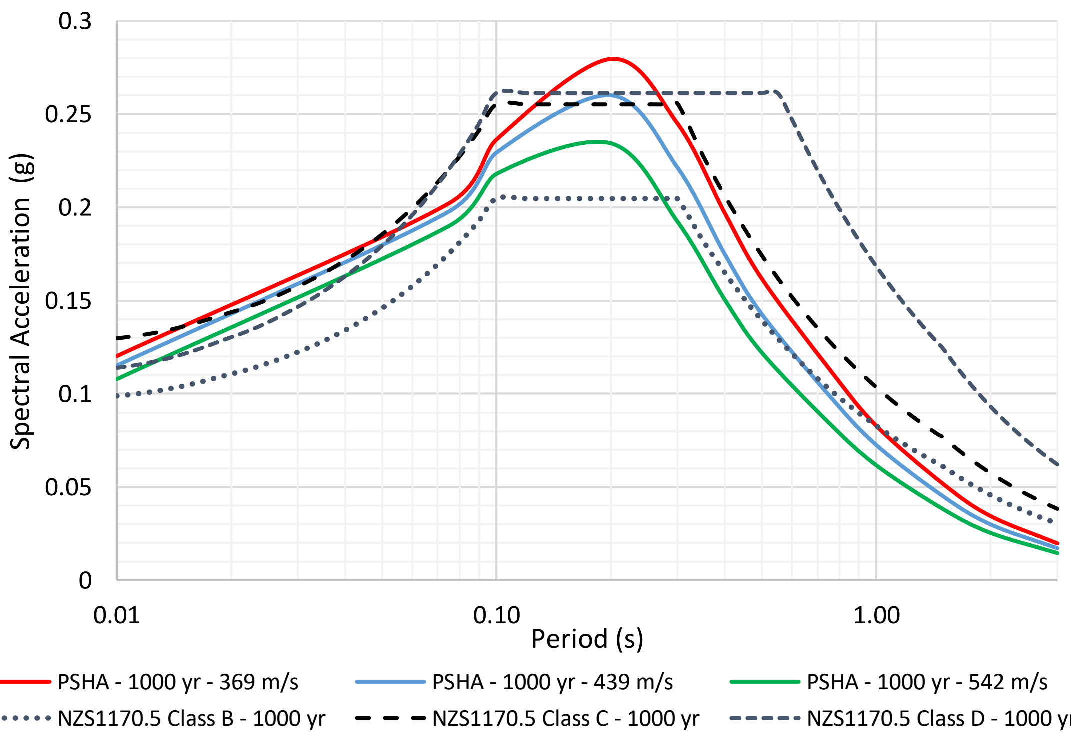

Figure 6.10: Sensitivity of the UHS for the 1000 year return period over the range of $V_{s,30}$ interpreted from downhole shear wave velocity testing. NZS1170.5 spectra are based on a Hazard Factor (Z) of 0.066.

7 Recommended design loads

7.1 Geotechnical design

The PGA and magnitude for liquefaction triggering assessment and the PGA for slope stability assessment and design of soil retaining structures are provided in below.

Table 7-1: Recommended PGA and magnitude for geotechnical design

Recommended parameters for Geotechnical Design		
Return Period	**PGA (g)**	**Magnitude**
25	0.01	5.7
500	0.19	6.5
1000	0.19	6.5
2500	0.19	6.5

- The PGA and magnitude determined from the PSHA and comparison with the Bridge Manual was discussed in Section 6.3.1.
- The PGA and magnitude determined from the PSHA are lower than the minimum criteria outlined in the Bridge Manual for all return periods. This is due to the Auckland Regional Landfill project being in an area of relatively low seismicity.
- As such, the minimum criteria outlined in the Bridge Manual is required to be adopted for ULS design (500, 1000 and 2500 years). However, the results determined from the PSHA mean that the PGA for the 2500 year return period can be reduced to the minimum criteria (PGA of 0.19 g) instead of using the higher PGA of 0.24 g that is determined in accordance with the Bridge Manual.

7.2 Structural design

The design spectra for Subsoil Class C determined in accordance with NZS1170.5 is recommended for structural design.

- The seismic hazard determined from the PSHA and comparison with NZS1170.5 was discussed in Section 6.3.2.
- The seismic hazard determined from the PSHA is consistently lower than the design spectra from NZS1170.5 for all return periods considered.
- However, again as the Auckland Regional Landfill project is in area of relatively low seismicity, the minimum hazard factor for the region outlined in NZS1170.5 will govern. The PSHA confirms that the site behaves as a Subsoil Class C.

8 Conclusions and recommendations

Waste Management NZ Ltd (WMNZ) has engaged Tonkin & Taylor Ltd (T+T) to undertake a probabilistic seismic hazard analysis (PSHA) to support resource consent application for the Auckland Regional Landfill project. The conclusions and recommendations from the SHA are outlined below.

1 The Auckland Regional Landfill project site is located in a region that is subject to a relatively low seismic hazard in New Zealand terms. The nearest active fault zones are the Kerepehi fault zone and the Wairoa North fault zone. These respective fault zones are approximately 70 km to 80 km south-south east of the project site.

2 A set of field investigations were undertaken by T+T for the Auckland Regional Landfill project between 26 February 2018 and 7 June 2018. The field investigations included six downhole shear wave velocity testing carried out by RDCL. The $V_{s,30}$ interpreted from the downhole shear wave velocity testing varied between 369 m/s to 542 m/s. A $V_{s,30}$ of 369 m/s has been adopted for the PSHA. The boreholes, in which the downhole shear wave velocity testing were undertaken, indicate generally similar ground conditions consisting of Pakiri Formation residual soil, typically between 5 m and 11 m below ground surface, overlying Pakiri Formation rock.

3 NZS1170.5 Clause 3.1.3.1 provides a hierarchy for site classification, which specifies measurement of shear wave velocity as the preferred method. As such, based on the shear wave travel time from top of rock to ground surface the Auckland Regional Landfill project site was classified as Subsoil Class C. The hazard spectra derived from the PSHA were generally found to be similar to the NZS1170.5-design spectra for Subsoil Class C confirming the Subsoil Class assessment that was undertaken in accordance with NZS1170.5.

4 The seismic hazard curves determined from the PSHA for PGA, SA (T = 0.4 s) and SA (T = 1.0 s) are presented in Section 6.1. UHS determined from the PSHA for 25, 500, 1000 and 2500 year return periods are provided in Section 6.2. The hazard curve for PGA has been compared against the PGA hazard curve determined in accordance with the Bridge Manual for Subsoil Class C. The hazard curves for SA (T = 0.4 s, 1.0 s) and the UHS for the return periods stated above that were determined from the PSHA have been compared against the hazard curves and design spectra determined in accordance with NZS1170.5.

