Bulletin of Earthquake Engineering
https://doi.org/10.1007/s10518-021-01177-y

ORIGINAL ARTICLE

A seismic risk classification framework for non-structural elements

Gerard J. O'Reilly[1] · Gian Michele Calvi[1]

Received: 29 October 2020 / Accepted: 5 July 2021

Abstract
The ability to quantify the seismic risk associated with structural and non-structural elements is a critical aspect of earthquake engineering. While methods to improve the understanding of structural response to earthquake shaking and how to quantify their risk have been studied, non-structural elements (NSEs) have more recently emerged as a crucial aspect to address given their pertinence in overall building performance and loss-related issues. This article describes the development of a risk quantification methodology for NSEs whereby the mean annual frequency of exceeding an NSE's damage state is computed and rated as part of a risk classification scheme using recently developed approaches. The basis of the methodology is described in detail followed by an example implementation, where the details surrounding hazard, structural and non-structural response are quantified consistently, ensuring that uncertainties are also incorporated to be in line with modern performance-based earthquake engineering. Discussion is provided surrounding the potential future use of such an NSE risk classification scheme for both structural engineers looking to improve the performance of their buildings via NSE performance and also manufacturers.

Keywords Non-structural elements · Seismic performance · Risk classification · Design · Assessment

1 Introduction

Non-structural elements (NSEs) are those which do not form part of a building's structural load-bearing system, but are nevertheless subjected to dynamic forces and deformations during ground shaking. Following the numerous earthquakes that have struck different regions around the world in past years, damage to NSEs has been noted to be a recurring theme (Filiatrault et al. 2001; Chock et al. 2006; Gupta and McDonald 2008; O'Reilly et al. 2018; Ricci et al. 2011; Perrone et al. 2018). It may be argued that this is a result of most design codes' attitude to seismic performance of buildings. The term 'building'

✉ Gerard J. O'Reilly
gerard.oreilly@iusspavia.it

[1] Centre for Training and Research on Reduction of Seismic Risk (ROSE Centre), Scuola Universitaria Superiore IUSS di Pavia, Pavia, Italy 27100

is noted as being distinct from 'structure' with the difference being that a structure comprises solely the load resisting system whereas a building denotes the final product of both structural and non-structural elements in addition to the building contents. In design codes such as Eurocode 8 (CEN 2004), ASCE 7–16 (ASCE 7–16 2016) and NZS 1170 (NZS 1170.5:2004 2004), precedence is typically given to satisfactory structural behaviour during strong ground motion events, with some checks on the imposed storey deformations and induced floor accelerations aiming to mitigate any excessive NSE damage during more frequent levels of ground shaking. However, Filiatrault and Sullivan (2014) have discussed how that even if the performance of a building's structural elements is satisfactory to permit continuous and immediate occupancy following a seismic event, the failure of architectural, mechanical or electrical elements that constitute the building as a whole can reduce the performance and functionality of the entire building system.

In terms of their role in building cost, Taghavi and Miranda (2003) have pointed out that for office, hotel and hospital buildings, NSEs make up approximately 82%, 87% and 92% of the total monetary investment, respectively, as shown in Fig. 1. Additionally, O'Reilly et al. (2018) have shown (Fig. 2) that for a typical school building in Italy, NSEs comprise the majority (> 60%) of the direct monetary losses induced at more frequent levels of ground shaking. These two points alone highlight the critical nature of NSEs both from an initial investment and potential monetary loss perspective. Furthermore, NSEs are known to also pose life safety risks due to falling objects or increase downtime due to issues like burst water pipes leaking. Considering that most design codes focus on the ultimate behaviour of a structure with checks and verifications for NSEs at more frequent levels of ground shaking, the actual margin of safety or quantification of NSE performance is not easily

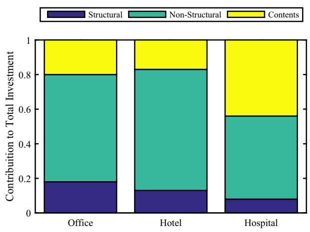

Fig. 1 Illustration of the relative monetary investments in different building typologies by Taghavi and Miranda (2003)

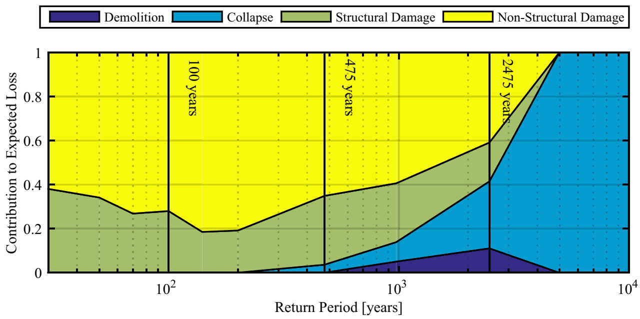

Fig. 2 Illustration of the relative contribution to the expected direct losses in an Italian school building with increasing return periods of ground shaking by O'Reilly et al. (2018)

obtained. Most codes prescribe methods to estimate the demands on the NSEs as a function of the main structure's response but given that code provisions tend to be conservative in their (deterministic) estimate of structural response via various design methods, no accurate quantification of the NSE performance can reasonably be obtained unless detailed non-linear dynamic analyses on both the main structure and its NSEs are performed. In order to quantify the risk, more probabilistic approaches should be adopted.

In 2017, a notable change in the characterisation of seismic risk was introduced in Italy in the form of the so-called *Sismabonus* guidelines (Calvi, Sullivan, and Welch 2014; Decreto Ministeriale 2017; Cosenza et al. 2018) whereby the seismic risk of buildings is classified into different ratings using the more critical of a collapse safety index (IS-V) and expected annual losses (EAL) ratio, as shown in Fig. 3. The definition of EAL and its role on characterising seismic vulnerability of buildings has been well-documented in the literature and implemented in the FEMA P-58 guidelines (FEMA 2012b), for example. The collapse safety index is essentially defined as the ratio between the existing building's actual peak ground acceleration capacity and the value that would be used in new design. These guidelines have allowed the seismic risk of existing buildings to be quantified (e.g. Polese et al. 2019) and provide a way in which the overall seismic resilience of buildings may be improved. They possess a number of simplifications and have the scope to be refined and improved with further research, such as the consideration of retrofit sustainability (Passoni et al. 2021), but the general idea has been introduced and is gaining increased attention. For example, it has recently been employed as part of a framework to consider the integrated seismic and environmental performance in Caruso et al. (2021) in addition to studies on decision frameworks comprising multi-criteria (Clemett et al. 2021; Carofilis et al. 2021). In addition, the Italian government has been providing financial incentives to building owners who improve the risk rating of their buildings through tax deductions of up to 85% of the costs, with a more recent decree (Decreto Ministeriale 2020) increasing that same deduction up to 110%.

The focus of this article is to explore the level and types of risk posed by various NSEs on the built environment. Available frameworks to characterise and quantify these risks for NSEs are discussed followed by a proposal in which the level of performance, or risk, of a certain NSE damage level within a given building typology can be quantified. This is tackled from both a new design and assessment of existing buildings perspective. It utilises detailed relationships to describe the structural response in a probabilistic manner in addition to available NSE fragility functions determined from experimental testing. The result of this is that the performance can be characterised quantitatively and different sources

EAL Rating (EAL)	Life Safety Index (IS-V)	Risk Rating
EAL ≤ 0.5%	100% ≤ IS-V	A+
0.5% < EAL ≤ 1.0%	80% ≤ IS-V < 100%	A
1.0% < EAL ≤ 1.5%	60% ≤ IS-V < 80%	B
1.5% < EAL ≤ 2.5%	45% ≤ IS-V < 60%	C
2.5% < EAL ≤ 3.5%	30% ≤ IS-V < 45%	D
3.5% < EAL ≤ 4.5%	15% ≤ IS-V < 30%	E
4.5% < EAL ≤ 7.0%	IS-V < 15%	F
EAL ≥ 7.0%		G

Fig. 3 Illustration of the *Sismabonus* seismic risk classification scheme introduced in Italy for buildings, where the overall risk rating is determined from the more critical of the EAL and IS-V ratings

of aleatory and epistemic uncertainty can be incorporated in the process. This method is initially presented in a *single structure-single NSE* context, but may also be extended to a regional scale whereby entire groups of NSEs could be evaluated for different structural typologies in a relatively simplified manner. The utilisation of such results within a tentative classification scheme similar to *Sismabonus* is also discussed to set the initial path for a more refined quantification of NSE performance. This may lead the way for the development of a NSE performance classification system whereby manufacturers can associate a given level of certified seismic performance to their product (for a range of structural typologies and locations), similar to what is currently done for buildings via *CasaClima* in Italy (Agenzia CasaClima 2019), for example.

2 Current guidelines for NSEs and protection of occupants

One of the more prominent guidelines when dealing with NSEs in earthquake engineering is FEMA E-74 (FEMA 2012a). It aims to gives practical guidance on how different risks posed by NSEs may be identified and reduced through various preventative measures. It divides the various risks posed by NSEs into 3 types, which are listed in Table 1. This division was an important step as it allows for a more refined prioritisation of which types of NSEs are more critical for a certain building occupancy type and performance level. For example, in a school building the life safety (LS) risk is clearly to be mitigated given its large density of people within the building. Property loss (PL) may be an important aspect to consider for a factory building, as the contents and products manufactured may represent a much greater monetary value than the structure itself. Examples of this were during both the 2012 Emilia-Romagna earthquake in Italy (Ioannou et al. 2012) and the 2014 earthquake in South Napa, California (Fischer 2014) where damage to the NSEs meant that the large amounts of *parmigiano-reggiano* cheese and wine produced in these regions, respectively, were lost. Functional loss (FL) would be considered vital in situations where perhaps the property inside the building is not of utmost importance, but its role during and/or after an earthquake is. An example of this would be a civil protection building whose operation following an event would be essential.

