# Site-specific hazard analysis for geotechnical design in New Zealand

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## ABSTRACT

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

*Keywords: Site-specific seismic hazard analysis, NZS1170.5:2004, response spectra, source deaggregation, ground motion selection.*

## 1 INTRODUCTION

A key requirement in the seismic design and assessment of geotechnical structures 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 (NZS 1170.5 2004a), New Zealand Transport Agency (NZTA 2013a, NZTA 2013b), and New Zealand Geotechnical Society (NZGS 2010). 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, and general conservatism may excessively impact the required financial costs for some projects. Generally such statements are interpreted as only applicable for the most high-importance high-cost projects (e.g. critical infrastructure), however simple economic arguments show that the cost of commissioning a site-specific hazard study relative to the potential cost savings through improved design efficiency make it useful for more conventional structures (multi-storey structures, multi-span bridges, among others). While 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 as a proportion than other countries with similar seismic hazard and economic conditions (e.g. USA, Canada).

The purpose of this paper is to summarize the role site-specific seismic hazard analyses can play in geotechnical design 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.

## 2 GROUND MOTION PRESCRIPTIONS IN NZ CODES, STANDARDS AND GUIDANCE DOCUMENTS

### 2.1 Structures loading standard, NZS1170.5:2004

NZS1170.5 (2004a) 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 (NZS 1170.5 2004a):

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

where $C$ is the design response spectral amplitudes (vibration period, $T$, dependent); $C_h$ is the spectral shape factor, which is a function of soil class; $Z$ 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 refereed to here as site-specific seismic hazard analysis, although no guidance is provided as to how this should be performed.

### 2.2 NZGS Liquefaction guidelines, 2010

The NZ Geotechnical Society (NZGS) provide guidelines (NZGS 2010) on the application of the simplified liquefaction triggering procedure, in which the design peak ground acceleration is needed to compute the cyclic stress ratio (CSR). This guideline provides 3 different approaches by which the design peak ground acceleration can be determined: Method 1 directly utilizes NZS1170.5 (2004), 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.

According to Method 1 (NZGS 2010), the design PGA is obtained as:

$$PGA = a_h = Z * R * C \tag{2}$$

where $Z$, $R$, and $C$ are the zone, return period, and soil class factors from NZS1170.5 (2004), (strictly speaking the values of $C$ are obtained from the spectral shape factor for a period of 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 (2004), using the McVerry et al. (2006) ground motion prediction equation, utilized a "magnitude factor" of $\left(\frac{M_w}{7.5}\right)^{1.285}$ (NZS 1170.5 2004b) 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 (Bradley 2012a, Bradley et al. 2014). Thus, the NZGS guidelines implicitly assume that the design PGA is for a Mw7.5 event, which often is a considerable source of conservatism as shown subsequently in ***Figure 4***a.

### 2.3 NZTA Bridge Manual

The NZTA Bridge Manual (NZTA 2013a, NZTA 2013b) provides prescriptions on the seismic design of transportation-related structures. Section 6.2 prescribes the design loading as:

$$PGA = C_{0,1000} * \frac{R_u}{1.3} * f * g \tag{3}$$

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 (2004) is that $C_{0,1000}$ represents the magnitude-unweighted PGA coefficient, as opposed to the magnitude-weighted 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 of magnitude weighting, as already noted. (NZTA 2013a, NZTA 2013b) 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.

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

### 3.1 Site-specific probabilistic seismic hazard analysis (PSHA)

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:

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

where $\lambda_{IM}(im)$ is the annual rate of $IM \geq im$ (the hazard curve); $\lambda_{Rup_k}$ and $N_{rup}$ are the annual rate of occurrence or 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 (2004) [or equivalently, NZGS (2010)] 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).

