Bilinear Probabilistic Models of the Seismic Response of a Low Ductility Reinforced Concrete Frame

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ABSTRACT
Probabilistic methods to evaluate the seismic vulnerability of reinforced concrete (RC) frames are largely used in the context of performance based design and assessment, often describing the structural response using global engineering demand parameters (EDPs) such as the interstory drift. While such EDPs are able to synthetically describe the structural behavior, the use of component level parameters of RC frames without seismic detailing can enable a more realistic and thorough description of failure mechanisms for structural vulnerability and loss assessment. This paper proposes a methodology for the probabilistic evaluation of seismic demand of low ductility RC frames by exploring a range of component level and global EDPs, identifying appropriate regression models and comparing performances of different ground motion intensity measures used in the probabilistic analysis. A realistic benchmark RC frame is used as a case study to identify key considerations in probabilistic seismic demand modeling.

1 INTRODUCTION

Reinforced concrete (RC) buildings constructed before the introduction of advanced seismic building design codes have suffered significant damage during past earthquakes due to lack of adequate ductility. As per the latest surveys, experts estimate that 25,000 to 30,000 non-ductile concrete buildings in California were constructed before the introduction of seismic building design codes. Additionally, gravity load design was historically the dominant consideration for design of RC buildings until recent decades. This underlines the need to develop reliable tools to assess the vulnerability of low ductility RC buildings in addition to estimating associated seismic losses. Performance Based Earthquake Engineering (PBEE) (Cornell and Krawinkler 2000, Moehle and Deierlein 2004) has gained momentum to support seismic risk mitigation decision-making by disaggregating individual elements of the risk assessment framework. Seismic fragility analysis is a key element of this process used to evaluate the performance of structures under earthquake events based on quantification of structural capacity limits and seismic demand. Probabilistic Seismic Demand Models (PSDMs) are often used to characterize the variation in demand on structures under seismic loading by providing a relationship between structural response and ground motion (GM) Intensity Measure (IM). Traditionally, structural response is measured by Engineering Demand Parameters (EDPs), such as the overall maximum interstory drift over the entire building. The use of this EDP is adequate to describe the seismic response of ductile frames designed by strength hierarchy rules, but may lead to a high approximation in the vulnerability evaluation (Freddi et al. 2012) and consequently in loss estimates since in this case there is not direct relation between local failure mechanism and global interstory drifts (Freddi et al. 2012). To obtain a more thorough characterization of the vulnerability of the structure, a multi-component fragility study is necessary, as suggested by Bai et al. 2011 and Ghosh and Padgett 2011.

In this study, different EDPs are considered in order to highlight the most significant failure modalities in RC low ductility frame buildings. PSDMs of single components are developed for various EDPs, and the viability of alternative IMs is explored. In particular PSDMs for component level EDPs are investigated since they serve as a basis for component level damage and loss assessment. This study provides insight into the form of regression model appropriate for such component level EDPs such as steel and concrete strains, moments and shears on beams and columns, and additional global EDPs such as base shear or story accelerations. Furthermore, several IMs are analyzed to identify which is most appropriate for Probabilistic Seismic Demand Analysis (PSDA) of component level EDPs for this type of structure on the basis of such characteristics as IM efficiency and sufficiency. All the considerations are based on the results of a PSDA performed on a case study. For case study purposes, a three-story ordinary moment resisting RC frame is adopted, which is representative of typical gravity load designed low-rise RC frames constructed in the Eastern and Central US. The case study frame was experimentally investigated extensively by Bracci et al. 1992a, 1992b and Aycardi et al. 1992, enabling validation of the Finite Element (FE) model and improved confidence in the global and local dynamic response estimates. The findings of this study can be used to support the formulation of demand models used in component level fragility analysis and loss estimation conducted within the PBEE framework.

2 PROBABILISTIC SEISMIC DEMAND ANALYSIS

Probabilistic Seismic Demand Analysis (PSDA) is one of the critical steps of the PBEE framework, and it is performed to determine the response of structures under different levels of ground excitation. The PSDA permits the definition of a PSDM, which is a mathematical model that relates the ground motion IM to the measure of structural response in terms of the chosen EDP. These models often provide the key link between a damage assessment and seismic hazard analysis. This section reviews the important properties of IMs and EDPs used to characterize the behavior of concrete buildings, the form of the regression model, and the statistical distribution typically adopted for PSDMs.