5 The PGA and magnitude determined from the PSHA for the Auckland Regional Landfill project is lower than the PGA and magnitude determined in accordance with the Bridge Manual for all return periods considered. The Bridge Manual, however, requires the consideration of a lower bound M_w of 6.5 and PGA of 0.19 for a Subsoil Class C site when designing for ULS effects. As the PGA and magnitude derived from the PSHA is lower than these lower bound values for all return periods considered, the lower bound values presented in the Bridge Manual will govern for ULS design. The PGA and magnitude recommended for design are provided in Section 7.

6 The seismic hazard curves for SA (T = 0.4 s) and SA (T = 1.0 s) and the UHS for 25, 500, 1000 and 2500 year return periods are consistently less than respective hazards determined in accordance with NZS1170.5. The difference is due to the Auckland Regional Landfill project site being located in a region of relatively low seismicity. For a region of low seismicity, NZS1170.5 specifies a minimum hazard factor (Z = 0.13) rather than a value that is probabilistically derived. In this PSHA a probabilistic hazard factor of 0.066 was estimated. Although the probabilistic hazard factor is lower than the hazard factor provided in NZS1170.5, when designing to NZS1170.5 the minimum hazard factor presented in NZS1170.5 is required to be applied. **As such, we recommend that the design spectra for Subsoil Class C be adopted for structural design.**

7 The assumptions that this PSHA is based on are presented in this report. The assumptions include:

- A minimum $V_{s,30}$ of 369 m/s has been adopted for the PSHA. The adopted $V_{s,30}$ is based on the lowest interpreted $V_{s,30}$ from downhole shear velocity testing. This is considered conservative based on the sensitivity assessment presented in Section 6.6.
- The PSHA has been undertaken for return periods of 25, 500, 1000 and 2500 years,
- The seismic hazard has been determined for period of 0 s (PGA), 0.4 s and 1.0 s,
- The PGA and SA presented are for the 'larger of two horizontal components'. This is based on the assumption that no azimuth-independent structures are proposed.

Where actual conditions deviate from the assumptions presented in this report (such as identified during design development in detailed design), a review of this PSHA is required.

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10 Applicability

This report has been prepared for the exclusive use of our client Waste Management NZ Ltd, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

Tonkin & Taylor Ltd

Report prepared by: Authorised for Tonkin & Taylor Ltd by:

B. Ananth S Eldridge

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Ananth Balachandra Simonne Eldridge

Geotechnical Engineer Project Director

Report reviewed by Jason Lim.

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Appendix A: Preliminary SSHA

Tonkin+Taylor

Job No: 1005069.1120
13 April 2018

Polaris Project
Confidential Project

Attention: Bruce Horide

Dear Bruce

Dome Valley Site

Preliminary assessment of design seismic ground motions and site-specific seismic hazard

Introduction

Tonkin & Taylor Ltd has been engaged by the Polaris Project to review the options for deriving design ground motion values including a Site-specific Seismic Hazard Assessment (SSHA) for the site located at Dome Valley, Auckland. The aim of an SSHA is to assess the seismic hazard for a specific site, incorporating recent advances in knowledge and the state of practice[1]. An SSHA provides earthquake design parameters values that may be used as an alternative to the loading parameters used for routine engineering design.

Methodology

MBIE provide guidelines[2] on the methods for selecting appropriate ground motion parameters at a site for design in earthquake geotechnical engineering (e.g. slope stability, liquefaction). These are;

- Method 1: Risk based method using the earthquake hazard presented in the NZTA Bridge Manual (2014),
- Method 2: Site-specific probabilistic seismic hazard studies (e.g. SSHA), and
- Method 3: Site-specific response analysis.

For routine engineering design projects Method 1 is usually sufficient to derive earthquake loads. These loads are based on a nation-wide hazard model (an update of the model used for the New Zealand earthquake loadings standard for structures, NZS1170.5:2004) and are presented in Table 1 for the Dome Valley site. These values are considered appropriate for preliminary design purposes.

[1] Bradley B. A. (2015). Benefits of Site-Specific Hazard Analyses for Seismic Design in New Zealand. *Bulletin of the New Zealand Society for Earthquake Engineering*. Copy attached for reference.
[2] NZGS and MBIE (2016) Guidelines for Earthquake Geotechnical Practise in New Zealand. Module 1: Overview of the guidelines.

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Table 1: Assessed earthquake ground motion parameter values based on Method 1 (NZTA Bridge Manual)

Seismic loading parameter	NZTA Bridge Manual (subsoil Class B - rock)	NZTA Bridge Manual (subsoil Class C – shallow soil)
Serviceability Limit State (SLS)		
50 year return period	0.04g M_w5.9	0.05g M_w5.9
150 year return period	0.07g M_w5.9	0.09g M_w5.9
Ultimate Limit State (ULS) (1)		
500 year return period	0.10g M_w5.9	0.13g M_w5.9
1000 year return period	0.13g M_w5.9	0.17g M_w5.9
Minimum ULS load(2)	0.14g M_w6.5	0.19g M_w6.5

Notes:

1. For the ULS case, the design ground motions shall be taken as the greater of the relevant return period or the minimum ULS load.
2. The minimum ULS load is based on the 84th percentile motion from a Mw6.5 normal-faulting earthquake at 20 km distance, Maximum Credible Earthquake (MCE).
3. The appropriate subsoil class for design should be informed by site-specific ground investigations and in accordance with NZS1170.5:2004.

Methods 2 and 3 are preferred for more significant projects or more complex sites. Method 2 (e.g. an SSHA) allows site-specific peak ground accelerations and/or spectra to be developed for the location of interest and will provide more accurate modelling of the earthquake loading, site effects and seismic response.

For example, the model used for Method 1 is based on a single earthquake attenuation equation. Recent studies have shown this to be outdated (Mak et al. 2018) and current practice is to use multiple international and local attenuation models to account for uncertainty. The model used for Method 1 also uses subsoil classes to estimate the local site effects, e.g. Class B – Rock, Class C – Shallow soil. This is a simplistic approach suitable on a nation-wide scale but can result in over/under-prediction for certain ground conditions. Recently developed models instead use the V_{s30} parameter (average shear wave velocity over the upper 30 m of soil and rock) which provides a sliding scale for the local site effects instead of the distinct subsoil class 'bins'.

