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SEISMIC RELIABILITY OF MULTI-STOREY BUILDINGS EQUIPPED WITH VISCOUS DAMPERS

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Abstract

Viscous dampers are energy dissipation devices widely employed for the seismic control of new and existing building frames. To date, the performance of systems equipped with viscous dampers has been extensively analyzed by employing deterministic approaches, i.e. by using a one-to-one relationship between the seismic intensity and a response parameters (usually coinciding with the mean response of a set of ground motions with a specific intensity level). However, these approaches neglects the response dispersion due to input uncertainties (the site seismic hazard condition and the record-to-record variability effects) as well as uncertainties of dampers properties. A more comprehensive performance assessment can be carried out by using probabilistic methodologies cable of fully accounting for different sources of uncertainty and the relevant effects on the structural reliability. This paper analyzes the probabilistic seismic performance of building frames equipped with viscous dampers by adopting a probabilistic methodology based on response hazard curves, providing the mean annual frequency (MAF) of exceedance of the response parameters of interest for the performance assessment of building structural and non-structural components and dampers. Finally, in order to quantify the difference between the demands evaluated by the probabilistic and deterministic approaches, ratios (R_{EDP}) between the probabilistic demand (characterized by different values of the MAF of exceedance) and the deterministic demand are evaluated. An application example is developed by considering a multi-storey steel frame selected from the SAC Phase II project and widely used in studies on seismic response control problems. A set of cases involving dampers with different exponents α designed for the same deterministic performance objective at a reference seismic intensity is considered. The probability distribution and the demand hazard of the response parameters of interest are evaluated and discussed in a range of variation of annual frequency of exceedance spanning from service to ultimate limit states, considering a range of the nonlinear exponent spanning from 0.15 to 1.00. Particular attention is focused on response parameters relevant to dampers (stroke and force). The ratios R are calculated from these parameters and the obtained values may be interpreted as amplification factors to be applied to the deterministic demand in order to obtain a desired safety level for the damper design. It is shown that the damper nonlinearity strongly affects the building and dampers performance and different trends are observed for different demand parameters. A comparison with code provisions shows that amplification factors currently used for the structural design provide non-homogeneous safety levels and should be improved by considering response variations due to the linear/nonlinear behaviour of the viscous dampers.

Keywords: seismic reliability, seismic protection, viscous dampers, nonlinear behaviour, probabilistic response

1. Introduction

Events have highlighted the inadequacy of the seismic design based on the "life safety" or "collapse prevention" concept, and the need of controlling the seismic performance in terms of structural and non-structural damage at multiple hazard scenarios, as also suggested by recent Performance-Based Design (PBD) guidelines [1, 2, 3].

In this context, base isolation [4] or passive energy dissipation [5, 6], have emerged as effective technologies that permit to improve the seismic performance of new and existing buildings by significantly reducing the damage to both structural and non structural components.

Among passive energy dissipation systems, fluid viscous dampers proved to have some performance advantages since they permit to reduce both displacements and accelerations simultaneously [7, 8]. This can be very significant for those structures (e.g., hospitals or electrical stations) whose contents and components are sensitive to deformations or accelerations. Another property of viscous dampers that makes them preferable to other types of damper is related to their velocity-dependent behaviour, which implies large energy dissipation also at small deformation levels .

The response of viscous dampers is proportional to a power-law of the velocity and can be linear or nonlinear depending on the value of the velocity exponent α, usually varying in the range between 0.15 and 1.00 in structural engineering applications [9].

The seismic response of building frames equipped with linear or nonlinear viscous dampers has been analyzed in many papers by using Single-Degree-Of-Freedom (SDOF) models [10, 11, 12, 13] as well as more complex structural models [8, 14, 15]. In general, these studies observed that nonlinear viscous dampers permit to achieve the same displacement reduction of linear viscous dampers but with lower damper forces.

Furthermore, in the nonlinear case the damper forces do not increase significantly for velocities increasing beyond the design value, thus avoiding potential overload in the dampers and in the system to which they are connected [16, 17].

