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On the utility of story loss functions for regional seismic vulnerability modeling and risk assessment

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Abstract

Vulnerability functions relate loss to seismic intensity and can be developed via several approaches. They are a fundamental part of seismic risk assessment on a regional level and support decision-making and intervention strategies aimed at reducing risk. This article discusses a prominent analytical approach to developing seismic vulnerability models for buildings based on equivalent single-degree-of-freedom (SDOF) modeling, fragility function, and damage-to-loss model integration. The fundamental assumptions are scrutinized, and their principal drawbacks are highlighted. An alternative approach also based on an equivalent SDOF modeling approach is discussed but instead capitalizes on story loss functions (SLFs) as a means to more accurately compute economic losses and their sources. The main benefit is that the contribution of floor acceleration-based losses can be directly considered, and the disaggregation of losses is fully represented. A case study comparison is presented to highlight the similarities and key benefits. It is seen that the SLF-based approach can provide a much more comprehensive means to compute and communicate loss contributions among different element groups (i.e., structural, non-structural, and contents) and individual stories along the building height. Existing models can simply be adjusted to this approach and provide a more holistic view of risk. The benefits and potential applications in the (re)insurance sector are also discussed.

Keywords

Seismic risk, vulnerability, loss, story loss function, fragility

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Introduction

Seismic risk assessment is vital in understanding and mitigating the potential impact of earthquakes on communities. By quantifying the likely consequences of seismic events, it is possible to identify the areas most susceptible, and thus where risk mitigation actions should be prioritized. Vulnerability functions play a crucial role in seismic risk assessment as they quantitatively measure how buildings and structures are likely to perform during an earthquake. These functions describe how the loss due to expected damage may vary based on the shaking intensity. By incorporating vulnerability functions into risk assessment models, the potential impact of earthquakes on buildings, infrastructure, and people can be better understood. This allows retrofitting efforts and available resources to be prioritized, facilitating the development of appropriate emergency plans, and making informed decisions on building codes and regulations to enhance resilience in earthquake-prone areas. Several methodologies to assess seismic vulnerability have been developed in past decades, with Calvi et al. (2006) providing a comprehensive overview of some key developments. Of note was the desire to utilize analytical methodologies to quantify the seismic fragility of structures. Methods such as the displacement-based assessment methodology described in the work of Calvi (1999) and later in other works by Crowley et al. (2004) and Borzi et al. (2008) are notable examples. These allow analysts to model the expected mechanical behavior of a building numerically and subsequently estimate its seismic vulnerability.

Further work was carried out during the late 2000s/early 2010s, and D'Ayala et al. (2015) formalized several approaches into a comprehensive set of guidelines. One notable method outlined in these guidelines was using equivalent single-degree-of-freedom (SDOF) systems to represent entire building structures or classes of buildings when working toward a larger and more regional-type seismic assessment. It has the advantage of excellent computational efficiency with a reasonable trade-off in accuracy. Further considerations surrounding this and other approaches are provided by Silva et al. (2019), highlighting potential areas of improvement along with the increasing and diverse needs of vulnerability modeling beyond simple loss estimates.

This article discusses the equivalent SDOF-based methodology and notes that while it largely delivers on its objective of providing reliable vulnerability functions for general use, this comes at the expense of several simplifying assumptions. The article starts with a quick overview of the equivalent SDOF modeling assumptions, followed by the widely used approach of fragility functions and damage-to-loss ratios to develop vulnerability functions. It then introduces an alternative methodology based on story loss functions (SLFs), which is advocated here. A simple case study example is presented to underline the similarities of the two methodologies, emphasizing the SLF-based approach's primary advantages. The article explores the broader benefits and impacts of using this approach, discussing its relevance in the context of regional seismic risk modeling.

Vulnerability assessment

Equivalent SDOF modeling

With a representative numerical model of the complete structure available, its seismic response with increasing intensity can be characterized. Several analysis methods exist: linear to non-linear, static to dynamic, and SDOF to multi-degree-of-freedom (MDOF) representations. For example, Silva et al. (2014b) investigated several non-linear static and dynamic analysis methods for deriving fragility functions. Likewise, empirical tools like SPO2IDA (Baltzopoulos et al., 2017; Nafeh et al., 2020; Vamvatsikos and Cornell, 2005) or similar derivatives utilizing more advanced intensity measures (IMs) (Nafeh and O'Reilly, 2022; Shahnazaryan and O'Reilly, 2024) have been developed. These tools may be considered advanced $R-\mu-T$ relationships, akin to those originally developed by Veletsos and Newmark (1960). Of course, a full non-linear dynamic analysis may be performed on MDOF models (e.g., Martins et al., 2015), but this is often too computationally demanding when applied at a large scale and difficult to implement considering the amount of structural information required to construct a detailed numerical model (e.g., O'Reilly et al., 2018).

An efficient alternative adopted in the past has been to use an equivalent SDOF oscillator to represent the non-linear structural behavior and to analyze this. This approach is used in many methods of seismic design (e.g., ASCE 7-22, 2022; CEN, 2004) and assessment (e.g., Fajfar, 2000; Priestley, 1997). It is based on the idea that the seismic response of an MDOF system can be approximated via its first mode response, as illustrated in Figure 1. This approximation holds well for low- to mid-rise buildings that are reasonably regular in plan and height.

The starting point for such an approach is often a bi-linear representation of the structure's expected lateral response, usually obtained from a static pushover analysis, but more detailed backbones may also be utilized (e.g., Nafeh et al., 2020). This is transformed to an equivalent SDOF oscillator by dividing the force and displacement values by the first mode participation factor, Γ, described in Equation 1, where m_i is the mass and $\phi_{1,i}$ is the normalized first mode shape value at floor level i. This backbone is denoted with the terms $F^* = F/\Gamma$ and $\Delta^* = \Delta/\Gamma$, where * denotes the equivalent SDOF system. The equivalent mass, m^*, is determined via Equation 2, and the period, T^*, is given by Equation 3. The ductility demand on the equivalent SDOF system is $\mu = \Delta^*/\Delta_y^*$, where Δ_y^* is the yield displacement shown in Figure 1, also denoted Sd_y. Likewise, the spectral acceleration at yield, also known as the base shear coefficient, Sa_y, may be determined by $Sa_y = F_y^*/m^* g$, where g is the acceleration due to gravity:

$$\Gamma = \frac{\sum_i m_i \phi_{1,i}}{\sum_i m_i \phi_{1,i}^2} \tag{1}$$

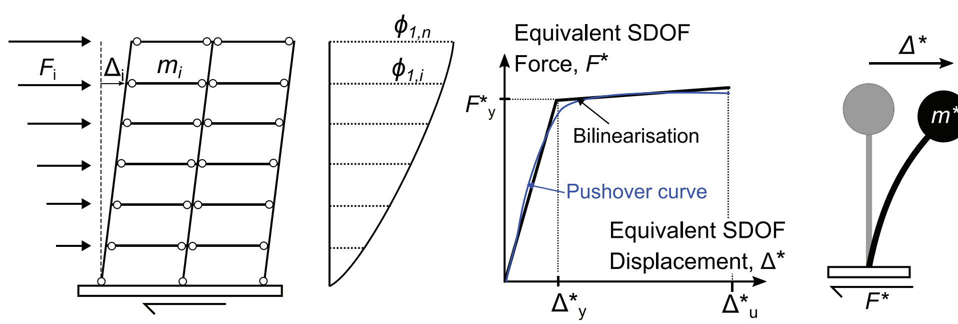

Figure 1. Basic steps in equivalent SDOF modeling. (a) MDOF properties. (b) First mode shape. (c) Equivalent SDOF response. (d) SDOF transformation.

$$m^* = \sum_i m_i \phi_{1,i} \quad (2)$$

$$T^* = 2\pi \sqrt{\frac{m^* \Delta_y^*}{F_y^*}} \quad (3)$$

It may be observed that with a simple estimate of the equivalent SDOF system's period, T^*, spectral acceleration at yield, Sa_y, and ultimate ductility capacity, μ_u, the response of a structure can be easily characterized via this equivalent SDOF system. This has been the basis for many past studies, where the first mode period of structures is established empirically (e.g., Crowley and Pinho, 2004; Ruggieri et al., 2022b). Other studies have described how to establish lateral strength and ductility capacity (e.g., Crowley et al., 2021) for different regions around the world.

