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SENSITIVITY OF PROBABILISTIC REGIONAL SEISMIC LOSS TO HAZARD AND VULNERABILITY MODELLING OPTIONS

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ABSTRACT

This paper investigates the sensitivity of probabilistic regional seismic loss assessment of reinforced concrete (RC) buildings to different ground motion intensity and vulnerability modelling options. To this aim, several variations of the loss assessment framework are tested to determine their effect in the resultant damage prediction, and the consequent loss estimation. Specifically, this study systematically assesses the sensitivity of losses to (1) ground motion intensity characterization (through selection of different types of intensity measures - IMs); and (2) assumptions in structural analysis (simplified nonlinear static-based procedures versus advanced nonlinear time-history analysis).

A synthetic portfolio of RC structures located in Avellino, Southern Italy, is chosen as a case-study. A site-specific hazard analysis is carried out, allowing the calculation of ground shaking footprints for different types of IMs, including conventional and advanced IM representations. Three regular 4-storey, 4-bay RC bare frame buildings, corresponding to three distinct vulnerability classes, compose the exposure database. Seismic fragility and vulnerability functions are derived for the considered building typologies, and finally, economic earthquake losses are estimated for all tested ground motion IMs and fragility derivation methods.

Results from this study highlight that the choice of the analysis method type has a significant impact in the overall loss estimation, as the proposed (more advanced) procedure is capable of capturing the building response and corresponding damage states more accurately. The choice of the IM type is also of high importance within the loss estimation process, due to the ability of advanced IMs to capture more information than standard IM, e.g. in terms of spectral shape.

Keywords: Seismic risk; Fragility curves; Seismic hazard; FRACAS; Portfolio loss

1. INTRODUCTION

Performance-based earthquake engineering (PBEE) aims to enable risk-informed decision-making (e.g., Deierlein et al. 2003). Over the past decade, several researchers have developed methods to extend PBEE to quantify the risks of earthquake-induced losses for a region or geographically distributed portfolio of buildings and/or critical infrastructure (e.g., lifelines). These methods predict the probability of exceeding any given level of loss to allow stakeholders, such as insurers and policy-makers, to make risk-informed decisions when investing in risk reduction and resilience-increasing solutions. However, the estimation of these losses is not straightforward due to the various sources of uncertainty affecting the process, including: uncertainties in the characteristics of future earthquakes (e.g., occurrence and source features), the characteristics of ground shaking at different sites, the building response and capacity, the fragility of building components, and the costs of repairing damage, among others (e.g., DeBock and Liel 2015). Although several studies have attempted to explore the effect of spatial correlations of ground motion and variability in exposure model within seismic loss framework (e.g., DeBock and Liel 2015; Weatherill et al. 2015), limited research has been conducted on the effect of the IM selection (Kohrangi et al. 2017) and structural analysis choices.

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The aim of this study is to assess, through simple yet realistic examples, the sensitivity of probabilistic regional seismic losses to different ground motion intensity and vulnerability modelling options. In addition, this paper also provides the fragility functions of typical mid-rise RC buildings that can be found in the Italian region.

2. PROBABILISTIC SEISMIC LOSS ASSESSMENT METHODOLOGY

The probabilistic loss assessment of spatially distributed building portfolios requires the definition of hazard, exposure and fragility/vulnerability components. The following subsection address the methodology used in the probabilistic assessment of hazard and fragility/vulnerability. The dataset of exposed buildings investigated in this study is further presented in section 3.

2.1 Hazard assessment

2.1.1 Spatial correlation and spectral cross correlation modelling

Previous studies have demonstrated that strong ground motion footprints of different IMs for a specific seismic event are spatially correlated (e.g. Boore et al. 2003; Sokolov et al. 2010), and also that the distances where substantial correlations may occur are usually greater for the long period characteristics of the ground motion (e.g., Jayaram and Baker 2009). Based on these points, several researchers attempted to create spatial cross-correlation models to characterize the correlation of ground motion IMs footprints (e.g., Goda and Atkinson 2009; Loth and Baker 2013). The inclusion of such models into the seismic risk framework for building portfolios has a significant impact, resulting in some cases an increase in the loss estimates, compared to the estimates obtained when no correlation or cross-correlation models are used. More in general, considering correlation allows one to capture more extreme loss cases, both higher and lower than average.

When dealing with heterogenous building portfolios where different IMs may be used in the fragility assessment of different building classes, the spatial cross-correlation of these IMs is typically considered in the simulation of correlated random variates assuming a joint normal distribution (Weatherill et al. 2015).
Existing ground motion predicting equations (GMPEs) account for the uncertainty in the estimation of IMs via the aleatory variability terms incorporated in the generic GMPE function, as shown below:

$$\log(IM_{ij}) = f(M_i, R_{ij}, \theta_{ij}) + \tau v_i + \sigma\varepsilon_{ij} \tag{1}$$

where IM_{ij} is the ground motion intensity value at a site j, which is situated at a distance R_{ij} from the seismic source, for an event of magnitude M_i; θ_{ij} is the parameter associated with the faulting types, site classes and other predictors (e.g., basin effect, etc.); and τv_i and $\sigma\varepsilon_{ij}$ components represent the inter-event and intra-event variability of the GMPE model respectively. v_i and ε_{ij} are in most cases assumed to be independent, normally distributed with zero mean and standard deviation τ and σ, respectively.

