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.

[1] *Associate Professor, University of Canterbury, Christchurch, brendon.bradley@canterbury.ac.nz (Member).*

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:

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

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

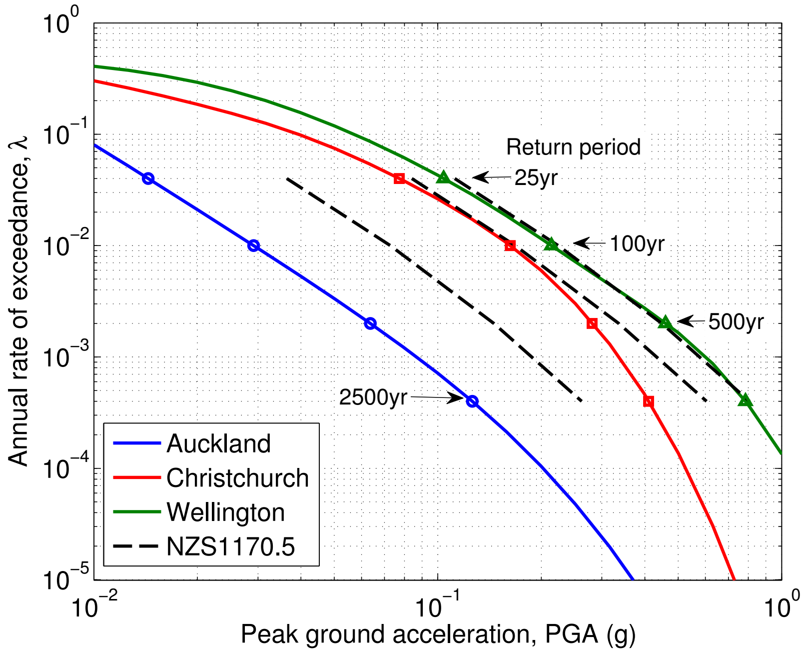

Figure 1: Site-specific seismic hazard curves forPGA at generic site class D sites in Auckland, Christchurch, and Wellington (obtained using OpenSHA [13]) in comparison with the NZS1170.5 design Z values (using Z = 0.13, 0.30, and 0.40, respectively). Amplitudes at the 25, 100, 500, and 2500 year return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

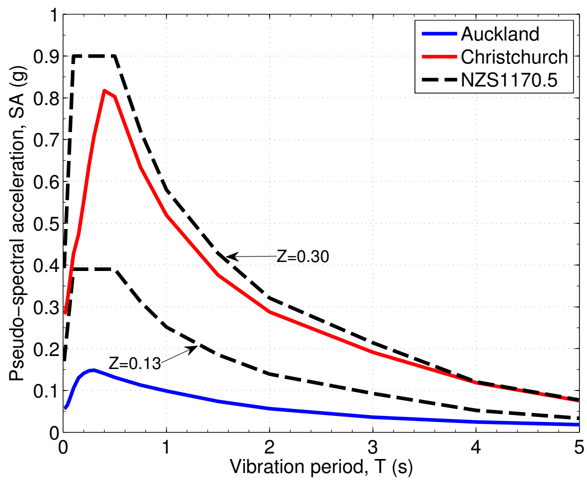

Figure 2: Site-specific UHS for the 500 year return period at generic site class D sites in Auckland and Christchurch in comparison with the NZS1170.5 design Z values (using Z = 0.13 and 0.30, respectively).

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.