Although all the above studies analyzed interesting aspects concerning the effectiveness of nonlinear viscous dampers in the design, they are all based on a "deterministic" measure of the seismic demand, i.e., they evaluate the seismic response by using a one-to-one relationship between the seismic intensity and a response parameters (usually coinciding with the mean response of a set of ground motions with a specific intensity level), and do not account for the response dispersion and the relevant effects on the structural reliability. The limits of this approach, currently employed in many seismic design guidelines [3, 18, 19, 20], have been highlighted in [21], where it has been stressed also the importance of a more comprehensive performance assessment through probabilistic methodologies capable of fully accounting for the effect of the uncertainty of the seismic input.

Some more recent works carried out the performance assessment of structural systems equipped with viscous dampers through probabilistic approaches accounting for the effect of input and/or model uncertainties in the framework of performance-based earthquake engineering (PBEE) [22, 23, 24, 25, 26, 27]. In particular, in [22] the authors carried out the vulnerability analysis of a r.c. buildings retrofitted with linear viscous dampers before and after retrofit, by evaluating the seismic performance in terms of inter-storey drift ratio (IDR).

Successive studies investigated also other response parameters relevant to the system performance such as the peak absolute accelerations [26], the residual storey drift, the peak plastic deformation of the frame resisting elements, the base shear, and the floor velocity [27].

However, these works considered only the case of dampers with linear behaviour and they mainly focused on the vulnerability problem, investigated through the development of fragility curves. Studies analysing and comparing the probabilistic response of system equipped with linear and non linear viscous dampers are not available in the technical literature. These studies are however a necessary step to evaluate the effective seismic performance of viscously damped structures and to properly measure the safety level, especially for very low velocity exponents, around 0.15-0.20, finding a growing interest in seismic applications. A first work on this topic was recently carried out in [11] by employing a SDOF system. The work considered both kinematic and dynamic response quantities (relative displacement, absolute acceleration, damper force). An extensive parametric analysis encompassing all the system characteristic (non-dimensional) parameters showed that the dispersion of each of these response quantities induced by the seismic input variability differently changes by varying the parameter α. Moreover, results concerning a case study showed the consequences of this effect on the structural safety, expressed in terms of risk of exceeding reference values of the response quantities of interest.

This paper focuses on the seismic reliability of multi-storey buildings equipped with viscous dampers, to the aim of evaluating the influence of the damper nonlinearity, measured by α, on the performance of both structure and dampers. This performance is described in terms of response hazard curves, providing the mean annual frequency of exceedance of the response parameters of interest for the performance assessment and obtained by combining the information on both seismic hazard and seismic vulnerability. In particular, an application example is developed by considering a multi-storey steel frame selected from the SAC Phase II project and widely used in studies on seismic response control problems [23, 24, 25]. The performance variations due to changes in the damper nonlinearity level are evaluated and highlighted by considering a set of cases involving dampers with different exponents α designed for the same deterministic performance objective at a reference seismic intensity. The probability distribution and the demand hazard of the response parameters of interest are evaluated and discussed in a range of variation of annual frequency of exceedance spanning from service to ultimate limit states, considering nonlinear exponents α ranging from 0.15 to 1.00.

Finally, an assessment of the seismic performance obtained by employing simplified code formulas for the damper design is carried out.

2. Probabilistic framework for performance assessment

In the context of PBEE, the assessment of the seismic performance of a structural system can be carried out at different levels (e.g., by quantifying the seismic demand, the seismic damage, or the direct and indirect losses), and can encompass different sources of uncertainty affecting e.g. the earthquake input, the model parameters or the direct and indirect losses estimation [28]. In this study, the focus is on the seismic demand considering the effects of the seismic input uncertainty only. In PBEE, the ground motion uncertainty is usually described by separating the randomness in the input intensity, described by the intensity measure IM (capital letter denotes random variables), from the randomness in the record characteristics (record-to-record variability). The IM randomness is described by the hazard curve, providing the mean annual frequency (MAF) of IM exceeding the value im, whereas the record-to-record variability is described by a representative ensemble of ground motions conditional on the considered IM level [21, 29]. Different choices can be made for the IM and for the sets of records, which should be ideally representative of the seismic threat at the different IM levels [29, 30].

