Risk-targeted framework for seismic design: Maintaining serviceability

A.M. Hulsey & K.J. Elwood
University of Auckland, Auckland.

N. Horspool & M.C. Gerstenberger
GNS Science, Lower Hutt.

ABSTRACT

Changes to how seismic loads are determined for building design are imminent, as necessitated by the ongoing National Seismic Hazard Model update. The authors recently proposed a framework to produce risk-targeted hazard spectra, replacing the existing uniform-hazard spectra for seismic design. Unique features of this framework include the full quantification of epistemic uncertainty in the seismic hazard and multiple risk targets for individual and societal risk at the building and city scales. This paper extends the framework from the life-safety objective to other performance objectives, focusing on that of maintaining building serviceability. Building design can be controlled by parallel damage-related limit states that serve as proxies for loss of serviceability. The building strength is controlled by a limit state for low structural damage, where a ductility factor of $\mu \leq 1.25$ maintains "essentially elastic" behaviour. The building stiffness is controlled by a limit state for low non-structural damage. The risk of drift-controlled component damage is assessed using single degree of freedom (SDOF) assumptions for effective drift and modal analysis assumptions for the critical story drift. The result is a pair of minimum strength and stiffness requirements for maintaining building serviceability. The local seismic hazard determines the relative influence of the serviceability requirements, with the stiffness requirement controlling in high seismic areas, such as Wellington, but not in low seismic areas, such as Auckland. As such, the stiffened buildings in regions with frequent low levels of shaking would be protected against frequent loss of serviceability.

1 INTRODUCTION: A RISK-TARGETED FRAMEWORK FOR NEW ZEALAND

Over the next few years, approaches to determining seismic design loads for building design will shift in response to the ongoing National Seismic Hazard Model (NSHM) update. Changes to the structure of the NSHM, particularly how it incorporates epistemic uncertainty in rupture source models and ground motion models, necessitate some level of change in how to translate the NSHM results into design loads. The Ministry of Business Innovation and Employment (MBIE)'s Building Performance group has tasked the Seismic Risk Working Group with considering the implications and opportunities this provides, including other features that could be updated simultaneously (MBIE, 2020). For example, the initial report noted that the current loading standards do not have mechanisms for quantifying/designing for seismic risk, explicitly addressing risk at a range of performance objectives, or the aggregation of risk in dense communities. In response, Horspool et al., (2021) adapted the internationally recognized risk-targeted hazard framework (Gkimprixis et al., 2019; Luco et al., 2007) to the New Zealand context, including the new NSHM's epistemic uncertainty and a focus on aggregated risk.

One significant contribution of the risk-targeted framework is the explicit recognition of uncertainty in building response. The current NZS1170.5 loading standards define design intensities based on an annual probability of exceedance (e.g., APoE=1/500, as marked on the grey hazard curves in Figure 1, commonly known as a 500-year return period), with no reference to the building's response to that shaking intensity. This conceptual framework places all the design attention on not exceeding the relevant limit state (LS) at the design intensity (i.e., ULS at APoE=1/500), with no explicit consideration of what would happen at higher intensities. As shown in Figure 1a, this deterministic assumption implies a deterministic response fragility (shown in black), with a 0 and 1 probability of exceeding the limit state before and after the design intensity, respectively. In contrast, the risk-targeted framework uses a probabilistic response fragility, recognizing that the intensity at which the limit state will be exceeded is uncertain, as indicated by the black line in Figure 1b. The full hazard curve (exposure) can then be combined with the fragility (vulnerability) to get the risk at each intensity (shown in red in Fig. 1b). The annual risk is the total area under this risk curve. The risk-targeted framework compares this risk to a target risk value, then shifts the fragility, and hence the associated design intensity, to the left (weaker) or right (stronger) to meet the target risk. Although the process may appear more complex, the end result is still a single response spectrum used for design, Sa(T), but now the intensity of this spectrum is reflective of the risk of exceeding a limit state over the entire hazard curve. Readers are referred to Horspool et al. (2021) for more details on the development of risk-targeted design spectra based on a target annual fatality risk.