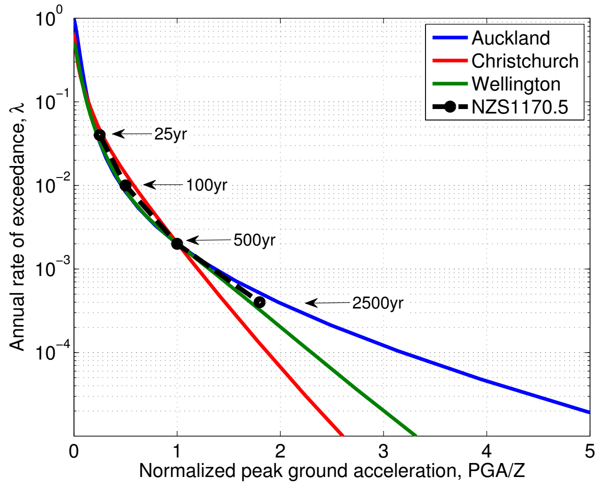

Figure 3: Normalization of the site-specific hazard results in Figure 1 by the 500 yr return period in order to illustrate the various shapes of the hazard curves in comparison with the return period factor, R, in NZS1170.5.

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

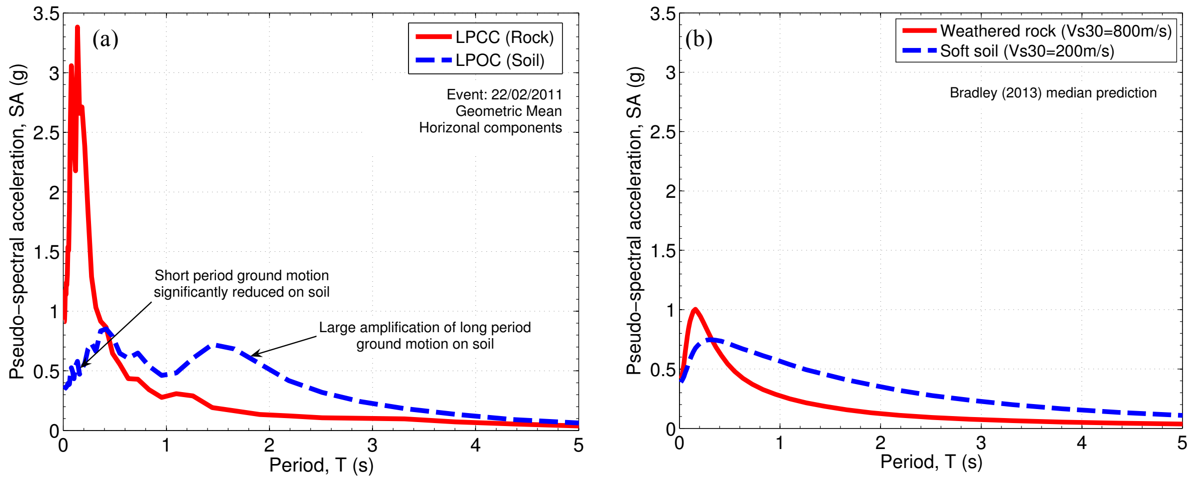

Figure 4: Illustration of the consideration of nonlinear site effects in empirical GMPEs: (a) observed horizontal response spectra at rock and soil sites in Lyttelton Port in the 22 February 2011 Christchurch earthquake [14]; and (b) nonlinear site effects based on the Bradley (2013) GMPE median prediction.

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 insuffiicent 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]