Vulnerability functions via fragility functions and damage-to-loss models

Fragility functions. Once the structural behavior can be condensed to an equivalent SDOF system, it is analyzed dynamically using ground motion records. The main assumption is that the entire building response can be accurately represented by a single displacement-based demand, capitalizing on the first mode-dominant behavior that holds well for low- to mid-rise buildings. For example, Martins and Silva (2021) utilized this approach for several types of structures worldwide to develop a comprehensive fragility model, as did Villar-Vega et al. (2017) for South America. While it was presented in a different context, the approach by Fajfar and Dolšek (2012), later improved by Nafeh et al. (2020), is similar when assessing infilled reinforced concrete (RC) frames as they allow fragility functions to be easily obtained. This works well in the case of displacement-based demands such as roof displacement or story drifts, which tend to be first mode-dominant. However, when strength-based quantities are required (e.g., peak floor acceleration (PFA)), this approach runs into difficulty since these structural demands can not be captured with the first mode of response alone and contain significant higher mode contributions. This is a major drawback of these methods that has persisted for many years, albeit without much major consequence.

With the response characterized with increasing seismic intensity, it is typical to identify several key damage states (DSs) and then identify their fragility functions. These may be fitted using various statistical approaches, but the end product is a set of median and dispersion values for the presumed lognormal distribution of the assumed DS definition. For example, in the work of Martins and Silva (2021), several displacement-based DS definitions were adopted based on the equivalent SDOF's backbone characteristics shown in Figure 1. These were $DS1 = 0.75\Delta_y^*$, $DS2 = 0.50\Delta_y^* + 0.33\Delta_u^*$, $DS3 = 0.25\Delta_y^* + 0.67\Delta_u^*$, and $DS4 = \Delta_u^*$. These were based on the recommendations of Villar-Vega et al. (2017), who extended previous definitions by Lagomarsino and Giovinazzi (2006), which were based on pushover curve considerations and comparisons to macro-seismic empirical data using engineering judgment.

Damage-to-loss models. Once the median and dispersion pairs for a discrete set of fragility functions are obtained, with some possible modification to account for additional uncertainties, it is typical to associate each DS with a loss ratio via what is termed a *damage-to-loss model*, or *consequence model*. It assumes that the structure has been damaged to a

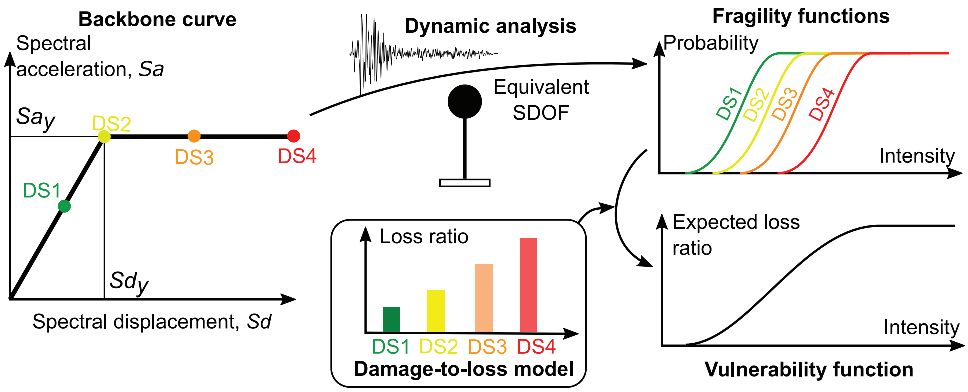

Figure 2. Development of vulnerability functions via fragility functions and damage-to-loss models.

fraction of its overall replacement value at each DS, and is denoted $E[L|DS_i]$ for DS DS_i. When combined with the fragility functions, the expected loss ratio versus intensity is obtained to give the vulnerability function via Equation 4, which is illustrated in Figure 2:

$$E[L_T|IM] = \sum_i E[L|DS_i]P[DS_i|IM] \tag{4}$$

Several past studies have looked to develop such damage-to-loss models using either analytical methods or post-earthquake observations. For example, Kappos et al. (2006) developed a model for Greece at five distinct DSs, whereas HAZUS (2003) reported a model for the United States, and Bal et al. (2008) developed a model for Turkey. These studies were primarily based on analytical observations with fixed ratios assigned to each DS, although some studies (e.g., Martins et al., 2015) did report the variability in these loss ratios. An earlier study by Di Pasquale and Goretti (2001) utilized empirical data in Italy to develop a damage-to-loss model. A similar approach was followed by Cosenza et al. (2018) using data from the 2009 L'Aquila earthquake as part of the well-known *Sismabonus* seismic risk classification guidelines in Italy (Decreto Ministeriale, 2017).

A general procedure to develop vulnerability functions is described by D'Ayala et al. (2015), and an online database of available models for different countries is maintained by the Global Earthquake Model (GEM) Foundation (Yepes-Estrada et al., 2016). A considerable degree of variability exists between each model, primarily due to each study using a slightly different set of criteria to identify the DS exceedance. For example, a study by Silva et al. (2014a) developed vulnerability functions for Portuguese RC moment frame structures with MDOF planar models using several DS criteria and associated loss ratios available in the literature at the time. Various approaches for defining the DSs were outlined, and some manual approaches to manually adjust these to account for issues like the presence of masonry infills were noted, underlining the subjectivity of such fragility models. A further study by Martins et al. (2015) again examined the damage-to-loss models used in developing vulnerability functions for the Portuguese context. The DSs were based on local damage criteria rather than global criteria adopted in other studies. For example, DS2 was described as when 10% or more of the beams or columns yielded. However, instead of directly assigning a loss ratio to each DS, the expected loss ratios at each DS were computed using the repair cost values based on the actions required to recover the building. This approach by Martins et al. (2015) was termed the "direct" approach in the guidelines published by D'Ayala et al. (2015), whereas the earlier approach used by Silva et al. (2014a), for example, was termed the "indirect" approach. A later study by Martins and Silva (2021) looked to provide a more harmonized approach for the most common building classes found worldwide. It followed the equivalent SDOF-based approach of developing fragility functions previously described in tandem with damage-to-loss models available as part of the online database described in Yepes-Estrada et al. (2016) to derive vulnerability functions.