Within FEMA E-74, these types of risks are then rated for a range of NSEs as low, medium or high risk and also as a function of the seismicity of the region. A number of assumptions were made in this classification scheme: no seismic provisions were utilised for the design of the NSEs; they are located near or at the ground level; and an ordinary occupancy is assumed. Depending on how important each of these risks are deemed to be, prioritisation schemes are suggested. While these guidelines are undoubtedly practical and accessible, quantifying the risks in such a general qualitative manner is not exactly in line with the objectives of modern performance-based earthquake engineering (Cornell and

Table 1 Types of risk for NSEs described in FEMA E-74 (FEMA 2012a)

Type of Risk	Description	Example
Life safety (LS)	Could anyone be hurt by this NSE in an earthquake?	School
Property loss (PL)	Could a large property loss result due to the loss of this NSE?	Warehouse
Functional loss (FL)	Could the loss of this NSE cause an outage or interruption to the functionality of this building?	Civil protection building

Krawinkler 2000) where risk is a measurable quantity that can be methodically reduced and managed. The FEMA E-74 approach will certainly help reduce risk but currently not on a measurable scale, as will be proposed in this study.

The Applied Technology Council (ATC) have published a set of recommendations for improved NSE performance in the ATC 120 (ATC 120 2018) report. It gives a relatively good and clear description of NSE performance and what to expect: *Building occupants and pedestrians outside are not threatened by falling hazards, egress is unimpeded, and hazardous materials are contained. Water intrusion is prevented, breathable air is provided, temperatures are maintained within an acceptable range, fire and smoke protection systems are in place and operable, and power, communications, and plumbing systems function as intended*. Should any of these conditions be violated, then it is deemed problematic. Like FEMA E-74, it provides many useful methods to reduce the vulnerability of NSEs and prescribes many preventative measures for general use. However, when coming to the issue of a quantitative description of what the performance objectives are, it too is insufficient. This is no fault of the document, but more a sign of how more thorough ways of addressing the issue of desired performance within earthquake engineering are yet to be put forward for NSEs and other secondary elements, in particular. Additionally, performance in ATC 120 is checked at a single level of seismic hazard (termed the 'design earthquake' in the US), as is typical with most codes, and a code-based design for new buildings is followed with the assumption that a design satisfying these requirements will implicitly lead to an acceptable level of performance at a lower 'serviceability' level. The problems in such assumptions when aiming to characterise the seismic risk of entire buildings are well-documented (e.g. Vamvatsikos et al. 2016) and are not discussed further here.

More recently, Sullivan et al. (2020) have proposed an NSE seismic rating framework where the storey drift or floor acceleration capacity is used to quantify the expected performance of each NSE in a building. It aligns well with the code-based limit state checks employed in the New Zealand context and offers a clear and simple way to rate the NSE performance for practitioners. Like the proposed framework described herein, it operates on the basis of a rating framework whereby different capacity limits are set by the analyst. Care must be taken when defining such limits for different NSE groups in order to ensure comparability between the expected performance of all NSEs examined. For example, if drift-based limits are set too conservatively and floor acceleration-based limits are set unconservatively by the analyst, then there may be a mismatch in the ratings assigned between NSE groups and the actual performance expected. The proposed framework discussed herein also operates on a user-defined rating scheme but argues that using risk as the measured quantity could help ensure comparability across all NSE groups.

3 Proposed risk quantification methodology

As shown in the previous sections, the quantification of NSEs within different structures has generally been addressed qualitatively with the aim of providing guidance on how to reduce their risk to incur harm, loss or interruption, but a methodology to systematically and accurately quantify the risk of these NSEs has received less direct attention. This section proposes a formulation to directly address this gap and describes a method to quantify and subsequently classify risk in a robust way for NSEs. It is also noted that these same approaches may also be extended to other secondary elements (i.e. elements not representing the principal lateral loading system) such as masonry gable walls or chimneys (e.g. Tomassetti et al. 2019; Kallioras et al. 2020).

3.1 Overview

For a given building, the seismic response with increasing intensity can be established from some kind of dynamic structural analysis procedure (e.g. incremental dynamic analysis, multiple-stripe analysis). This means that the general relationship between structural demand, D, and seismic intensity, s, is known for both storey drift and floor acceleration, which is herein termed a *demand-intensity* model (O'Reilly and Calvi 2020). The seismic intensity measure (IM) is the first mode spectral acceleration, $Sa(T_1)$, and the structural demands can be initially taken as the maximum of the peak storey drifts, θ_{max}, and peak floor accelerations, a_{max}, along the height. The maximum value along the height is referred to here for simplicity but it may also refer to specific location's demand (i.e. the roof or the ground storey) as will be discussed later. Certain situations may arise where the structural demands are impacted by the response of the NSE and the subsequent response (e.g. sloshing of water in tanks at the roof or presence of masonry infills). Also, situations where the NSE response is limited by the presence of the structure (e.g. limited clearance of a ceiling system to move freely) are issues to be given due care. These can be incorporated in the demand-intensity model through specific numerical modelling approaches but the details are not discussed here. For instance, the presence of masonry infill panels, which is a common structural typology found across the Mediterranean region, the same approach described herein may be adopted but it is important to note that the presence of the masonry infill panels must be accounted for in the numerical modelling (e.g. Mohamed and Romão 2021) and subsequent analysis results derived. Also, in such a situation, analysts should reconsider the IM adopted as recent research (O'Reilly 2021) has shown that using $Sa(T_1)$ (used as an example here) may not be the best IM to use for infilled frames and other IMs based on an averaging of spectral accelerations (Eads et al. 2015; 2016; Kohrangi et al. 2017) can avoid problems of bias. Similarly for the case of PFA, where past research (Iervolino and Manfredi 2008; Kohrangi et al. 2016) has indicated that $Sa(T_1)$ may not be the optimal IM but rather PGA, for example. Further considerations on which IM or demand parameter to use for different structural typology demands can be dealt with on a case-by-case basis by analysts as the framework proposed herein may be modified to suit these needs and $Sa(T_1)$ is described herein as an illustrative example.

Simplifying relationships have been proposed to describe demand-intensity models mathematically, with Cornell et al. (2002) noting that a linear model in logspace is generally sufficient for drift-based quantities. To deal with the acceleration-sensitive components, which play an important role in NSE response, O'Reilly and Monteiro (2019) have more recently shown that a bilinear model in logspace is more suited for acceleration-based quantities given the role of structural non-linearity on the propagation of floor accelerations along a building height (Calvi and Sullivan 2014). These relationships essentially mean that for a given value of structural demand, D, the intensity required to exceed that in a building can be computed, as illustrated in Fig. 4. Knowing this intensity and the site hazard model determined from probabilistic seismic hazard analysis (PSHA), the mean annual frequency of exceeding (MAFE) that level of demand can be computed in a closed-form solution. If the structural demand is set as the NSE limit state capacity, C, then the MAFE,

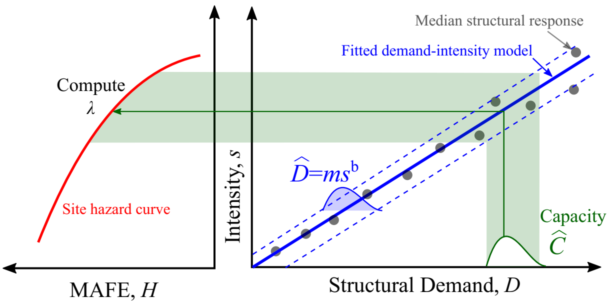

Fig. 4 Illustration of MAFE computation for a NSE capacity, C, using a structure's demand-intensity model and site hazard model, where the uncertainty surrounding the capacity and the demand are explicitly considered (O'Reilly and Calvi 2020) Note: all axes in the diagram are plotted in logspace

λ, of that NSE's damage state being exceeded is estimated. If this is inverted, it gives the return period, T_R, in years (Eq. 1). It is noted that in the capacity-based framework proposed by Sullivan et al. (2020), this NSE limit state capacity, C, is set as a deterministic values, whereas this study considers the uncertainty in this.

$$T_R = \frac{1}{\lambda} \tag{1}$$

Based on either λ or T_R, a rating system (e.g. A+, A, B, C etc.) may be defined to classify the NSE performance for a given structural typology and site location. The input requirements for this would therefore be:

- Site location and a suitable hazard model (e.g. SHARE model (Woessner et al. 2015));
- Structural typology to characterise its demand-intensity models required for that NSE, defined in terms of the intensity measure and demand parameter deemed most suitable;
- Fragility of non-structural element (e.g. PACT (FEMA 2012c) or experimental testing (Davies 2010));
- Decision framework to assign a risk rating.

Using such an approach illustrated in Fig. 5, its output would be the MAFE for a given NSE, structural typology and location, which can then be used to quantify and classify the risk of NSEs in a building. This would not be too different to the *Sismabonus* risk classification system for buildings recently introduced in Italy (Decreto Ministeriale 2017), but explicitly for NSEs that is specifically tailored based on the type of NSE risk. Furthermore, a map of the expected risk class of an NSE could also be generated and used by manufacturers to show the risk category of their product for different locations and building class in addition to their anticipated location along the building height, for example, which would be an invaluable tool for the industry from both a design and assessment perspective.

