Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 58, No. 1, March 2025

SEISMIC HAZARD WITH DETERMINISTIC MAXIMUM LIMITS: CONSIDERATIONS IN A NEW ZEALAND-SPECIFIC CONTEXT

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(Submitted *July 2024*; Reviewed *August 2024*; Accepted *August 2024*)

ABSTRACT

This paper outlines the consideration of deterministic limits on maximum ground motion levels within seismic design codes and standards. The specific motivation is to outline the basis for the exclusion of such a limit in the 2024 draft Technical Specification for NZS1170.5 [1], despite the presence of such limits in NZS1170.5:2004 [2]. An overview of the historical consideration of so-called 'deterministic' and probabilistic seismic hazard analysis methods is provided, as well as how they have translated into contemporary seismic design codes and standards in New Zealand (NZ) and internationally. The fundamental issues with deterministic maximum limits are outlined through the use of examples in a NZ-specific context. The underlying reason 'well above average' ground-motion intensity levels (for a given earthquake scenario) are prevalent in regions of high seismicity is discussed, as well as other common misconceptions that lead to the use of deterministic limits to achieve apparently realistic design ground motion intensities. Finally, in the vein of the hazard-risk separation principle, sentiments are expressed for achieving economic and resilient seismic design in regions of high seismicity without resorting to implementing deterministic limits.

https://doi.org/10.5459/bnzsee.1706

INTRODUCTION

Contemporary seismic design codes and standards adopt an array of modifications to the underpinning results based on probabilistic seismic hazard analysis (PSHA) [3] to arrive at design ground motion intensities. One such modification includes limiting the maximum design spectral amplitudes based on a 'deterministic' scenario, and are colloquially referred to as 'deterministic limits' or 'deterministic caps'.

A deterministic *minimum* level of seismic loading [*e.g.* 2,4] is relevant in low seismicity regions to guard against the low, but non-zero, probability of moderate ground-motion shaking resulting from a proximal moderate magnitude, or distant larger magnitude, earthquake. Such a lower limit of seismic load is relatively uncontentious, and will not be discussed further in this paper. A maximum level of seismic loading is typically determined based on a 'maximum considered earthquake' magnitude on a proximal fault and a specific percentile from the resulting ground-motion prediction distribution.

The current seismic loading standard in New Zealand, NZS1170.5:2004 [2], utilises such a (maximum) deterministic limit for all locations nationwide based on a M8.1 scenario. The NZ Society for Large Dams (NZSOLD) guidelines [5] also indicate that a site-specific seismic hazard study should undertake PSHA as well as deterministic seismic hazard analysis. In the United States of America (US), ASCE 7-22 also adopts a deterministic limit, based on the seismogenic potential of nearby faults.

Despite the use of deterministic limits in the aforementioned codes and standards, there are fundamental issues with their development and the implications of their adoption. This is a widely held view in academic literature [see 3, Section 6.10.3] and even in code revision cycles, *e.g.*, Stewart *et al.* [6]. In such context, this paper outlines the issues with a deterministic treatment of seismic hazard, its consideration for seismic design in NZ, and ultimately why its omission in the draft Technical Specification for 1170.5 (TS1170.5:2024; [1]) is, in the author's opinion, the right approach toward respecting the hazard-risk separation principle.

HISTORY OF PROBABILISTIC AND DETERMINISTIC SEISMIC HAZARD ANALYSIS

To appreciate the current presence of deterministic limits in seismic design codes and standards, it is instructive to understand the historical estimation of seismic hazard using probabilistic and deterministic seismic hazard analysis, as well as the specification of seismic loads in said codes.

During the period 1909-1930's[1], seismic design loads in different countries were originally specified as constant multipliers on building weight. These load levels were largely based on precedent without any formal basis in analysis (owing to the lack of actual instrumental observations of building responses at the time and inability to model complex structural response). In the 1960's, several seismic design codes (or proposals for inclusion in codes) began to associate such ground-motion intensity levels with average return periods (RPs), but the technical basis for how these were determined was far from clear [8].

A turning point was the publication of what is now referred to as *probabilistic seismic hazard analysis* by Cornell [8] and Esteva [9], and popularised through the computer program of McGuire [10]. PSHA formalised the quantification of a relation-

[1]End date varies by country, but most seismically-active countries adopted such regulations before the end of the 1930's [7].

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ship between ground-motion intensity and its likelihood (*i.e.*, mean annual rate of exceedance, or its reciprocal, RP) [11]. The first national seismic hazard maps directly using PSHA were in Mexico in 1970 [12]. Subsequently, PSHA-based maps have been adopted nearly globally, including in the US in 1976, and NZ in 1985 [13].

The ubiquitous adoption into national seismic design and assessment mapping, and site-specific assessments for critical infrastructure, is empirical evidence toward a preference for a predominantly PSHA-based approach to seismic hazard quantification. Nonetheless, there remain skeptics of the PSHA method [*e.g.* 14–16], mostly because of an incorrect conflation of the overall method (which is simply an application of the total probability theorem), and the specific assumptions that are used for the constituent seismic source characterisation (SSC) and ground-motion characterisation (GMC) models within the method [*e.g.* 17–19]. These sentiments are discussed further in Baker *et al.* [3, Chapter 12], where emphasis is given to the common misconceptions and analytical mistakes as to how earthquake and ground-motion observations are compared with PSHA results and the underlying SSC and GMC predictions.

This is not to imply that all the sentiments that are considered in a 'deterministic' approach are without merit, of course. Bommer [20] and McGuire [21] both compare and contrast the two approaches, and note that they share some similarities. Anderson [22] also points to the benefit of scenario ground-motion maps, and suggests that 'scenario hazard' is a more accurate word than 'deterministic hazard' to reflect the actual calculations that are being undertaken (and also that deterministic procedures typically have a probabilistic element to them in the heuristic decision to use the 84^{th} percentile of the ground-motion intensity distribution [20]). Importantly, Table 1 of McGuire [21] illustrates the array of different earthquake decisions that are commonly considered, and which method is more prevalent in use. Specifically, PSHA is routinely adopted for quantitative-oriented decisions such as setting seismic design for new structures, retrofit of existing structures, and (re)insurance pricing; whereas earthquake scenarios are insightful for planning for emergency response and post-earthquake recovery.