2.1 Seismic Intensity Measure

The characteristics of a GM record may be synthesized by an IM. Ideally, an appropriate IM should be able to capture the amplitude, frequency content and duration properties of GM which significantly affects the elastic and inelastic response of the structure. An IM should be practical, efficient, sufficient, as well as "predictable" through a seismic hazard analysis. The *practicality* characteristic is a measure of the sensitivity of an EDP with respect to the IM (Padgett et al. 2008). An IM is *efficient* if it reduces the amount of dispersion in the estimated demand (Giovenale et al. 2004). *Sufficiency* of the IM is the property that makes the structural response conditionally statistically independent of GM characteristics, such as earthquake magnitude (M) and source-to-site distance (R) (Padgett et al. 2008, Luco and Cornell 2007). *Hazard computability* refers to the effort required to assess the probabilistic seismic hazard or availability of hazard curves (Giovenale et al. 2004).

The choice of IM is a critical step in developing a viable PSDM and has been widely investigated in the literature. However, the characteristics of an IM were usually investigated by focusing on the behavior of global EDPs such as the maximum interstory drift.

2.2 Engineering Demand Parameters for low ductility RC frames

"Engineering Demand Parameters (EDPs) are structural response quantities that can be used to predict damage to structural and non structural component systems" (ATC-58 2004) and can generally be used to investigate a range of potential inadequacies of structures. EDPs selected should correlate well with a measure of damage of the structure as well as with decision variables, such as, direct dollar losses and duration of downtime (Medina and Krawinkler

2004). Thus, appropriate response indicators of the structure can be chosen based on the observation of failure modes highlighted in past earthquake events. Many common failure modes are attributed to deficiencies associated with reinforcements of non-ductile RC buildings, moreover, from a global point of view, irregularities in strength and stiffness either in elevation or in plan have been identified as one of the main cause of failure.

Structural damage can be estimated in different ways. Global deformational parameters, such as story displacement and interstory drift are often used to estimate overall structural damage. Residual story displacements can be used as EDPs to investigate the structural stability of the system. The use of global EDPs is also suggested from some contemporary codes (HAZUS MH-2.0, FEMA 356), but their correlation to some component level failures may be limited and their use may introduce a high level of uncertainty. Failure of elements subjected to flexural and axial action may be estimated through various approaches. Local EDPs may include fiber stress and strain for steel (σ_s ,ε_s) and concrete (σ_c ,ε_c) for columns and beams and principal tension and compression stresses for joints ($\sigma_{j,tens}$,$\sigma_{j,comp}$). Intermediate EDPs such as forces and deformations, including axial forces, shear forces, moments, curvatures and rotations, may be adopted to evaluate element failure. The use of local EDPs may be preferred with respect to intermediate EDPs since they have the advantage of enabling the consideration of flexural and axial interaction. Moreover, the use of intermediate EDPs implies either an approximation in the evaluation of component vulnerability or added complexity in capacity estimation. However, curvature and rotation are widely used EDPs. Interstory drift, story accelerations and velocity are often used to evaluate the building contents and non-structural damage. Moreover they are used as EDPs to evaluate the human comfort for high-rise structures when subjected to environmental loads.

2.3 Probabilistic Seismic Demand Model

Cornell et al. 2000 presented the basis for a formal probabilistic framework for seismic design and assessment of structures. A closed form solution to define fragility curves is achieved by analytical approximation of the demand representation. The relationship between median structural demand $\hat{D}$, and IM was proposed to be approximated by a power model:

$$\hat{D}(IM) = aIM^b \tag{1}$$

where a and b are regression coefficients. In order to complete the probabilistic representation, the demand has traditionally been assumed as lognormally distributed with logarithmic standard deviation, β_D. It is calculated from the error of the mathematical demand model with respect to the j^{th} corresponding realizations from the NTHA as shown:

$$\beta_D = \sqrt{\frac{\sum_{j=1}^{N}\left(\ln\left(D_j(IM)\right) - \ln\left(\hat{D}(IM)\right)\right)^2}{N-2}} \tag{2}$$