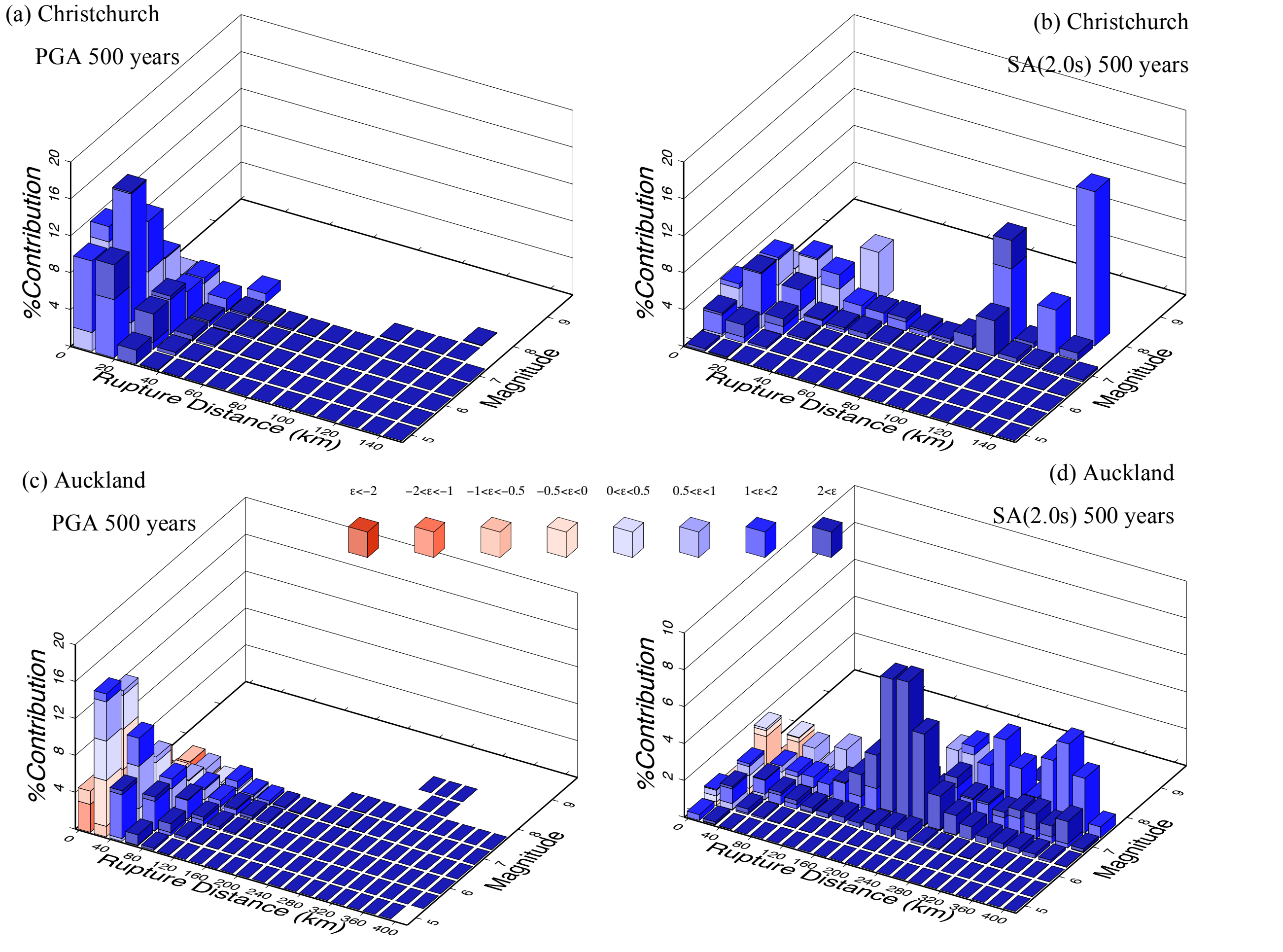

Figure 5: Seismic hazard deaggregation illustrating the dominant seismic sources contributing to the total seismic hazard: (a) Christchurch – PGA; (b) Christchurch SA(2.0s); (c) Auckland – PGA; and (d) Auckland SA(2.0s). It can be seen that small magnitude close proximity sources dominate the PGA hazard,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

REFERENCES

1 SNZ (2004). "*Structural Design Actions, Part 5: Earthquake Actions - New Zealand*". Standards New Zealand, Wellington, New Zealand. 82 pp.

2 NZTA (2013). "*Bridge Manual: SP//M/022. Ch5: Earthquake Resistant Design of Structures*". New Zealand Transport Agency, 23 pp.

3 NZTA (2014). "*Bridge Manual: SP//M/022. Ch6: Site-Stability, Foundations, Earthworks, and Retaining Walls*". New Zealand Transport Agency, 42 pp.