The SSHA would confirm or otherwise the appropriate minimum design loads for detailed design. Based on previous studies in the Auckland region, the site-specific hazard is likely to be similar to or slightly lower than the hazard based on the simplified Method 1. However, this would be dependent on a number of factors including the results of the site-specific ground investigations (e.g. downhole shear wave velocity measurements).

An SSHA would also provide a site-specific acceleration spectra suitable for use either in structural design applications or for the selection and scaling of earthquake records for time history analysis, e.g. advanced slope stability modelling or site-specific ground response analysis (Method 3).

The use of either Method 1 values or those from Method 2 (detailed SSHA) has opportunities and risks, as summarised in Table 2. We recommend that the project team explore the potential significance of these opportunities and risks as they relate to the proposed Dome Valley site as the project progresses.

Table 2: Summary of identified opportunities and risks.

Method 2 (SSHA) OPPORTUNITIES	
Description	**Possible next steps to explore opportunity**
The detailed SSHA finds that seismic design loadings required to meet the criteria of the building code are less than the routine design values of Method 1 (NZTA Bridge Manual).	Assess the extent to which the reduced design loadings from the SSHA might allow savings in the design.
The SSHA provides reassurance that the Method 1 earthquake design loadings would be appropriate for use on this project if desired, being similar or higher than found by the SSHA.	Consider whether there are advantages to using the standard values, and how these balance against potential savings in the design by using the lower SSHA values. Even if the Method 1 values are used as the primary basis of design, the SSHA can be used to demonstrate that the design is robust.
The SSHA can provide site-specific acceleration spectra suitable for time history selection and scaling for use in time history analyses (e.g. advanced slope stability analysis or ground response analysis).	Consider whether there may be advantages or the requirement to undertake advanced slope stability modelling during detailed design.
Method 2 (SSHA) RISKS	
Description	**Possible next steps to manage risk**
The detailed SSHA results indicate higher design values than the standard Method 1 values which changes expected design cost and timeframes.	Project team to consider this risk and consider impact on the project delivery.
The Building Consent Authority (BCA) may require additional information before they accept the SSHA as the basis of design. This may result in additional cost, delay or design changes compared to using the routine Method 1 design actions.	Discuss with BCA to understand their requirements, and how this could impact design programme. Consider commissioning a peer review of the SSHA which can be provided to support the building consent application.

Applicability

This letter has been prepared for the exclusive use of our client Polaris Project, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

The results presented in this letter are preliminary in nature only and do not fully consider all of the aspects required for a detailed SSHA study. As such, the results presented are subject to change with more detailed assessment.

We would be happy to work with you and the design team to continue to explore the opportunities and risks in undertaking a SSHA as an alternative basis for design and decide on the next steps.

Yours sincerely,

Simonne Eldridge
Project Director

13-Apr-18
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Attached:

Bradley B. A. (2015). *The Benefits of Site Specific Hazard Analysis for Geotechnical Design in New Zealand*. Bulletin of the New Zealand Society for Earthquake Engineering.

Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 48, No. 2, June 2015

BENEFITS OF SITE-SPECIFIC HAZARD ANALYSES FOR SEISMIC DESIGN IN NEW ZEALAND

Brendon A. Bradley[1]

(Submitted *September 2014*; Reviewed *November 2014*; Accepted *March 2015*)

ABSTRACT

This paper summarizes the role site-specific seismic hazard analyses can play in seismic design and assessment in New Zealand. The additional insights and potential improvements in the seismic design and assessment process through a better understanding of the ground motion hazard are examined through a comparative examination with prescriptive design guidelines. Benefits include the utilization of state-of-the-art knowledge, improved representation of site response, reduced conservatism, and the determination of dominant seismic source properties, among others. The paper concludes with a discussion of these relative benefits so that the efficacy of site-specific hazard analysis for a particular project can be better judged by the engineer.

INTRODUCTION

A key requirement in the seismic design and assessment of structural and geotechnical systems is the determination of the inherent seismic hazard at the site due to earthquake-induced ground motions and consequent geo-hazards (fault rupture, slope stability, and liquefaction, among others). In the overwhelming majority of cases, ground motion intensities for such purposes are obtained from prescriptive design standards and guidance documents developed by authorities such as Standards New Zealand [1], New Zealand Transport Agency [2, 3], and New Zealand Geotechnical Society [4]. Such prescriptions allow for a time-efficient determination of seismic hazard, which is of sufficient accuracy for many conventional geotechnical structures. However, the standardization process required in the development of such prescriptions leads to both a significant loss of information, and a general insertion of conservatism in the quantification of the seismic hazard. This loss of information may have a significant impact on obtaining a fundamental understanding of seismic performance of the system considered, and general conservatism may excessively impact the required financial costs and even project viability. While such statements have previously been interpreted as only applicable for the most high-importance high-cost projects (e.g. critical infrastructure), the cost of commissioning a site-specific hazard study relative to the potential cost savings through improved design efficiency demonstrate its utility for more conventional structures (multi-storey structures, multi-span bridges, among others).

Despite the fact that the use of site-specific seismic hazard analyses is increasing in NZ (particularly following the 2010-2011 Canterbury earthquakes), their utilization is still significantly lower in proportion to other countries with similar seismic hazard and economic conditions (e.g. USA, Canada). The purpose of this paper is therefore to summarize the role site-specific seismic hazard analyses can play in seismic design and assessment in NZ. A summary of ground motion prescriptions in NZ seismic design standards and guidelines is first provided. The basic features of site-specific seismic hazard analyses are then summarized, as well as their relationship to informing design standards and guidelines. The various benefits of site-specific seismic hazard analyses are then enumerated within the context of several examples for NZ's major cities.

GROUND MOTION PRESCRIPTIONS IN NZ STANDARDS AND GUIDANCE DOCUMENTS

Structures Loading Standard, NZS1170.5 (2004)

NZS1170.5 [1] is the principal document in NZ providing quantitative prescriptions for design ground motion intensities. Because NZS1170.5 was exclusively developed as a loadings standard for the design of structural systems, it provides ground motion intensity in the form of design response spectra according to the following equation [1]:

$$C(T) = C_h(T) * Z * R * N(T, D) \tag{1}$$

where C is the design response spectral amplitudes; C_h is the spectral shape factor, which is a function of soil class and vibration period, T; Z is the zone factor; R is the return period factor; and N is the near-fault factor.

As suggested by Equation (1), the simplification of the design response spectrum into four factors requires several gross simplifications which are elaborated upon subsequently. NZS1170.5 also allows for "special studies", i.e., what is refered to here as site-specific seismic hazard analysis, although no guidance is provided as to how this should be performed.