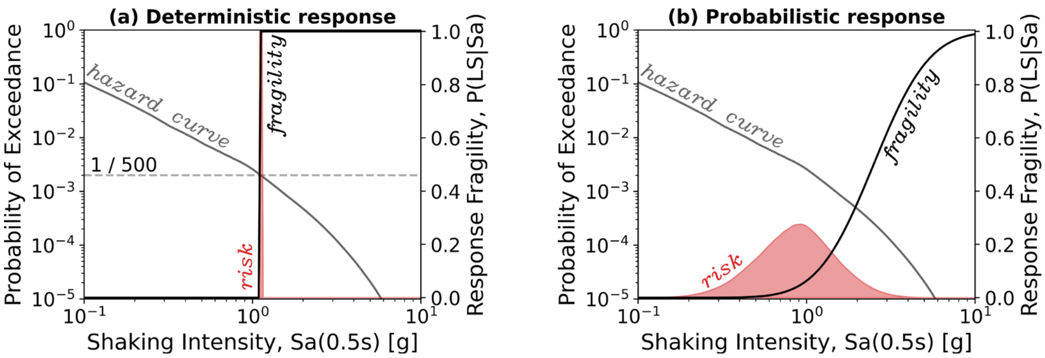

Figure 1: The contribution of shaking intensities to the total risk (red), given the hazard probability of exceedance (grey) and the building response (black), considering (a) a deterministic response fragility or (b) a probabilistic response fragility, where the fragility is the probability of exceeding a relevant limit state, LS, given the shaking intensity: P(LS|Sa).

Current design requirements for most buildings consider two limit states (Ultimate, ULS, and Serviceability, SLS). However, the Seismic Risk Working Group noted the importance of a range of performance objectives (MBIE, 2020). Horspool et al. (2021) demonstrated how ULS can be based on a life-safety risk target and suggested that such a risk-targeted framework could be extended to other performance objectives as well. Below is a possible list, intended to be revised through broader discussions among the engineering community:

- Risk of individual fatality (equivalent to ULS, see Horspool et al. (2021)).
- Risk of needing structural repair.
- Risk of long and highly uncertain building recovery times.
- Risk of losing serviceability (equivalent to SLS, discussed further below).

This paper continues the work by Horspool et al. (2021) in two ways. First, it demonstrates how the risk-targeted framework enables a better appreciation for the implication of epistemic uncertainty in hazard and introduces how aggregated risk in a large building or urban region can be explicitly accounted for in determining the risk-targeted design spectrum. Second, it extends the framework from life-safety to serviceability-related performance objectives. To do so, the risk-targeted framework is reformulated to consider drift-based hazard intensities, in addition to the acceleration hazard curves typically used in risk-targeted hazard frameworks. This reformulation also lays the groundwork for extensions to other performance objectives, such as limiting the need for structural repair or addressing building recovery time frames.

2 FRAMEWORK FOR LIFE-SAFETY DESIGN REQUIREMENTS

The risk-targeted framework for New Zealand's life-safety target is, in many ways, a direct extension of the collapse-prevention framework originally developed in the United States for the ASCE7 design hazard maps (Luco et al., 2007). The local seismic hazard is characterized by a hazard curve for the annual probability of exceedance of acceleration-based intensity measures, while the building performance is modelled as a logarithmic fragility function for the collapse limit state. Together, the hazard and performance produce the annual risk of collapse. The adaptation for the New Zealand context follows other studies in reframing the risk as the individual risk of fatality, conditioned on the risk of building collapse (Silva et al., 2016). The following sections briefly describe New Zealand-specific nuances in the parameter selection and two significant adaptations for considering epistemic uncertainty and aggregated risk.