As discussed above, it is now well-established in the literature that the intra-residual component is spatially correlated, i.e. the coefficient of correlation, $\rho_h(T)$, of the intra-residuals at two different locations is a function of the separation distance, h, between the two locations. An exponential functional form is used in most spatial correlation models (e.g., Goda and Atkinson 2009; Jayaram and Baker 2009), as shown below:

$$\rho_h(T) = \exp\left[a(T)h^{b(T)}\right] \tag{2}$$

Where $\alpha(T)$ and $b(T)$ are period-dependent model parameters associated with the attenuation level of the spatial correlation with distance. Additionally, the intra-residual term is also known to be cross-correlated; the coefficient of correlation of the intra-residual term of spectral acceleration at two different periods at the same site will decay as the period difference increases (Baker and Cornell 2006; Baker and Jayaram 2008).

The standard decomposition method is used here to simulate the spatially correlated random fields of GMPE residual values that are not conditioned to any observation. The multivariate Gaussian distributed random field, $\mathbf{Y}$, which is a vector consisting of a set of ground motion residual values for N sites, is assumed to follow a joint log-normal distribution, and is defined as:

$$\mathbf{Y} = \mu + \mathbf{LZ} \tag{3}$$

In Equation (3), μ is a zero N-element vector, $\mathbf{Z}$ is a vector of independent Gaussian distributed random variates, and $\mathbf{L}$ is the lower triangular matrix as obtained from Cholesky decomposition, where $\mathbf{C}$ is the N by N positive-definite correlation matrix:

$$\mathbf{C} = \mathbf{LL}^T = \begin{bmatrix} 1 & \rho(h_{1,2}) & \cdots & \rho(h_{1,N}) \\ & 1 & \cdots & \rho(h_{2,N}) \\ & & \ddots & \vdots \\ sym & & & 1 \end{bmatrix} \tag{4}$$

In Equation (4), $\rho(h_{i,j})$ represents the coefficient of correlation of residuals at two locations separated by a distance $h_{i,j}$. For the case of homogenous spatially distributed building portfolios, where a single IM is used (e.g. PGA or Spectral acceleration at a single period), one can use Equation 3 directly to determine the IM spatial correlation at different locations. Next, the logarithmic IM is computed as shown in Equation 1, i.e., the residuals are multiplied with the aleatory variability term, as obtained from the GMPE and added to the GMPE expected value. In the case of heterogenous building portfolios, the aforementioned approach becomes more complex, as the spatial cross-correlation structure needs to be maintained while co-simulating the Gaussian random fields of IMs. To this aim, a number of spatial cross-correlation approaches have been proposed in the existing literature, such as Oliver 2003; Iervolino et al. 2010; Loth and Baker 2013. A thorough review of such approaches can be found in Weatherill et al. (2015). This study showed that the methodologies which account for spatial cross-correlation and inter-event residual correlation are more favorable to calculate accurately the losses for a heterogenous building portfolio. For this reason, the full-block cross-correlation (FBCC) methodology (Oliver 2003) that incorporates both spatial cross-correlation and inter-event residual correlation is used herein to represent the state-of-the-art approach. In addition to the advantages discussed above, the adoption of this approach also provides the flexibility to utilize correlation models that may encompass regional characteristics of the site of interest. The FBCC is combined with the period-dependent spatial correlation model proposed by Jayaram and Baker (2009). With regard to the spectral correlation approach, Goda and Atkinson (2009) model is used.

2.1.2 Considered IMs

In most building codes and modern catastrophe risk models, seismic hazard is defined in terms of ground-motion IMs. An IM is a scalar ground motion parameter that is considered to be representative of the earthquake damage potential with respect to the specific structure. Some of the most commonly used IMs include peak ground responses (PGA, PGV and PGD) and spectral acceleration at the first period, $S_a(T_1)$, for 5% damping. However, these standard IMs poorly predict the structural response of mid- to high-rise moment resisting frames (MRFs), although $S_a(T_1)$ can sufficiently capture the elastic behavior of first-mode dominated structures, characterized by low-to-moderate fundamental periods (Shome et al. 1998). An alternative to standard IMs, are IMs that can account for an enhanced range of periods, hereafter advanced IMs. Several studies showed that advanced IMs perform better over a range of criteria when selecting an optimal IM, therefore are more suitable for predicting the seismic response of structure, and eventually the seismic fragility and loss (e.g. Ebrahimian et al. 2015; Kazantzi and Vamvatsikos 2015). In addition, Minas and Galasso (2018) performed a comparative study to rank a number scalar IMs (including standard and advanced IMs) based on their performance in terms of efficiency, sufficiency/relative sufficiency and hazard computability. This study showed that the spectral-shape proxy I_{Np}, proposed by Bojórquez and Iervolino (2011) is considered to be the optimal IMs for fragility analysis of mid-rise RC buildings, with $S_a(T_1)$, being the most favorable standard IM. Both $S_a(T_1)$ and I_{Np} are utilized herein as the reference representations for standard and advanced IMs respectively. Specifically, I_{Np} is based on $S_a(T_1)$ and the parameter N_p, defined as:

$$I_{Np} = S_a(T_1) N_p^{a} \quad , \quad N_p = \frac{S_{a,avg}(T_1,...,T_N)}{S_a(T_1)} = \frac{\left[\prod_i^N S_a(T_i)\right]^{1/N}}{S_a(T_1)} \tag{5}$$

where T_N corresponds to the maximum period of interest and lays within a range of 2 and $2.5T_1$, as suggested by the authors α parameter is assumed to be $\alpha = 0.4$. However, a recent study by the authors showed that α parameter should be calibrated in accordance to the vulnerability of the building of interest (Minas and Galasso, 2018). Table 1 below shows the α parameter values calibrated for mid-rise RC buildings of different vulnerability classes.

Table 1. Calibrated parameter a for mid-rise RC buildings of different vulnerability classes

Vulnerability Class	**α-parameter**
Pre-code	0.7
Low-code	0.8
Special-code	0.2

In this study T_1 is assumed to be equal to 1 second for all the considered buildings, as this is the representative natural period for mid-rise building class in catastrophe modelling (also for the case of $S_a(T_1)$), T_N is assumed to be equal to $2T_1$ (i.e. 2 seconds).

2.2 Fragility and Vulnerability assessment

2.2.1. Analysis methods

Another essential step towards the development of an analytical fragility function and consequently a vulnerability function is the choice of the analysis method to calculate the structural response. Although conventional linear elastic analysis is appropriate for design purposes, inelastic approaches are preferred for fragility and loss assessment as they better depict the inelastic seismic behavior of structures, particularly under severe ground motions, and therefore, are characterized by lower uncertainty. However, the degree of uncertainty within inelastic analysis methods varies substantially, depending on the level of detailing of the structural model and the representation of the hazard. Two main types of inelastic analysis exist and are widely used in practice, namely nonlinear dynamic analysis (NDA) and nonlinear static procedures (NSPs).

NDA is considered the most reliable and accurate method to determine the seismic performance of structures. This method implements very detailed structural models, and uses a series of natural or simulated ground motions of varying intensities as an input. As a result, the dynamic behavior is in theory more realistically represented and the seismic response of a structure subjected to ground motion time history is more efficiently estimated. However, the complexity of NDA is translated into a significant increase in the computation demand and can sometimes lead to convergence difficulties. The high computation effort and the convergence issues associated with NDA, rendered this method to be impractical for portfolio of structures (as in the case of catastrophe models) and highlighted the need of using more rapid and simple approaches, such as NSPs. NSPs rely on simplifying assumptions, such that the response of a complex multi-degree-of-freedom (MDOF) system is represented by an equivalent single-degree-of-freedom (SDOF) system, and that elastic response spectra are used to characterize the seismic demand. Other limitations associated with most NSPs are that they fail to identify alternative failure mechanisms and they do not account for record-to-record variability (Fragiadakis et al. 2014).

In contrast to other NSPs, the recently proposed FRACAS method (Rossetto et al. 2016) is a variant of capacity spectrum method (CSM) which utilizes natural (unsmoothed) spectra in order to account for record-to-record variability in the evaluation of performance of a single or population of structures. Previous studies showed that FRACAS provides a reasonable estimate of the seismic performance predicted by NDA for mid-rise RC buildings (average error is less around 25% across the considered models), particularly when nonlinear structural response is considered (average error is around 15% across the considered models). NDA and FRACAS are the analysis methods tested herein.

2.2.2. Development of fragility functions

There are a number of procedures available in the literature for estimating the parameters to construct analytical fragility functions. Fragility functions are defined as continuous relationships between the ground motion IM and the probability that a specified asset will reach or exceed predefined damage states (DS) in accordance to their structural response. The structural response of buildings, as obtained from the chosen analysis method discussed in the previous section, is expressed in terms of engineering demand parameters (EDPs), e.g., maximum interstorey drift ratio, denoted as MIDR, and compared to EDP thresholds associated with each DS. Statistical methods are employed to characterize this response probabilistically as a function of seismic hazard, and build the conditional distribution of an EDP given the IM. One commonly used approach is the Cloud analysis (e.g., Bazzurro et al. 1998), where the structure is subjected to a series of ground motion time-histories associated with respective IM values in order to estimate the associated seismic response. The resultant peak values of EDP for given IM levels form a scatter of points, the so-called “cloud”. Least-squares regression is then used to fit a simple model to the cloud of data points. The EDP is considered to vary as a power-law of the form αIM^{b} such that, after taking logarithms, the relationship can be expressed as:

$$\ln(EDP) = \ln(\alpha) + b\ln(IM) + e \tag{6}$$

where EDP is the conditional median of the demand given the IM, a , b are the parameters of the regression, and e is a zero mean random variable representing the variability of $\ln(EDP)$ given the IM. The cloud approach has the advantages of simplicity and rapidity over alternative fragility assessment methods, but also has some restrictions. Firstly, the assumption that the relationship between IM and EDP is represented by a linear model in the log space may be valid for a short range of IM and EDP combinations but not for the entire cloud response space. Additionally, the cloud method requires a large number of earthquake records to be used as an input, and the accuracy of the approach is highly dependent on the record selection process followed (e.g., Jalayer et al. 2014).

The cloud analysis is utilized in this study for the generation of fragility functions. Other well-established methods for fragility assessment also exist, such as Incremental Dynamic Analysis (Vamvatsikos and Cornell 2002) and Multiple Stripes Analysis (Jalayer and Cornell 2009), but are not considered here due to the excessive computational resources required for the nature of this application.

3. CASE-STYDY APPLICATION

3.1 Exposure model

A synthetic exposure model representing the town of Avellino in the Campania region of Southern Italy is used here. The area is chosen because it is characterized by high seismicity, as two major events occurred in the last 90 years (i.e., the Mw 6.72 on 27/7/1930 and Mw 6.89 on 23/11/1980).

According to recent field surveys (Ricci 2010; Del Gaudio 2015), the predominant construction material in the town is reinforced concrete, representing more than 80% of total building inventory, the majority of which fall within mid-rise category (~63% RC). For simplicity, it is assumed that the building stock only consists of mid-rise RC buildings, an assumption deemed appropriate for the nature of this study. The buildings are spatially distributed according their exact locations, as they are extracted directly from the OpenStreetMap database, hereafter OSM, (OpenStreetMap 2017) as illustrated in Figure 1.

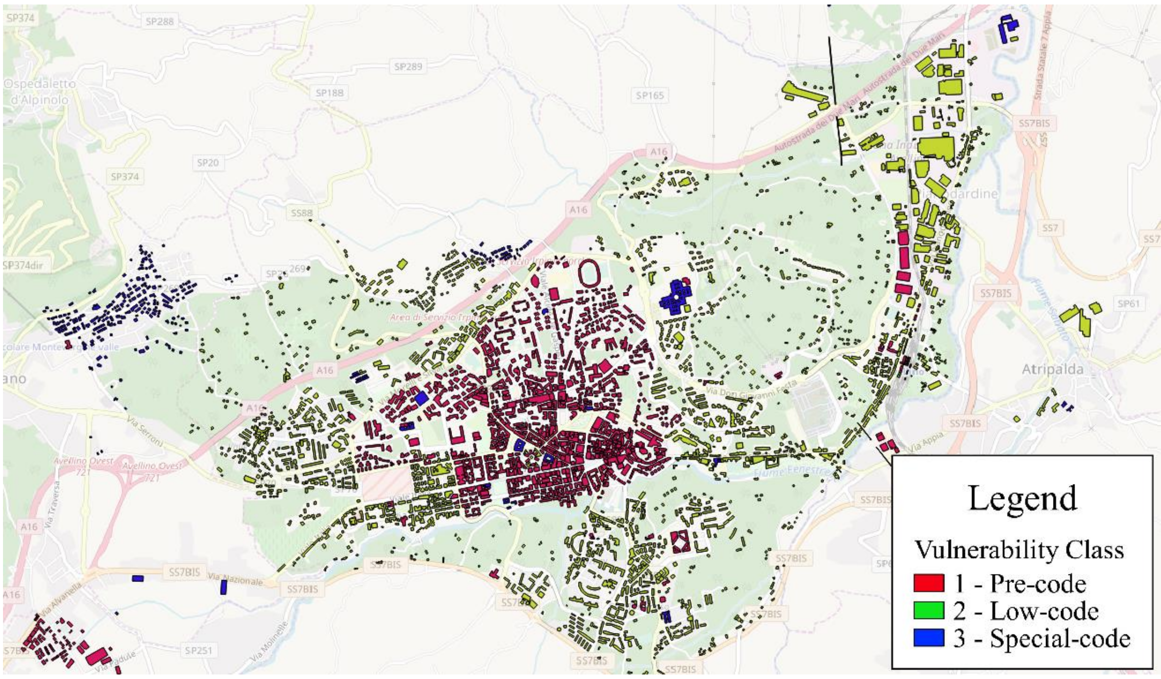

Figure 1. The map of Avellino town, and the associated building portfolio (OpenStreetMap 2017)

The buildings are organized based on their year of construction, and associated design-code level into three main classes, namely Pre-code (constructed before 1972, representing 32% of the total population), Low-code (erected between 1973 and 2005, representing 60% of the total population) and Special-code buildings (built after 2006, representing 8% of the total population). The OSM databases do not include any information about the year of construction of each building, therefore, a large concentration of Pre-code buildings is assumed at the center of Avellino encircled by Low-code buildings, while Special-code buildings are scattered in the outskirts of the town.