3.2 Computation of mean annual frequency of exceedance

The performance of an NSE can be quantified as the probability that the demand, D, exceeds the damage state capacity, C, of that NSE for a given intensity of shaking, s,

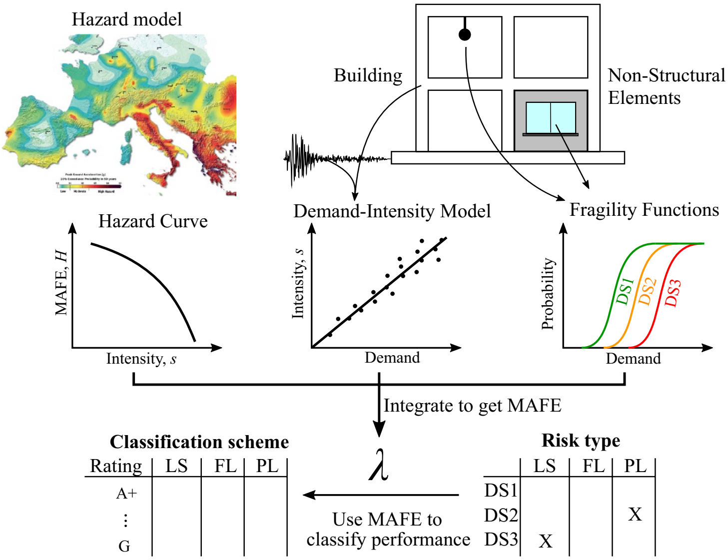

Fig. 5 Illustration of proposed risk quantification and classification for NSEs, where the seismic hazard illustrated has been adapted from the SHARE model (Woessner et al. 2015))

described by Eq. (1). The MAFE of ground shaking intensity s is described by the site hazard curve, $H(s)$, determined from PSHA. When integrated with Eq. (1) for all intensities, it gives the MAFE that the NSE capacity is exceeded, as per Eq. (2). The local rate of the demand exceeding the capacity is given by Eq. (3), which when integrated will result in an expression for the MAFE and Eq. (4). If the demand on an NSE being transmitted from a structure is described by a lognormal distribution with a median, η_D, and dispersion, β_D, and the capacity of the NSE (i.e. its fragility function) is defined in a similar manner (i.e. with η_C and β_C), then the MAFE is computed using Eq. (5), where the term $\Phi(\bullet)$ represents the cumulative distribution function of a normal distribution and the demand and capacity are assumed to be uncorrelated.

$$P[D > C|s] \tag{2}$$

$$\lambda = \int_0^{+\infty} P[D > C|s]\,|\mathrm{d}H(s)| \tag{3}$$

$$\mathrm{d}\lambda = P[D > C|s]\,|\mathrm{d}H(s)| \tag{4}$$

$$\lambda = \int_0^{+\infty} \frac{\mathrm{d}P[D > C|s]}{\mathrm{d}s} H(s)\mathrm{d}s \tag{5}$$

$$\lambda = \int_0^{+\infty} \frac{\mathrm{d}}{\mathrm{d}s}\left[1 - \Phi\left[\frac{\ln\eta_C - \ln\eta_D(s)}{\sqrt{\beta_D^2 + \beta_C^2}}\right]\right] H(s)\mathrm{d}s \tag{6}$$

Although the format of Eq. (6) appears rather theoretical, some assumptions can be made via demand-intensity models to facilitate a direct and closed-form implementation more suitable for NSE classification framework aimed towards practitioner. Similarly, the capacity parameters are simply the specific NSE fragility function determined from experimental testing, or similar, and the hazard information is identified from the available PSHA model. With these three pieces of information, the MAFE can be computed in a simple closed-form expression. This represents a step forward for the risk classification of NSEs, which have to date been focussed on qualitative prescription and secondary code prescription. The details of this for both drift-sensitive and acceleration-sensitive NSEs are described in the following subsections.

One of the first pieces of information required is the hazard model, which so far has been described as $H(s)$. This is an empirical output of PSHA in discrete points rather than a continuous distribution and typically needs to be fitted with an expression to simplify and avoid the need to numerically integrate to compute the MAFE. Numerous formulations for closed-form expressions have been proposed over the years. Vamvatsikos (2013) extended the power law by Sewell et al. (1991) to provide a quadratic fit in logspace (Eq. 7) and is used herein, where the coefficients k_0, k_1 and k_2 are fitted to the PSHA data.

$$H(s) = k_0 \exp\left(-k_1 \ln s - k_2 \ln^2 s\right) \tag{7}$$

3.3 Storey drift-sensitive elements

In the case of the drift-sensitive elements, the objective is to estimate the MAFE of a certain NSE damage state capacity, described by a fragility function with median η_C and dispersion β_C. In terms of demand, the median structural response of a building is predicted using a demand-intensity relationship, which can be represented as linear in logspace. This is described in Eq. (8) where θ_{max} represents the maximum peak storey drift (MPSD) along the building height in the direction of interest. The empirical coefficients m_θ and b_θ are determined from structural analysis (e.g. O'Reilly and Calvi 2020; Gaetani d'Aragona et al. 2019; Gaetani d'Aragona et al. 2020; Sullivan et al. 2021) or from empirical relationships depending on the characteristic of the building in question, such as those proposed by Orumiyehei and Sullivan (2020), for example, and s is the intensity measure. For infilled frames other the hand, a simple linear model in logspace may not be sufficient and a bilinear model such as the one described by O'Reilly and Monteiro (2019) could be easily adopted instead, with the same framework outlined herein equally applicable. It is recalled that the purpose of Eq. (8) is to predict the demand with increasing intensity and the coefficients represent a means with which this can be achieved, with varying degrees of accuracy characterised via dispersion β_D.

$$\theta_{max} \approx m_\theta s^{b_\theta} \tag{8}$$

It should be noted that using MPSD in this manner inherently assumes that: 1) the NSE is uniformly placed along the height the building (and this approach will signal the first exceedance); 2) that all NSEs of the same typology at a given storey level will be damaged to the same degree as they are characterised by the same fragility function and subject to the same level of demand; and 3) NSEs will be damaged by demand in one direction (i.e. gypsum partition positioned in the longitudinal and transverse direction of a building). For the first point, if a particular NSE is present at only one specific storey level, then the demand parameter should be the peak storey drift at that specific storey level and not the maximum value for all storeys along the height. Also in the case of torsional response, specifically tailored demand-intensity models could be easily developed using a numerical model that accounts for it to describe the demands at different sides of the building. The relationship is flexible and can be adapted for more specific situations when needed without any major problem. The purpose of the discussion here is to outline the fundamental steps involved as MPSD in systems with a linear log model is used as an example.

Using such a demand-intensity model, the closed-form expressions to compute the MAFE for MPSD, λ_{θ}, are given in Eq. (9) where the φ'_{θ} term is described by Eq. (10) (Vamvatsikos 2013). The dispersion terms β_{D} and β_{C} represent the uncertainty in the structural demand and the NSE capacity, respectively, and may consist of both aleatory and epistemic sources. These terms are left generic here and it is envisaged that all pertinent sources of uncertainty will be accounted for during their characterisation.

$$\lambda_{\theta} = \sqrt{\phi'_{\theta}} k_0^{1-\phi'_{\theta}} H\left(\left(\frac{\eta_C}{m_{\theta}}\right)^{\frac{1}{b_{\theta}}}\right)^{\phi'_{\theta}} \exp\left[\frac{k_1^2 \phi'_{\theta}}{2b_{\theta}^2}\left(\beta_D^2 + \beta_C^2\right)\right] \tag{9}$$

$$\phi'_{\theta} = \frac{1}{1 + \frac{2k_2}{b_{\theta}^2}\left(\beta_D^2 + \beta_C^2\right)} \tag{10}$$

3.4 Floor acceleration-sensitive elements

In the case of the acceleration-sensitive elements, the objective is again to estimate the MAFE of a certain NSE damage state, again described by a fragility function with median η_{C} and dispersion β_{C}. The maximum of the peak floor accelerations (MPFA), a_{max}, is a demand parameter typically used for acceleration-sensitive components. NSEs sensitive to MPFA generally cannot be separated into unique directions of response but rather the maximum acceleration experienced in any direction. In this case, the demand parameter would almost certainly need to be defined as the maximum component experienced in any horizontal direction, which is not necessarily one of the two orthogonal directions of the building. For this, some combinations like those discussed for drift demands previously should be considered. Furthermore, some acceleration-sensitive NSEs may be loosely correlated to PFA (e.g. flexible or multi-modal NSEs) and their damage may be better characterised by floor response spectra (e.g. Chalarca et al. 2020) but many rigid NSEs can be adequately assessed using PFA (Welch and Sullivan 2017). These situations are not dealt with further here but will be addressed in future applications as again, the purpose is to illustrate the application and the specific tailoring of the intensity measures or demand parameters is not elaborated.