NZGS Liquefaction Guidelines (2010)

The New Zealand Geotechnical Society (NZGS) provide guidelines [4] on the application of the simplified liquefaction triggering procedure, in which the design horizontal peak ground acceleration (PGA) is utilized to compute the cyclic stress ratio. This guideline provides three different approaches by which the design PGA can be determined: Method 1 directly utilizes NZS1170.5, Method 2 is based on site-specific seismic hazard analysis (as discussed in the next section); and Method 3 combines site-specific seismic hazard analysis with a site-specific response analysis of the surficial soils.

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According to Method 1 [4], the design PGA is obtained as:

$$PGA = a_h = Z * R * C \quad (2)$$

where Z, R, and C are the zone, return period, and soil class factors from NZS1170.5, (strictly speaking the values of C are obtained from the spectral shape factor for *T*=0).

For liquefaction evaluation applications, it is critical to understand that Method 1 and NZS1170.5 provide no information on the causal magnitudes which the design PGA corresponds to, and hence, no magnitude scaling factor can be considered. While the development of NZS1170.5, using the McVerry et al. [5] ground motion prediction equation, utilized a "magnitude factor" of $\left(\frac{M_w}{7.5}\right)^{1.285}$ [6], it should be emphasised that this is not a conventional "magnitude scaling factor" used for liquefaction triggering (where the magnitude dependent exponent is generally on the order of 2.5), and was utilized to correct for the known over-prediction bias of the McVerry et al. model at small vibration periods [7, 8]. Thus, the NZGS guidelines implicitly assume that the design PGA is for a moment magnitude (Mw) 7.5 event, which often is a considerable source of conservatism.

NZTA Bridge Manual – 3rd Edition (2013, 2014)

The NZTA Bridge Manual – 3rd Edition [2, 3] provides prescriptions on the seismic design of transportation-related structures, specifically Section 5.0 and 6.0 for the design of structural and geotechnical systems, respectively. Section 5.2 prescribes the design loading by directly referring to NZS1170.5, with only two exceptions: (1) the zone factor, Z, is reduced below the NZS1170.5-minimum of 0.13 for the Auckland/Northland region (but the combination of $Z * R$ must still exceed 0.13 for the ultimate limit state); and (2) the return period factor for ULS design is based on specifics in the NZTA bridge manual rather than NZS1170.5 (since the latter is focused on buildings). Section 6.2 prescribes the design loading as:

$$\text{PGA} = C_{0,1000} * \frac{R_u}{1.3} * f * g \quad (3)$$

where $C_{0,1000}$, R_u, and f are the PGA coefficient, return period factor, and site class factor, respectively, and g is the acceleration of gravity. The principal difference of Equation (3) from NZS1170.5 is that $C_{0,1000}$ represents the magnitude-unweighted PGA coefficient, as opposed to the 'magnitude-factored' value of Z in NZS1170.5. The return period factor, R_u, in Equation (3) is obtained directly from NZS1170.5, and thus since R_u=1.3 for a 1000 year return period the factor $C_{0,1000}/1.3$ is analogous to NZS1170's Z – with the exception the 'magnitude factor', as already noted. NZTA [2, 3] also allows for site-specific hazard analysis ("special studies") to be conducted and provides brief guidance in this regard. For large projects (>$7M), site-specific analyses are required.

SITE-SPECIFIC HAZARD ANALYSES AND BASIS FOR NZS1170.5:2004

Site-specific Probabilistic Seismic Hazard Analysis (PSHA)

Seismic Hazard Curve

The prescriptions underlying the seismic design standards and guidelines mentioned in the previous section are based on the results of site-specific probabilistic seismic hazard analysis (PSHA), which are then summarized in a codified form. Seismic hazard analyses involve two key ingredients: (1) an earthquake rupture forecast (ERF) which provides the location, characteristics, and rate of occurrence of all potential earthquakes in the region of interest; and (2) a ground motion prediction equation (GMPE) which provides the distribution of some measure of ground motion intensity at a given site from a given earthquake rupture. The principal output of PSHA is the seismic hazard curve, which provides the annual rate of exceedance of a particular ground motion intensity measure, and is obtained from [9]:

$$\lambda_{IM}(im) = \sum_{k=1}^{N_{rup}} P(IM > im|Rup_k) * \lambda_{Rup_k} \quad (4)$$

where $\lambda_{IM}(im)$ is the annual rate of $IM \geq im$ (the hazard curve); λ_{Rup_k} and N_{rup} are the annual rate of occurrence of earthquake rupture k and the number of earthquake ruptures, respectively (both from the ERF); and $P(IM > im|Rup_k)$ is the probability that the occurrence of earthquake rupture Rup_k will produce a ground motion at the site of interest with an intensity $IM \geq im$.

Figure 1 provides an example illustration of the seismic hazard curves (i.e. Equation (1)) obtained from site-specific seismic hazard analyses at generic site class D sites in Auckland, Christchurch and Wellington. For comparison, the design PGA values based on NZS1170.5 [or equivalently, NZGS [4]] are also provided. It can be seen that the design values based on NZS1170.5 have a significantly varying proximity to the 'exact' site-specific values, with variations being both a function of location, and also of the return period of interest. The results of Figure 1 are elaborated upon subsequently, however it is important to mention from the outset that the comparison observed is representative for the PGA hazard only and gives little insight into similar comparisons for other ground motion intensity measures (e.g., SA at different vibration periods).

Uniform Hazard Spectra (UHS)

One way in which the results of PSHA for spectral accelerations, SA, can be expressed in a compact manner is to create a uniform hazard spectrum (UHS). A UHS represents a locus of spectral accelerations at various vibration periods which have the same annual frequency of exceedance (or equivalently, return period). Figure 2 provides an example illustration of a UHS at the 500 year return period from site-specific PSHA at generic site class D sites in Auckland and Christchurch. For comparison, the design spectra based on NZS1170.5 are also provided. It can be seen that the NZS1170.5 spectrum for Christchurch is similar to one published model for the post-Canterbury earthquake sequence hazard [10] (another being Gerstenberger et al. [11]) at long vibration periods, but becomes increasingly conservative as the vibration period reduces – particularly for T<0.5s. In the case of Auckland, it can be seen that the NZS1170.5 hazard is significantly higher than the site-specific seismic hazard, although this is because the deterministic hazard from a Mw6.5 earthquake at R_{rup}=20km dominates in the NZS1170.5 values in this region [6].