DETERMINISTIC LIMITS IN PSHA-BASED SEISMIC LOADING STANDARDS

As mentioned in the prior section, PSHA now overwhelmingly dominates the quantification of seismic hazard for the development of seismic design loads for both conventional structures (*i.e.*, in national seismic hazard maps [*e.g.* 23,24]) and safety-critical facilities [25]. Despite this, fragments of the deterministic hazard method remain inter-twined in many seismic design standards and codes of practice. Given the arc of history discussed in the prior section, this progressive unravelling of a deterministic mindset toward the consideration of seismic hazards is somewhat logical. However, there are also indications that the inclusion of deterministic limits on maximum shaking have been introduced as a means for dealing with otherwise increasing ground-motion intensities as a result of an update to a national seismic hazard map. For example, Stewart *et al.* [6] indicate this was the rationale for the introduction of deterministic limits in the 1997 NEHRP provisions [26]. In this section, the specific prescriptions of deterministic limits in relevant codes and standards are briefly addressed, as well as the points in favour of such approaches as voiced by their proponents.

Deterministic Limits in ASCE 7-22

As a relevant international example, ASCE-7-22 [4] adopts deterministic limits on maximum ground motion intensities (see C21.2.2 of ASCE 7-22). The determination of deterministic ground-motion limits in ASCE 7-22 adopts several changes from ASCE 7-16 [27], as discussed in a subsequent section of this paper, that (ironically) further align the determination of the associated scenario earthquakes to those based on PSHA results. The deterministic limits are nominally obtained on a site- and fault-specific basis. However, the calculated value must be at least as large as tabulated values that are prescriptively specified based on 84^{th} percentile ground-motion intensities from a moment magnitude M8.0 shallow crustal earthquake at a source-to-site distance of approximately $R = 12.5$ km[2], which geographically represents a northern San Andreas rupture in downtown San Francisco (*i.e.*, a nominal repeat of the 1906 event).

Deterministic limits on maximum ground motions were first introduced into US-based standards in 1997 [26]. As noted by Stewart *et al.* [6], in 1997 the RP for design ground-motions was increased from a nominal value of 475 years to 2475 years, in combination with a $2/3$ 'margin of safety' factor. The resulting "two-thirds of 2475-year" design ground motions were similar to pre-1997 levels away from major faults in active regions like California, but resulted in larger values in many California locations. Most likely with a preference toward *stability* of design loads, the notion of deterministic limits on maximum ground motion was introduced. It was argued that designing for larger ground motions was impractical and unjustifiably costly (which, as discussed later, is generally not supported by evidence).

While deterministic limits remain in ASCE 7-22, there is increasing awareness of the problems with their use. In particular, Stewart *et al.* [6] elaborate on discussions in the development of ASCE 7-22 that explicitly considered how to remove the current use of such limits, and proposed several alternative approaches that could be adopted to phase out their use.

Deterministic Limits in NZS1170.5:2004

NZ first introduced deterministic limits in NZS1170.5:2004 [2], seven years after its adoption in US standards in 1997. The documentation on its inclusion is relatively minimal [28], but explicit reference is made to the adopted approach being based on international precedents.

The deterministic limit for maximum ground-motion intensity is based on a near-source Alpine Fault rupture scenario, specifically M8.1 at a distance of $R = 0$ kilometres. Thus, similar to ASCE 7-22, the limit is based on a specific rupture scenario. However, the +0.1 unit larger magnitude and the use of an $R = 0$ distance (*vs.* 12.5 km in ASCE 7-22) both lead to larger deterministic limiting motions. Lastly, again similar to ASCE 7-22 (and prior US codes), the 'margin of safety' factor of 1.5 is used to reduce the directly computed scenario ground-motion intensity in order to arrive at the deterministic limit. The result of the above considerations is a maximum limit on the product of the NZS1170.5:2004 parameters $ZR_u = 0.7$, where Z is the hazard factor, and R_u is the return period factor for the ultimate limit state.

In NZS1170.5:2004, the deterministic limit does not have an

[2]This is referred to as the 'deterministic lower limit'. ASCE 7-22 allows for calculation of a deterministic limit via a site-specific hazard analysis, but such a process is involved to the point that the prescriptively provided values are likely adopted for the majority of use cases that avoid a site-specific hazard study.

effect on seismic design levels for ordinary (Importance Level 2) structures, in contrast to its pervasive impact in the US, as elaborated upon by Stewart *et al.* [6]. This is because such structures have their ultimate limit state design for a 500 year RP (for which $R_u = 1$), and the maximum seismic hazard region in NZ is $Z = 0.6$ (locations: Arthurs Pass/Otira). In contrast, for high-importance structures (Importance Level 4, $R_u = 1.8$), the limit of $ZR_u = 0.7$ implies that $Z_{max} = 0.383$, and this deterministic maximum limit therefore governs for such structures in the lower North Island, within approximately 50 km of the Alpine Fault, and in the southern portion of the Marlborough Fault System [2].

The reason that the deterministic maximum limit is less impactful for ordinary structures in NZ, as compared to the US, is principally due to two factors: (i) the greater deterministic maximum limit due to the larger magnitude and smaller source-to-site distance adopted; and (ii) that ultimate limit state design is based on the 500 year return period, which is parametrically 1.8 times smaller than the 2500 year loading, *i.e.*, the factor 1.8 is 20% larger than the 1.5 margin of safety factor used to reduce the deterministic loading. However, it is also worth noting that the PSHA results in the most recent 2022 NZ National Seismic Hazard Model (NSHM) [29] suggest an increase in hazard on the order of several 10's of percent for the majority of locations in NZ, so such deterministic limits, if considered, could have a greater impact at limiting PSHA-based design ground-motion levels[3].

Proponent Arguments for Deterministic Limits on Maximum Design Ground Motions

Numerous arguments are offered by proponents in support of the use of deterministic limits on maximum ground motions [*e.g.* 6,15,16,20]. In a code-based context, the clearest arguments are [6]:

1. Deterministic limits are based on faults that produce frequent large-magnitude earthquakes, which can be readily identified;
2. The 'characteristic' magnitude of these faults can be established with reasonable certainty; and
3. The ground motions estimated from the 84^{th} percentile of the prediction distribution will be large enough, when used to design a structure, to avoid catastrophic consequences if the event occurs.