3.2 Hazard model and ground motion simulation

With regard to the hazard estimation, the output of the Earthquake Model for the Pan-European

Region developed by AIR-worldwide[5] (further referred to as AIR) is used. Seismicity in the Pan-European region can be attributed to crustal earthquakes on known faults, large subduction interface earthquakes, large subduction zone earthquakes, and shallow background and deep seismic activity. In the AIR Earthquake Model for the Pan-European Region, large earthquakes associated with subduction zones and active crustal faults are modelled as characteristic events. The seismicity of smaller magnitude earthquakes on subduction zones and crustal faults and the seismicity of regions not captured by these sources are represented by background seismicity. A smoothed background-seismicity distribution, implemented on a three-dimensional grid, is used to capture the small-to-moderate events that occur on as yet unknown or unmapped faults. The model domain is divided into two depth layers based on historical activity and the tectonics. Shallow events such as those affecting the region investigated occur at depths of 0-25 km.

The hazard component of the Pan-European Earthquake Model was used to develop a stochastic catalogue of events with 10,000 year simulations of seismicity, incorporating a total of 817,251 simulated events. It should be noted that a 10,000-year catalogue may not be long enough to obtain a stable exceedance probability curve (in terms of hazard and loss). For this reason, AIR developed a 100,000-year catalogue to which an optimization procedure was applied, creating an optimized 10,000-year sample of the 100,000 year catalogue that ensures the accuracy of hazard and loss results, while optimizing the computational effort.

The ground motion footprint of three different IMs, namely $S_a(0.3s)$, $S_a(1.0s)$ and $S_a(2.0s)$ were generated for each event in the catalogue, using designated GMPEs for Italy and Mediterranean region. It is noted that, for this example, spectral acceleration for natural period of one second is used ($S_a(1.0s)$) (a natural period of one second is assumed in catastrophe models for mid-rise RC buildings), while all three available spectral accelerations are used to calculate the corresponding value of I_{Np}. In this way, $S_a(0.3s)$ can capture the effect of the higher modes (characterized by periods less than T_1), which are explicitly modeled when performing NDA.

3.3 Vulnerability model

Three RC moment resisting frame (MRF) buildings are used to populate the vulnerability model for this study. The buildings share the same geometry and are regular in plan and elevation, but they represent distinct vulnerability classes, as they are designed according to different code provisions. The first frame is designed to only sustain gravity loads following the Italian Royal Decree n. 2239 of 1939 (Regio Decreto 16/11/1939 n. 2229 1940) that regulated the design of RC buildings in Italy up to 1971, hereafter called the Pre-code building; the second frame was designed according the Decreto Ministeriale of 1972, hereafter Low-code building (Decreto Ministeriale del 30/05/1972 1972); the last frame is designed according to the latest Italian seismic code (or NIBC08; Decreto Ministeriale del 14/01/2008 (2008)), fully consistent with Eurocode 8 (EC8; EN 1998-1 (2004)), following the High Ductility Class (DCH) rules, hereafter called the Special-code building. Interstorey heights, the spans of each bay and cross-sections dimensions for each case-study building are reported in Figure 2.

The nonlinear response of the buildings is estimated with the aim of NDA and FRACAS analysis methods over a large set of ground motions suitable for displacement-based design and assessment, selected from the SIMBAD database (*Selected Input Motions for displacement-Based Assessment and Design*; (Smerzini et al. 2013)). More details about the building models, the ground motion selection and the analysis processes can be found in Minas and Galasso (2018).

[5] http://www.air-worldwide.com/publications/brochures/documents/air-earthquake-model-for-the-pan-european-region