Basis for NZS1170.5:2004

The results of PSHA in the format of a UHS provide the basis for the prescriptions in NZS1170.5, and by reference, those in NZGS [4] and NZTA [2, 3]. McVerry [12] discusses details of the progression from site-specific results

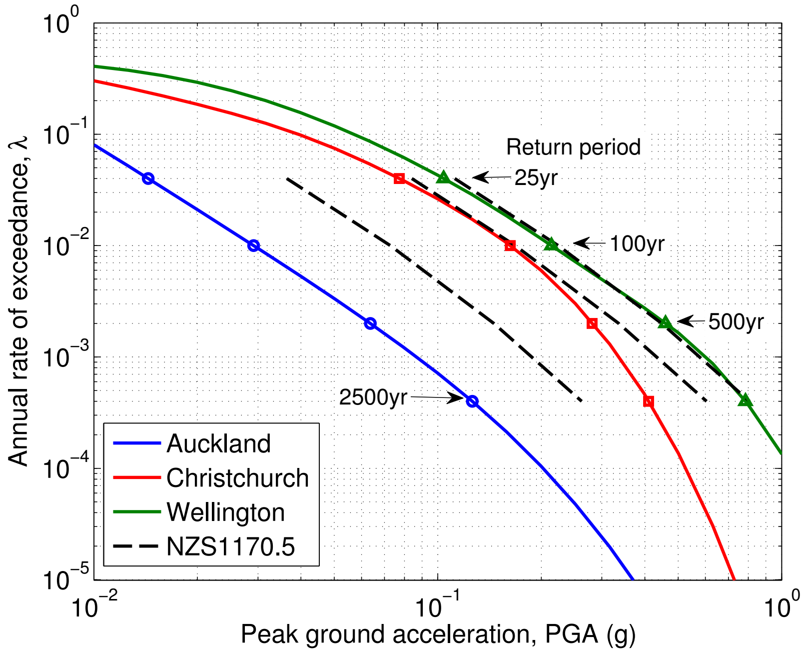

Figure 1: Site-specific seismic hazard curves forPGA at generic site class D sites in Auckland, Christchurch, and Wellington (obtained using OpenSHA [13]) in comparison with the NZS1170.5 design Z values (using Z = 0.13, 0.30, and 0.40, respectively). Amplitudes at the 25, 100, 500, and 2500 year return periods are annotated with markers.

obtained throughout NZ into a codified format for NZS1170.5. As already alluded to, the simplification of site-specific seismic hazard analysis results throughout NZ into the form given by Equation (1) entails a significant amount of information loss, and generally associated conservatism. In particular:

- The effects of surficial soils on surface ground motions is grossly simplified into 4 different soil classes (through soil-class dependent spectral shape factors)
- The spectral shape factor, C_h, which defines the shape of the response spectrum, is constant for all locations throughout NZ
- The return period factor, R, which defines the variation in seismic hazard with changes in return period (the inverse of exceedance rate) is constant throughout NZ.

BENEFITS AND INSIGHTS FROM USING SITE-SPECIFIC SEISMIC HAZARD ANALYSES

Site-specific Representation of Design Ground Motion Amplitudes and Reduced Conservatism

In comparison to the bulleted list in the previous section it should be clear that: (1) site response effects are much more complicated than the discrete division into soil classes; and (2) the spectral shape and its variation for different return periods are location-specific as a result of the site-specific features of the earthquake rupture forecast (e.g. nearby seismic sources), ground motion prediction equation (e.g. region-specific wave propagation effects), and site-specific surficial soil response including nonlinearity.

Site-specific Spectral Shape

Figure 2 clearly illustrates that the spectral shape of site-specific UHS vary significantly from the assumed NZS1170.5 shape, and vary from location to location based on soil conditions and the fact that the potential seismic ruptures in the region dominate the short and long vibration period hazard differently. This has also been illustrated by McVerry [12].

Site-specific 'Return Period Factors'

Figure 1 also illustrated that the slope of the hazard curves at specific sites differ from each other. This implies that the ratio of ground motion amplitudes at two different exceedance rates (or return periods) is not constant. Figure 3 provides a summary of the 'shape' of the seismic hazard curves by normalizing the results in Figure 1 by the 500 yr return period value. As also noted in Section C3.3 of NZS1170.5 [6], it can be seen that the hazard curve shapes for the three regions are quite different (a function of the characteristics and frequency of occurrence of the dominant seismic sources). At the 2500yr return period, in particular, it can be seen that the ratios range from 1.5-2.0, as compared to the NZS1170.5 value of 1.8. This 25% difference is clearly significant in the assessment of a system's performance for this return period, which is being increasingly considered to test structural robustness.

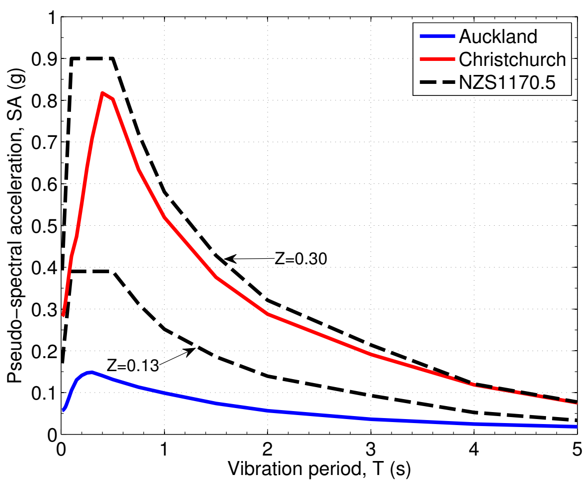

Figure 2: Site-specific UHS for the 500 year return period at generic site class D sites in Auckland and Christchurch in comparison with the NZS1170.5 design Z values (using Z = 0.13 and 0.30, respectively).

Near Source Factor

NZS1170.5 accounts for forward-directivity effects from near-fault ground motions by providing an amplification to response spectral ordinates at periods greater than T=0.5s for sites located near major faults [1]. One of the critical limitations of this prescription is that it is only considered for faults of larger magnitude with frequency recurrence intervals. The limitation of this approach is evident in the large forward directivity ground motions observed in the 2010-2011 Canterbury earthquakes [14, 15], which NZS1170.5 neglects because these causative faults are not among the 11 listed major faults in Table 3.6 of NZS1170.5. To put this in further context, there are over 500 mapped faults in the most recent version of the NZ seismic source model [16], the majority of which are located onshore.