Thus, while some of the sentiments enumerated in the above list may have intuitive appeal and apparent logic; observational evidence, and extrapolation based on first-principles modelling, contradicts these points as discussed in the next section.

ISSUES WITH MAXIMUM DETERMINISTIC LIMITS

As discussed in Baker *et al.* [3, Sections 1.3 and 6.10.3] and Stewart *et al.* [6], there are several conceptual issues with the use of deterministic limits on maximum ground-motion intensities. These are enumerated below, and then elaborated on sequentially in the following subsections via illustrative examples.

1. The identification of major active faults is non-trivial, and poorly-mapped faults can produce greater-than-design level ground-motions at the site of interest;
2. The magnitude of future earthquakes on major earthquake sources has a significant degree of variability, such that 'characteristic' earthquakes are generally not practically identifiable;
3. Ground motions from future earthquakes will have a wide range of intensities, and use of the 84^{th} percentile of the prediction distribution leaves potential for appreciably greater intensities.
4. In combination, these assumptions result in a deterministic ground-motion limit that is not a *worst-case* scenario, and also has an unknown likelihood of being exceeded.

Identification of Major Faults

Traditional deterministic seismic hazard analysis [*e.g.* 30] considers the 'controlling' ground-motion from all potential seismic sources. However, the modern adaptation in seismic design codes for buildings (*e.g.*, ASCE 7-16 [27]) does not consider all possible ruptures, but instead restricts attention to 'major' active faults, which leaves open the interpretation of what level of seismic activity is necessary for a fault to be considered as major? [6]. More recently, ASCE 7-22 [4] further adjusted the definition of major sources to be those sources that contribute more than 10% of the largest contributor to the total hazard at the site (as determined via disaggregation), such that no subjective assessment of 'major' is needed.

A one-sided argument can be made that this major active fault definition (*i.e.*$>$ 10% disaggregation contribution) avoids relatively lower-slip rate faults (and even ruptures on unmapped faults that are accounted for via distributed seismicity sources), thus avoiding the controlling scenario being a distributed seismicity source, of the maximum allowable magnitude, directly underneath the site; presumably because it is subjectively considered unlikely and too conservative [see 3, Chapter 1]. However, the counter argument is that such an approach will miss earthquake sources that pose a clear hazard to the site and can produce ground motions that are greater than current seismic design levels. A case in point is the three major earthquakes of the 2010-2011 Canterbury earthquake sequence (CES) - 4 September 2010, 22 February 2011, and 13 June 2011 - all three of which produced ground-motion spectral amplitudes in urban Christchurch that exceeded the 500 year RP ultimate limit state design spectral amplitudes [31–33]. Notably, these CES events produced ground-motion intensities that exceed those anticipated from large earthquakes on major faults in the South Alps and associated foothills, such as the Alpine, Hope, and Porters Pass faults [*e.g.* 34,35]. However, all of these CES events would *not* be considered as scenarios in a deterministic analysis, because they did not occur on explicitly modelled fault sources. In contrast, they are all considered implicitly within the distributed seismicity model used in PSHA [36], which accounts for potential future earthquakes on unmapped faults.

Significant Degree of Variability in the Magnitudes of Major Fault Ruptures

A central basis of deterministic seismic hazard analysis is the notion that a characteristic magnitude can be established with reasonable certainly for each major fault source. It is important to be mindful of the era in which deterministic limits were first introduced into modern seismic design codes (*e.g.*, 1997 and 2004 in the US and NZ, respectively). At this time the char-

[3] The degree of impact would also depend on revised ground-motion predictions for the same deterministic maximum scenario, including revision to reflect the subduction interface scenarios that are presently ignored.

acteristic earthquake hypothesis and fault segmentation[4] were still firmly held paradigms in seismic hazard analysis [39,40]. Presently, however, there is ample evidence from global observations of complex multi-segment fault rupture (*e.g.*, the 2010 Darfield and 2016 Kaikōura earthquakes as NZ examples), to illustrate that fault structures exist as fractal-like components of a greater fault network. And while geometric complexities in fault networks may provide preferential settings to inhibit through-going rupture, the characteristic rupture and fault segmentation paradigms, in the limit, were largely just a convenient modelling assumption to simplify the possible set of potential ruptures in a fault system. Most importantly, the development of inversion methods [41,42] to determine the numerous rupture distributions of interconnected fault networks (typically in high seismicity regions) that generalise beyond simple characteristic or Gutenberg-Richter distributions has enabled seismic hazard analysis to generalise seismic source characterisation beyond these paradigms.

Wide recognition that specific fault segments participate in earthquakes with a range of magnitudes is reflected in the adopted language of more recent codes that still retain deterministic maximum limits, such as ASCE 7-22, which notes "characteristic earthquakes are no longer specified because they are considered to be inconsistent with recent earthquakes" [4, Section C21.2.2]. Instead ASCE 7-22 defines *scenario earthquakes* as a substitute for characteristic earthquakes, and assigns them as having a magnitude equal to the mean of the disaggregation distribution for the associated PSHA results for that specific source.

Figure 1 illustrates two example magnitude disaggregation distributions from the 2022 NZ NSHM (obtained from https://nshm.gns.cri.nz/Disaggs) for Otira and Wellington. The hazard in Otira is dominated by active shallow crustal ruptures on the Alpine Fault, while the hazard in Wellington is contributed to by crustal, subduction interface and slab ruptures.

With these two NZ examples in mind, there are several important sentiments in terms of: (i) the ASCE 7-22 definition of scenario earthquakes for deterministic limit estimation; and (ii) the M8.1 Alpine Fault scenario that the NZS1170.5:2004 deterministic limit is based on. First, significant ruptures on the Alpine Fault that affect the hazard in Otira range from approximately M6.6-M8.55, with a mean source magnitude of M7.98. Therefore, according to ASCE 7-22, M7.98 would be adopted as the scenario magnitude for a deterministic maximum limit. However, Figure 1 illustrates that up to M8.55 events are modelled to occur on the Alpine Fault. To provide a geographical perspective on this difference, Figure 2 illustrates two potential ruptures on the Alpine Fault that are within the 2022 NZ NSHM (available at: https://nshm.gns.cri.nz/RuptureMap). The top panel is an example of the fault sections that participate in a M8.1 rupture, the same magnitude as adopted in NZS1170.5:2004 for the deterministic limit calculation. The bottom panel illustrates the largest modelled rupture that includes the Alpine Fault section closest to Otira, which results in a M8.55 rupture that is nearly three times longer than the M8.1 scenario illustrated in the top panel.