2.1 Selecting parameters for the New Zealand context

The basic risk-targeted framework requires three parameters: a risk target for the annual probability of exceeding acceptable performance as defined by a limit state (LS), the logarithmic standard deviation of the limit state fragility curve, β, and an anchor point linking the design intensity (IM_D) to a probability of exceeding the limit state, $P(LS|IM_D)$.

The commentary to New Zealand's design standards cites 10^{-6} as the target annual individual fatality risk (AIFR) to building occupants (NZS1170.5, 2016). Unlike the ASCE7 target, which focuses on the probability of collapse, a fatality risk target requires an additional parameter linking the probability of fatality to the collapse limit state, $P(F|C)$. This value is taken as 10%, per the NZS1170.5 commentary and other precedents (Silva et al., 2016). The uncertainty in the collapse limit state, β, is assumed to be 0.6, as used in the US framework. With the risk target and β known, the fragility function is analytically shifted with respect to the hazard curve to obtain an optimized fragility that achieves the risk target ($AIFR=10^{-6}$). The final parameter is the anchor point, which is used to extract a design intensity from the optimized fragility function. In the US, this anchor point is taken as $P(C|MCE_R)=10\%$, with a 2/3 factor applied to determine the design intensity ($IM_D = 2/3\ MCE_R$, where MCE_R is ASCE 7's Risk-targeted Maximum Considered Earthquake). The proposed New Zealand framework calibrates the anchor point to minimize the average change from the current concept of APoE=1/500 design intensities, resulting in $P(C|IM_D)=10^{-4}$ for the selected β value. Conditioning the anchor point on a selected β value within the range $0.4 \leq \beta \leq 0.8$ reduces the sensitivity of the resulting IM_D values to the assumed β. This result highlights that the primary benefit of introducing a probabilistic response fragility (i.e., the ability to consider the entire hazard curve) is not contingent on precise knowledge of the response fragility. Together, these three parameters ($AIFR=10^{-6}$, $\beta=0.6$, and $P(C|IM_D)=10^{-4}$) produce a design intensity, IM_D, which could be incorporated into NZS1170.5 via location-specific risk-coefficients for the current Z-based design spectra or multi-period design spectra, depending on the broader recommendations of the Seismic Risk Working Group.

2.2 Accounting for epistemic uncertainty

The in-progress update to the National Seismic Hazard Model (NSHM) is a primary driver for a new framework for translating the hazard to the seismic design loads. One of the most significant new features is the addition of epistemic uncertainty, using multiple rupture source models and ground motion models to best estimate the seismic hazard. Therefore, the NSHM is actually a weighted logic tree representing an ensemble of possible model combinations (Marzocchi et al., 2015). As the NHSM update is not yet complete, the results below are representative, based on the 2010 NSHM's single source model and a modern logic tree for the ground motions. Each logic tree branch, *i*, is represented by a unique hazard curve, leading to a unique design intensity, IM_{D_i} (Fig. 2a, where IM_{D_i} represents the risk-targeted ULS intensity, ULS_R). The result is a weighted distribution of possible design intensities for each location (Fig. 2b), rather than a unique value as in the current NSHM and the resulting Z-factors for location-specific design spectra. Regardless of the method of translating from a hazard curve to a design intensity (the current aPoE or the proposed risk-targeted framework), it would be challenging to rationalise the selection of any specific design intensity within this distribution of possible outcomes without some form of risk quantification. A benefit of this framework is that the epistemic uncertainty of the risk associated with any potential design intensity, IM_{D_i}, can be explicitly assessed by re-calculating the risk for all the possible hazard curves in the logic tree. This provides a quantification of the likelihood that the actual risk for a selected design intensity (IM_{D_i}) is in fact larger than the risk target, thus supporting broader discussions on acceptable risk levels among the committee debating where to set the seismic design intensity, and the public in general. For example, Figure 2c shows the likelihood that the actual risk associated with a building design will be greater than the risk target, AIFR, given the selected design intensity. There is a 35% and 20% chance that a design intensity selected as the median of the ULS_R distribution for Wellington and Dunedin, respectively (purple and orange lines, at 0.5 on the x-axis), will result in over 1.5 x 10^{-6} (1.5 x AIFR). The grey dashed line shows the equivalent result for 1 x AIFR. The risk-targeted framework ensures that this 1:1 inverse relationship is true for all locations. A similar plot for the distribution of APoE intensities would not follow this trend. This uncertainty analysis would not be required by design engineers but rather could inform the ongoing discussion of how to formulate seismic design loads from the new NSHM.