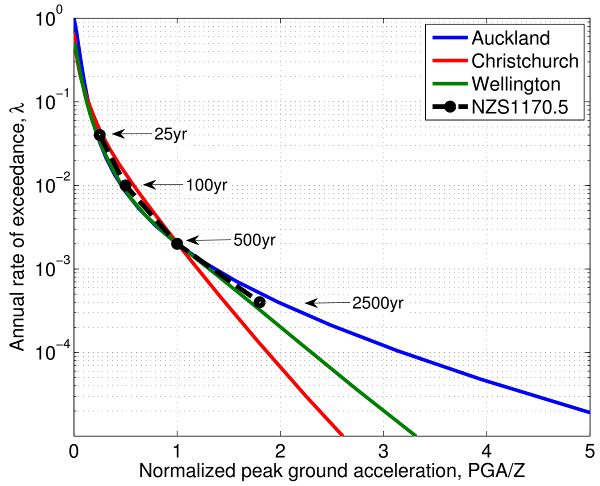

Figure 3: Normalization of the site-specific hazard results in Figure 1 by the 500 yr return period in order to illustrate the various shapes of the hazard curves in comparison with the return period factor, R, in NZS1170.5.

Sources of Conservatism

As referred to in previous sections, the codification of site-specific seismic hazard analyses within some parametric framework naturally results in a loss of information, and as a corollary the introduction of conservatism on average. With reference to NZS1170.5, in particular, conservatism is introduced in the following ways:

- The spectral shape factor is assumed constant for all locations throughout New Zealand, and the adopted spectral shape functional form is generally developed to conservatively envelope the results of site-specific hazard spectra [12]
- The spectral shape factor is constant for all levels of ground motion intensity, i.e. no nonlinear site effects are considered in the parameterization, which results in the spectral shape factor being a conservative 'envelope'.
- The return period factor, R, is constant for all locations in New Zealand [6], and for all vibration periods.

Current vs. 15-year-old Knowledge of Seismic Sources and Ground Motion

One obvious benefit in the use of site-specific seismic hazard analyses is that they employ the best available knowledge at the present time. In contrast, the science underpinning NZS1170.5 (and as a result, NZGS [4] and NZTA [2, 3]) is approximately 15 years old. While NZS1170.5 was published in 2004, the seismic hazard analysis results it is based on are those from Stirling et al. [17], which uses a seismic source model finalized in 2000, and a ground motion prediction equation developed in 1997 (although published in the public domain in 2006 as McVerry et al. [5]).

Significant progress has been made in better characterizing seismic sources and ground motion modelling in NZ over the past 15 years. The latest nationwide update to the NZ seismic source model in Stirling et al. [16] includes further mapping of 200 onshore and offshore faults from the model a decade earlier [17], as well as a significantly improved characterization of important large faults such as the Wellington Fault, Hikurangi subduction zone, and Alpine Fault. In terms of ground motion modelling, the commencement of the GeoNet programme (www.geonet.org.nz) has resulted in a significant increase in the quality and quantity of recorded strong ground motions in NZ which form the basis of empirical ground motion prediction equations. For example, Bradley [18, 19] developed NZ-specific ground motion models based on this significantly improved NZ dataset. The occurrence of the 2010-2011 Canterbury earthquakes also provided a significant dataset to blindly validate that model, as documented elsewhere [8, 14, 15], as well as the observed strong motions enabling the computation of region-specific site effects [20, 21].

The recent 2010-2011 Canterbury and 2013 Seddon earthquake sequences also highlight the importance of understanding the time-dependent effect of aftershock decay sequences on seismic hazard over 50 year time horizons of interest to infrastructure seismic design [10, 11], which can be directly considered within site-specific seismic hazard analyses.

Improved Representation of Site Response

As noted already, NZS1170.5 provides an overly simplistic representation of local site effects through the classification of 3 soil and one rock class. As a result, there is both a large variation in *actual* site response effects for soil deposits that would fall under the same broad site classes, as well as a large step-change in the implied site response for soil deposits falling into different site class categories, even if such soils may have similar site responses. Site-specific seismic hazard analyses offer several options for the consideration of site effects which can be more general than those in NZS1170.5, as discussed below.

Site Response Parameters in Empirical Ground Motion Prediction Models

Empirical GMPEs include variables to represent properties of surficial soil deposits. While such variables are still a highly simplified representation of surfical site effects (see next section) they allow for an improved representation as compared to the site class definition and spectral shape factors in NZS1170.5. For example, it is now conventional for GMPEs to represent the very near surface soils through the use of the 30-m time averaged shear wave velocity, Vs30, as well as deeper soil properties from depths to specific levels of shear wave velocity (Vs), such as the depth to Vs=1000m/s, $Z_{1.0}$, or depth to Vs=2500m/s, $Z_{2.5}$. For example, the NZ-specific GMPE of Bradley [18, 19] uses Vs30 as well as $Z_{1.0}$, while the NGA model of Campbell and Bozorgnia [22] uses Vs30 as well as $Z_{2.5}$. As noted by Seyhan et al. [23], other less common site classification options include site period, which is strongly correlated with Vs30, and depth to bedrock – although this is ill defined based on the vague definition of "bedrock".

One critical shortcoming in NZS1170.5 is that response spectra amplitudes at all vibration periods scale uniformly with the return period factor, *R*, implying that site response effects are linear in nature. In contrast, it is well known that under strong ground motion shaking, soft surficial soils will deform nonlinearly and affect the surface ground motion. Figure 4a illustrates the significant reduction in short-period spectral ordinates on soft soil sites observed in Lyttelton Port during the 22 February 2011 Christchurch earthquake [14]. Similarly, Figure 4b illustrates the modelled effect of nonlinear site response using the Bradley [19] GMPE for a generic weathered rock and soft soil site. While it can be clearly seen that the median empirical prediction does not capture the significant short-period rock acceleration (a systematic feature at the LPCC site [20]) or the longer period spectral peak at the LPOC site (and hence the benefit of site response analyses discussed subsequently), the modelled nonlinear reduction at very short periods on the soft soil site is clearly seen.

Because of the fact that NZS1170.5 chooses to use an amplitude-independent spectral shape factor, the adopted factors need to be appropriate for both small and large amplitude ground motions, for which nonlinear site effects differ. As a result, the utilized spectral shape factors are a conservative “envelope” of both extreme cases and therefore imply that soils on site class D/E will have higher SA values over the full spectrum of vibration periods compared with site class B (i.e. rock) conditions. While this is likely true for small amplitude motions, Figure 4 illustrates the incorrectness of this assumption for larger amplitude motions, and this generally results in NZS1170.5 yielding a significant over-prediction of short period spectral amplitudes on soft soil sites for large ground motion shaking (as seen in Figure 2).