As a second example, the magnitude disaggregation of the hazard in Wellington illustrated in Figure 1 contains a significant contribution from ruptures on the Hikurangi subduction interface. The mean interface magnitude is M8.25, but ruptures range from approximately M6.6-M9.6. Similar to the example of the Alpine Fault, Figure 3 illustrates an example rupture geometry of

[4]That is, faults have a preference for repeated rupturing of approximately the same fault segments with a near constant magnitude [37,38].

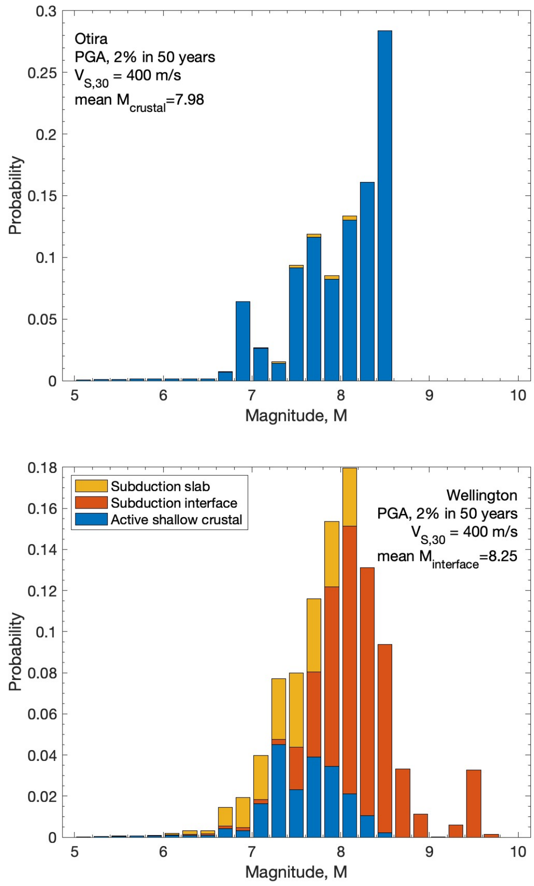

Figure 1: Magnitude disaggregation distribution for Otira (top panel) and Wellington (bottom panel) from the 2022 NZ NSHM for the 2% in 50 year exceedance probability of** PGA **on** $V_{S,30} = 400$ **m/s site conditions.

a M8.25 earthquake (*i.e.*, that which would be adopted following the ASCE 7-22 ‘scenario mean magnitude’ logic) as well as a full rupture of the interface in a M9.6 event. The associated rupture area of these two cases differs by a factor of nearly 20, and clearly the associated ground motions from these events would be different, particularly for structures sensitive to long vibration periods.

The subduction interface example is also useful to point out that neither NZS1170.5:2004 nor the prescriptive ASCE 7-22 provisions refer to subduction earthquake sources when setting the respective M8.1 and M8.0 scenarios that they adopted. This is likely because: (i) the threat of ruptures on the Hikurangi interface was not well understood in NZ at the time of the development of NZS1170.5:2004 (underpinned by the 2002 NZ NSHM of Stirling *et al.* [40]); and (ii) the ASCE 7-22 deterministic lower limit provisions are principally of relevance for California, where crustal faults dominate the hazard.

These two examples in Otira and Wellington have illustrated that the distribution of earthquake magnitudes on crustal (*e.g.*, Alpine Fault) and subduction (*e.g.*, Hikurangi interface) sources varies appreciably, and the use of any single magnitude (*e.g.*, the mean) will provide an incomplete assessment of the resulting hazard that is actually posed to the site. As further explored in the next subsection, in some instances the difference in ground-motion intensity from the range of source magnitudes may be significant, particularly so for long-period spectral amplitudes.

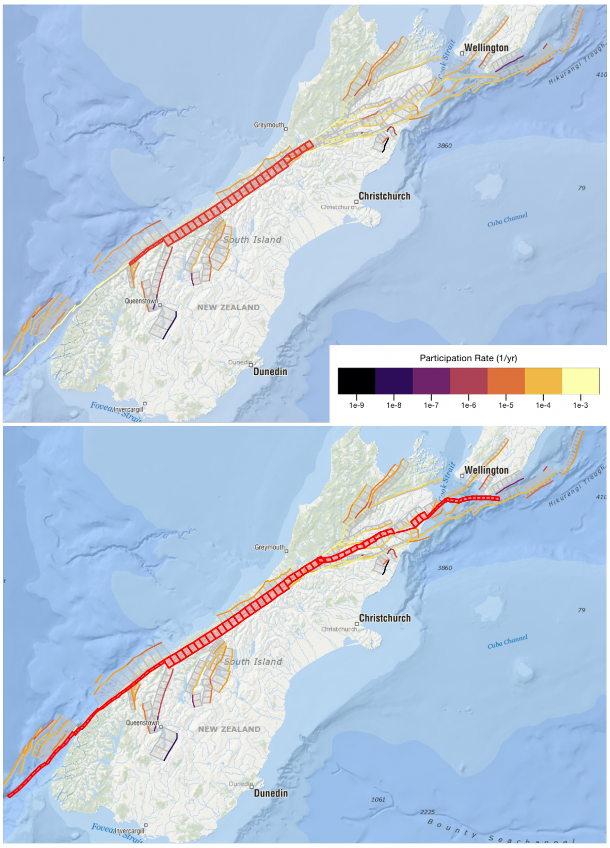

Figure 2: Fault segments in the 2022 NZ NSHM that participate in rupture scenarios that include the Jacksons-to-Kanerie section of the Alpine Fault. Colors on the fault surface trace indicate the participation rate. Top panel: a 357 km-long M8.1 rupture (annotated in red), which is the same magnitude assigned in NZS1170.5:2004 for the deterministic maximum limit. Bottom panel: a 977 km-long M8.55 rupture (annotated in red).