where N represent the total number of j^{th} realizations. Homoscedasticity of the demand is often practically assumed. Under these assumptions for the form of regression of the median, and the distribution of demands, the probability that a certain value of the demand (D) exceeds the capacity (C) can be written as:

$$F(IM) = 1 - \Phi\left(\frac{\ln\left(\hat{C}\right) - \ln\left(\hat{D}(IM)\right)}{\sqrt{\beta_D^2 + \beta_C^2}}\right) \tag{3}$$

where $\Phi(\cdot)$ is the standard normal cumulative distribution function, $\hat{C}$ is the median value of the structural capacity, and β_C is the logarithmic standard deviation, or dispersion of the capacity. Several authors have used this approach to develop fragility curves. Other authors that developed PSDMs have found that linear regression of the demand in the log-log space was not accurate enough to represent the demand response either with local and global EDPs. In some cases they found that good fit of the demand can be obtained by adopting a bilinear regression. Hence, investigation of the goodness of these two different regression models is performed in the following sections.

3 OPTIMAL PSDM OF LOW DUCTILITY RC FRAMES

A three-story ordinary RC moment resisting frame experimentally tested by Bracci et al. 1992a and 1992b has been chosen as the case study structure. The building has been designed for gravity loads only and without any seismic detailing, by applying the design rules existing before the introduction of modern seismic provisions. This case study has been selected because experimental results concerning local behavior are available for a 1:3 reduced scale model of the frame and of its subassemblages (Bracci et al. 1992, Aycardi et al. 1992). This allows an accurate validation of the FE model and permits a reliable test of the probabilistic study developed in the following sections. Figure 1 contains the general layout of the structure including the indications for beams (B), columns (C) and joints (J). A detailed description and validation of the FE model is reported in Freddi et al. 2012.

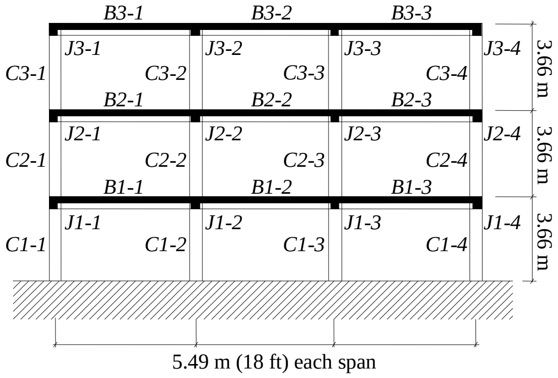

Figure 1. General layout of the structure.

Amongst others, only the effect of record-to-record variability is considered in this study since the effects of model parameter uncertainty and epistemic uncertainty are usually less notable and are often introduced a posteriori. The uncertainty affecting the ground motion input is taken into account by selecting a set of natural GM records that reflect the variability in duration, frequency content, and other characteristics of the input expected to act on the system. The validated FE model of the prototype structure is hence defined as deterministic and is used to explore several IM-EDP pairs by building PSDMs. The IMs investigated in this study are shown in Table 1 and are chosen among the more popular IMs and other scalar IMs. Besides being relatively easy to use, seismic hazard curves for these IMs are either readily available or computable with a reasonable effort. The use of vector valued IMs may be interesting for future investigation but are not considered in this study since they open a full range of alternative model forms, combinatorial expansion of the problem considering IM pairs, and practical challenges in implementation in a risk assessment. A set of 240 GMs from Baker et al. 2011 has been used in the non-linear dynamic analyses. The GMs used in this study are representative of a wide range of variation in terms of source to site distance (R) (from 8.71 to 126.9 km) and soil characteristics with an average shear wave velocity in the top 30 m (V_{s30}) that range from 203 to 2016.1 m/sec while the magnitude (M) of the GMs range from 5.3 to 7.9. Pulse like records are not included.

Table 1. Intensity Measures (IMs).