Examining the relative accuracy of vulnerability functions computed using these different damage-to-loss models is not the focus of this study, but rather the various approaches that can be adopted to arrive at the same output. However, past works have indicated that they can notably impact the resulting vulnerability functions. While this subjectivity and sensitivity can appear to be a negative facet of this approach to deriving vulnerability functions, it is important to note that these have been widely used with success on a regional scale in different contexts around the world (e.g., Silva et al., 2020; Silva and Horspool, 2019).

Vulnerability functions via SLFs

The previous section highlighted a simple and effective means to quantify fragility and vulnerability models for seismic risk assessment. It was seen how discrete DS definitions were not always harmonized among different studies worldwide. In addition, they were almost always displacement-based and ignored the contributions from acceleration-sensitive non-structural elements and contents. Therefore, when examining the vulnerability function of a structure, it is not possible to identify from which source (i.e., displacement-based or acceleration-based demands and structural or non-structural elements) the expected economic losses are being generated. This presents a significant disadvantage when identifying effective retrofitting strategies that can reduce the overall risk, as will be discussed further in later sections.

An alternative procedure to overcome some of these issues when deriving vulnerability functions is presented here. It is based on an extension of similar concepts presented previously, whereby the structure's seismic response is assumed to be represented by an equivalent SDOF system. The key difference is that instead of assuming that discrete DSs can be used to develop fragility functions, which are then related to a loss ratio, and subsequently developed into a vulnerability function via Equation 4, some refinements are introduced. The benefits of these refinements will become evident in subsequent discussions. The general procedure to follow is illustrated in Figure 3 and essentially comprises two distinct parts: computing the expected peak story drift (PSD) and PFA demands at a given intensity, followed by their respective losses at each story level of the building and damageable component group, shown next in Figure 4. This distinction allows for a much more holistic view of the source of losses and can aid risk-reduction strategies without much additional computational effort compared with the previously described approach. It is important to note that although an equivalent SDOF system-based approach is used, which inevitably comprises its own limitations, the expected PSD and PFA demands at a given intensity can equally be computed via a more refined approach, should the analyst deem it necessary. Hence, this article presents one approach, but alternatives are available.

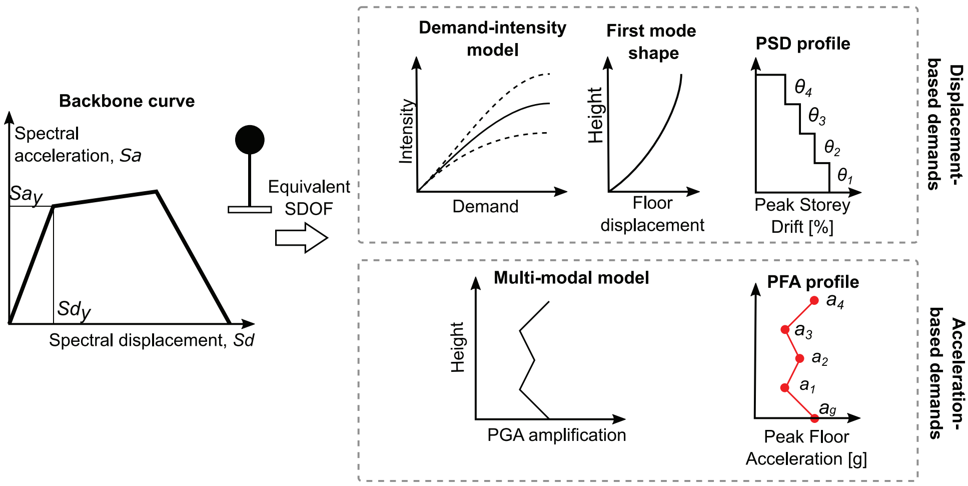

Figure 3. Story-based approach to estimating demands.

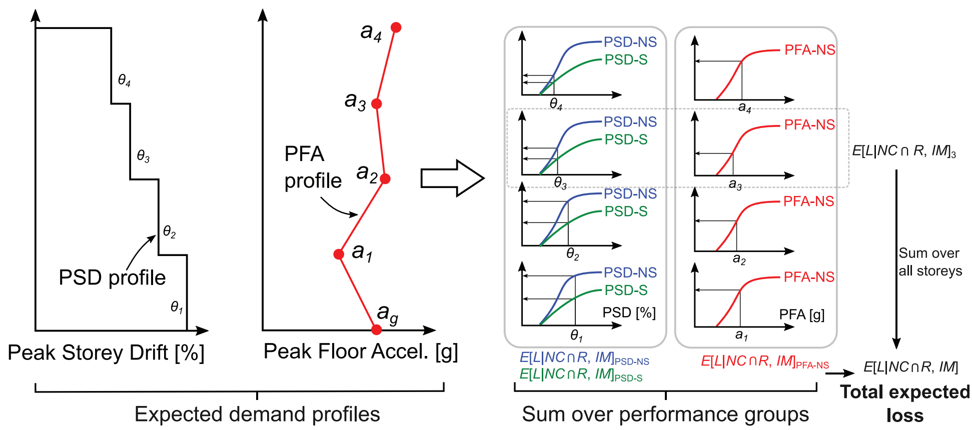

Figure 4. Estimating expected repair costs using SLFs.

PSD demands. As per Figure 3, the first step is to estimate the PSD demand profile for a given intensity. For a SDOF system with period, T^*, the dynamic strength ratio (i.e., the median intensity required to exceed that level of μ) can be identified from $R-\mu-T$ relationships (Miranda, 2000; Newmark and Hall, 1982; Vamvatsikos and Cornell, 2005; Veletsos and Newmark, 1960; Vidic et al., 1994). A key assumption of these $R-\mu-T$ relationships is that the IM is the spectral acceleration, $Sa(T^*)$, since R is by definition:

$$R = \frac{Sa(T^*)}{Sa_y} = \frac{F^*}{F_y^*} \tag{5}$$

Research over the past decade or so (e.g., Eads et al., 2015; Kazantzi and Vamvatsikos, 2015; Kohrangi et al., 2016; O'Reilly, 2021) has advocated for a shift in paradigm in seismic fragility analysis, arguing that $Sa(T^*)$ possesses several issues related to IM efficiency and bias. Recent work (Nafeh and O'Reilly, 2022; Shahnazaryan and O'Reilly, 2024) has instead opted to use average spectral acceleration, $Sa_{avg}(T^*)$, defined as follows:

$$Sa_{avg}(T^*) = \left(\prod_{i=1}^{N} Sa(c_i T^*) \right)^{1/N} \tag{6}$$

where $N = 10$ and c_i is a linearly spaced coefficient ranging from 0.2 to 2.0, although this may also be defined as 0.2–3.0 following Eads et al. (2015)'s definition. $Sa_{avg}(T^*)$ is used in place of $Sa(T^*)$ herein, but this has the consequence that the $R - \mu - T$ relationship traditionally used is now replaced by $\rho - \mu - T$ relationships, where ρ is defined as follows:

$$\rho = \frac{Sa_{avg}(T^*)}{Sa_y} \tag{7}$$

Using these $\rho - \mu - T$ relationships for an equivalent SDOF system with period T^*, the ductility demand, μ, can be determined at a given intensity $Sa_{avg}(T^*)$ using the *xgb-rhomut* package developed in Python by Shahnazaryan and O'Reilly (2024), for example, available at https://pypi.org/project/xgb-rhomut. Once this ductility demand is known in the equivalent SDOF system for a given intensity, the MDOF system's displacement profile may be estimated by a simple transformation factor that assumes first mode-dominant behavior for a given typology (Equation 8) and the corresponding PSD profile, θ_i, is computed via Equation 9 where h_i is the height of story i. The result of this step is that for assumed seismic intensity $Sa_{avg}(T^*)$, the expected PSD profile, θ, across all stories of the structure is obtained. Again, this means that estimating the PSD demand profile for a given intensity is just one approach described and others may be adopted if the assumptions used to develop these simple tools (e.g., backbone curve and ground motion records used) should no longer be deemed suitable:

$$\Delta_i = \phi_{1,i} \mu \Delta_y^* \Gamma \tag{8}$$

$$\theta_i = \frac{\Delta_{i+1} - \Delta_i}{h_i} \tag{9}$$

PFA demands. The previous section discussed the simplified estimation of PSD-based demands in a structure at a given intensity. The corresponding PFA-based demands also need to be computed across the building height for acceleration-sensitive elements. The principal difference here is that the PFA at a given level, a_i, tends to possess notable contributions for several modes of response. Therefore, the assumption of a first mode-dominated response is no longer sufficient. However, several studies have sought to overcome such a limitation by proposing semi-empirical relationships that describe the expected PFA profile over the height of a building. For example, FEMA P-58 (FEMA, 2012) proposed a means to estimate the ratio Ω_i (see Equation 10) at each floor level i for frame, wall, and braced frame structures. Other works (e.g., Sullivan et al., 2013; Merino et al., 2020; Welch and Sullivan, 2017) looked at ways to more accurately quantify the entire floor response spectra of demands on non-structural elements. A recent study by Muho et al. (2021), however, has proposed a means to estimate PFA profiles based on the expected level of drift demand in the structure and is adopted herein.

The method begins with estimating the maximum PSD along the structure height, θ_{max}. Knowing the structural typology, a functional form is identified based on the number of stories, and a series of coefficients are determined from Table 1 (Muho et al., 2021). The PFA profile can then be determined from the ratio Ω_i times the peak ground acceleration (PGA), a_g, where:

$$\Omega_i = \frac{a_i}{a_g} = \alpha_1 \theta_{max}^{\alpha_2} T^{\alpha_3} \tag{10}$$

Again, it is at the analyst's discretion whether these studies are suitable for their specific building typology and future developments are certainly possible on this topic. The slight inconsistency in adopting this method is that the vulnerability function will be constructed for discrete values of $\mathrm{IM} = Sa_{avg}(T^*)$. In contrast, this method by Muho et al. (2021) requires a value of PGA to estimate the PFA profile. This can be overcome by simply finding the corresponding value of PGA for a given value of $Sa_{avg}(T^*)$ by comparing their respective hazard curves. Another aspect worth noting is that in order to estimate PFA demands with such a simplified method, knowledge of the building typology (e.g., moment frame) is required. While this may appear to be a disadvantage since it requires further details sometimes not needed for PSD demand estimation, it is worth recalling that to estimate the Γ parameter in Equation 1, some knowledge of the first mode shape is also needed, which is also typology-specific.

Estimating repair costs. Once the expected PSD and PFA profiles are estimated for a given intensity, the losses may be obtained to create a vulnerability function. This is a key difference of the approach outlined here, whereby instead of assuming that the structure's total economic loss can be summarized at discrete displacement-based DSs, the individual PSD and PFA demands are used to tabulate the expected structural and non-structural contributions to the loss in a more refined manner. It requires a means to estimate repair costs based on the expected level of story drift or floor acceleration. These repair costs are a function of the structural and non-structural damageable elements assumed to be located in the building.

Ramirez and Miranda (2009) proposed a solution using what are known as SLFs, relating the level of structural demand to an expected monetary loss due to repair costs at level i as $E[L|NC \cap R, \theta_i \vee a_i]_{PG,i}$, where the terms NC and R denote that the building is in a non-collapsed state and repairable, and the operator $\vee$ denotes that either θ_i or a_i is used based on the performance group (PG) being considered. A PG in this context refers to a group of elements, either structural (S) or non-structural (NS), which are sensitive to PSD or PFA demands. Generally, the three PGs are PSD-S, PSD-NS, and PFA-NS and are

Table 1. Coefficients proposed by Muho et al. (2021) to estimate Ω_i profiles

Point	2–4 stories			5–20 stories		
	α_1	α_2	α_3	α_1	α_2	α_3
A	0.046	–0.770	0.297	0.135	–0.537	–0.025
B	0.230	–0.355	0.213	0.254	–0.340	–0.026
C				0.636	–0.151	–0.154
D				0.923	–0.029	–0.100

illustrated in Figure 4. Several studies have examined the use of such SLFs (e.g., Esmaili et al., 2018; Papadopoulos et al., 2019; Poveda and O'Reilly, 2024; Ramirez and Miranda, 2009), with Shahnazaryan et al. (2021) proposing a toolbox for their development. These SLFs depend on the assumed damageable inventory for the building and will nonetheless be a function of the quantities, seismic fragility, and repair costs of the different components. Shahnazaryan et al. (2021) demonstrated that with respect to the more refined component-based approach advocated in FEMA (2012), using SLFs yielded essentially the same result.

The obvious drawback with this approach is that the damageable components, their fragility, and repair costs need to be known to develop the SLFs. Typically, the required information is rather plentiful when examining the North American context, with databases available in FEMA (2012) providing much of the information needed, for example. Analysts should exercise caution when extending such databases and data to other regional contexts where the structural and non-structural components may behave differently, or the repair costing and quantities information varies notably. However, a recent study by Nafeh and O'Reilly (2024) examined over 105 infilled RC building models, varying the number of stories, global dimensions, occupancy type, structural typology, and other specific architectural features, such as infill locations. They found that for a given typology and assumed building occupancy type (e.g., residential and commercial), the SLFs' *relative* trend tends to be relatively stable, but their absolute values may change depending on the story value. This is because the repair costs tend to be broadly similar and essentially scaled up and down as a function of the damageable inventory's relative magnitude. O'Reilly et al. (2023) proposed the use of *generalized* SLFs, whereby the functions are normalized by their total PG value at each story. These generalized SLFs are defined as follows:

$$E\left[\tilde{L}|NC \cap R, \theta_i \vee a_i\right]_{PG,i} = \frac{E[L|NC \cap R, \theta_i \vee a_i]_{PG,i}}{\sum_{PG} E[L|NC \cap R, \infty]_{PG,i}} \tag{11}$$

where $E[L|NC \cap R, \infty]_{PG,i}$ represents the expected loss among all components of a PG at story level i at infinite demand. This essentially means the total value of $E[\tilde{L}|NC \cap R, \theta_i \vee a_i]_{PG,i}$ for all PG at each level i should sum to unity. Esmaili et al. (2018) briefly mentioned a similar approach for use in regional analysis, although in that study the SLFs are normalized by the replacement cost of each story, whose exact computation was not clear. With a set of generalized SLFs, which ultimately represent the trend in accumulation of losses for increased PSD and PFA demands with intensity, a user can specify their different PG's relative contribution to $E[\tilde{L}|NC \cap R, \theta_i \vee a_i]_{PG,i}$ and proceed from there. Summing these together for each story, the total losses due to repair costs, $E[L|NC \cap R, IM]$, are then computed at a given IM value as follows:

$$E[L|NC \cap R, IM] = \sum_{i} \sum_{PG} E[L|NC \cap R, \theta_i(IM) \vee a_i(IM)]_{PG,i} \tag{12}$$

where the θ_i and a_i are computed for each level of IM, as per the previous sections. This means that an analyst may simply adopt generalized SLFs from a known database, provided they accept that the relative trend of repair costs with increasing story drift or floor acceleration is indeed representative of their building typology and region of interest. These databases may be extended in future studies, as is currently done using damage-to-loss models. They will instead have the added benefit of more detailed information within this model using actual demand parameters, which can be adjusted if needed.