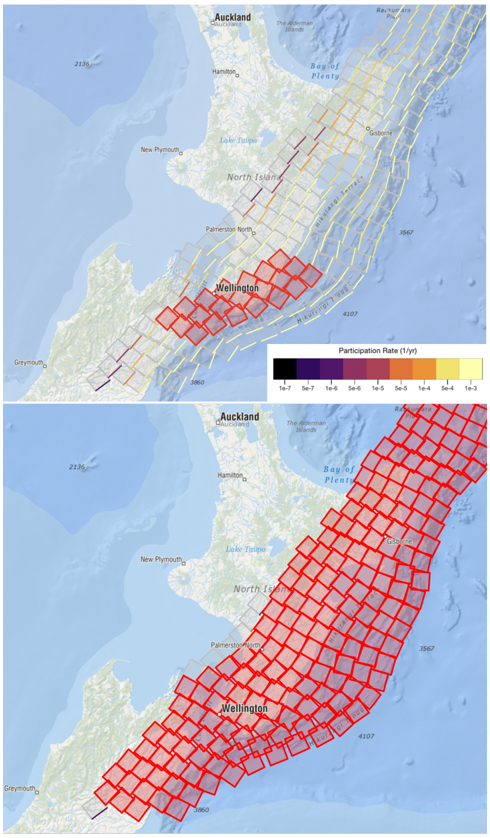

Figure 3: Fault segments in the 2022 NZ NSHM that participate in rupture scenarios that include the Hikurangi subduction interface sources. Colors on the fault section edges indicate the participation rate. Top panel: a 1.89×10^4 ***km***^{2} ***M8.25 rupture (annotated in red). Bottom panel:*** 3.99×10^5 ***km***^{2} ***M9.6 rupture of the entire interface northward into the Kermadec Trench (annotated in red).***

Significant Variability in Predicting Ground-Motion Amplitudes for a Given Scenario Earthquake

Observed ground motions for a given magnitude, source-to-site distance, and similar site conditions, exhibit appreciable variability in their intensity. As a result, the explicit consideration of apparent aleatory variability in ground-motion models is considered as an indispensable component of ground-motion modelling and seismic hazard analysis [see 3, Section 4.4.3]. In a deterministic hazard analysis, the single value corresponding to the 84^{th} percentile of the ground-motion intensity distribution has been historically utilized, and this is what is adopted as the basis for deterministic limits in ASCE 7-22 and NZS1170.5:2004.

Why the 84^{th} percentile, in particular? Statistical examination of ground motions suggests that the lognormal distribution is phenomenologically appropriate for several standard deviations above the mean [*e.g.* 43], and there is no robust basis for truncation of ground-motion predictions less than, at least, three standard deviations above the mean [*e.g.* 44]. That the 84^{th} percentile represents exactly one standard deviation above the mean of the modelled lognormal distribution should trigger the reader's suspicion that it is largely a subjective precedent, rather than having a theoretical basis. Proponents for deterministic hazard analysis would mention something to the effect that using the median (50^{th} percentile) would be inappropriately unconservative, because half of observed ground motions would exceed that adopted intensity; but to use something more than the 84^{th} percentile, *e.g.*, the the 98^{th} percentile (two standard deviations above the mean), would be introducing too much conservatism.

Unfortunately for proponents of the deterministic approach, the subjective adoption of a specific percentile has an appreciable impact on estimated ground-motion intensities. Figure 4 illustrates the predicted response spectra of major Alpine Fault ruptures at a source-to-site distance approaching $R = 0$ km[5], consistent with the seismic hazard in Otira depicted in Figure 1. Figure 4 depicts the median (50^{th} percentile), as well as the one and two standard deviation predictions from the median. It also depicts the prediction for two magnitudes - the 'mean scenario magnitude' of M7.98, consistent with the ASCE 7-22 deterministic lower limit definition, and the maximum magnitude rupture considered in the 2022 NZ NSHM. The effect of the adopted percentile clearly has a major impact on the resulting ground motion - with a PGA value of 1.22g for the 84^{th} percentile, but values ranging from 0.748g - 1.99g for the 50^{th} and 98^{th} percentiles. A similar range in intensities is observed for the prediction of $SA(3s)$ spectral amplitudes. There is also an effect from the assumed magnitude of the scenario - *e.g.*, an approximately 10% difference in the median prediction for $SA(3s)$, but a negligible difference for the prediction of PGA.

The Otira ground-motion intensity example of Figure 4 illustrates

[5] Based on the Bradley [45] ground-motion model, which was one of the several active shallow crustal models adopted in the 2022 NZ NSHM [46].

the significant effect of which percentile of the ground-motion distribution is adopted. While there is a consistent precedent for the adoption of the 84^{th} percentile of this distribution in deterministic hazard analysis practice, including its use in seismic design codes, the fact that this percentile choice is arbitrary undermines the credibility of the resulting intensity values that are obtained. In PSHA, the entire prediction distribution that reflects this ground-motion variability is considered via the probability of the ground-motion intensity measure being exceeded. Thus, PSHA considers ‘average’ levels of ground-motion intensity that are frequently exceeded, as well as a ‘rare’ ground-motion intensity that is infrequently exceeded - all weighted probabilistically by their likelihood of occurring.

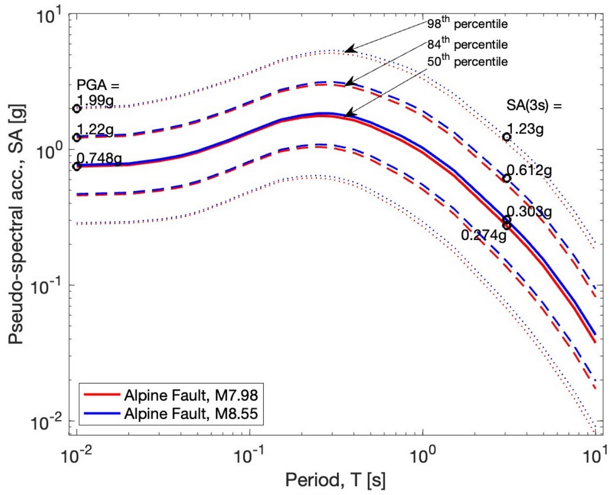

Figure 4: Predicted ground-motion pseudo-acceleration response spectra for a near-source ($R_{Rup} \to 0$ km) Alpine Fault rupture on a $V_{S,30} = 400$ m/s site. Two magnitudes are predicted - corresponding to the ‘mean magnitude scenario’ definition in ASCE 7-22, and the maximum magnitude from the disaggregation in Figure 1, as well as the 50, 84, and 98^{th} percentiles, which correspond to $\varepsilon = 0$, 1, and 2 standard deviations above the distribution logarithmic mean.