IM	Description
Structure Dependent IM	
S_a	Spectral Acceleration at T_1
S_v	Spectral Velocity at T_1
S_d	Spectral Displacement at T_1
S_{aC}	S_a Predictor [Cordova et al. 2000]
S_{N1}	S_a Predictor [Lin et al. 2011]
Structure Independent IM	
PGA	Peak Ground Acceleration
PGV	Peak Ground Velocity
PGD	Peak Ground Displacement
$S_{a\text{-}02s}$	Spectral Acceleration at 0.2 sec
$S_{a\text{-}1s}$	Spectral Acceleration at 1 sec
I_a	Arias Intensity
I_v	Velocity Intensity
CAV	Cumulative Absolute Velocity
CAD	Cumulative Absolute Displacement

In order to investigate all the possible failure modes, 12 EDPs are considered as shown in Table 2. While local and intermediate EDPs are used to capture the seismic behavior and vulnerability at component level, global EDPs are considered since they are commonly used to assess the global behavior of the building and simultaneously permit inclusion of non-structural building component responses.

Table 2. Engineering Demand Parameters (EDPs).

EDP	Description
Local EDPs	
$\varepsilon_{s,max}$	Rebar strain
$\varepsilon_{c,max}$	Concrete strain
ϕ_{max}	Curvature
$\sigma_{j,tens,max}$	Joint tensile stress
$\sigma_{j,compr,max}$	Joint compressive stress
Intermediate EDPs	
V_{max}	Shear
M_{max}	Moment
Global EDPs	
$V_{b,max}$	Base shear
$\Delta_{i,max}$	Story displacement
$\theta_{i,max}$	Interstory drift
$St.Vel_{i,max}$	Story velocity
$St.Acc_{i,max}$	Story acceleration

PSDMs for all the considered IM-EDP pairs are developed by using the dynamic responses from the 240 NTHA in what is often termed a “cloud analysis”. The frame model is considered to be deterministic and the variability in local, intermediate, and global responses captured in the PSDA reflects the propagation of GM variation alone. Optimal regression form of PSDMs for response quantities of interest in vulnerability modeling of low ductility RC frames is described as follows. The PSDMs using $S_a(T_1)$ as the IM are shown as an example to explore the regression form, however, all EDP-IM pairs were explored confirming that the behavior in terms of viability of linear versus bilinear regression (in log-log space) is consistent across all IMs.

Figure 2a illustrates the PSDMs constructed in the transformed space considering as EDP the interstory drift at the 1st level (θ_1). The results reveal that linear regression of the structural demands relative to $S_a(T_1)$ provides a good fit for the drift, similar results have been obtained by considering the top story velocity and the top story acceleration. In contrast, as shown in Figure 2b, bilinear regression is needed to obtain a better fit of the analyses results for the base shear (V_b) since it is a force-based demand measure. Indeed, after the elastic limit of the force is exceeded, the slope of the regression is lower capturing the post yielding behavior of the structure. PSDMs for local and intermediate EDPs have been developed for all critical sections of the structure. For most of the sections, which exhibit significant non-linear behavior, the bilinear regression is indispensable to adequately represent the demand for local and intermediate EDPs. Figure 2c and 2d show the curvature and bending moment PSDMs for upper column C1-1. Since rebar and concrete maximum strains are strictly correlated with the sectional curvature their behavior is similar. For these EDPs, bilinear regressions of the demand are found to be the best fits to build PSDMs reflecting typical stress-strain bilinear behavior of the materials and typical moment-curvature bilinear behavior for sections.