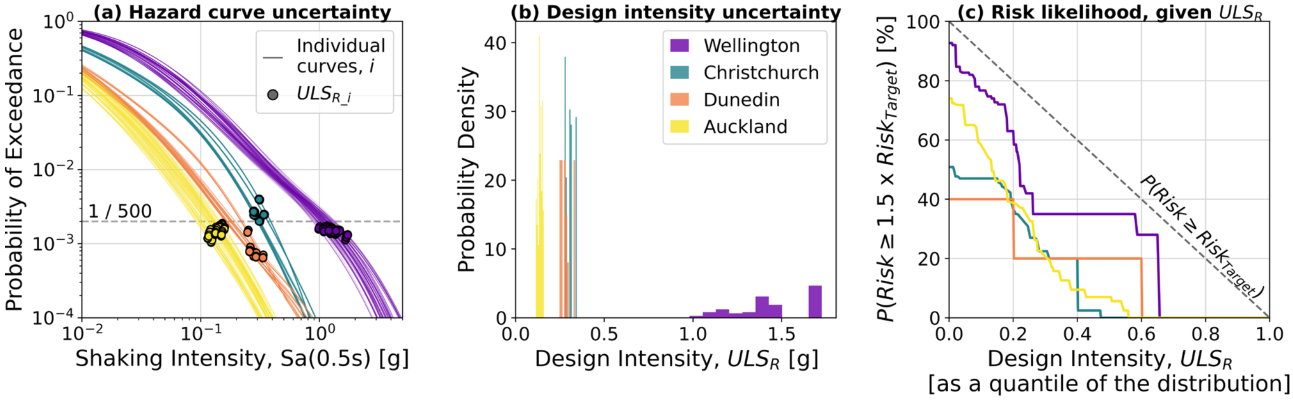

Figure 2: Epistemic uncertainty in design intensities. (a) A set of unique hazard curves for each location, representing all the model combinations in the logic tree. The dots show the risk-targeted ULS design intensity, ULS_R, for each curve. The dashed line shows the 1/500 annual probability of exceedance, for reference. (b) Histogram of design intensities for four locations, based on uncertainty in the underlying hazard curves. (c) Probability that the selected design intensity (shown on the x-axis as a quantile of the distributions for each location in b) will result in more than 1.5 times the annual individual fatality risk target of 10^{-6} (AIFR). The dashed grey line is the equivalent for 1 x AIFR.

2.3 Considering aggregated risk in high-density communities

Design standards have long recognised the need to account for aggregated risk in buildings with large numbers of occupants (i.e., Importance Level 3), and there has been a growing sense that the standards should also need to account for aggregated risk in densely populated urban regions (MBIE, 2020). The proposed risk-targeted framework allows for a rational means of selecting such factors by estimating the aggregated fatality risk via F-N curves, which represent the annual probability (or frequency, F) of exceeding N fatalities within a community. In this context, the "community" is defined as either the occupants of an individual building or the population in a dense central business district (CBD). The individual fatality risks are aggregated into an F-N curve for this community, using an algorithm that is depicted visually in Figure 3a. If the baseline design intensity, IM_D, produces an F-N curve that exceeds the F-N limit, the design intensity is increased by a "societal factor," thereby shifting the F-N curve to satisfy the limit. The societal factor (solid line in Fig. 3b) is taken as the greater of the occupancy factor (circles) and population factor (horizontal line), which define the community as either a single high-occupancy building or the surrounding high-density central business district (CBD), respectively. The F-N limit for an individual building is calibrated to produce an average occupancy factor of 1.3 for 5000 occupants, in keeping with NZS1170.5's current factor for Importance Level 3 (IL3, buildings with over 5000 occupants, shown in Fig. 3b and c). Meanwhile, the population factor is constant, depending only on the CBD's population, with Figure 3c based on approximate population values and F-N limits to demonstrate the framework. Once again, this analysis would not become part of the design process, but rather could inform tabulated factors for each location and number of building occupants. The high population factor for Auckland as compared to other locations (Fig. 3c) is consistent with current intuition that NZS1170.5's Z-floor for minimum seismic hazard informally accounts for aggregated risk due to Auckland's population density. Analysis of the floor's implied population factor for Auckland is 1.8 (calculated as the ratio of Z for Auckland from NZS1170.5 over ½ Sa(0.5) for APoE=1/500 from the 2010 NSHM), equivalent to the value calculated through the more formal risk aggregation depicted in Figure 3c.