Deterministic Limits are not Worst-Case Scenarios - So What is their Likelihood of Exceedance?

As alluded to in the prior three subsections, the deterministic approach to seismic hazard analysis requires subjective decisions about which earthquake sources to consider, what magnitude to assign to them, and what percentile of the resulting ground-motion intensity distribution to consider. What should also be clear is that deterministic limits are not obtained based on the ‘worst case’ earthquake and resulting ground motion. Hence, it is implicitly accepted that the ground-motion intensity obtained from a deterministic maximum limit will have some probability of being exceeded during the design life of the structure being considered.

As an illustrative example, Figure 5 depicts the PSHA-based seismic hazard curves from the 2022 NZ NSHM (obtained from https://nshm.gns.cri.nz/Hazardcurves) for Otira and annotates the specific points on the seismic hazard curves that correspond to the results from the 50, 84, and 98^{th} percentiles from the M8.55 Alpine Fault scenario shown in Figure 4. The M8.55 Alpine Fault 84^{th} percentile ground-motion intensity values for PGA and SA(3s), in particular, correspond to annual probabilities of approximately 1×10^{-3} and 6.3×10^{-4} from the mean seismic hazard curves. These represent return periods of approximately 1000 and 1600 years, respectively. It is also evident from the other black points annotated in Figure 5 that the 50, 84, and 98^{th} percentile scenario values give a wide range of associated annual probabilities of exceedance spanning a factor of approximately 15.

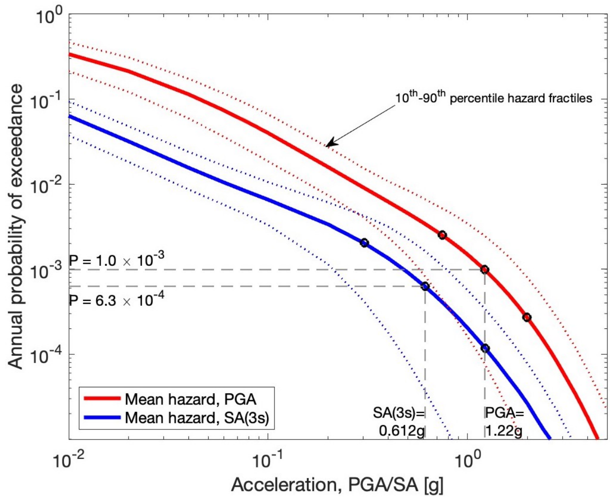

Figure 5: Comparison of PSHA results for PGA and SA(3s) from the NZ NSHM for $V_{S,30} = 400$ m/s in Otira with the deterministic limit values based on the near-source M8.55 Alpine Fault scenario. Results are shown for both the mean hazard, as well as the 10^{th} and 90^{th} fractiles of the hazard.

Like ground-motions calculated using a deterministic framework, the values obtained from PSHA are also not worst-case intensities, and also can be exceeded. The main difference is that through explicitly considering all potential ruptures on seismic sources, their likelihood of occurrence, and the consequent distribution of resulting ground motions, PSHA-based seismic design ground-motion intensities have an explicitly-determined probability of being exceeded[6]. In contrast, the likelihood of exceeding the deterministic limit is not computed, and can only be ascertained by comparing it with the results from PSHA, as shown from the example in Figure 5. Said another way, when a deterministic limit governs the resulting seismic design loading, it is equivalent to adopting a different probability of exceedance for the specified seismic loading.

Policy-Related Issues with Deterministic Limits on Maximum Ground-Motion Intensity

As elaborated upon by Stewart *et al.* [6], the principal policy-related issues that result from the use of deterministic limits on maximum ground-motion intensity are:

1. A non-uniform hazard[7] that is not understood by the public and is difficult to explain to asset owners.
2. As a result of non-uniform hazard, all other things equal (*i.e.*, building fragilities), different levels of seismic protection are enforced in different regions.
3. Deterministic limits increase seismic risk by an unspecified amount, and hence there is a lack of transparency.

[6]This quantification does have epistemic uncertainty associated with it, due to imperfect models/knowledge, but there are structured processes by which such uncertainty is considered and propagated in the hazard calculation [see 3, Chapter 6].

[7]Stewart *et al.* [6] refer to *risk* in place of the use of the word *hazard* adopted here, because ASCE 7-22 directly adopts a risk-based determination of seismic loads, whereas NZS1170.5 [12] uses a hazard-based specification.

Stewart *et al.* outline two approaches as alternatives to deterministic limits. Both proposed approaches seek to explicitly reflect the actual results from PSHA by either accepting higher probabilities of exceeding the design ground motions everywhere in the US, or specifically in the vicinity of major faults. Although it was not discussed directly by Stewart *et al.*, presumably because it was perceived to be untenable in a US code context, another option is to avoid deterministic limits, but without changing the design return periods (or equivalent probabilities of exceedance), thus, accepting the seismic hazard for what it is, and instead pushing the focus to appropriate seismic design and analysis techniques that can achieve acceptable levels of risk in the face of such a hazard. Neither of the alternative options outlined in Stewart *et al.*were pursued in a NZ context, because design return periods (or, equivalently, probabilities of exceedance) are set separately in NZS1170.0: 2002 [47], and hence, were beyond the scope of the TS1170.5 development.

DISCUSSION

In this discussion section, the fundamental reason why such high levels of ground motion result from PSHA is presented, followed by a rational discussion about achieving acceptable seismic risk in high seismic hazard regions. Collectively, in conjunction with the prior NZ-specific context, they provide a coherent argument against the consideration of deterministic maximum limits.

Why do high seismic regions typically have 'high-epsilon, ε' design-level ground motions?