Assembling the vulnerability function. The previous section described how the expected loss in a building due to repairs, $E[L|NC \cap R, IM]$, may be estimated. It is important to recall that this represents just one possible source of loss. The other prominent sources that should be accounted for are the possibility that the building has collapsed or is damaged to such an extent that it will require demolition rather than repair. In short, economic loss may be viewed as the sum of three mutually exclusive, collectively exhaustive events: non-collapse requiring repair (R), non-collapse requiring demolition (D), and total replacement due to collapse (C). When combined, the vulnerability function is assembled as follows:

$$\begin{aligned} E[L_T|IM] = & \underbrace{E[L|NC \cap R, IM](1 - P[D|NC, IM])(1 - P[C|IM])}_{\text{Non-collapse requiring repair}} + \\ & \underbrace{E[L|NC \cap D]P[D|NC, IM](1 - P[C|IM])}_{\text{Non-collapse requiring demolition}} + \underbrace{E[L|C]P[C|IM]}_{\text{Collapse requiring replacement}} \end{aligned} \tag{13}$$

where $P[D|NC, IM]$ is the probability of requiring demolition for a non-collapsed structure. Ramirez and Miranda (2012) provided a means of estimating this based on the expected residual drifts in the structure, although future work is needed to understand this issue in greater detail. Also, demolition may also be the simplest option if the owner deems it more worthwhile to start with a new building, rather than repair the existing one. For example, Kim et al. (2017) discussed this for the case of New Zealand following the 2011–2012 Canterbury earthquakes, where several factors were noted to play an important role. However, if this issue can be ignored and the non-collapsed building is assumed always to be repairable (i.e., $P[D|NC, IM] = 0$), Equation 13 simplifies to:

$$E[L_T|IM] = \underbrace{E[L|NC, IM](1 - P[C|IM])}_{\text{Non-collapse requiring repair}} + \underbrace{E[L|C]P[C|IM]}_{\text{Collapse requiring replacement}} \tag{14}$$

where the $P[C|IM]$ denotes the collapse probability at the given intensity level. This may be computed directly from the collapse fragility function provided by the *xgb-rhomut* tool previously described. It is worth noting that integrating the collapse fragility function with seismic hazard enables the determination of collapse risk, similar to how the vulnerability function is used to derive expected (or average) annual losses.

Case study example

The fragility-based and SLF-based approaches outlined previously were implemented in a case study application for a comparative assessment. In particular, the SLF-based approach relies on a few definitions of building properties, such as number of stories, story heights, structural typology, occupancy type, and so on. The complexity of the building definition may increase based on the needs of the designer (i.e., construction year), but for the sake of simplicity, the case study structure was assumed to be an RC moment-resisting frame (MRF) system with 4 stories of height 3.5 m for the ground and 3.0 m for the typical stories. The following properties were assumed for the case study structure's backbone curve: a secant-to-yield period of 1.0 s, a ductility of 4.0, a base shear coefficient of 0.2, a hardening ratio of 0.02, and a softening slope of –0.5. To maintain consistency for comparative purposes, it is crucial to highlight that the resulting equivalent SDOF's backbone curve is used in the fragility-based approach.

The resulting equivalent SDOF oscillator with $T^* = 1s$ was modeled using the *Hysteretic* uniaxial material in OpenSeesPy (Zhu et al., 2018) to be used during the fragility-based approach. Committed stiffness proportional to Rayleigh damping of 5.0% was considered. Both post-yield hardening and post-peak degrading branches were considered leading to the creation of a tri-linear degrading backbone curve of Figure 5a, where PSD-based DSs are denoted. This curve was considered to better reflect the nature of common structural systems encountered in seismic design and assessment over a simpler bi-linear model without degradation.

Fragility functions and loss ratios approach

First, vulnerability assessment was carried out using fragility functions, and the damage-to-loss model of Martins and Silva (2021) (Figure 5b) considered for almost 500 building typologies. The damage-to-loss model was based on four DSs ranging from slight (DS1) to complete damage (DS4). To accommodate damage onset in non-structural components like infill walls, slight damage was assumed to start at 75% of the yielding displacement, while complete damage was assumed at the ultimate displacement capacity of the structure. Intermediate DSs were assumed to be evenly spaced between the initial and ultimate DS. To derive the fragility functions, non-linear time history analyses were conducted following a cloud analysis approach (Jalayer and Cornell, 2009). As mentioned previously, $Sa_{avg}(T^* = 1.0\text{s})$ was chosen as the IM for the case study application, although not strictly

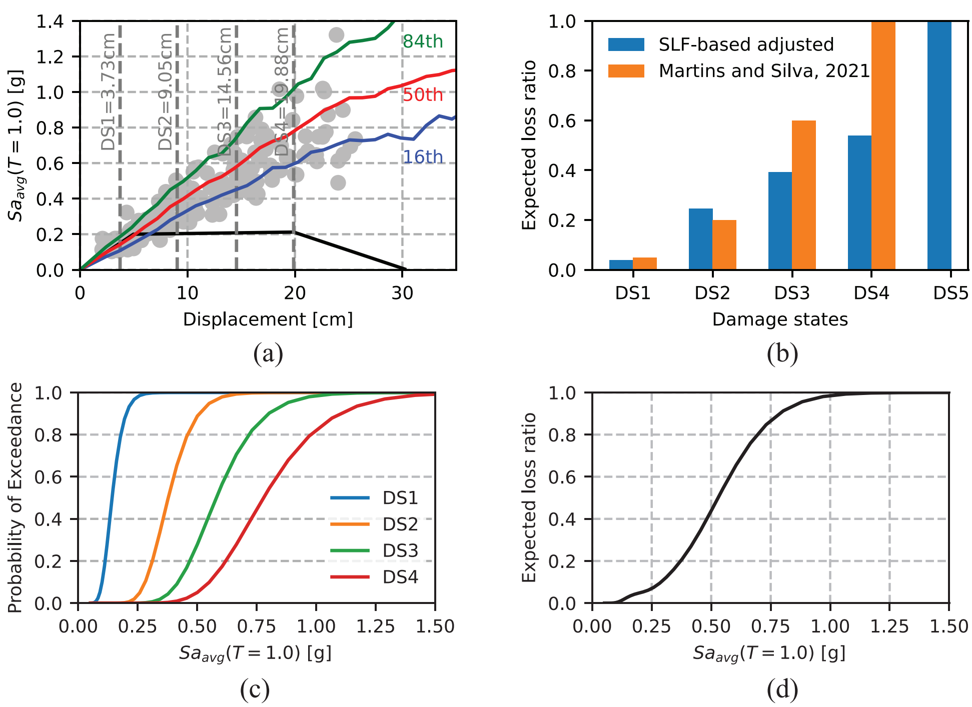

Figure 5. (a) Damage-to-loss models adopted; (b) backbone curve, cloud analysis results, and *xgb-rhomut* 16th, 50th, and 84th percentile predictions; (c) fragility functions; and (d) vulnerability function of fragility-based assessment.

required. To perform cloud analysis, record selection needed to be carried out. Following the approach of Martins and Silva (2021), 10 bins of IM were considered between 0.1 and 2.0 g, and 30 records were randomly selected for each bin from the NGA-West2 database (Ancheta et al., 2014). As a result, 300 records were selected, where some of the records were scaled by a maximum scaling factor of 2.0 to populate the more intense IM bins. Figure 5a depicts the cloud analysis results, indicating that most records exceeding an $Sa_{avg}(T^* = 1.0\text{s})$ of 1.0 g led to numerical instabilities due to lateral collapse. With the damage-to-loss model defined, the fragility function parameters were computed using the maximum likelihood estimation method and are shown in Figure 5c. As discussed previously, once the discrete set of fragility functions associated with each DS and respective expected loss ratio were obtained, the vulnerability function was computed via Equation 4 and is plotted in Figure 5d.