4 NZGS (2010). "*Geotechnical Earthquake Engineering Practice: Module 1 - Guideline for the Identification, Assessment and Mitigation of Liquefaction Hazards*". New Zealand Geotechnical Society. 34 pp.

5 McVerry GH, Zhao JX, Abrahamson NA and Somerville PG (2006). "New Zealand Acceleration Response Spectrum Attenuation Relations for Crustal and Subduction Zone Earthquakes". *Bulletin of the New Zealand Society for Earthquake Engineering*; **39**(1): 1-58.

6 SNZ (2004). "*Structural Design Actions, Part 5: Earthquake Actions - New Zealand - Commentary*". Standards New Zealand, Wellington, New Zealand. 86 pp.

7 Bradley BA (2012). "*Ground Motion and Seismicity Aspects of the 4 September 2010 and 22 February 2011 Christchurch Earthquakes*". Technical Report Prepared for the Canterbury Earthquakes Royal Commission. 62 pp.

8 Bradley BA, Quigley MC, Van Dissen RJ and Litchfield NJ (2014). "Ground Motion and Seismic Source Aspects of the Canterbury Earthquake Sequence". *Earthquake Spectra*; **30**(1): 1-15. doi:10.1193/030113eqs060m

9 Cornell CA (1968). "Engineering Seismic Risk Analysis". *Bulletin of the Seismological Society of America*; **58**(5): 1583–1606.

10 Bradley BA (2014). "*Seismic Hazard Analysis for Urban Christchurch Accounting for the 2010-2011 Canterbury Earthquake Sequence*". Technical report prepared for the New Zealand Earthquake Commission (EQC) and Tonkin & Taylor Ltd. 20 pp.

11 Gerstenberger M, McVerry G, Rhoades D and Stirling M (2014). "Seismic Hazard Modeling for the Recovery of Christchurch". *Earthquake Spectra*; **30**(1): 17-29. doi:10.1193/021913eqs037m

12 McVerry G (2003). "From Hazard Maps to Code Spectra for New Zealand". in *2003 Pacific conference on earthquake engineering*: Christchurch, New Zealand, 9.

13 Field EH, Gupta N, Gupta V, Blanpied M, Maechling P and Jordan TH (2005). "Hazard Calculations for the Wgcep-2002 Forecast Using Opensha and Distributed Object Technologies". *Seismological Research Letters*; **76** 161-167.

14 Bradley BA and Cubrinovski M (2011). "Near-Source Strong Ground Motions Observed in the 22 February 2011 Christchurch Earthquake". *Seismological Research Letters*; **82**(6): 853-865. doi:10.1785/gssrl.82.6.853

15 Bradley BA (2012). "Strong Ground Motion Characteristics Observed in the 4 September 2010 Darfield, New Zealand Earthquake". *Soil Dynamics and Earthquake Engineering*; **42** 32-46. doi:10.1016/j.soildyn.2012.06.004

16 Stirling M, McVerry G, Gerstenberger M, Litchfield N, Van Dissen R, Berryman K, Barnes P, Wallace L, Villamor P, Langridge R, Lamarche G, Nodder S, Reyners M, Bradley B, Rhoades D, Smith W, Nicol A, Pettinga J, Clark K and Jacobs K (2012). "National Seismic Hazard Model for New Zealand: 2010 Update". *Bulletin of the Seismological Society of America*; **102**(4): 1514-1542. doi:10.1785/0120110170

17 Stirling MW, McVerry GH and Berryman KR (2002). "A New Seismic Hazard Model for New Zealand". *Bulletin of the Seismological Society of America*; **92**(5): 1878–1903.

18 Bradley BA (2010). "*NZ-Specific Pseudo-Spectral Acceleration Ground Motion Prediction Equations Based on Foreign Models*". Report No.2010-03, Department of Civil and Natural Resources Engineering, University of Canterbury: Christchurch, New Zealand. 324 pp.