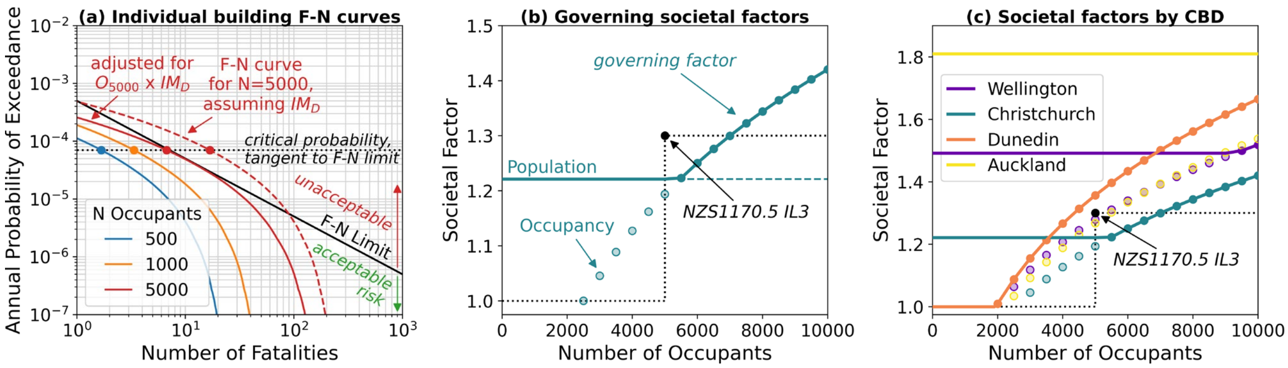

Figure 3: Societal factors based on aggregated risk: (a) Individual building F-N curves based on the number of occupants (blue, orange, and red). The dashed red curve for 5000 occupants is based on the baseline design intensity, IMD, while the solid red curve is adjusted by an occupancy factor to satisfy the black F-N limit. (b) The governing societal factor (solid line) is the greater of the occupancy factor (circles) and the population factor (horizontal line). The black dot shows the step change at the current NZS1170.5 factor for Importance Level 3 buildings (IL3, with over 5000 occupants). (c) The governing factors for each of four central business districts (CBD), with the minimum value depending on the CBD population.

3 FRAMEWORK FOR MAINTAINING SERVICEABILITY DESIGN REQUIREMENTS

The aftermath of the Christchurch and Kaikōura Earthquakes has highlighted a potential discrepancy between societal expectations for buildings and current building performance (Hare, 2019). The following discussion focuses on maintaining serviceability after an earthquake, though similar ideas could be used for other relevant performance objectives. The proposed framework focuses on building damage as a typical reason for loss of serviceability. A limit state for low structural damage (low enough to not require post-earthquake inspection) controls the building strength requirement, while a non-structural limit state addresses building stiffness. The previous discussions of epistemic uncertainty and aggregated risk (in the sense of limiting the total number of buildings that lose serviceability) are also applicable here.