SLF-based approach

As the name suggests, the generalized SLFs, developed using the toolbox presented by Shahnazaryan et al. (2021), were required prior to initiating the vulnerability assessment. These are illustrated in Figure 6, which denote the $E[\tilde{L}|NC \cap R, \theta_i \vee a_i]_{PG,i}$ previously presented in Equation 11 and are simply adopted from an example presented in Shahnazaryan et al. (2021). It is important to note that the developed SLFs are not meant to be representative for the specific building typology and occupancy examined here, but rather an arbitrary set of functions for demonstrative purposes. The SLFs were assumed to have different relative values at each story (i.e, $\sum_{PG} E[L|NC \cap R, \infty]_{PG,i}$ from Equation 11) with the ratios 2:3:3:3.5 for PSD-NS, 3:2.5:2.5:1.5 for PSD-S, and 4:1:1:2 for PFA-NS used to provide variability and were all normalized to the total value of unity for the entire building.

To identify the demand-intensity model through the *xgb-rhomut* tool, the equivalent SDOF representation was derived following the procedure outlined previously. Figure 5a also incorporates the *xgb-rhomut* predictions in terms of displacement and IM, where they are essentially the same meaning that non-linear dynamic analysis with ground motion records are not needed. Using this estimated value of μ for a given $Sa_{avg}(T^* = 1.0\text{s})$ value, the PSD profile was estimated from Equation 9. For what concerns the floor accelerations, the PFA profile was estimated using Equation 10.

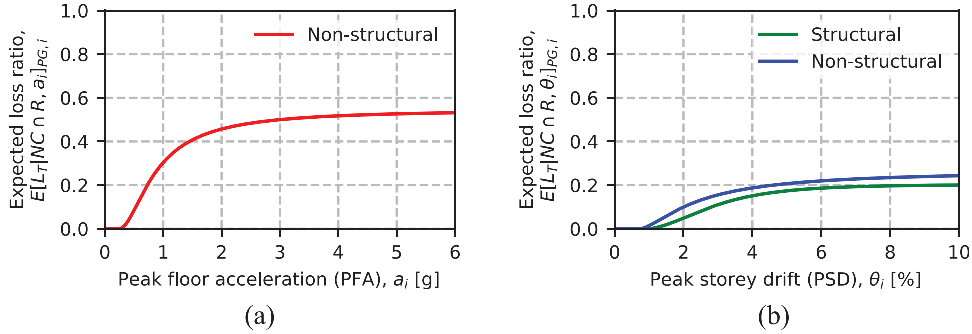

Figure 6. Generalized SLFs for the case study structure: (a) PFA-sensitive and (b) PSD-sensitive.

As a result, the PSDs and PFAs associated with different levels of intensity allowed the computation of repair costs using the SLFs of Figure 6 and Equation 12. It should be noted that these simplified methods to estimate the PSD and PFA demands are generally based on empirical formulae calibrated from past numerical analysis results. This may introduce some additional uncertainties into calculations, especially when extending the SLF-based approach to large-scale risk analysis. However, the magnitude of these uncertainties with respect to other sources is not expected to be significant. Besides, this estimation approach can be seamlessly substituted in the present approach for a more refined means of estimating demands. The other prominent sources of loss include the non-collapse requiring demolition and total replacement costs due to collapse. Even though the demolition costs could be easily incorporated by considering residual drifts or assuming a demolition capacity of a structure, as suggested in Ramirez and Miranda (2009), these were neglected within this case study application to maintain a comparative basis with the fragility-based approach. For what concerns the losses associated with the collapse of the structure, the collapse fragility function was estimated using the *xgb-rhomut* tool, which was then used to estimate the probability of collapse at each IM level. The resulting vulnerability function was estimated using Equation 14 and is plotted in Figure 7. Also shown is the disaggregation of the vulnerability function for each PG along with the contribution of collapse.

Comparison of results

The vulnerability functions obtained through the fragility- and SLF-based approaches are compared here to investigate the similarities and important differences between the two methods. Figure 8 shows this comparison, where the immediate observation is that the two methods are notably different, albeit within the same order of magnitude. This is not a surprising observation given that the damage-to-loss models used to develop these vulnerability functions were different in both cases. The structural demands were the same, however, as illustrated in Figure 5a; hence, any discrepancy here comes from the difference in the loss model. An immediate observation is that the SLF-based vulnerability function is notably higher at lower intensities than the fragility-based function. This is simply due

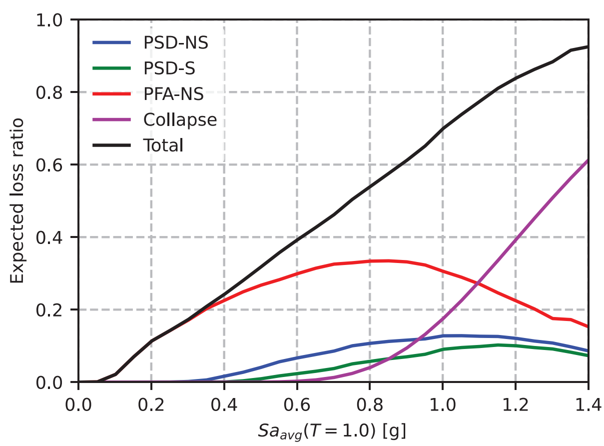

Figure 7. Vulnerability functions showing the breakdown between different contributors following the SLF-based approach.

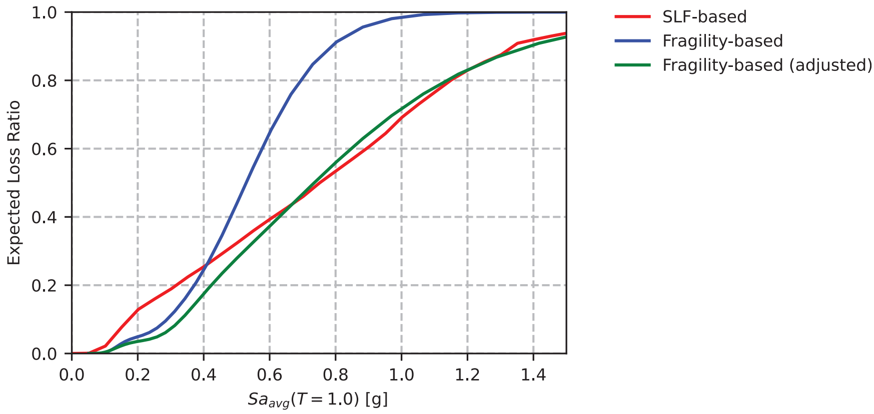

Figure 8. Comparison of the fragility-based and SLF-based vulnerability functions.

to the loss models being different in this specific scenario and does not imply that SLF-based functions are expected to be higher.