19 Bradley BA (2013). "A New Zealand-Specific Pseudospectral Acceleration Ground-Motion Prediction Equation for Active Shallow Crustal Earthquakes Based on Foreign Models". *Bulletin of the Seismological Society of America*; **103**(3): 1801-1822. doi:10.1785/0120120021

20 Bradley BA (2015). "Systematic Ground Motion Observations in the Canterbury Earthquakes and Region-Specific Non-Ergodic Empirical Ground Motion Modeling". *Earthquake Spectra*; (in press). doi:10.1193/053013eqs137m

21 Bradley BA (2012). "Ground Motions Observed in the Darfield and Christchurch Earthquakes and the Importance of Local Site Response Effects". *New Zealand Journal of Geology and Geophysics*; **55**(3): 279-286. doi:10.1080/00288306.2012.674049

22 Campbell KW and Bozorgnia Y (2008). "NGA Ground Motion Model for the Geometric Mean Horizontal Component of PGA, PGV, PGD and 5% Damped Linear Elastic Response Spectra for Periods Ranging from 0.01 to 10 s". *Earthquake Spectra*; **24**(1): 139-171.

23 Seyhan E, Stewart JP, Ancheta TD, Darragh RB and Graves RW (2014). "NGA-West2 Site Database". *Earthquake Spectra*; **30**(3): 1007-1024. doi:10.1193/062913EQS180M

24 Bradley BA, Cubrinovski M, Dhakal RP and MacRae GA (2010). "Probabilistic Seismic Performance and Loss Assessment of a Bridge-Foundation-Soil System". *Soil Dynamics and Earthquake Engineering*; **30**(5): 395-411. doi:10.1016/j.soildyn.2009.12.012

25 Hancock J (2006). "*The Influence of Duration and the Selection and Scaling of Accelerograms in Engineering Design and Assessment*". Civil and Environmental Engineering, Imperial College: London, 442 pp.

26 Raghunandan M and Liel AB (2013). "Effect of Ground Motion Duration on Earthquake-Induced Structural Collapse". *Structural Safety*; **41**(0): 119-133. doi:http://dx.doi.org/10.1016/j.strusafe.2012.12.002

27 Bradley BA (2010). "A Generalized Conditional Intensity Measure Approach and Holistic Ground-Motion Selection". *Earthquake Engineering & Structural Dynamics*; **39**(12): 1321-1342. doi:10.1002/eqe.995

28 Bradley BA (2012). "A Ground Motion Selection Algorithm Based on the Generalized Conditional Intensity Measure Approach". *Soil Dynamics and Earthquake Engineering*; **40**(0): 48-61. doi:10.1016/j.soildyn.2012.04.007

29 Chandramohan R, Baker JW and Deierlein GG (2014). "Hazard-Consistent Ground Motion Duration: Calculation Procedure and Impact on Structural Collapse Risk". *Proceedings of the Tenth U.S. National Conference on Earthquake Engineering*: Anchorage, Alaska, 10.

30 Oyarzo-Vera CA, McVerry GH and Ingham JM (2012). "Seismic Zonation and Default Suite of Ground-Motion Records for Time-History Analysis in the North Island of New Zealand". *Earthquake Spectra*; **28**(2): 667-688. doi:10.1193/1.4000016

31 Bradley BA (2012). "Ground Motion Comparison of the 2011 Tohoku, Japan and 2010-2011 Canterbury Earthquakes: Implications for Large Events in New Zealand". *New Zealand Society of Earthquake Engineering Annual Conference*: Christchurch, New Zealand, 8 pp.

32 Abrahamson NA, Atkinson GM, Boore DM, Bozorgnia Y, Campbell KW, Chiou B, Idriss IM, Silva WJ and Youngs RR (2008). "Comparisons of the NGA Ground-Motion Relations". *Earthquake Spectra*; **24**(1): 45-66.