Ignoring that deterministic limits on maximum ground motions have been adopted in order to obtain seismic design levels that are similar to prior codes and standards, another common rationale for their consideration is that "ground motion intensities from PSHA are at the tails of the prediction distribution and are therefore not representative for design". This section explains why this is the case for common return periods considered in seismic design in high seismicity regions, building on the examples that have been presented thus far.

From a mathematical perspective, if the ground-motion exceedance rate considered is λ_{SA}; a given rupture contributes a percentage P_{rup} to the hazard (obtained from disaggregation); and the occurrence rate of that rupture is λ_{rup}, then the 'epsilon' value, ε, of that rupture will be (see Appendix):

$$\varepsilon = -\Phi^{-1}\left[\frac{\lambda_{SA}P_{rup}}{\lambda_{rup}}\right] \quad (1)$$

The ε value reflects the number of standard deviations that the ground-motion intensity value is from the mean of the ground-motion model distribution. Values of $\varepsilon = 0, 1, 2, 3$ correspond to the 50, 84, 98, and 99.9th percentiles of the distribution. In order to illustrate some typical values for high seismicity regions, consider $\lambda_{SA} = 1/2475 = 4.04 \times 10^{-4}$, with a dominant fault rupture that comprises P_{rup}=30% of the total hazard, and has a recurrence interval of 500 years (*i.e.*, $\lambda_{Rup} = 1/500 = 0.002$). Then $\varepsilon = \Phi^{-1}[4.04 \times 10^{-4} \times 0.3/0.002] = 1.55$, which is the 94th percentile of the ground-motion model distribution.

Now consider the same problem framing, but in a low seismicity setting where the occurrence rates of all seismic sources are 10 times smaller. The same calculation as above, but with $\lambda_{Rup} = 0.0002$, then becomes, $\varepsilon = \Phi^{-1}[4.04 \times 10^{-4} \times 0.3/0.0002] = -0.27$ (the 39th percentile of the distribution, *i.e.*, below the median prediction). Hence, one can see that in low seismicity regions, for the same ground-motion exceedance rate (or equivalent return period), the ground motions will generally be associated with smaller ε values (*i.e.*, lower percentiles of the prediction distribution). Of course, it is still possible to get high ε ground-motion levels in low seismicity regions, you just need to consider smaller exceedance rates - *e.g.*, considering the 24,750 year return period in the given example would again produce $\varepsilon = 1.55$.

Does PSHA give larger results because it considers multiple high-intensity ground motions from different ruptures during the design life of the structure?

Another point of confusion, although less heard, is that "PSHA considers multiple large rupture sources, and therefore produces larger hazard values because it is considering multiple large ground motions occurring during the design life of the structure". There are two key aspects to clarify this confusion. The first is pertaining to how the rupture rate is considered in the PSHA calculation, and the second is differentiating between the hazard curve for 'one or more' occurrences of the ground-motion level and specifically one occurrence.

First, seismic design is strictly based on ground motions at the site of interest, not the implicit earthquakes that cause them (though the latter, obviously, are causally linked to the former). A thought experiment is useful to illustrate this point. Consider a site with a single seismic source that has a rupture rate of λ_{rup_0}. Now consider a second site with two rupture sources that have rupture rates of λ_{rup_1} and λ_{rup_2}, respectively, such that $\lambda_{rup_0} = \lambda_{rup_1} + \lambda_{rup_2}$. If all three ruptures produce the same ground-motion intensity distribution at the sites of interest, then the seismic hazard at both sites will be identical (as per Equation 2 in the Appendix), irrespective of the fact that the first site is subjected to a single earthquake source and the second site is subject to two sources. That is, the hazard is solely a function of the rate of exceedance of the ground-motion intensity at the site, and not by how many different potential earthquake ruptures can cause this ground-motion intensity.

Second, the seismic hazard is commonly presented in terms of the mean annual rate of exceedance of the ground-motion intensity measure, and denoted with the symbol, λ (see Equation 2 in Appendix). The reciprocal of this is the return period. In order to determine the probability that the ground-motion level occurs during a specific time period, T, the Poisson distribution is most commonly invoked (which assumes time independence). As noted in Equation 9 in the Appendix, the probability of one or more occurrences is given by the common formula: $P(Y \geq 1) = 1 - e^{-\lambda T} \approx \lambda T$. For example, for the 2475 year return period ($\lambda = 1/2475$), and a $T = 50$ year design life, $P(Y \geq 1) = 0.2$ (the approximation gives 0.202). But, this is the probability of *one or more* occurrences, how likely is it to have more than one occurrence? (*i.e.*, the design ground motion level be exceeded two of more times in the service life of the structure). From Equation 11 in the Appendix, this is $P(Y \geq 2) \approx \frac{(\lambda T)^2}{2}$, which for the example above is $P(Y \geq 2) \approx 2 \times 10^{-4}$, or 1% of the value of $P(Y \geq 1)$. Said another way, if there is one or more occurrences, then there is a 99% chance of only one, and only a 1% chance of two or more. Hence, for typical likelihoods considered in seismic design, the probability of more than one occurrence is sufficiently low that it can be ignored.

Seismic ground-motions for design that honour the hazard-risk separation principle

Marzocchi and Meletti [48] discuss that PSHA-based hazard results can be regarded as a best scientific estimate of the truth, in so far as they are a probabilistic forecast of the future. They emphasise that it is critical to honour the hazard-risk separation principle [49] - *i.e.*, the hazard is what it is, whereas we can design the built environment in order to accept varying levels of risk as a result of this (unchangable) hazard. The use of deterministic limits of maximum ground motions that arbitrarily reduce the effective return period of ground-motion intensities used in seismic design are the equivalent of "putting one's head in the sand" with regard to an externality that cannot be controlled.