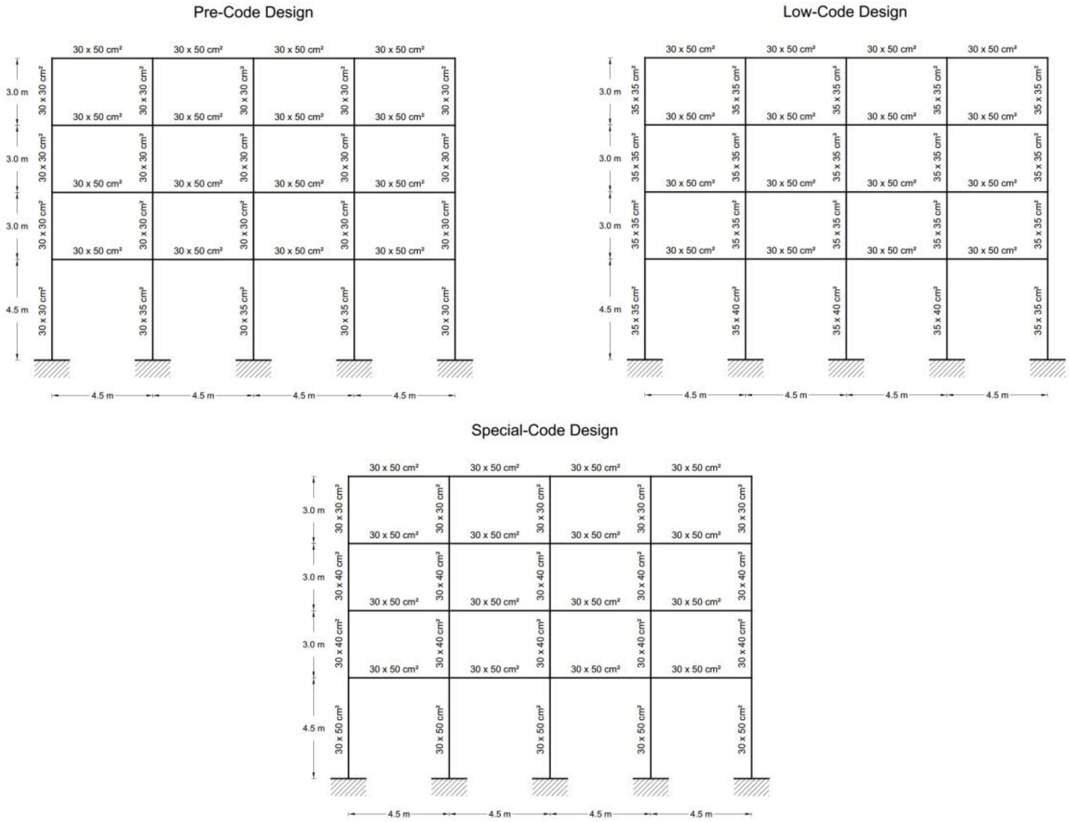

Figure 2. Elevation dimensions and members cross-sections of the considered RC frames

3.3.1 Fragility analysis for mid-rise RC building populations in Italy

In this section, twelve sets of fragility functions are presented. These twelve fragility curves correspond to the three different building types presented in Section 3.3, analyzed with the aim of NDA and FRACAS analysis, and expressed in terms of the standard IM, $S_a(T_1)$, and the advanced IM, I_{Np}. The fragility curves are derived by adopting thresholds of MIDR to define three damage states, discussed in detail by Rossetto et al. (2016). Figure 3 illustrates the median fragility curve sets expressed in terms of $S_a(1.0s)$ and I_{Np}, and their associated 95% confidence intervals estimated using the bootstrap technique. The resultant fragility curves are then combined with the damage to loss model designated for Italy (Di Pasquale and Goretti 2001) to develop a vulnerability for each of the above twelve combinations.

4. SENSITIVITY OF SEISMIC LOSS ESTIMATION TO INPUT ASSUMPTIONS

4.1 Impact of Cross-correlation of ground motion intensity in seismic risk analysis

To understand the impact of the different input assumption on the risk analysis, a comparison of the estimated seismic losses for each modelling option is carried out. Four modelling options are considered here as obtained from combining the two different types of IMs discussed above (standard IM – $S_a(1.0s)$, and advanced IM – I_{Np}) and the two types of analysis (simplified analysis – FRACAS, and advanced analysis – NDA). The seismic losses are then computed for these four modelling options. A probabilistic event-based approach was used to evaluate the seismic losses to