To provide a more compatible comparison of the two methods, the DS loss ratios required for the fragility-based method were back-calculated from the results of the SLF-based approach. That is, the fragility functions shown in Figure 5c were integrated with the red line shown in Figure 8. This is described by Equation 15, and the resulting loss ratios are shown in Figure 5b as *SLF-based adjusted*. An additional DS of collapse was added with a loss ratio of 1.0 for consistency. The vulnerability function using these adjusted loss ratios is shown in Figure 8 alongside the original fragility-based and SLF-based approaches. Despite the blaring disparity between the vulnerability functions of the fragility-based and the SLF-based approaches, it does not necessarily reflect negatively on either methodology, as the initial set of loss ratios was not consistent with the SLF functions (Figure 6) employed. This is underlined when compatible loss ratios were derived and used within the fragility-based method, shown as the green line. There is still some discrepancy, however, but further investigation showed that this was a consequence of using just four DSs in the fragility-based method. If many more DSs were added, the fragility-based and SLF-based approaches' results (i.e., the red and green lines in Figure 8) coincided perfectly. Hence, the discrepancy shown in Figure 8 is a result of discretization error and may be theoretically reduced by adding further DSs. However, doing so would likely run into practical issues beyond the scope of interest for this study. The main message here is that for a compatible set of loss ratios and a large enough number of DSs, the discrete fragility-based approach theoretically converges toward the more continuous SLF-based approach. This is an encouraging observation as it essentially means the two are equivalent, with one offering more outputs and insight than the other:

$$E[L|DS_i] = \int_0^{+\infty} P[DS_i|IM] \left(\frac{dE[L_T|IM]}{dIM} \right) dIM \tag{15}$$

To further underline the enhanced output offered via the SLF-based approach, Figure 9a provides the relative contributions of different PGs and collapse losses to the vulnerability functions as a function of IM. At lower levels of IM, repair (non-collapse) losses, particularly attributable to PFA-sensitive non-structural PG, emerge as the primary

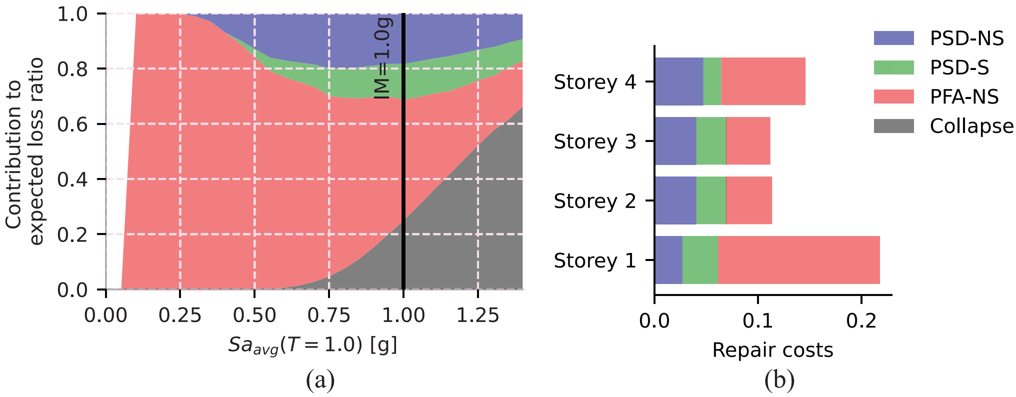

Figure 9. (a) Relative contribution to expected loss with respect to increasing IM and (b) repair cost contributors along the structure's height at the IM of 1.0 g.

contributors. As intensity increases, the contributions from PSD-sensitive non-structural and structural PGs gradually rise while remaining relatively stable. Conversely, the contribution from collapse to the expected loss ratio starts increasing at mid-to-high intensities, eventually becoming the dominant factor in overall loss. Another significant advantage of the SLF-based approach lies in its ability to disaggregate losses along the height of the building, as shown in Figure 9b. Here it is clear which PG is contributing most to the losses at each individual level, and not just for the entire building. Again, this specific example has been chosen to illustrate the potential of the approach and does not imply that this is the expected distribution of losses for this typology. Only through accurate and more specific data will it be possible to observe general trends. The point is to show some of the key benefits of using the proposed SLF-based approach with respect to the fragility-based approach of SDOF systems, as disaggregation of losses allows the designer or stakeholders to identify and address vulnerable components within the building without necessitating extensive analysis.

Discussion

Benefits of SLF-based approach

The previous section presented a case study example to show the SLF-based approach's compatibility with the widely used fragility-based approach. No extensive non-linear dynamic analysis or validation of the loss estimates was conducted, mainly to avoid scope creep and make this study unnecessarily long. Such analyses can easily be carried out in future case studies. It was shown that when the damage-to-loss ratios are tailored to align with the SLF values, the two approaches lead to essentially the same vulnerability function. This underlines the conceptual similarity of the methodologies, and the SLF-based approach may be considered a refinement, or more detailed approach, based on similar principles. Again, future comparisons with detailed numerical validation analyses may shed light on where these approaches can be adjusted and refined, but within the scope of this study, the conceptual differences were made clear. This brings with it several advantages and future opportunities.

The main advantage is that the economic losses can be disaggregated into their principal contributors, whereas this was not previously possible with traditional damage-to-loss models. This is useful because the principal drivers of losses in certain contexts may differ among building typologies and can impact the retrofitting strategies pursued. This is true not just for the PGs (i.e., PSD-S, PSD-NS, and PFA-NS) but also along the different stories of the building, as illustrated in Figure 9b. Adopting the typical approach of increased strength and stiffness of a structure is not always guaranteed to be a prudent strategy. This is because past work by O'Reilly and Sullivan (2018), for example, has shown that while increased strength and stiffness may improve collapse safety, it may increase the expected losses rather than decrease them. This is primarily due to whether the dominant loss sources are drift-sensitive or acceleration-sensitive. Increasing strength and stiffness will reduce the drift demands but at the same time increase the floor acceleration demands. Hence, there is a trade-off that one needs to be aware of. Utilizing SLFs to generate vulnerability functions can provide valuable information in this regard since it disaggregates the losses in a way that is more useful to practitioners and stakeholders when setting out to take risk-reducing measures.

Also worth mentioning is that the collapse fragility function obtained from a tool like *xgb-rhomut* can be used to compute the collapse risk. This may be used to provide a more explicit estimate of expected casualties, and decisions may be made on how to prioritize occupant life safety. Also, situations where buildings are so heavily damaged that they will likely require demolition can be useful to decision-makers. With this information, building typologies that are so deficient that retrofitting measures may not suffice can be identified. Any attempted retrofitting actions may be costly and futile, and the more prudent choice could be to replace the structure or explore proactive insurance policies. These kinds of features may be directly incorporated into the vulnerability models utilized around different regions depending on a variety of factors discussed by Kim et al. (2017) in New Zealand, for example, different attitudes to repair and recovery may exist worldwide.