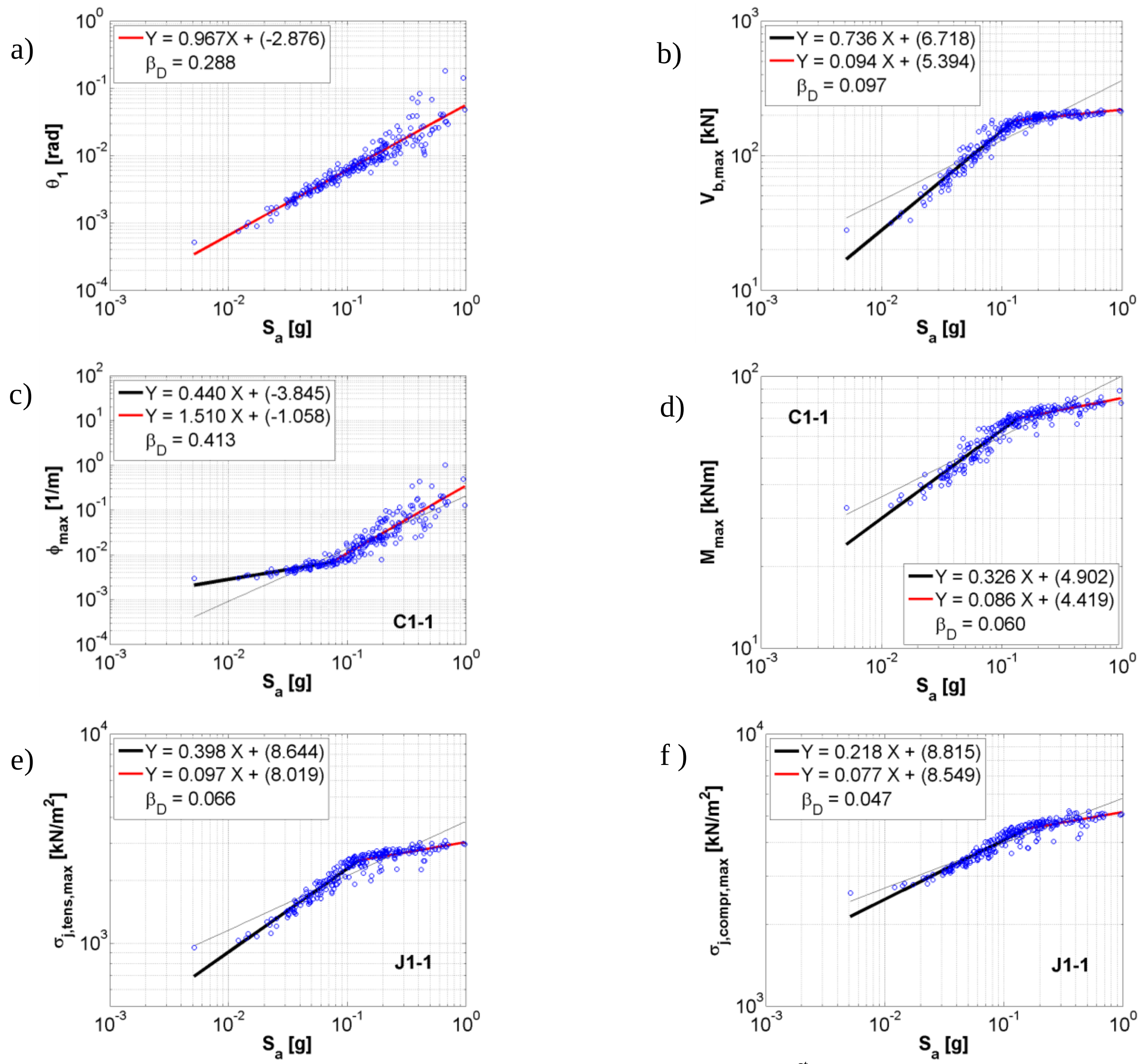

Figure 2. PSDMs and Linear/Bilinear Comparison for: a) Interstory drift for the 1st story; b) Base Shear; c) Curvature for the upper section of column C1-1; d) Moment for the upper section of column C1-1; e) Tensile stress for joint J1-1; f) Compressive stress for joint J1-1;

Also the beams and columns shear PSDMs and the tensile and compressive stress PSDMs for the joints (Figures 2e and 2f) revealed the improvement of the bilinear versus linear regression.

4 INTENSITY MEASURE COMPARISON

Given the form of regression identified for PSDA of different EDPs, a comparison of alternative GM IMs is conducted to select the ideal independent variable for the PSDM regression. To identify the "best" IM, conditions of practicality, sufficiency, hazard computability and efficiency are evaluated. All the IM-EDP pairs evaluated in this paper are considered practical and amongst others, the efficiency of an IM is considered as the main decision parameter for IM selection. Moreover, an IM should be "good" for all the components (and hence, all the EDPs) interested in the probabilistic procedure since both minor and major damage of the considered components can lead to a failure condition. When local EDPs are used, the amount of components involved in the procedure is very large and hence the average measure of each IM characteristic is provided as an efficient way to screen the overall ability of the IM for the entire structure. The following sections investigate the characteristics of an ideal IM and present the results of the IM comparison.