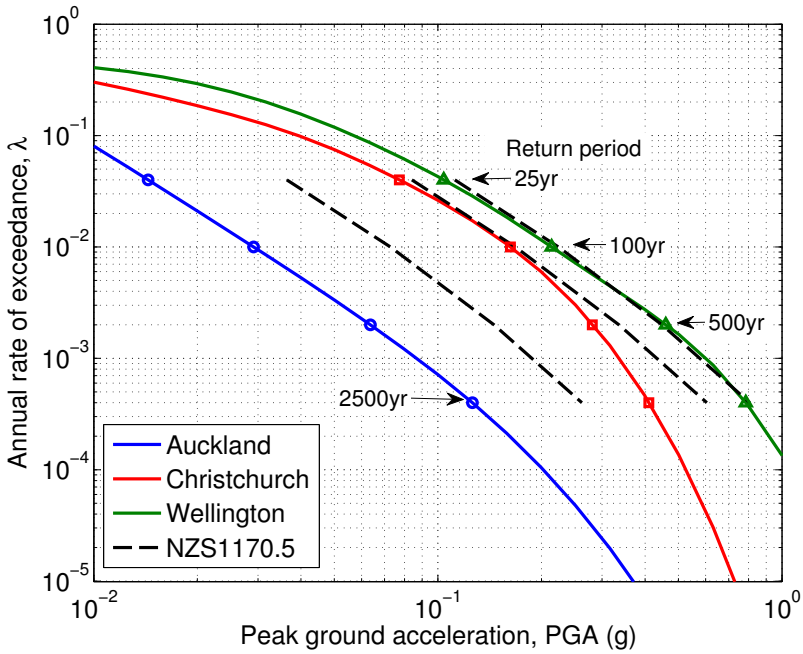


*Figure 1. Site-specific seismic hazard curves for peak ground acceleration at generic site class D sites in Auckland, Christchurch, and Wellington in comparison with the NZS1170.5 (2004) design values (using Z values of 0.13, 0.30, and 0.40, respectively). Amplitudes at the 25, 100, 500, and 2500 year return periods are annotated with markers.*

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. ***Figure 2*** provides an example illustration of uniform hazard spectra 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 (2004) are also provided.

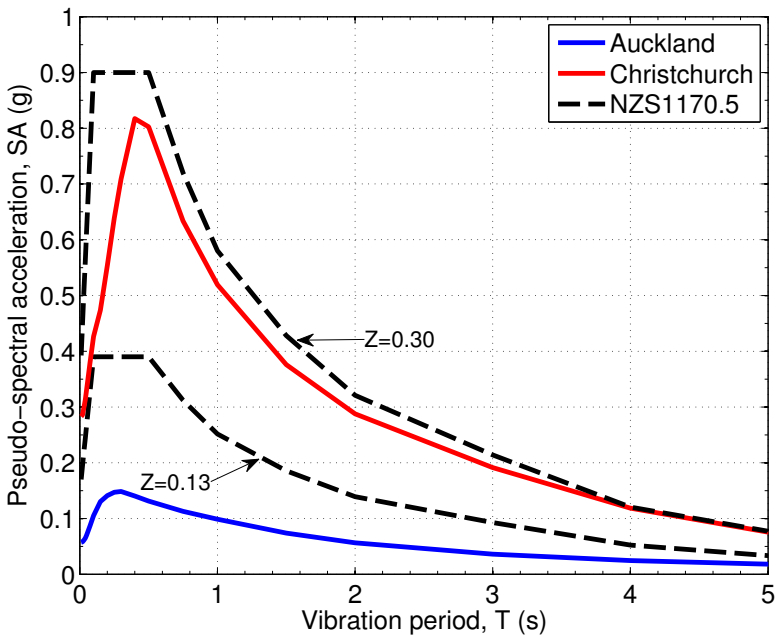


*Figure 2. Site-specific uniform hazard spectra for the 500 year return period at generic site class D sites in Auckland and Christchurch in comparison with the NZS1170.5 (2004) design values (using Z values of 0.13 and 0.30, respectively).*

The results of PSHA in the format of a UHS form the basis for the prescriptions in NZS1170.5 (2004), and by reference those in NZTA (2013) and NZGS (2010). McVerry (2003) discusses details of the progression from site-specific results obtained throughout NZ into a codified format. 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.

## 4 BENEFITS AND INSIGHTS FROM USING SITE-SPECIFIC SEISMIC HAZARD ANALYSES

### 4.1 Site-specific representation of design ground motion amplitudes and reduced conservatism

In comparison to the bulleted list in the previous section is should be clear that: (1) site response effects are much more complicated than the division into three soil classes and one rock class; and (2) the spectral shape and variation in hazard for different return periods (i.e. as defined by $C_h$ and $R$ respectively) vary at different locations as a result of the site-specific features of the earthquake rupture forecast (i.e. nearby seismic sources), ground motion prediction equation (i.e. region-specific wave propagation effects), and site-specific surficial soil response including nonlinearity.