Regarding actual computation, no real increased computational cost or difficulty is associated with the SLF-based approach. Implementation and analysis time for the case study presented were more or less identical. Examining Figure 3, it is clear that several tools, methods, and assumptions are required. It is argued, however, that this is no more computationally demanding than the approach presented in Figure 2 since ground motion modeling and record selection were not required. For what concerns input data, an added degree of detail is required to derive SLFs. Still, it is envisaged that the extra effort is worth the increased quality of the output decision variables shown in Figure 7. The main point is that SLFs will need to be developed for several structural (e.g., RC, masonry, and steel) and occupancy (e.g., residential and commercial) typologies, or building taxonomies, in different regions of the world. While this may seem a mammoth task, several tools and simplifications may be adopted. The first is via the toolbox developed by Shahnazaryan et al. (2021) where a user can develop their own SLFs from scratch, which has already been implemented by some studies around the world (e.g., Nafeh and O'Reilly, 2024; Poveda and O'Reilly, 2024). Another option is to use generalized SLFs, where the general shape and horizontal scale of the SLFs shown in Figure 6 can be assumed to be constant among different building taxonomies (e.g., for a given level of story drift θ, $X\%$ of the story loss has been reached), but the vertical scale is normalized and subsequently adjusted based on the assumed relative values of repair costs between each story. Hence, if one were to have a reasonable estimate of the relative values of the PGs, a set of SLFs could be easily obtained via Equation 11. This approach is considered much more intuitive than the current damage-to-loss model approach, which tends to involve empirical calibration to DSs whose precise definition may be somewhat subjective. SLFs would, of course, need to be calibrated, but the fact they can be disaggregated makes a more appealing strategy.

Usability in insurance industries

The potential utility of SLFs is not limited to estimating economic losses in single and multi-story buildings quickly and relatively accurately, as illustrated in Figure 9b. It can provide a useful means to estimate relative losses within a building when little data is available. Consider if an insurer has underwritten policies of different assets (i.e., paintings, specific equipment, and machinery) located in different stories of a multi-story building. In this situation, an insurer may obtain an estimate, albeit relative, of the expected level of damage to different component groups across the building height. This kind of breakdown can be useful since it offers a story-by-story view of the situation. It also has the potential to allow the development of premium tariffs for other non-seismic natural catastrophe perils in which the story location is the distinguishing factor (e.g., floods). It is advantageous since most of the current risk and vulnerability models describe buildings as single-whole assets, offering some distinction between structural and non-structural elements and building contents, but not at a story-by-story level. This is especially relevant considering the seismic demands are not uniform along the building height. For example, Poveda and O'Reilly (2024) recently showed this via a component-based loss assessment of a single building in Ecuador. Hence, there is a gap when disaggregating the global loss estimate of a building into its relative stories and component groups. It is here that SLFs can be of potential assistance assist.

An example scenario is described to illustrate this point. Consider the distribution of expected losses per story shown in Figure 9b. Here, it can be clearly seen what the relative risk between each PG and story is for that intensity of shaking. Therefore, if an insurer needs to determine tariffs and underwrite risk for the respective owners of apartments or offices at each story level, this information can be used to assign relative quotas, recognizing that damage and losses are known to be non-uniform along building height. This can also account for the evolution of damage patterns typically expected for low-intensity earthquakes versus more damaging high-intensity earthquakes. Previously, without such a breakdown, an insurer may be inclined to divide the tariffs equally among all occupants, regardless of the relative damage expected. In the specific case of Figure 9b, it is clear that the tariffs should be higher on their ground story and the top level compared with the other stories, for example. This is discussed merely from the perspective of risk estimation when determining tariffs and not for calculating actual losses and deciding payouts. For this, existing methods can still be followed and even serve as a cross-validation check for the SLF-based estimate of losses.

Future directions

While the SLF-based methodology offers several advantages that can be beneficial in several respects, some possible future developments can be pursued. The first concerns developing SLF data for different typologies. Much of the data can be derived directly based on costing data or from actual cost data following major earthquakes. Nevertheless, reformatting to fit this context is needed. Following on from this, and with a view to making this approach more standardized, governing bodies can adopt generalized SLFs that have been standardized based on available data. This way, the subjectivity element of the process is removed.

Another possible future extension surrounds using SDOF models in Figure 2. While the methods adopted to estimate PSD and PFA demands relied on empirical approximations, equivalent MDOF models such as those proposed by Gaetani d'Aragona et al. (2020) or Ruggieri et al. (2022a) may also be utilized when needed. It is also important to note that the methods outlined previously provided an estimate of median structural demand for a given intensity. The variability due to ground motions or other sources was not directly incorporated here but may be easily incorporated via Monte Carlo simulations. Likewise, the variability in the SLFs could be accounted for this way, but both were omitted from the previous discussions to avoid over-complicating the text and taking away from the main discussion.

Regarding the IM used in assessment, this approach directly adopted the more advanced average spectral acceleration in its methodologies and assessments. It is envisaged that this IM is a much more powerful predictor of structural response and hence ought to be adopted. When applying this SLF-based approach with $Sa_{avg}(T^*)$ on a more regional scale, this will invariably lead to difficulties in assessing rupture scenarios for entire regions via ground motion fields or ShakeMaps (Silva et al., 2019). Ground motion and correlation models will be required, which recent research by O'Reilly et al. (2024) has recently begun addressing, for example, although there is nothing preventing the described approach from being used with a simpler IM like $Sa(T^*)$ in place of $Sa_{avg}(T^*)$.

Summary and conclusions

This article presented a comparative study of two methods to develop vulnerability functions for regional seismic risk modeling. These methods were based on analytical methods and involved several key assumptions to make them widely applicable and computationally highly efficient. The first of these methods was based on the fragility analysis of displacement-based demands on an equivalent SDOF model, which, in tandem with loss ratios, returns vulnerability functions. A second and alternative approach described here utilizes an equivalent SDOF model to estimate both displacement- and acceleration-based demands at each building level. Combining these demands with SLFs leads to the estimation of vulnerability at each story and, ultimately, the vulnerability function of the building. The two methods were described in detail, and a comparative case study example was presented to scrutinize their similarities and differences. Some final remarks on the wider implications and possible future directions were also presented.

Based on the work presented here, the following conclusions can be drawn:

- The SLF-based approach is a more detailed and robust methodology compared with the widely used fragility and damage-to-loss ratio-based approach. The SLF-based approach is more detailed but does not carry the burden of excessive extra computation or effort. In fact, when simplified, the SLF-based approach can be reduced to give essentially the same result as the fragility-based approach.
- The SLF-based approach directly considers both PSD- and PFA-based demands at all stories of the building. This breakdown allows for a more detailed consideration of non-structural elements and identifies which PGs at which building location contribute most to the expected loss.

- The issue of collapse and demolition are handled more directly, which can be useful for decision-making in issues beyond economic losses, such as estimating casualties or identifying when retrofitting interventions may be futile.
- While SLFs are needed for different building taxonomies, the generalized format may be a prudent strategy given that a physics-based approach to modeling the building component fragility can be adopted, followed by a rational approach to estimating the expected repair costs.
- The uses of this approach extend beyond vulnerability modeling and can also aid insurance companies in deciphering and establishing a relative scale of expected damage and loss distribution within a building to determine more reasonable tariffs.

Overall, this extension of the existing to the proposed approach is without much additional computational cost and can represent an enhanced level of quality in the output decision variables used in regional seismic risk assessment.

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