Unlike MPSD, MPFA is a quantity that tends to saturate with increasing intensity as a result of the structure yielding. The result of this is that a single linear fit in log-space for the demand-intensity model is no longer sufficient over the entire range of structural response. To overcome this, O'Reilly and Monteiro (2019) recent developed a bilinear demand-intensity model described by Eq. (11) and shown in Fig. 6, where s_{lim} represents the intensity at which the structure is expected to yield. This allows the limit state exceedance rates of acceleration-sensitive NSEs located at any part of the building

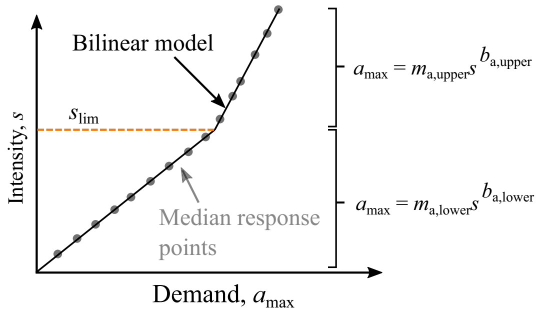

Fig. 6 Illustration of the bilinear demand-intensity utilised for floor accelerations, where the limiting intensity where the structure begins to yield is noted as s_{lim}

to also be quantified. Assuming first-mode dominated response, this limiting intensity may be estimated as the ratio between the base shear and modal mass if using spectral acceleration as an intensity measure. The coefficients $m_{\text{a,lower}}$, $m_{\text{a,upper}}$, $b_{\text{a,lower}}$ and $b_{\text{a,upper}}$ (Fig. 6) are again coefficients quantified from response analysis results (e.g. O'Reilly and Calvi 2020; Welch 2016; Gaetani d'Aragona et al. 2020; Perrone et al. 2020), or similar.

$$a_{max} \approx \begin{cases} m_{a,lower} s^{b_{a,lower}}, s < s_{lim} \\ m_{a,upper} s^{b_{a,upper}}, s \geq s_{lim} \end{cases} \tag{11}$$

As in the case of the drift-sensitive components, using MPFA in this way assumes that components are uniformly located at each floor and will be damaged simultaneously. For instances where this is not the case (i.e. a cooling tower located only at the roof), then the demand parameter should be switched to the demand at that specific level and not the maximum over the height. The same approach as that followed drift should be followed in cases where uniform damage for all NSEs is not reasonable to expect.

Using this bilinear demand-intensity model, the MAFE, λ_{a}, was derived in O'Reilly and Monteiro (2019) and described by Eq. (12).

$$\lambda_a = F_{lower}(s) G_{lower} + \left[1 - F_{upper}(s)\right] G_{upper} \tag{12}$$

where $F_{\text{lower}}(s)$ and $F_{\text{upper}}(s)$ are the lognormal cumulative density function values with corresponding mean values of μ_{lower} and μ_{upper} and standard deviations of σ_{lower} and σ_{upper}, respectively, (i.e. the command LOGNORM.DIST(s; μ; σ; TRUE) in MS Excel) which when using the respective coefficients in Eq. (11) and shown in Fig. 6 are described by Eqs. (13)–(20).

$$\mu_{lower} = \phi'_{a,lower} \left(\frac{\left(\ln \eta_C - \ln m_{a,lower}\right)}{b_{a,lower}} - \frac{k_1\left(\beta_D^2 + \beta_C^2\right)}{b_{a,lower}^2} \right) \tag{13}$$

$$\mu_{upper} = \phi'_{a,upper} \left(\frac{\left(\ln \eta_C - \ln m_{a,upper}\right)}{b_{a,upper}} - \frac{k_1\left(\beta_D^2 + \beta_C^2\right)}{b_{a,upper}^2} \right) \tag{14}$$

$$\sigma_{lower} = \frac{\left(\beta_D^2 + \beta_C^2\right)\sqrt{\phi'_{a,lower}}}{b_{a,lower}} \tag{15}$$

$$\sigma_{upper} = \frac{\left(\beta_D^2 + \beta_C^2\right)\sqrt{\phi'_{a,upper}}}{b_{a,upper}} \tag{16}$$

$$\phi'_{a,lower} = \frac{1}{1 + \frac{2k_2}{b_{a,lower}^2}\left(\beta_D^2 + \beta_C^2\right)} \tag{17}$$

$$\phi'_{a,upper} = \frac{1}{1 + \frac{2k_2}{b_{a,upper}^2}\left(\beta_D^2 + \beta_C^2\right)} \tag{18}$$

$$G_{lower} = \sqrt{\phi'_{a,lower}}k_0^{1-\phi'_{a,lower}} H\left(\left(\frac{\eta_C}{m_{a,lower}}\right)^{\frac{1}{b_{a,lower}}}\right)^{\phi'_{a,lower}} \exp\left[\frac{k_1^2\phi'_{a,lower}}{2b_{a,lower}^2}\left(\beta_D^2 + \beta_C^2\right)\right] \tag{19}$$

$$G_{upper} = \sqrt{\phi'_{a,upper}}k_0^{1-\phi'_{a,upper}} H\left(\left(\frac{\eta_C}{m_{a,upper}}\right)^{\frac{1}{b_{a,upper}}}\right)^{\phi'_{a,upper}} \exp\left[\frac{k_1^2\phi'_{a,upper}}{2b_{a,upper}^2}\left(\beta_D^2 + \beta_C^2\right)\right] \tag{20}$$

4 Development of an NSE classification framework

For NSEs, a tentative classification scheme is outlined using the performance metrics quantified in Sect. 3 (i.e. MAFE of specific damage states). This can be done by creating a scoring system based on the MAFE value computed in order to classify the performance into a letter-based system, as done with domestic appliances for their energy consumption in Europe (EU Council 1992), for example. This would be in line with the *Sismabonus* guidelines illustrated in Fig. 3 as they would give a letter-based rating, but would differ since *Sismabonus* considers the performance of a building as a whole and the overall monetary costs associated with its repair following earthquake damage and the life safety of the occupants due to loss of structural capacity. Instead, the classification scheme for NSEs will focus much more on mitigating the immediate impacts and consequences of earthquake shaking due to the failure of certain NSE elements on the building and its occupants. FEMA E-74 has already provided some kind of differentiation among NSEs and which type of risks they pose, which were summarised in Table 1. For each risk type, different kinds of acceptable MAFE or return periods of failure could be assigned. Utilising the methodology outlined in Sect. 3, the compliance or violation of these limits could be examined and an individual risk rating assigned to each NSE. Establishing these limits is clearly not an simple task and collaborative research would be needed to identify reasonable values, as in the case of acceptable fatality risk models to be used in structural design discussed by Sinković and Dolšek (2020), and future work through extensive case studies

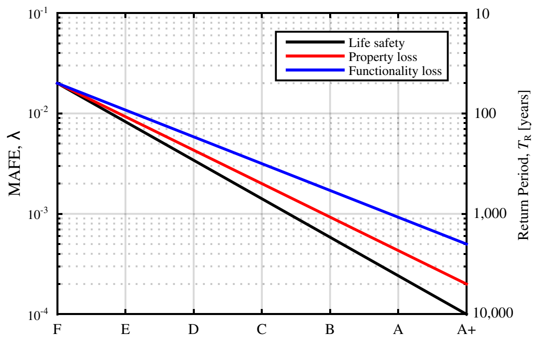

Fig. 7 Illustration of hypothetical risk classification system for NSEs based on the type of risk a certain damage state poses within a particular structure

and quantification of suitable limits would be needed. This kind of approach is argued to be a much more thorough and meaningful way to classify and rank the performance of NSEs compared to more typical demand/capacity ratios that current codes tend to use, also featured in the approach by Sullivan et al. (2020).

As a demonstrative exercise, Fig. 7 illustrates some sample values of acceptable failure rates for the three different types of risks identified in FEMA E-74. It is underlined that these values are not intended for immediate use but rather that illustrate what this framework could potentially look like once suitable values are established. Each risk type starts off by having a minimum protection return period, which in this case is set to 50 years considering the nominal life of typical constructions, and varies linearly in logspace up to different maximum levels of protection, although it doesn't necessarily need to be and the longer nominal life of other types of constructions may be considered. The A + rating for life safety has been tentatively set as 10^{-4} in order to correspond to the target collapse risk values used in risk-based seismic design of structures (e.g. Žižmond and Dolšek 2019; Shahnazaryan and O'Reilly 2021). Other limits and trends with respect to risk classes could be adopted and proposed once it is fully understood what the general risk for typical components in different buildings currently is in seismic regions. Based on these limits, a letter-based scheme could be developed to score the NSE being examined. Another task to perform for each NSE is to associate a risk type to each of the damage states, as shown in Fig. 5. For example, collapse of a heavy ceiling system may pose a life safety risk and should be treated as such. However, loss of a piping system that provides fresh water to a building would be considered a functionality risk, for example. Two works that have made progress in this regard are the FEMA P-58 guidelines and more recently the REDi rating system (Arup 2013) and could be integrated with the proposed framework.

5 Example application

A framework to estimate the MAFE of a certain damage state for both storey drift-sensitive and floor acceleration-sensitive NSEs has been outlined previously. In this section, an illustrative example of how this may be computed for a single structure is described.