Direct Site Response Analysis Modelling

While empirical GMPEs that use Vs30 and basin depth parameters ($Z_{1.0}$, $Z_{2.5}$), and explicitly consider nonlinear site response provide an improved estimate of surficial site effects over the NZS1170.5 site classes, they still represent an average representation of near surface site effects. Sites which have atypical soil profiles (e.g. velocity inversions), and/or very soft soil deposits where significant cyclic softening or liquefaction is likely under strong shaking will benefit greatly from the direct modelling of near surface site effects through wave propagation analyses. In NZGS [4] this is referred to as the “Method 3” approach to determine design ground motion amplitudes. Such analyses can be 1D/2D/3D in nature and consider the constitutive (stress-strain) response of the soils using equivalent-linear, nonlinear total stress, or nonlinear effective stress approaches. While a detailed discussion of each of these possibilities is beyond the scope of this paper, it should be clear that such site-specific modelling will provide significant insights into the role of the subsurface soils on the surface ground motion, as well as providing explicit estimates of ground displacements, plastic localization phenomena (including potential liquefaction), and the potential benefits of ground improvement.

Intensity Measures other than PGA or Spectral Acceleration, SA

NZS1170.5, and by reference NZGS [4] and NZTA [2, 3], provide seismic hazard information for PGA and response spectral ordinates (SA) only. However, other measures of ground motion can be particularly useful in seismic design and assessment. For example, the peak shear strain, γ_{max}, in a soil deposit is known to be directly related to the peak ground velocity, PGV, through the relationship $\gamma_{max} \sim PGV/\acute{V}_s$ (where $\acute{V}_s$ should be a strain-consistent Vs, and not the linear elastic Vs).

Given that ground motion severity is, in general, a function of amplitude, frequency content and duration, then the consideration of PGA and SA (peak response of a linear elastic single-degree-of-freedom) really provide little insight into the cumulative effects of a ground motion which can be important for degrading systems (e.g., cyclically softening plastic soils and liquefiable soils, as well as degrading structural systems). For example, there is increasing empirical evidence to support the obvious influence of ground motion duration on the collapse of structures and the likelihood of liquefaction [24-26]. The determination of additional ground motion intensity measures in addition to PGA and SA is also important in the selection of ground motions for use in seismic response analyses [27-29].

Dominant Seismic Sources from Hazard Deaggregation

An understanding of the seismic sources which dominate the seismic hazard is of critical importance in order to have a thorough understanding in relation to: (1) determination of magnitude scaling factors for liquefaction triggering analyses (as emphasised previously documents such as NZGS [4] conservatively assume that the PGA hazard is for Mw7.5) and; (2) selection of ground motion time series for use in seismic response analyses (e.g., site response analyses or other geotechnical/structural analyses). Because PSHA is obtained by summing over all of the seismic sources which pose a threat to the site, then the ‘total’ seismic hazard is the sum of

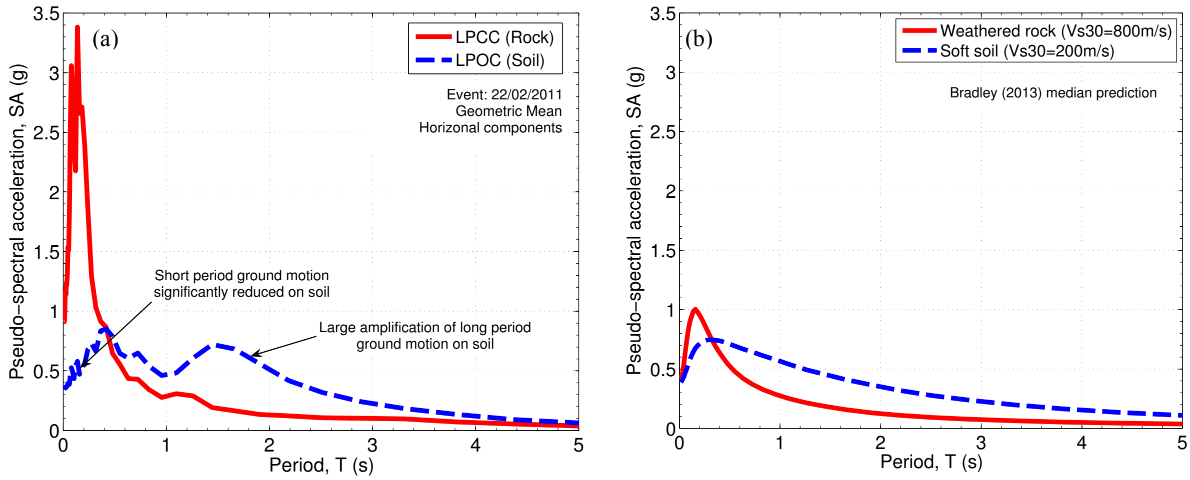

Figure 4: Illustration of the consideration of nonlinear site effects in empirical GMPEs: (a) observed horizontal response spectra at rock and soil sites in Lyttelton Port in the 22 February 2011 Christchurch earthquake [14]; and (b) nonlinear site effects based on the Bradley (2013) GMPE median prediction.

the hazard from each source (i.e., Equation (1)). Seismic hazard deaggregation is the terminology used to depict the 'total' seismic hazard deaggregated into the contributions from each source. Figure 5 provides an example illustration of seismic hazard deaggregation results for Christchurch and Auckland. It is important to note that the seismic deaggregation results are a function of: (1) the site location; (2) the return period of interest; and (3) the intensity measure considered. The fact that site location affects the seismic hazard should be obvious because it changes the sites proximity to nearby faults, and hence those that contribute the most to the total hazard. The deaggregation is a function of return period because of the different occurrence rates of the sources, and their potential to produce large and small ground motions. Finally, Figure 5 directly illustrates the effect of intensity measure on the deaggregation, where it can be seen that small-magnitude close-proximity sources tend to dominate the PGA hazard, while faults with greater magnitudes and high rupture rates at large distances dominate the SA(2.0s) hazard. Hence, while sporadic deaggregation information can be found in papers published in literature [e.g. 17, 30, 31] they are generally insuffcent for use at a site-specific location and intensity measure of interest.

Scenario-based Seismic Hazard Analysis

As already alluded to, design ground motion intensities in NZS1170.5 are based on PSHA [12]. However, because PSHA combines both the distribution of ground motion for a given event with the rate of occurrence of the event itself, it does not allow one to explicitly answer the question of "what will be the ground motion intensity if a particular earthquake rupture occurs?" Such questions are particularly informative in several circumstances, in particular for regions where the dominant fault sources have recurrence intervals which are larger than the typical design return periods. The 22 February 2011 Christchurch earthquake provides a classic example, where the observed ground motions produced were consistent with what would be expected from a Mw6.2 event in the near-source region [14, 19], but that significantly exceeded the 500yr return period design spectra.