3.1 Determining the building strength requirement

The procedure and necessary parameters for determining the building strength are identical to the life-safety framework described above, except the risk target is related to serviceability, such as a 50% chance of loss of serviceability over a 50-year design life. The “essentially elastic” limit state, quantified as a ductility factor of $\mu = 1.25$, is used as a proxy for loss of serviceability. There is no precedent in risk-targeted analyses for this limit state’s fragility function uncertainty, β. Therefore, β will be selected through nonlinear single degree of freedom (SDOF) analysis, considering record-to-record variability and modelling uncertainty. The value will likely be $0.3 \leq \beta \leq 0.4$. The final parameter for linking the optimized fragility function to a design intensity, $P(LS|IM_D)$, will be assessed through sensitivity analysis, starting with a value of $P(LS|IM_D) = 0.25$, as shown in Figure 4a. This starting point was selected because it is not sensitive to the assumed beta value and results in a design intensity that is typically close to the 1/50 annual probability of exceedance (Fig. 4b), consistent with a recent recommendation to increase the Serviceability Limit State (SLS1) return period from 25 to 50 years to attain better building performance (Pettinga et al., 2019). As with the ULS_R design intensities above, this analysis results in a distribution of possible design intensities (Fig. 4c) that could be further assessed for the epistemic uncertainty of the actual risk of exceeding the target risk, given the selected SLS_{R_i}. Once again, the analysis could all be in the background, informing tabulated design spectra.

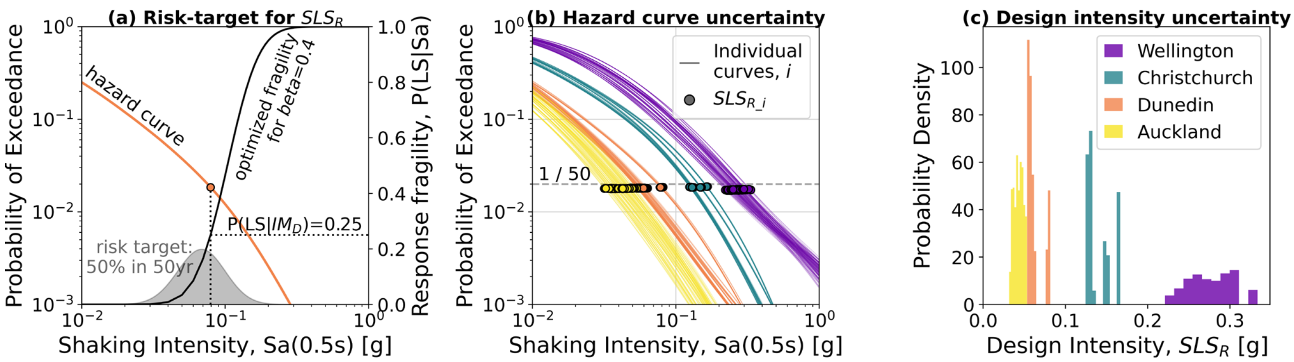

Figure 4: The risk-targeted framework for the serviceability strength requirement. (a) visual description of the three parameters: the risk target of 50% chance in 50 years, fragility uncertainty of β=0.4, and an anchor point of $P(LS|IM_D)=0.25$. The orange line is the hazard curve, the black line is the fragility, shifted left or right (weaker or stronger) to produce an area under the grey risk curve equal to the target risk, and the orange marker is the resulting SLS_R design intensity. (b) The hazard curves for four locations, considering epistemic uncertainty. The dots show the resulting SLS_R design intensities, in comparison with the dashed line for 1/50 annual probability of exceedance. (c) A histogram of the distribution of SLS_R design intensities for each location.