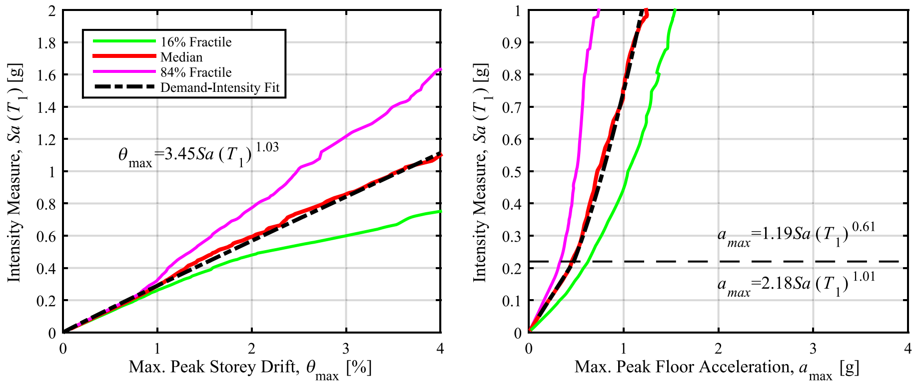

Fig. 8 Illustration of the IDA results of a 4 storey RC frame structure for both MPSD and MPFA, where the fitted demand-intensity models are also shown

The structure discussed herein is a 4 storey RC moment frame building taken from a previous study by Haselton and Deierlein (2007) and examined in detail in O'Reilly and Calvi (2020). It comprises a perimeter frame structure designed to form a ductile beam-sway mechanism following modern code provisions, therefore avoiding any unwanted mechanisms such as column or joint failure. No infill panels were considered in the design or numerical modelling of the structure. A numerical model of the structure was constructed in OpenSees (McKenna et al. 2010) using lumped plasticity element models described by Haselton et al. (2008) and second order geometry effects were considered via a P-Delta leaning column. Its dynamic behaviour was quantified using incremental dynamic analysis (IDA) and the results, characterised by the median, 16% and 84% fractiles, are plotted in Fig. 8. As shown, the seismic intensity measure, s, was chosen as the spectral acceleration at the first mode period of vibration of the structure, $Sa(T_1)$, although other could have been adopted. The demand-intensity model coefficients were found to be $m_\theta = 3.45$, $b_\theta = 1.03$, $m_{a,lower} = 2.18$, $m_{a,upper} = 1.19$, $b_{a,lower} = 1.01$, $b_{a,upper} = 0.61$ and the limiting intensity $s_{lim} = Sa_y(T_1) = 0.22$ g. The building is situated in a location whose seismic hazard is characterised via the coefficients $k_0 = 7\text{e-}4$, $k_1 = 2.0$ and $k_2 = 0.3$ and the corresponding hazard curve is plotted in Fig. 9.

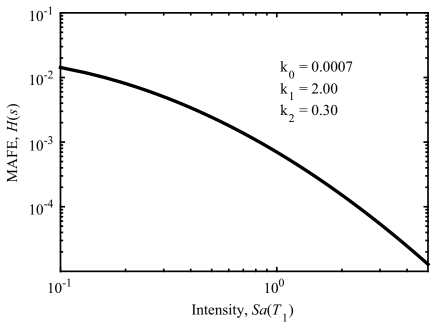

Fig. 9 Site hazard curve used to evaluate NSE risk

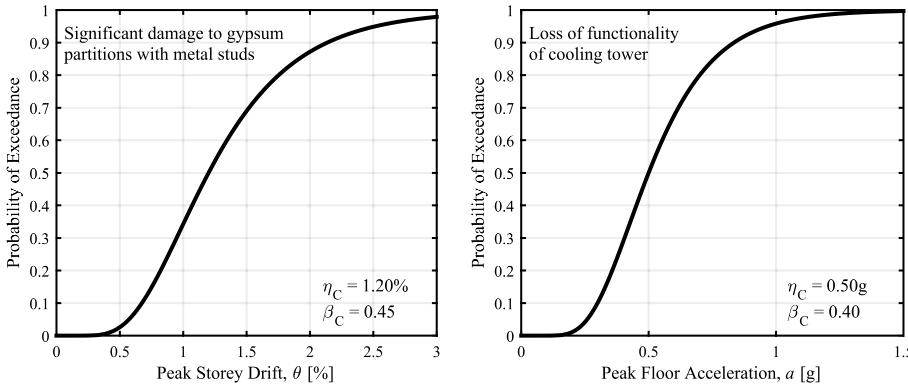

Fig. 10 Illustration of the fragility functions for the NSEs being evaluated: gypsum partition with metal studs (left) and cooling tower (right)

Table 2 Computation of the MAFE and return period of significant damage to the gypsum partitions with metal studs

Description	Reference	Source	Value (s)
Demand-intensity model	Equation (8)	Structural Analysis	$m_{\theta}=3.45$, $b_{\theta}=1.03$, $\beta_{D}=0.30$
Site hazard model	Equation (7)	PSHA	$k_0=7e\text{-}4$, $k_1=2.0$ and $k_2=0.3$
NSE fragility function	Figure 10	Experimental test data etc	$\eta_C=1.2\%$, $\beta_C=0.45$
MAFE	Equation (10)	Calculated	$\varphi'_{\theta}=0.86$
	Equation (8)*		$Sa(T_1)=0.36$ g
	Equation (7)		$H(Sa(T_1))=3.97e\text{-}3$
	Equation (9)		$\lambda_{\theta}=4.61e\text{-}3$
Return period	Equation (1)		$T_R=217$ years
Rating	Figure 7	Output	D

*Refers to the inverted form of Eq. (8)

To evaluate the performance of NSEs in this particular building, two cases were considered: a drift-sensitive and an acceleration sensitive NSE. For the first case, gypsum partitions with metal studs were considered and for the second case, a cooling tower was examined. The "significant damage" damage state of the partitions and the "loss of functionality" damage state of the cooling tower were analysed, which were both deemed "Functionality loss" risk types according to Fig. 7. The definitions of these two damage states and their corresponding fragility functions were taken from FEMA P-58-3 (FEMA 2012c) and are plotted in Fig. 10. With this information, the median capacity and dispersion of the NSEs' damage states were known quantities. Additionally, the hazard and demand-intensity model terms were known for the case study building, as described above. Following the expressions outlined in Sects. 3.3 and 3.4, the MAFE of the damage states was computed for the NSEs. These calculations are described in Tables 2 and 3 for both drift and acceleration-sensitive elements, respectively, where the tentative classification scheme plotted in Fig. 7 was also utilised to assign a risk class for demonstration.

From the results shown in Table 2 and *Refers to the inverted form of Eq. (11)

Table 3 Computation of the MAFE and return period of loss of functionality of a cooling tower

Description	Reference	Source	Value(s)
Demand-intensity model	Equation (11)	Structural analysis	$m_{a,lower}=2.18$, $m_{a,upper}=1.19$, $b_{a,lower}=1.01$, $b_{a,upper}=0.61$, $\beta_D=0.30$
Site hazard model	Equation (7)	PSHA	$k_0=7e\text{-}4$, $k_1=2.0$ and $k_2=0.3$
NSE fragility function	Figure 10	Experimental test data etc	$\eta_C=0.50$ g, $\beta_C=0.40$
MAFE	Equation (17, 18)	Calculated	$\varphi'_{a,lower}=0.87$, $\varphi'_{a,upper}=0.71$
	Equation (13, 14)		$\mu_{lower}=-1.70$, $\mu_{upper}=-1.36$
	Equation (15, 16)		$\sigma_{lower}=0.23$, $\sigma_{upper}=0.35$
	Equation (11)*		$Sa(T_1)_{lower}=0.23$ g, $Sa(T_1)_{upper}=0.24$ g
	Equation (7)		$H(Sa(T_1)_{lower})=6.83e\text{-}3$, $H(Sa(T_1)_{upper})=6.55e\text{-}3$
	Equation (19, 20)		$G_{lower}=7.30e\text{-}3$, $G_{upper}=7.58e\text{-}3$
			$F_{lower}=0.79$, $F_{upper}=0.33$
	Equation (12)		$\lambda_a=1.08e\text{-}2$
Return period	Equation (1)		$T_R=92$ years
Rating	Figure 7	Output	E

*Refers to the inverted form of Eq. (11)

Table 2, it can be seen that the MAFE of particular NSE damage states located in certain buildings can be computed in a relatively simple manner. One of the main advantages of expressing the performance this way is that NSEs can be evaluated simultaneously and their relative risks compared. Additionally, the level of risk of a certain damage state was evaluated and quantified in a probabilistic manner. This means that the uncertainties present in the characterisation of both NSE capacity and structural seismic behaviour can be effectively propagated and taken into account in decision-making, which is currently unaccounted for in other NSE classification frameworks. This represents a marked improvement to existing pass/fail methods of evaluation or quantifying NSE performance discussed in Sect. 2, or the approach of Sullivan et al. (2020). Furthermore, this kind of quantification allows the relative risk to be managed more efficiently through the systematic reduction of the different NSE element MAFE, depending on which are deemed more critical, allowing prioritisation schemes to be easily developed. Lastly, the simplified means with which this methodology was implemented means that alternative NSE retrofitting solutions (e.g. inclusion of isolators underneath the cooling tower) simply means that the fragility information needs to be updated and the impact on reducing the MAFE can be easily examined.

6 Potential usage of an NSE risk classification framework

The previous sections have highlighted a means to compute the MAFE of an NSE damage state in a building as part of tentative risk classification framework. This section provides some discussion on the uses of such an NSE risk classification framework in seismic design and assessment and the potential future applications of it. The scope is to make it clear to engineers and stakeholders involved in the design of NSEs why such an approach is worth pursuing, both on an individual and regional level.