An increase in seismic design loads (that will occur where deterministic maximum limits that govern design loads are removed) naturally provoke responses to the effect of (i) "the associated cost implications will be too high", or (ii) "such loads cannot be designed for". Point (i) may seem logical, but more thorough analyses have time-and-time again shown that increases in design loads have a none-to-minor impact on actual construction costs, as discussed by Porter [50] (and references therein), and NZ-specific anecdotes noted by Hare [51]. Point (ii) does have practical merit for many of the simplified methods of seismic analysis that are used in conventional engineering design. In particular, pseudo-static approaches, which treat seismic loads as an equivalent static force, are particularly problematic when the level of seismic loads increases. This clearly requires advancements in engineering science and research in order to develop new simplified analysis methods that are not as 'brittle' with regard to the level of seismic loads being imposed. But it is also presents a challenge and opportunity for engineering practice in order to embrace more advanced analysis methods that seek to actually model deformations that occur during seismic response.

CONCLUSIONS

This paper has examined the use of deterministic maximum limits on ground-motion intensity in contemporary seismic design codes and standards. With reference to the history of seismic hazard analysis, and also examples in a NZ-specific context, the fundamental issues with deterministic seismic hazard analysis, and thus deterministic limits on maximum ground motions were outlined. In particular, deterministic limits require subjective decisions about which earthquake sources to consider, what magnitude to assign to them, and the adoption of an arbitrary percentile of the resulting ground-motion intensity distribution. As a result, deterministic ground motion limits result in ground-motion intensities that have an unquantified likelihood of exceedance.

The draft Technical Specification of NZS1170.5:2024 [1] does not utilize deterministic limits on maximum ground motions, unlike NZS1170.5:2004. While this results in greater ground-motion intensities, this better reflects our best scientific estimate of the truth in terms of forecasted future ground motions in NZ. While the cost implications of designing to higher ground-motion intensities are largely not supported by evidence, addressing higher seismic hazard loading for seismic design does require new simplified analysis methods that are not 'brittle' in the face of high seismic loads, and also presents a challenge to the practicing profession to make use of more advanced methods that actually seek to model deformations.

ACKNOWLEDGEMENTS

This paper benefited from constructive peer-review comments from John Hooper and Jonathan Stewart. This work was partially funded by the NZ Ministry of Business, Innovation, and Employment and Te Hiranga Rū QuakeCoRE: The NZ Centre for Earthquake Resilience. This is QuakeCoRE publication number 990.

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APPENDIX

Derivation of Equation 1

Consider the general equation for the seismic hazard curve [3, Equation 6.1]:

$$\lambda(SA > x) = \sum_{i=1}^{n_{rup}} P(SA > x|rup_i, site)\lambda(rup_i) \tag{2}$$

where $\lambda(SA > x)$ is the mean annual rate of the ground-motion pseudo-spectral acceleration, SA, exceeding the value x; $\lambda(rup_i)$ is the mean annual rate of occurrence of the earthquake rupture, rup_i; $P(SA > x|rup_i, site)$ is the probability that the ground-motion intensity, $SA > x$, due to rupture, rup_i, at the site of interest; and n_{rup} is the number of (independent) ruptures that pose a hazard to the site in the region of interest.

For a ground-motion intensity level, x, seismic hazard disaggregation [3, Chapter 7] indicates that a particular rupture i contributes $P_{rup_i|SA>x}$ to the total hazard. As a result:

$$\lambda(SA > x)P_{rup_i|SA>x} = P(SA > x|rup_i, site)\lambda(rup_i) \tag{3}$$

Noting that $P(SA > x|rup_i, site) = 1 - \Phi[\varepsilon]$ is obtained from a ground-motion model (where $\Phi[\varepsilon]$ is the normal distribution cumulative distribution function), which is universally assumed to be lognormally distributed [3, Chapter 4], for which:

$$\varepsilon = \frac{\ln(x) - \mu_{\ln SA|rup_i}}{\sigma_{\ln SA|rup_i}} \tag{4}$$

By rearranging Equation 3:

$$\varepsilon = \Phi^{-1}\left[1 - \frac{\lambda(SA > x)P_{rup_i|SA>x}}{\lambda(rup_i)}\right] \tag{5}$$

Which, due to the anti-symmetric property of $\Phi(\dot{})$, is equivalent to:

$$\varepsilon = -\Phi^{-1}\left[\frac{\lambda(SA > x)P_{rup_i|SA>x}}{\lambda(rup_i)}\right] \tag{6}$$

With the short-hand notation in the body of the article:

$$\varepsilon = -\Phi^{-1}\left[\frac{\lambda_{SA}P_{rup}}{\lambda_{rup}}\right] \tag{7}$$

which is Equation 1.

Properties of the Poisson Distribution

The probability mass function of the Poisson distribution is given by:

$$P(Y = y) = \frac{\lambda^y e^y}{y!} \tag{8}$$

where λ is the rate of occurrence of the event, and $Y = y$ is the number of occurrences. Using the common seismic hazard notation of λ is the mean annual rate of ground-motion exceedance, and T is the time period under consideration - the rate in the time period considered is λT.

The probability of *one or more* occurrences of the ground-motion intensity level is computed from:

$$\begin{aligned} P(Y \geq 1) &= 1 - P(Y = 0) \\ &= 1 - \frac{(\lambda T)^0 e^{-\lambda T}}{0!} \\ &= 1 - \left[1 + \frac{(-\lambda T)}{1!} + \frac{(-\lambda T)^2}{2!} + \ldots\right] \\ &= \lambda T - \frac{(\lambda T)^2}{2} + \ldots \\ &\approx \lambda T \end{aligned} \tag{9}$$

where the first-order approximation is valid for $\lambda T \ll 1$ [3, Section 6.2.2].

The probability of only one occurrence is:

$$\begin{aligned} P(Y = 1) &= \frac{(\lambda T)^1 e^{-\lambda T}}{1!} \\ &= (\lambda T)\left[1 + \frac{(-\lambda T)}{1!} + \frac{(-\lambda T)^2}{2!} + \ldots\right] \\ &= \lambda T - (\lambda T)^2 + \ldots \\ &\approx \lambda T \end{aligned} \tag{10}$$

and the probability of two or more occurrences is:

$$\begin{aligned} P(Y \geq 2) &= P(Y \geq 1) - P(Y = 1) \\ &= \left[\lambda T - \frac{(\lambda T)^2}{2} + \ldots\right] - \left[\lambda T - (\lambda T)^2 + \ldots\right] \\ &= \frac{(\lambda T)^2}{2} + \ldots \\ &\approx \frac{(\lambda T)^2}{2} \end{aligned} \tag{11}$$