***Figure 2*** clearly illustrates that the spectral shape of site-specific uniform hazard spectra 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 (2003).

***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. Comparing the 2500yr and 500yr return periods, in particular, the ratios range from 1.5-2.0, as compared to the NZS1170.5 (2004) value of 1.8. This 25% difference is clearly significant in the assessment of a systems performance under this return period, which is being increasingly considered to test structural robustness.

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 (2004), 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 (McVerry 2003)
- 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' as noted in section 4.3.1.
- The return period factor, R, is constant for all locations in New Zealand, which is conservative for regions of high seismic hazard (NZS 1170.5 2004b).

### 4.2 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 (2004) (and as a result, NZGS (2010) and NZTA (2013)) 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. (2002), 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. (2006)).

Significant progress has been made in better characterizing seismic sources and ground motion modelling in NZ over the past 15 years. The latest nation-wide update to the NZ seismic source model in Stirling et al. (2012) includes further mapping of 200 onshore and offshore faults from the (2002) model, 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 (2010, 2013) 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 (Bradley 2012c, Bradley and Cubrinovski 2011, Bradley et al. 2014), as well as the observed strong motions enabling the computation of region-specific site effects (Bradley 2012b, Bradley 2014).

### 4.3 Improved representation of site response

As noted already, NZS1170.5 (2004) 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 (2004), as discussed below.

#### 4.3.1 Site response parameters in empirical ground motion prediction models

Empirical ground motion prediction equations (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 (2004). For example, it is now conventional for GMPEs to represent the very near surface soils through the use of the 30-m averaged shear wave velocity, Vs30, as well as deeper soil properties from depths to specific levels of shear wave velocity, 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 (2010, 2013) uses Vs30 as well as $Z_{1.0}$, while the NGA model of Campbell and Bozorgnia (2008) uses Vs30 as well as $Z_{2.5}$. As noted by Seyhan et al. (2014), 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 omission 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 3***a illustrates the significant reduction in short period spectral ordinates on soft soil sites observed in Lyttelton Port during the 22 February 2011 Christchurch earthquake (Bradley and Cubrinovski 2011). Similarly, ***Figure 3***b illustrates the modelled effect of nonlinear site response using the Bradley (2013) 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 (Bradley 2014)) or the longer period spectral peak at the LPOC site (and hence the benefit of site response analyses discussed subsequently), the nonlinear reduction at very short periods on the soft soil site is clearly seen.