6.1 Relevance in design, assessment and retrofitting

The direct consideration of NSE performance may allow for more suitable decisions to be made in the design of new buildings and the retrofitting of existing ones. In the case of new design, engineers will know the typology of building that they are designing. If they know what kind of performance they seek in order to comply with a rating system such as that plotted in Fig. 7, the level of resistance to storey drift or floor acceleration can be estimated, while at the same time accounting for the uncertainties in structural demand and NSE response. This way, the path followed to arrive at feasible NSE designs that conform to a risk-oriented performance classification scheme is apparent. For example, an engineer wishes to have an A-rated NSE system for life safety placed within the building. They wish to know what the drift limits that they need to respect are when designing the structure, which could be done as follows. Using the tentative classification system in Fig. 7, it can be seen that to have an A-rated system, its MAFE of the life safety risk should be at $\sim 2.4 \times 10^{-4}$ or lower. Knowing the site hazard, the drift capacity required for this MAFE can be identified by inverting Eq. (9), which is akin to following the green arrow backwards from left to right in Fig. 4 and used to verify that the structural performance, described via its demand-intensity model, will actually result in the desired NSE rating.

As previously mentioned, adapting such an approach in limiting the storey drifts or floor accelerations in design and assessments allows users to explicitly consider the uncertainty in the NSE and structural response in addition targeting a specified MAFE. These specific features are not integrated into the Sullivan et al. (2020) approach, for example, but the actual NSE capacities are considered in a more direct manner than existing code approaches.

More specifically, an NSE manufacturer could use the proposed risk classification system to explain differences in the seismic performance of products to a building owner using simple but meaningful language. For example, a manufacturer sells two types of base support systems for mechanical equipment to be used in a factory building: fixed anchors and a type of isolation system. The rigid anchors are cheap whereas the isolators seem rather expensive in comparison, making the former seem like a much more competitive option. Phrasing like increased resilience of the mechanical equipment on isolators to seismic shaking due to its reduced vulnerability may not be so convincing to a building owner that would typically not have an understanding of the effects of ground shaking on a structure or the meaning of fragility functions. However, if a manufacturer were simply able to 'tag' their isolator product as Class A for this building scenario (i.e. its typology and site location), whereas the rigid anchors would correspond to a Class D, the advantage would be much clearer to the building owner. Various assumptions would need to be made to arrive at the point where such a statement could be made but at least the general concept would be clear to the people making the decision.

Similar in the design of retrofits, where a building owner may discover that their building is not at a high risk of collapse but is prone to accumulating large economical losses characterised via EAL. O'Reilly and Sullivan (2018) demonstrated that in situations where collapse performance is not an issue, the retrofitting of NSEs can have a much bigger effect on reducing the EAL of building when compared to traditional structural interventions (which in some cases actually increased the EAL due to excessive strengthening and stiffening). In this regard, how are designers to know what kind of NSE retrofitting is required? If the proposed scheme were adopted and it were determined using this framework that the

NSEs should all be improved to at least a Class B performance, for example, the increased resistance to storey drift or floor acceleration required from each NSE could be computed. Following design methods such as Filiatrault et al. (2018) or Steneker et al. (2020), for example, the required improvements to the NSEs could then be established.

6.2 Implementation on a regional scale

In addition to focussing on a single structure like the example presented in Sect. 5, the proposed classification framework could also be extended to a regional scale. That is, if the hazard data for numerous locations in a given region are known and the demand-intensity model coefficients can be quantified for a range of building typologies, then the process outlined in Sect. 3 may be implemented. This way, the expected failure rate of a certain NSE across an entire region could be mapped. This may be considered an improvement on existing methods to assess seismic performance over entire regions, since the NSE performance is considered explicitly.

An example of regional assessment is using the OpenQuake engine (GEM 2016). In this type of regional study, much more attention is given to the economic losses associated with damage to buildings than the performance of individual buildings or their elements themselves. For example, to estimate the losses associated with a certain building typology, the approach adopted by OpenQuake utilises a library of fragility functions available as part of the risk modeller's toolkit (Silva et al. 2017) for a series of damage states that each have an associated repair cost, which describe the vulnerability of the stock (i.e. the expected losses versus intensity) (Silva et al. 2014; Villar-Vega et al. 2017; Calderon and Silva 2019). The level of ground shaking can be estimated from seismic hazard analysis and an exposure model mapping the various building typologies spatially is used. Using a library of fragility functions for each building's damage state and their associated losses, the expected losses for entire building portfolios can be estimated.

This is a fine method that works well for the assessment of entire regions and delivers on its goals to communicate risk on a larger scale to the relevant stakeholders. However, its extension to NSEs is a little problematic since it is not formulated in an overly convenient manner. It typically utilises a bilinear single degree of freedom (SDOF) oscillator to represent the structural response and the damage states are defined as a function of the displacement demand on the SDOF, as shown in Fig. 11 where N_{storey} is the number of storeys, h_{storey} is the typical storey height, θ_{global} is the global drift associated with the damage state, Γ is the modal participation factor and T_{y} is the first mode period of vibration. By considering many variations of this SDOF to account for uncertainties and running numerous dynamic analyses, the fragility functions for entire buildings are quantified. This approach may be able to reasonably estimate the displacement response demands on NSEs when the underlying assumptions of first mode-dominated response and temporally unvarying mode shapes still hold, but its output may still be considered approximate. No attention is given to acceleration-based damage states which are of undoubted importance in the assessment of NSEs. In summary, this type of global approach may not be particularly well-suited to assessing individual types of NSEs. The HAZUS approach (HAZUS 2003) is somewhat similar as it provides values for the different damage states for both drift-sensitive and acceleration-sensitive NSEs to be used in a simplified manner to compute the probabilities of each NSE damage state. While not formulated in the same way as Sect. 3, much of the information provided may be adopted here, although these are yet to be thoroughly evaluated through detailed studies.

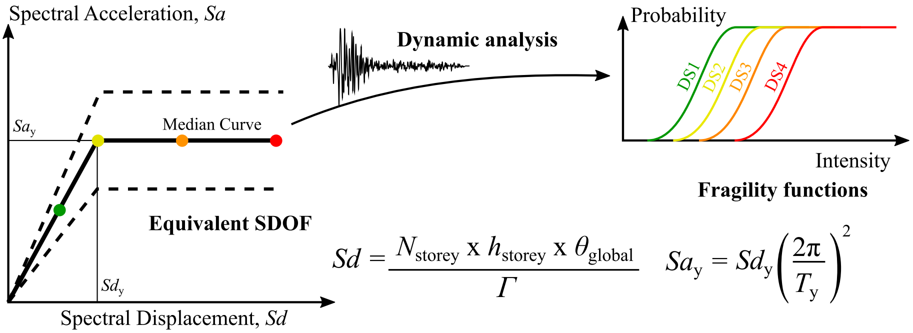

Fig. 11 Derivation of fragility functions using simplified equivalent SDOFs to represent the structural behaviour and characterise different damage states in regional assessment

Instead, what may be suggested is rather than providing sets of fragility and consequence functions for each building typology, its demand-intensity model could be provided. This way the procedure described in Sect. 3 could be directly implemented to compute the MAFE of an NSE. This would mean that a new library of demand-intensity models would be required for different typologies. This would not be exact but it may offer an improved estimate of performance over the current SDOF oscillator approach and offer much more flexibility as opposed to assigned global damage states that reflective the overall state of the structure rather than specific components within it. Some key parameters affecting these parameters (i.e. m_θ, b_θ, m_a and b_a) would need to be identified in addition to issues surrounding intensity measure and ground motion selection. For example, the lateral strength capacity and number of storeys are parameters that will certainly influence the fitting coefficients for drift-sensitive damage.

Figure 12 shows the variation of the m_θ and b_θ coefficients for non-ductile RC frame buildings in Italy (O'Reilly and Sullivan 2018), for example. The relationship between the parameter m_θ and the number of storeys is clear from Fig. 12, as is the relative insensitivity of the parameter b_θ and its closeness to 1.0, which has been reported by others (Cornell et al. 2002) for medium to long period structures and most recently examined by Orumiyehei and Sullivan (2020). Fitting predictive equations to these relationships would be necessary and would not be very dissimilar to the fitting of empirical period-height relationships (Crowley and Pinho 2004) as is currently done for regional studies. With respect to current regional approaches, the extension of the proposed methodology would contain some principal differences. First, empirical period-height relationships would no longer be needed in addition to the use of SDOF oscillators. The behaviour of building typologies characterised via full models, complete with strength and stiffness degradation, higher mode amplification and P-Delta effects accounted for, would all be captured within the demand-intensity model utilised. The need for building damage states as a function of global behaviour would also be removed and both drift and acceleration-sensitive NSEs could be directly considered. Again, these are potential aspects that may be investigated as part of future work but the purpose here is to highlight the potential usage of the NSE risk classification scheme presented here.