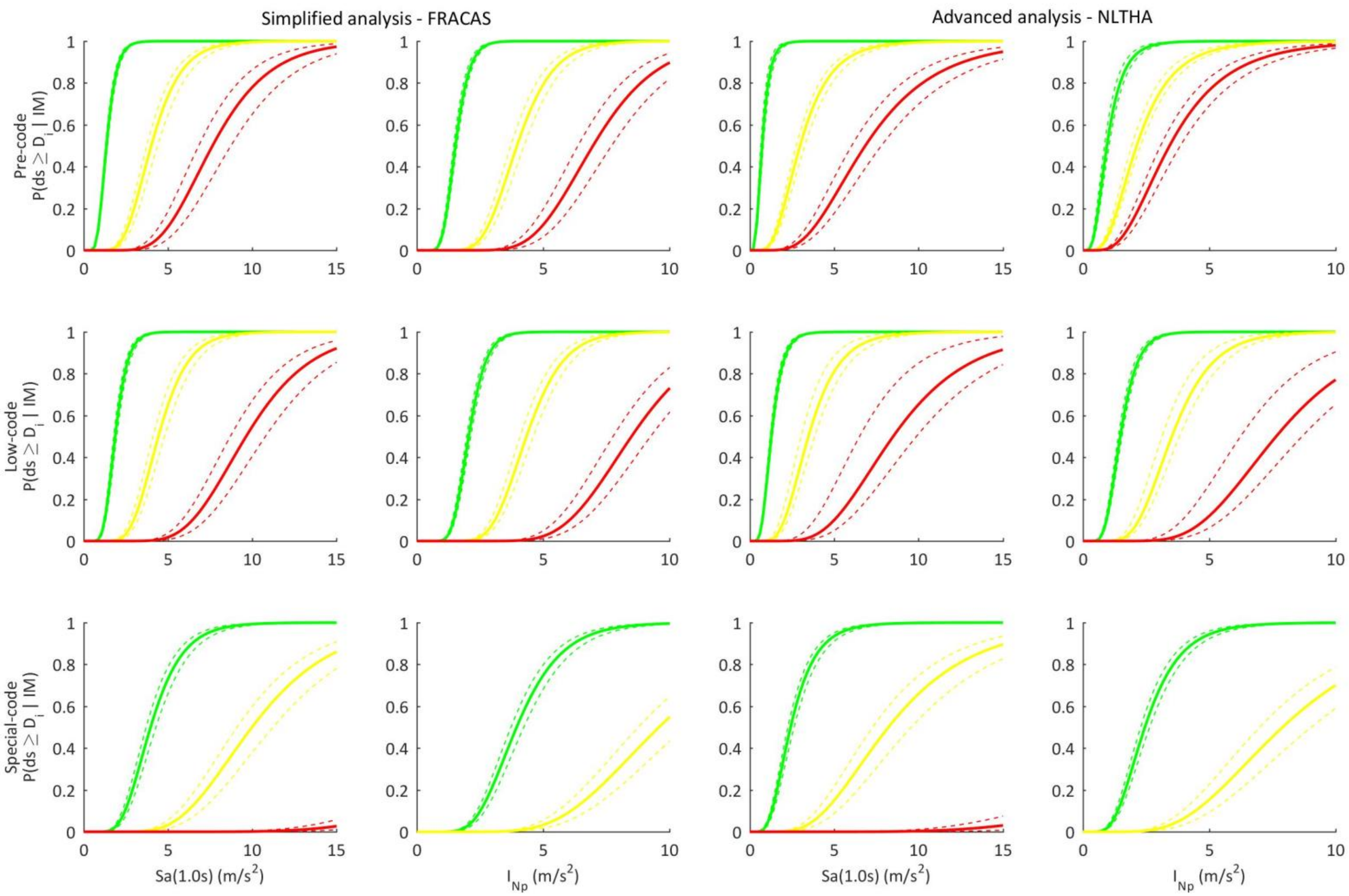

Figure 3. Fragility curves for all the twelve combinations, including: three vulnerability classes, namely Pre-, Low- and Special-code (corresponding to first, second and third row respectively); two analysis types, namely FRACAS and NDA (corresponding to columns 1-2 and 3-4 respectively); two types of IMs, $S_a(1.0s)$ and I_{Np} (corresponding to columns 1,3 and 2,4 respectively

each of the portfolios, using the aforementioned stochastic catalogue of events with 10,000 year simulations of seismicity. Aggregated losses were evaluated for each simulated earthquake event. The resulting set of portfolio event losses is commonly referred to as the "event loss table", which was used to evaluate the annual probability of achieving or exceeding any possible level of loss. This information is used to build the so-called "loss exceedance probability curve" (or simply EP curve). EP curves describe the probability that various levels of loss will be exceeded over a specific timeframe (usually one year), and are obtained by transforming the "event loss table" into a "year loss table". More specifically, the total loss for each year is computed as the sum of the losses for the events simulated in that year. As a result, it is possible to evaluate the annual frequency of exceedance of any loss by ranking the year loss values in descending order and dividing each rank by the length of the catalogue (e.g. the highest year loss has rank equal to one, and a frequency of exceedance of 1/10,000, whereas the lowest year loss has a rank equal to 10,000 and an annual exceedance frequency of 10,000/10,000 = 100%). Figure 4 illustrates the aggregated EP curves for the four tested options.