4.1 Efficiency

Efficiency indicates the amount of variability of an EDP given an IM and can be quantified by the dispersion, β_D (Equation 2). Identification of the best IM based on reduced β_D is challenging since the level of dispersion may vary for different components. To facilitate the comparison, a statistic of the dispersion values (mean) among all the components and all the EDPs for each IM is reported in Table 3. Structure dependent IMs have the lower dispersion and hence are much more efficient relative to the structure independent IMs. Among the structure dependent IMs, $S_a(T_1)$ and $S_d(T_1)$ are found to be the best IMs consistently with previous results from other studies. The optimal IM in terms of efficiency does not tend to show dependence upon EDP of interest, and consistent results can be observed looking each EDP independently.

4.2 Sufficiency

All the IMs are evaluated for sufficiency in terms of conditional statistical independence of the response from magnitude (M) and distance (R) (Padgett et al. 2008). It is acknowledged that sufficiency with respect to other characteristics such as epsilon or duration is also desirable, but these extended tests are beyond the scope of the current study. Residuals from the PSDMs, $\varepsilon|IM$, are considered in a linear regression with M and R. Hypothesis tests of residual independence from M or R are conducted resulting in p-values (Hines et al. 2003) used to assess the sufficiency, where smaller p-values indicate an insufficient IM. The p-value is defined as the probability of rejecting the null hypothesis in an analysis of variance, where the null hypothesis states that the coefficient of regression is zero. Smaller p-values indicate stronger evidence for rejecting the null hypothesis and evidence of an insufficient IM (Padgett et al. 2008).

Table 3. Mean values of β_D across all components and all the EDPs used to evaluate IM efficiency (cut off of the mean values of β_D equal to 0.30). (Note: Bold values indicate inefficient IM.)

	Structure dependent IMs					Structure independent IMs								
	S_a	S_v	S_d	S_{aC}	S_{N1}	PGA	PGV	PGD	$S_{a\text{-}02s}$	$S_{a\text{-}1s}$	I_a	I_v	CAV	CAD
β_D	0.20	0.21	0.20	0.23	0.22	**0.42**	0.27	**0.36**	**0.47**	0.28	**0.35**	**0.40**	**0.38**	**0.39**

Table 4. Check of the sufficiency hypothesis test with respect distance (R) and magnitude (M): Fraction of components where it is satisfied (cut off of the p-value equal to 0.025). (Note: Bold values indicate insufficient IM.)

	Structure dependent IMs					Structure independent IMs								
	S_a	S_v	S_d	S_{aC}	S_{N1}	PGA	PGV	PGD	$S_{a\text{-}02s}$	$S_{a\text{-}1s}$	I_a	I_v	CAV	CAD
R	0.83	0.71	0.83	0.76	0.69	0.99	0.94	**0.16**	1.00	0.97	0.90	**0.21**	0.75	**0.22**
M	0.95	0.90	0.95	0.99	0.96	**0.07**	0.96	0.96	**0.03**	0.82	0.88	1.00	0.93	1.00

Table 4 contains the average values of the fraction of components where the hypothesis test is satisfied weighted for the number of components of each EDP. P-value lower of an assumed cut off of 0.025 indicate that the sufficiency hypothesis test is rejected.

Among all of the IM-EDP pairs for the RC frame, PGD, I_v and CAD are found to be insufficient respect to distance, while PGA and $S_{a\text{-}02s}$ are found to be insufficient with respect to magnitude. PGV is found to be the IM that best satisfies the sufficiency hypothesis test with respect to both R and M while all the others IMs are considered equally sufficient. Consistent results are also obtained using different values of the statistical significance level.

4.3 Hazard Computability

Among the IMs considered, hazard information is readily available across the United States for PGA, PGV, PGD, and specific spectral quantities corresponding to 0.2 sec and 1.0 sec ($S_{a\text{-}02s}$ and $S_{a\text{-}1s}$), from such entities as the US Geological Survey. For the structural dependent IMs considered in this paper, hazard curves can be approximated with a reasonable level of effort while, the hazard curve definitions for the other structural independent IMs (I_a, I_v, CAD and CAV) are practicable but require considerable efforts.