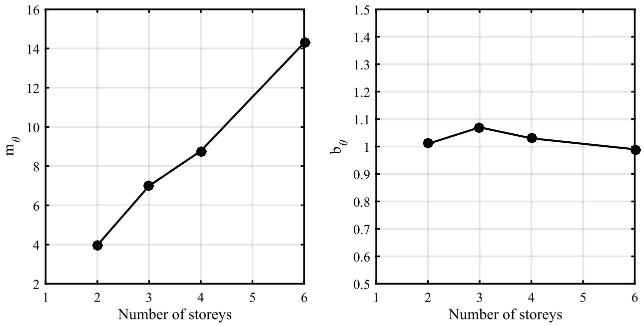

Fig. 12 Illustration of the relationship between the m_θ and b_θ coefficients for existing non-ductile RC frame buildings in Italy studied by O'Reilly and Sullivan (2018)

7 Summary

The development of a risk classification framework for non-structural elements (NSEs) has been described. Existing guidelines specifically related to the protection of NSEs were reviewed in terms of their ability to classify performance and quantitatively demonstrate improvement. A comprehensive but simple methodology to quantify the performance of both storey drift-sensitive and floor acceleration-sensitive NSEs was outlined whereby the mean annual frequency of exceeding (MAFE) a given damage state is determined. This utilises information from seismic hazard analysis, dynamic structural analysis and also NSE behaviour to characterise the performance in a consistent manner, while at the same time incorporating the uncertainties involved to be in line with modern performance-based earthquake engineering. A tentative classification scheme to rank the performance in a simplified manner similar to the existing energy rating system *CasaClima* or seismic risk for buildings *Sismabonus* used in Italy was proposed. An example implementation of the methodology was described for two kinds of NSE to illustrate its simplified nature and potential application. Lastly, the possible benefits and required future developments of using this proposed methodology for engineers and manufacturers were discussed in addition to its extension to a more regional level.

Author contribution **Gerard J. O'Reilly**: Conceptualisation; Validation; Supervision; Writing—Initial drafting, editing and reviewing. **Gian Michele Calvi**: Conceptualisation; Validation; Supervision; Writing—editing and reviewing.

Funding The work presented in this paper has been developed within the framework of the project "Dipartimenti di Eccellenza", funded by the Italian Ministry of Education, University and Research at IUSS Pavia.

Declarations

Conflicts of interest The author declares that they have no conflicts of interests.

References

Agenzia CasaClima. 2019. "Certificato Energetico CasaClima." 2019. https://www.agenziacasaclima.it/it/certificazione-edifici/certificato-energetico-casaclima-1281.html

Arup. 2013. "The Resilience-Based Earthquake Design Initiative (REDi^{TM}) Rating System. URL: ." https://www.arup.com/perspectives/publications/research/section/redi-rating-system?query=redi

ASCE 7–16. 2016. "Minimum Design Loads for Buildings and Other Structures." American Society of Civil Engineers. Reston, VA, USA

Calderon A, Silva V (2019) Probabilistic seismic vulnerability and loss assessment of the residential building stock in costa Rica. Bull Earthq Eng 17(3):1257–1284. https://doi.org/10.1007/s10518-018-0499-1

Calvi PM, Sullivan TJ (2014) Estimating floor Spectra in multiple degree of freedom systems. Earthq Struct 7(1):17–38. https://doi.org/10.12989/eas.2014.7.1.017

Calvi GM, Sullivan TJ, Welch DP (2014) A seismic performance classification framework to provide increased seismic resilience. In: Ansal A (ed) Perspectives on European earthquake engineering and seismology. Springer, pp 361–400. https://doi.org/10.1007/978-3-319-07118-3_11

Carofilis W, Clemett N, Gabbianelli G, O'Reilly GJ, Monteiro R (2021) Influence of parameter uncertainty in multi-criteria decision-making when identifying optimal retrofitting strategies for RC buildings. J Earthq Eng (Under Review)

Caruso M, Pinho R, Bianchi F, Cavalieri F, Lemmo MT (2021) Integrated economic and environmental building classification and optimal seismic vulnerability/energy efficiency retrofitting. Bull Earthq Eng. https://doi.org/10.1007/s10518-021-01101-4

CEN. 2004. "Eurocode 8: Design of Structures for Earthquake Resistance - Part 1: General Rules, Seismic Actions and Rules for Buildings (EN 1998–1:2004)." Comité Européen de Normalisation. Brussels, Belgium

Chalarca B, Filiatrault A, Perrone D (2020) Seismic demand on acceleration-sensitive nonstructural components in viscously damped braced frames. J Struct Eng 146(9):04020190. https://doi.org/10.1061/(ASCE)ST.1943-541X.0002770

Chock, G., I. Robertson, P. Nicholson, H. Brandes, E. Medley, P. Okubo, B. Hirshorn, et al. 2006. "Compilation of Observations of the October 15, 2006, Kiholo Bay (Mw 6.7) AAnd Mahukona (Mw 6.0) Earthquakes, Hawaii." Oakland, California

Clemett N, Carofilis W, O'Reilly GJ, Gabbianelli G, Monteiro R (2021) Optimal retrofitting of existing buildings considering environmental impact. Eng Struct (accepted)

Cornell CA, Krawinkler H (2000) Progress and challenges in seismic performance assessment. PEER Center News 3(2):1–2

Cornell CA, Jalayer F, Hamburger RO, Foutch DA (2002) Probabilistic basis for 2000 sac federal emergency management agency steel moment frame guidelines. J Struct Eng 128(4):526–533. https://doi.org/10.1061/(ASCE)0733-9445(2002)128:4(526)

Cosenza E, Del Vecchio C, Di Ludovico M, Dolce M, Moroni C, Prota A, Renzi E (2018) The italian guidelines for seismic risk classification of constructions: technical principles and validation. Bull Earthq Eng 16(12):5905–5935. https://doi.org/10.1007/s10518-018-0431-8

Council EU (1992) Council Directive 92/75/EEC of 22 September 1992 on the Indication by Labelling and Standard Product Information of the Consumption of Energy and Other Resources by Household Appliances. Belgium, Brussels

Crowley H, Pinho R (2004) Period-height relationship for existing european reinforced concrete buildings. J Earthquake Eng 8(Suppl1):93–119. https://doi.org/10.1080/13632460409350522

Davies RD (2010) Seismic evaluation parameterization and effect of light-frame steel studded gypsum partition walls. State University of New York at Buffalo, NY

Eads L, Miranda E, Lignos DG (2015) Average spectral acceleration as an intensity measure for collapse risk assessment. Earthquake Eng Struct Dynam 44(12):2057–2073. https://doi.org/10.1002/eqe.2575

Eads L, Miranda E, Lignos D (2016) Spectral shape metrics and structural collapse potential. Earthquake Eng Struct Dynam 45(10):1643–1659. https://doi.org/10.1002/eqe.2739

Ioannou, Ioanna, Randolph Borg, Viviana Novelli, José Melo, David Alexander, Indranil Kongar, Enrica Verrucci, Bryan Cahill, and Tiziana Rossetto. 2012. "The 29th May 2012 Emilia Romagna Earthquake." EPI-FO-290512. London, UK. http://www.ucl.ac.uk/~ucestor/research-earthquake/EPICentre_Report_EPI-FO-290512.pdf

FEMA. 2012a. "FEMA E-74: Reducing the Risks of Nonstructural Earthquake Damage—A Practical Guide." FEMA E-74. Washington, DC, USA

FEMA (2012b) FEMA P58-1: Seismic Performance Assessment of Buildings Performance Assessment Calculation Tool (PACT), vol 1. Washington, DC