The model using the advanced IM and the advanced analysis method (i.e., I_{Np} combined with NDA), is assumed as a benchmark for the presented comparison. A close inspection of Figure 4 reveals that, as expected, the losses associated with the advanced analysis (NDA) are in both cases considerably higher than the associated losses obtained from the simplified analysis (FRACAS). This reflects some of the limitation of the pushover-based approaches, including FRACAS, thoroughly discussed in Fragiadakis et al. (2014). For instance, NSPs cannot account for the contribution of the higher modes. In addition, FRACAS utilizes simple hysteretic models and capacity curve idealizations, and models failure mechanism in a simplified manner. The choice of a given IM is also of high importance, as significant difference in the loss estimation is observed when different types of IMs are used. As opposite to $S_a(T_1)$, I_{Np} can account for the spectral shape in a range of periods, potentially resulting in more accurate structural response and damage/loss estimates. This discrepancy is further amplified by the fact that the ground motion records used in the derivation of fragility functions by both FRACAS and NDA have not been selected based on the actual site-specific hazard. Specifically, the IM footprints used in this study were generated combining the stochastic catalogue and a set of GMPEs for Italy and Mediterranean region, while the fragility curves were developed utilizing 150 unscaled records arbitrarily selected from the SIMBAD database (recorded worldwide); see Rossetto et al. (2016) for details. This observation is consistent with the study of Kohrangi et al. (2017), which shows a non-negligible influence of the site-specific hazard (and corresponding ground motion selection) on the fragility/vulnerability derivation, the degree of which depends on the IM adopted for assessment. Advanced IMs can considerably reduce such dependence.

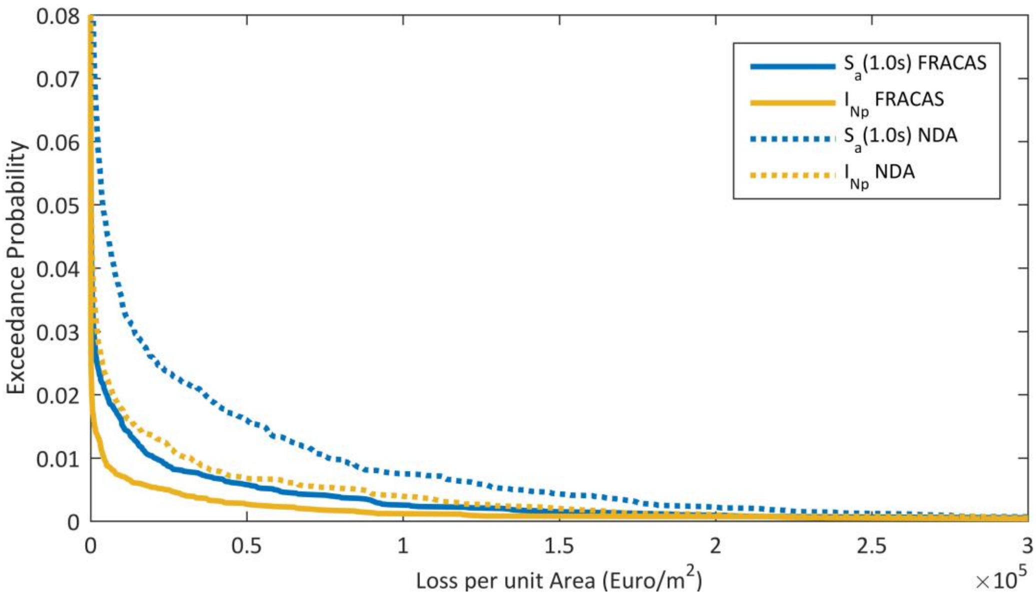

Figure 4. Aggregated exceedance probability loss curves for the four tested modelling options

5. CONCLUSIONS

This paper investigates the impact of ground motion intensity and vulnerability modelling options to the estimation of probabilistic regional seismic loss assessment of reinforced concrete (RC) buildings. An idealized, heterogeneous portfolio of buildings located in Avellino, Southern Italy, is chosen as a case-study. A site-specific hazard analysis accounting for uncertainty in the factors affecting ground motions is carried out, allowing the calculation of IM footprints for different types of IMs, including conventional and advanced IM representations. The exposure database consists of three regular mid-rise RC bare frame buildings corresponding to three distinct vulnerability classes, which are spatially distributed to realistically simulate the typical Italian exposure. These buildings are analyzed with the aim of NDA and the simplified analysis procedure FRACAS to generate sets of seismic fragility functions expressed in terms advanced and standard IMs. The probabilistic regional economic earthquake losses are estimated and presented in the form of EP curves for all tested hazard and vulnerability combinations.

Results from this study highlight that the choice of the analysis method type has a significant impact in the loss estimation. The NDA can account for the effect of higher modes and hysteretic behavior, and can determine the failure mechanisms more precisely. Therefore, the estimated losses are substantially higher to the ones associated with the simplified procedures. The choice of IM type is also of high importance within the loss estimation process, due to the ability of advanced IMs to capture more information than standard IM, e.g. spectral shape in a range of periods. In addition, it is observed that the use of advanced IMs can significantly reduce the effect of the ground motion characteristics to the predicted structural response, and consequent loss estimates, which is consistent to recent studies.

The present research work constitutes the first step of a more detailed systematic assessment on the sensitivity of seismic losses to: (1) hazard characterization, (2) assumptions in structural analysis, (3) the choice of different fragility assessment approaches (including the advanced Bayesian emulation-based approach (BEA) recently proposed by the authors), (4) choice of correlation models (spatial, spectral and cross-correlation), and (5) the grid resolution.

6. ACKNOWLEDGMENTS

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