3.2 Determining the building stiffness requirement

Most non-structural damage is due to building displacements, an aspect that is better controlled by building stiffness than by building strength. The proposed framework reconfigures the risk-targeted methodology to consider damage due to drift, while remaining similar to collapse due to acceleration in the life-safety context. Typically, component damage is assessed through performance-based engineering methods such as

FEMA P-58 (FEMA, 2012), with a building response model that converts acceleration-based hazard intensities into engineering demand parameters (EDP, e.g., story drift ratios), then using a building performance model that simulates damage, using component fragilities representing the probability of component damage, P(DS|EDP), where DS is a component damage state. While this type of detailed risk analysis can be used in design, it is resource intensive, such that most projects employ a more prescriptive, code-based approach. The simplified analysis of the proposed framework allows the risk component damage to inform the loading standards, without requiring a full FEMA P-58 analysis for every project.

The key step in reconfiguring the risk-targeted framework is to quantify the hazard intensity as displacements, rather than accelerations. The hazard curve is converted from spectral acceleration, $Sa(T)$, to spectral displacements, using the fundamental relationship, $Sd(T) = 4\pi^2/T^2 \cdot Sa(T)$, where T is the building period. This is shown in Figure 6a for 5-story reinforced concrete buildings with a frame or shear wall system (solid and dash-dot lines, respectively). These displacements can be further expressed as a drift at the effective height by taking the ratio with respect to the effective height (H_e) of an equivalent SDOF: $Sd(T)/H_e$. The next step is to find the story drift ratio at the critical story, where component damage is likely to first occur. The critical story will have a higher drift than the effective drift of the building and its location along the height of the building will depend on the structural system. At low levels of damage, the critical story will depend primarily on linear modal analysis. The proposed framework uses a simplified MDOF model to approximate scale factors for the story drift ratio (SDR) at the critical story. The model uses a single parameter, α, for the structural system's contribution of shear versus flexure deformations (Miranda & Taghavi, 2005) to provide a displacement profile, based on the modal shapes for α (Table 1 and Fig. 5a), which can then be converted into story drift ratios (SDR) over the height of the building. The critical story factor is taken as the maximum SDR for any story (an upper story for walls or a lower story for frames), normalized by the SDOF drift, as depicted in Figure 5b. The SDOF drift hazard curve is then scaled by the relevant factor to obtain a hazard curve for the SDR at the critical story where damage is most likely to first occur (Fig. 6b).

Table 1: Structural system assumptions

System	**alpha, α**	**Critical story factor, C_{cr}**
Frame	20	1.4
Wall	0	1.55

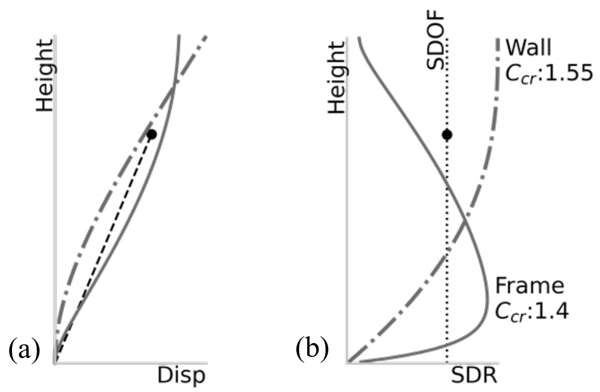

Figure 5: Engineering demand parameter profiles over the height of a building, considering a frame (solid) or wall (dash-dot) structural system. The black dot shows an equivalent SDOF system. (a) displacements, (b) story drift ratios (SDR).