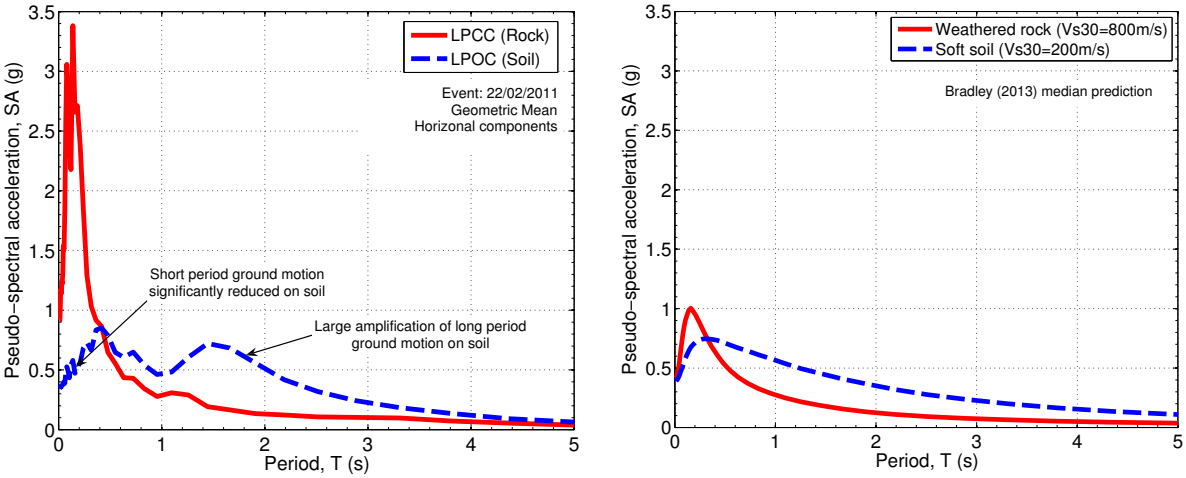


*Figure 3. Illustration of the consideration of nonlinear site effects in empirical ground motion prediction models: (a) observed horizontal response spectra at rock and soil sites in Lyttelton Port in the 22 February 2011 Christchurch earthquake (Bradley and Cubrinovski 2011); and (b) nonlinear site effects based on the Bradley (2013) GMPE median prediction.*

Because of the fact that NZS1170.5 (2004) 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 3*** 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***).

#### 4.3.2 Direct site response analysis modelling

While empirical ground motion models that use Vs30, basin depth parameters ($Z_{1.0}$, $Z_{2.5}$) and consider nonlinear site response provide an improved estimate of surficial site effects over the NZS1170.5 (2004) 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 (2010) 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 document 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.

### 4.4 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 through understanding as well as in relation to: (1) determination of magnitude scaling factors for liquefaction triggering analyses (as emphasised previously documents such as NZGS (2010) conservatively assume that the PGA hazard is for Mw7.5); (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 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 4***

provides an example illustration of seismic hazard deaggregation results for Christchurch. 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 difference occurrence rates of the sources, and their potential to produce large and small ground motions. Finally, ***Figure 4*** directly illustrates the effect of intensity measure on the deaggregation, where it can be seen that small magnitude close proximity sources dominate the PGA hazard, while greater Mw faults at large distances dominate the SA(2.0s) hazard.

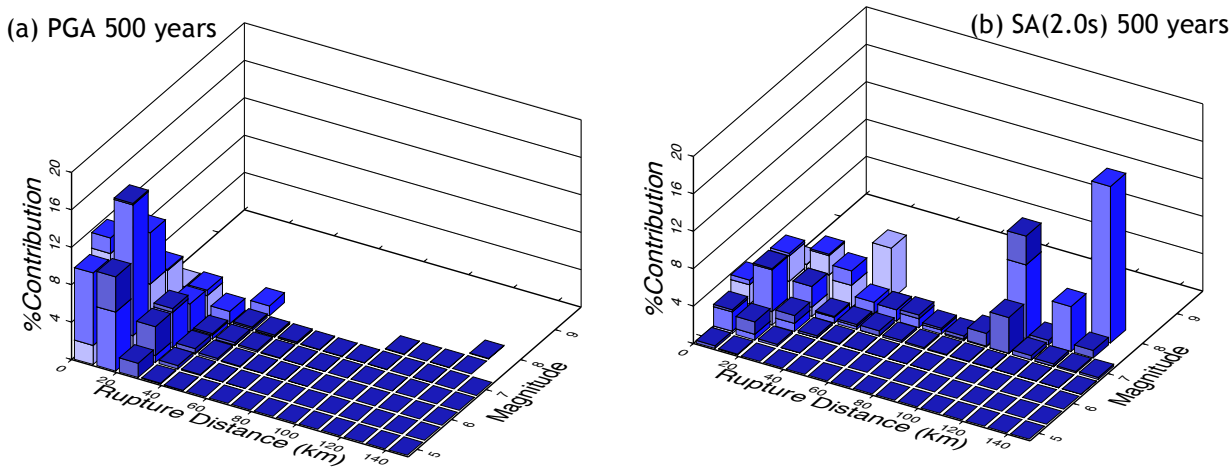


*Figure 4. Seismic hazard deaggregation illustrating the dominant seismic sources contributing to the total seismic hazard: (a) Christchurch – PGA; and (b) Christchurch SA(2.0s).*

## 5 DISCUSSION: IMPLICATIONS FOR GEOTECHNICAL EARTHQUAKE ENGINEERING DESIGN AND ASSESSMENT

The use of site-specific seismic hazard analyses offers several benefits for geotechnical earthquake engineering. 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. Intensity measures other than PGA and SA can also be obtained (e.g. PGV, significant duration), 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 (2004) 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 NZS1170.5:2004 values may become feasible.

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