FEMA (2012c) FEMA P58-3: Seismic Performance Assessment of Buildings Performance Assessment Calculation Tool (PACT), vol 3. Washington, DC
Filiatrault A, Sullivan T (2014) Performance-based seismic design of nonstructural building components: the next frontier of earthquake engineering. Earthq Eng Eng Vib 13(1):17–46. https://doi.org/10.1007/s11803-014-0238-9
Filiatrault, A., C. M. Uang, B. Folz, C. Christopoulos, and K. Gatto. 2001. "Reconnaissance Report of the February 28, 2001 Nisqually (Seattle-Olympia) Earthquake." Structural Systems Research Project Report No. SSRP-2000/15,. San Diego, La Jolla, CA
Filiatrault A, Perrone D, Merino RJ, Calvi GM (2018) Performance-based seismic design of nonstructural building elements. J Earthq Eng. https://doi.org/10.1080/13632469.2018.1512910
Fischer, Erica. 2014. "Learning from Earthquakes: 2014 Napa Valley Earthquake Reconnaissance Report."
Gaetani d'Aragona M, Polese M, Cosenza E, Prota A (2019) Simplified assessment of maximum interstory Drift for RC buildings with irregular infills distribution along the height. Bull Earthq Eng 17(2):707–736. https://doi.org/10.1007/s10518-018-0473-y
Gaetani d'Aragona M, Polese M, Prota A (2020) Stick-IT: a simplified model for rapid estimation of IDR and PFA for existing low-rise symmetric infilled RC building typologies. Eng Struct 223(November):111182. https://doi.org/10.1016/j.engstruct.2020.111182
GEM. 2016. The OpenQuake-engine User Manual. Global Earthquake Model (GEM) Technical Report 2016-03, pp 189. https://doi.org/10.13117/GEM.OPENQUAKE.MAN.ENGINE.1.9/01
Gupta, A., and B. M. McDonald. 2008. "Performance of Building Structures during the October 15, 2006 Hawaii Earthquake." In: 14th World Conference on Earthquake Engineering. Beijing, China
Haselton, Curt B., C. A. Goulet, J. Mitrani Reiser, James L. Beck, Gregory G. Deierlein, K. A. Porter, J. P. Stewart, and E. Taciroglu. 2007. "An Assessment to Benchmark the Seismic Performance of a Code-Conforming Reinforced Concrete Moment-Frame Building." PEER Report 2007/12
Haselton, C. B., A. B. Liel, S. Taylor Lange, and G. G. Deierlein. 2008. "Beam-Column Element Model Calibrated for Predicting Flexural Response Leading to Global Collapse of RC Frame Buildings." PEER Report 2007/03
HAZUS (2003) Multi-hazard loss estimation methodology —earthquake model. DC, USA, Washington
Iervolino I, Manfredi G (2008) "A Review of ground motion record selection strategies for dynamic structural analysis. Modern Test Tech Struct Anal. https://doi.org/10.1007/978-3-211-09445-7_3
Kallioras S, Correia AA, Graziotti F, Penna A, Magenes G (2020) Collapse shake-table testing of a clay-URM building with chimneys. Bull Earthq Eng 18(3):1009–1048. https://doi.org/10.1007/s10518-019-00730-0
Kohrangi M, Bazzurro P, Vamvatsikos D (2016) Vector and scalar IMs in structural response estimation: part II—building demand assessment. Earthq Spectra 32(3):1525–1543. https://doi.org/10.1193/053115EQS081M
Kohrangi M, Bazzurro P, Vamvatsikos D, Spillatura A (2017) Conditional spectrum-based ground motion record selection using average spectral acceleration. Earthquake Eng Struct Dynam 46(10):1667–1685. https://doi.org/10.1002/eqe.2876
LazarSinković N, Dolšek M (2020) Fatality risk and its application to the seismic performance assessment of a building. Eng Struct 205(February):110108. https://doi.org/10.1016/j.engstruct.2019.110108
McKenna F, Scott MH, Fenves GL (2010) Nonlinear finite-element analysis software architecture using object composition. J Comput Civ Eng 24(1):95–107. https://doi.org/10.1061/(ASCE)CP.1943-5487.0000002
Decreto Ministeriale. 2017. "Linee Guida per La Classificazione Del Rischio Sismico Delle Costruzioni - 58/2017." Il Ministero Delle Infrastrutture e Dei Trasporti. Rome, Italy
Ministeriale D (2020) Misure Urgenti in Materia Di Salute, Sostegno Al Lavoro e All'economia, Nonche' Di Politiche Sociali Connesse All'emergenza Epidemiologica Da COVID-19 - 34/2020. Italy, Rome
Mohamed H, RomÃo X (2021) Robust calibration of macro-models for the in-plane behavior of masonry Infilled RC frames. J Earthquake Eng 25(3):407–433. https://doi.org/10.1080/13632469.2018.1517703
NZS 1170.5:2004. 2004. "Structural Design Actions Part 5: Earthquake Actions—New Zealand." NZS 1170.5:2004. Wellington, New Zealand
O'Reilly GJ (2021) Limitations of Sa(T1) as an intensity measure when assessing non-ductile infilled RC frame structures. Bull Earthq Eng 19(6):2389–2417. https://doi.org/10.1007/s10518-021-01071-7
O'Reilly GJ, Calvi GM (2020) Quantifying Seismic risk in structures via simplified demand-intensity models. Bull Earthq Eng 18(5):2003–2022. https://doi.org/10.1007/s10518-019-00776-0
O'Reilly GJ, Monteiro R (2019) Probabilistic models for structures with bilinear demand-intensity relationships. Earthquake Eng Struct Dynam 48(2):253–268. https://doi.org/10.1002/eqe.3135
O'Reilly GJ, Sullivan TJ (2018) Probabilistic seismic assessment and retrofit considerations for Italian RC frame buildings. Bull Earthq Eng 16(3):1447–1485. https://doi.org/10.1007/s10518-017-0257-9

O'Reilly GJ, Perrone D, Fox M, Monteiro R, Filiatrault A (2018) Seismic assessment and loss estimation of existing school buildings in Italy. Eng Struct 168(August):142–162. https://doi.org/10.1016/j.engstruct.2018.04.056

Orumiyehei A, Sullivan TJ (2020) Quantifying the likelihood of exceeding a limit state via the displacement-based assessment approach. J Earthquake Eng. https://doi.org/10.1080/13632469.2020.1828200

Passoni C, Marini A, Belleri A, Menna C (2021) Redefining the concept of sustainable renovation of buildings: state of the art and an LCT-based design framework. Sustain Cities Soc 64(January):102519. https://doi.org/10.1016/j.scs.2020.102519

Perrone D, Calvi PM, Nascimbene R, Fischer EC, Magliulo G (2018) Seismic performance of non-structural elements during the 2016 central italy earthquake. Bull Earthq Eng. https://doi.org/10.1007/s10518-018-0361-5

Perrone D, Brunesi E, Filiatrault A, Nascimbene R (2020) Probabilistic estimation of floor response spectra in masonry infilled reinforced concrete building portfolio. Eng Struct 202:109842. https://doi.org/10.1016/j.engstruct.2019.109842

Polese, Maria, Marco Gaetani d'Aragona, Marco Di Ludovico, and Andrea Prota. 2019. "Simplified Assessment of Seismic Performance for RC Building Classes towards Preliminary Applications of SISMABONUS Incentive at the Community Scale." In: 13th International Conference on Applications of Statistics and Probability in Civil Engineering (ICASP13). Seoul, South Korea. https://doi.org/10.22725/ICASP13.340

Recommendations for Improved Seismic Performance of Nonstructural Components." Gaithersburg, MD. https://doi.org/10.6028/NIST.GCR.18-917-43

Ricci P, De Luca F, Verderame GM (2011) 6th April 2009 L'Aquila Earthquake, Italy: reinforced concrete building performance. Bull Earthq Eng 9(1):285–305. https://doi.org/10.1007/s10518-010-9204-8

Sewell RT, Toro GR, McGuire RK (1991) Impact of ground motion characterisation on conservatism and variability in Seismic Risk Estimates—NUREG/CR-6467. Washingtion, DC

Shahnazaryan D, O'Reilly GJ (2021) Integrating expected loss and collapse risk in performance-based seismic design of structures. Bull Earthq Eng 19(2):987–1025. https://doi.org/10.1007/s10518-020-01003-x

Silva V, Crowley H, Varum H, Pinho R, Sousa L (2014) Investigation of the characteristics of portuguese regular moment-frame rc buildings and development of a vulnerability model. Bull Earthq Eng 13(5):1455–1490. https://doi.org/10.1007/s10518-014-9669-y

Silva, Vitor, Chiara Casotto, Dimitrios Vamvatsikos, Anirudh S Rao, and Mabe Villar Vega. 2017. "Presentation of the Risk Modeller's Toolkit, the Open–Source Software for Vulnerability Assessment of the Global Earthquake Model." In: 16th World Conference on Earthquake Engineering. Santiago, Chile

Steneker P, Filiatrault A, Wiebe L, Konstantinidis D (2020) Integrated structural-nonstructural performance-based seismic design and retrofit optimization of buildings. J Struct Eng 146(8):04020141. https://doi.org/10.1061/(ASCE)ST.1943-541X.0002680

Sullivan TJ, Dhakal RP, Stanway J (2020) A framework for the seismic rating of non-structural elements in buildings. In: 17th world conference on earthquake engineering. Sendai, Japan

Sullivan TJ, Saborio-Romano D, O'Reilly GJ, Welch DP, Landi L (2021) Simplified pushover analysis of moment resisting frame structures. J Earthquake Eng 25(4):621–648. https://doi.org/10.1080/13632469.2018.1528911

Taghavi, Shahram, and Eduardo Miranda. 2003. "Response Assessment of Nonstructural Building Elements." PEER Report 2003/05

Tomassetti U, Correia AA, Graziotti F, Penna A (2019) Seismic vulnerability of roof systems combining URM gable walls and timber diaphragms. Earthq Eng Struct Dynam. https://doi.org/10.1002/eqe.3187

Vamvatsikos D (2013) Derivation of new SAC/FEMA performance evaluation solutions with second-order hazard approximation. Earthquake Eng Struct Dynam 42(8):1171–1188. https://doi.org/10.1002/eqe.2265

Vamvatsikos D, Kazantzi AK, Aschheim MA (2016) Performance-based seismic design: avant-garde and code-compatible approaches. ASCE-ASME J Risk Uncertain Eng Syste Part a Civil Eng 2(2):C4015008. https://doi.org/10.1061/AJRUA6.0000853

Villar-Vega M, Silva V, Crowley H, Yepes C, Tarque N, Acevedo AB, Hube MA, Gustavo CD, María HS (2017) Development of a fragility model for the residential building stock in South America. Earthq Spectra 33(2):581–604. https://doi.org/10.1193/010716EQS005M

Welch, David P. 2016. "Non-Structural Element Considerations for Contemporary Performance-Based Earthquake Engineering." Pavia, Italy: PhD Thesis, IUSS Pavia

Welch, David P., and Timothy John Sullivan. 2017. “Illustrating a New Possibility for the Estimation of Floor Spectra in Nonlinear Multi-Degree of Freedom Systems.” In: 16th World Conference on Earthquake Engineering. Santiago, Chile

Woessner J, Laurentiu D, Giardini D, Crowley H, Cotton F, Grünthal G, Valensise G et al (2015) The 2013 European seismic hazard model: key components and results. Bull Earthq Eng 13(12):3553–3596. https://doi.org/10.1007/s10518-015-9795-1

Žižmond J, Dolšek M (2019) Formulation of risk-targeted seismic action for the force-based seismic design of structures. Earthquake Eng Struct Dynam 48(12):1406–1428. https://doi.org/10.1002/eqe.3206