5 CONCLUSIONS

This paper proposes a methodology for the probabilistic seismic demand analysis of low ductility RC frame buildings, to support multi-component vulnerability assessment of such structures that exhibit susceptibility to damage under earthquake loads. In particular, this study employs not only global EDPs but includes also the use of intermediate and local EDPs able to provide a more realistic and thorough description of the failure mechanisms for structural vulnerability and loss assessment. However, the global EDPs have been included in the study since they are the only available parameters able to relate the non-structural and contents damage. The paper explores the appropriate form of PSDM in terms of the regression model and analyzes the performance of alternative IMs, when not only global EDPs are used, on the basis of such criteria as model efficiency. As a case study, a typical gravity load designed low ductility RC frame is chosen and validation of the Finite Element model is performed using published experimental data prior to conducting the numerical simulations for PSDA. The obtained results are limited to the investigated case study; however, since it is considered representative of a building typology, it is expected that similar results can be found on structures having similar characteristics. Nevertheless, an extension of the investigation to other case studies is needed in order to provide more generalizable recommendations. Among the traditional and advanced GM IMs, 14 structure dependent and structure independent IMs are assessed to identify IMs that "best" respect the requirements of practicality, sufficiency, hazard computability and efficiency. Twelve EDPs indicative of damage potential to RC buildings are considered that span the categories of local, intermediate, and global response quantities. To construct the PSDMs for all IM-EDP pairs and structural components, non-linear dynamic analysis is conducted on the validated model using a set of 240 GMs.

Optimal PSDMs confirm that linear regression (in the logarithmically transformed space) provides a good fit of the demand for conventionally used global EDPs, while it is found that for local and intermediate EDPs, such as curvature, shear, joint stresses, or material strains, a bilinear regression is required.

Assessment of the demand dispersions indicates that structure dependent IMs are much more efficient for all considered EDPs relative to the structure independent IMs, with approximately 50%-75% lower β_D. Among the structure independent IMs, PGV and $S_{a\text{-}1s}$ are the most efficient while $S_{a\text{-}02s}$ and PGA produce the largest values of dispersion. Among the structure dependent IMs, $S_a(T_1)$ and $S_d(T_1)$ have the lowest β_D, while $S_v(T_1)$, $S_{N1}(T_1)$ and $S_{aC}(T_1)$ are all relatively efficient. The sufficiency test of each IM with respect to magnitude (M) and source to site distance (R) indicates that among all considered IM-EDP pairs, PGA and $S_{a\text{-}02s}$ are insufficient with respect to magnitude, while PGD, I_v and CAD are found to be insufficient with respect to distance for most of the EDPs. PGV best satisfies the sufficiency hypothesis with respect to both distance and magnitude, while all other IMs were found to be equally sufficient.

Overall, $S_d(T_1)$ and $S_a(T_1)$ are found to best satisfy the requirement of practicality, sufficiency, hazard computability and efficiency across the range of EDPs. This finding is consistent with other studies performed considering global EDPs only, moreover, well known deficiencies of the IMs based on the first structural period (i.e. $S_a(T_1)$) are not observed since the considered case is such that higher vibration modes do not strongly affect the response and since near-source GMs are not considered. Moreover, the use of a bilinear regression can improve the description of local and intermediate EDP behavior, regardless of the IM adopted.

These findings can support the probabilistic assessment of low ductility RC frames, forming the foundation for enhanced component and system level seismic fragility assessment and loss estimation for these types of structures. In particular, the use local and intermediate demand parameters such as concrete strain (ε_c) can be used in order to evaluate the components' behavior and damage evaluation. Moreover, the use of bilinear regression models, with IMs such as $S_d(T_1)$ or $S_a(T_1)$ is suggested for such local and intermediate level EDPs in order to reduce uncertainties and to improve the predictive capabilities of the demand model and confidence in the risk assessment. This components-based approach, although more cumbersome, can provide a more comprehensive understanding of the structural behavior, thorough assessment of the impact of mitigation strategies, and accurate evaluation of the seismic losses.

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