Once the hazard curve is defined in terms of the critical SDR, it can be used to estimate the risk of drift-based damage to non-structural components, such as wall partitions. FEMA P-58 provides component fragilities for a variety of non-structural components, many of which are conditioned on SDR as the engineering demand parameter (EDP). The hazard and the fragility are combined for the annual risk, shown in Figure 6b, as 50% and 32% in 50 years for the frame and wall systems, respectively. Unlike the building fragilities for collapse or exceeding the essentially elastic response, these fragilities cannot be shifted for improved performance (though many components could be substituted for a less damageable version). As such, the optimization for achieving the risk target is based on shifting the hazard curve, rather than the fragility. This can be done by stiffening the building, reducing the period and thereby reducing the hazard. This is depicted in Figure 6c, where the risk decreases with the period. The vertical lines show the assumed periods for the 5-story frame and wall systems (0.75 and 0.5 seconds, solid and dash-dot, respectively). This analysis could be reflected in the design standard as a minimum stiffness for a 5-story building, tabulated either as a maximum period or a maximum drift. For example, if the risk target were 50% chance of wall partition damage in 50 years, the maximum period for Wellington would be 0.75 seconds. The figure shows the benefit of shear walls as compared to less stiff frame systems, with recognition that the engineer can adjust the design to achieve better performance within a given system. The figure also demonstrates the relative influence of the serviceability design requirements in various locations. Wellington's high seismicity produces relatively higher shaking intensities at high probabilities of exceedance (e.g., at the 1/50 probability of exceedance, see Fig. 4b), which increases the risk of losing serviceability over the lifetime of the building. For Dunedin and Auckland, however, this risk is much lower (orange and yellow lines are less than 10% for all periods in Fig. 6c) such that the stiffness requirement for serviceability would not control the design.

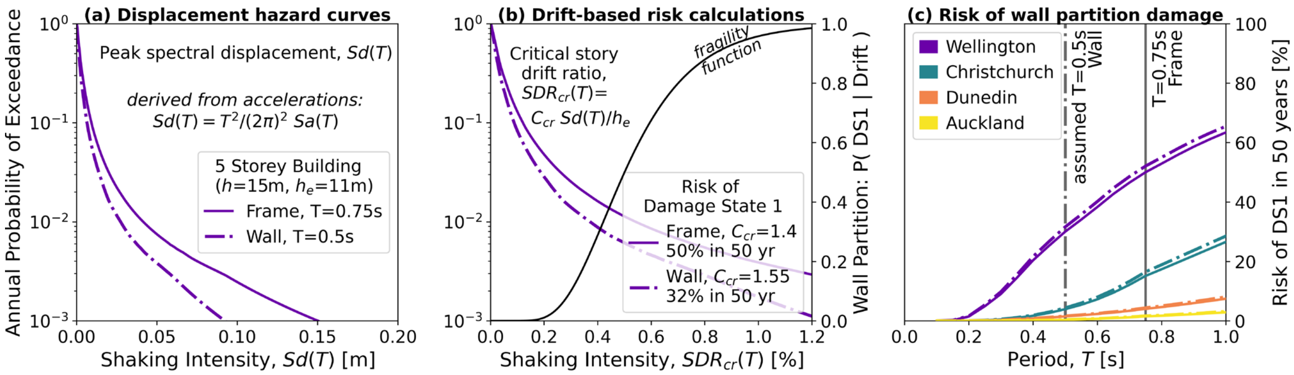

Figure 6: Risk-targeted framework for drift-controlled damage. (a) The acceleration-based hazard curve is translated into displacements, shown here for a 5-story building with a frame (solid line) or wall (dash-dot) structural system. (b) An effective height and critical story factor are applied to get a hazard curve for the critical story drift ratio. The black fragility function shows the probability of damage for a wall partition, given the story drift. The legend quantifies the annual risk of damage for each system. (c) The risk for each location, shown as a function of period. The vertical lines show the assumed period for the frame and wall systems, recognizing that the engineer can adjust the period within a given system to achieve better performance.

4 CONCLUSIONS

The proposed framework extends the traditional risk-targeted methodology to incorporate additional features that are relevant to New Zealand, such as accounting for epistemic uncertainty in the hazard model and considering aggregated risk in high density communities. Furthermore, the framework is applied to serviceability design to address the growing sense that New Zealand's current design requirements are not aligned with societal expectations for building performance. Limit states for structural and non-structural damage lead to building strength and stiffness requirements, respectively. The influence of these requirements on the final design depends on the local seismic hazard, with serviceability stiffness requirements controlling for high seismic areas like Wellington but not for Auckland.

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