Because NZS1170.5 does not provide any deaggregation information on which seismic sources dominate the seismic hazard, then such insight is not possible, however, it is something which can be easily performed within a site-specific PSHA.

It is also important to emphasise that in high seismic regions, NZS1170.5 caps ground motion intensities based on the so-called "MCE motions". By definition, this is considered as 2/3 of the 84th percentile ground motion of the dominant nearby fault. Hence while the name "Maximum considered earthquake (MCE)" is used, the stated phrase "represents the maximum motions … likely to be experienced in New Zealand" in NZS1170.5 is simply not correct. By definition, there is a 16% probability that 2/3 times the MCE level ground motion will be exceeded should the dominant event occur – which for a typical ground motion variability of 0.6 [32]

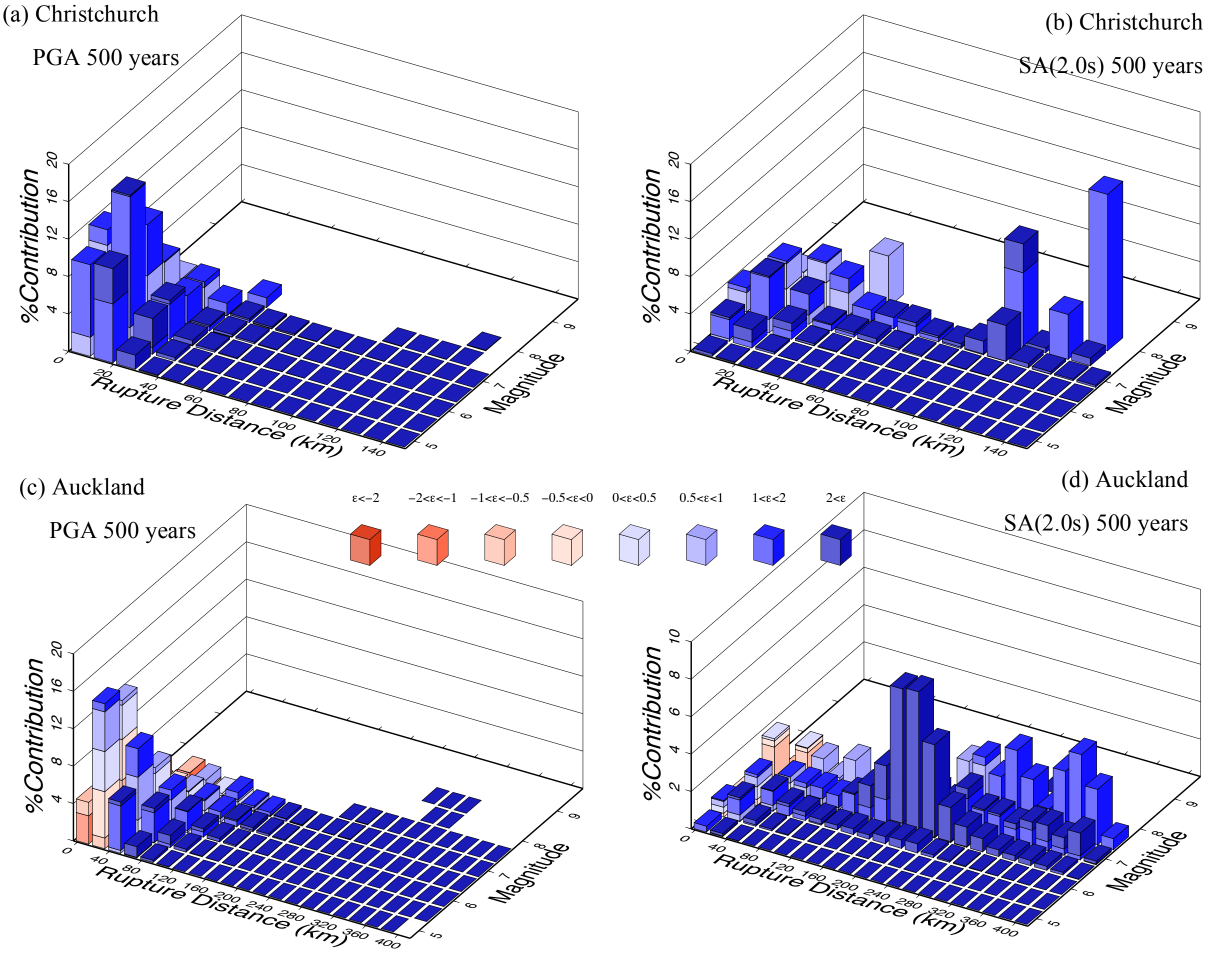

Figure 5: Seismic hazard deaggregation illustrating the dominant seismic sources contributing to the total seismic hazard: (a) Christchurch – PGA; (b) Christchurch SA(2.0s); (c) Auckland – PGA; and (d) Auckland SA(2.0s). It can be seen that small magnitude close proximity sources dominate the PGA hazard, while faults with greater magnitudes and high rates at large distances dominate the SA(2.0s) hazard.

means there is a 37% probability of the MCE level of ground motion being exceeded should the rupture event occur. Furthermore, this does not account for the possibility of events greater than those considered in the hazard model (e.g., as was the case in the 2011 Mw9.0 Tohoku, Japan earthquake).

CONCLUSIONS

The use of site-specific seismic hazard analyses offers several benefits for seismic design and assessment in New Zealand. The ability to understand the seismic sources which dominate the hazard allows a direct determination of magnitude scaling factors for liquefaction triggering analyses, as well as criteria for the appropriate selection of ground motion time series for seismic response analyses. Dominant seismic sources are also an important factor in understanding the ground motion hazard associated with the rupture of specific seismic sources (so-called scenario seismic hazard analysis). Intensity measures other than PGA and SA can also be obtained (e.g., PGV, significant duration, Arias intensity), which maybe particularly useful in some analysis procedures. Site-specific hazard analyses also allow for an improved representation of local site effects, either via GMPEs; or explicitly using site-specific response analyses. The inherent conservatism in NZS1170.5 also means that, on average, site-specific seismic hazard analyses will result in lower seismic demands. Not only does this mean that a given design or mitigation measure could be less expensive, but also that design/mitigation measures which are impractical based on NZS1170.5 values may become feasible.

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