The seismic demand can be monitored by a number of response parameters (engineering demand parameters EDPs) relevant to the performance assessment of the building. As for the case of the IM, the EDP variability can be described by the demand hazard curve, providing the MAF of exceedance of a specific level of seismic demand edp and computed as:

$$\nu_{EDP}(edp) = \int_0^\infty P_{EDP|IM}(edp \mid im) \cdot \left| d\nu_{IM}(im) \right| \tag{1}$$

where $P_{EDP|IM}(edp|im)$ denotes the probability that $EDP > edp$ given $IM = im$ and depends on the record-to-record variability of the response. The probability $P_{EDP|IM}(edp|im)$ can be estimated at different IM levels by performing multi-stripe analysis (MSA) or incremental dynamic analysis (IDA) [29].

These analyses consist in performing a series of simulations of the response of the structure subjected to a set of input ground motions scaled to a common IM level, for different IM levels. A common assumption introduced in PBEE to simplify the assessment of $P_{EDP|IM}(edp|im)$ and to make possible analytical calculations, is that the distribution of the demand conditional to the IM, $f_{EDP|IM}(edp|im)$, is lognormal. This assumption is appropriate also for the case of structural systems with nonlinear viscous dampers [11]. The lognormal distribution parameters $\mu_{lnEDP|IM}(im)$ and $\sigma_{lnEDP|IM}(im)$, the former denoting the lognormal mean, and the latter the lognormal standard deviation (dispersion), can be evaluated from the response samples and they vary with the *IM* level considered [31].

It is noteworthy that in the case of a lognormal distribution, the relation between the mean response $\mu_{EDP|IM}(im)$, adopted in the deterministic approach, the lognormal mean $\mu_{lnEDP|IM}(im)$ and lognormal standard deviation $\sigma_{lnEDP|IM}(im)$ is:

$$\mu_{EDP|IM}(im) = \exp\left[\mu_{\ln EDP|IM}(im) + \frac{\sigma^2 \ln EDP \mid IM(im)}{2}\right] \tag{2}$$

Also the response percentiles can be easily evaluated once the distribution parameters have been calculated.

In the following, the term *deterministic demand* denotes the mean value measured for a set of seismic inputs with the same intensity [21]. The term "deterministic" underlines that there is a one-to-one relationship between the seismic input and the structural demand and information about the dispersion of the response is not considered. In this context, the MAF of exceedance of the deterministic demand coincides with $v_{IM}(im)$.

3. Probabilistic seismic performance assessment of frames equipped with viscous dampers

This section illustrates, through an example, the methodology adopted to evaluate: 1) the influence of the damper nonlinearity on the performance of frames equipped with viscous dampers and 2) the differences between the seismic demand estimated by the deterministic and the probabilistic approach. The comparison is performed by considering a family of case studies designed to achieve the same deterministic seismic performance but involving dampers with different values of the nonlinear parameter α. The methodology is articulated into three-stages, each one corresponding to a section.

3.1 Hazard scenario and design of dampers

The case study consists of a 9-storey steel moment-resisting frame building (Fig.1) designed as part of the SAC steel project and located in the same site in the Los Angeles area. The structural system consists of steel perimeter moment frames and interior gravity frames with shear connections and was designed in compliance with local code requirements and design practices for office building, by considering the gravity, wind and seismic load. Detailed descriptions of the structure are provided in many other works [24].

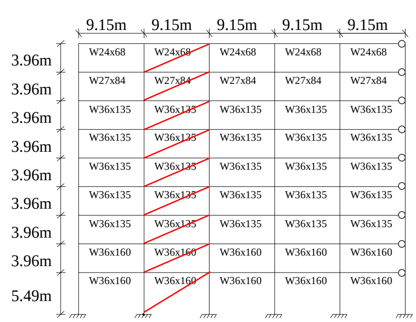

Fig. 1 – Building model description

The structural model is developed in Opensees [32] by adopting the general criteria used in [24, 25]. The structure is modelled as two-dimensional frames describing half of the symmetric buildings in the north–south direction. The inelastic members behaviour has been described by using distributed plasticity fiber element models with nonlinear material properties. The nonlinear geometrical effects induced by the vertical loads acting on both the interior frames and the modelled frame are included in the analysis by employing an elastic P-delta column with high axial stiffness and negligible bending stiffness, so that it does not contribute to resist the seismic induced loads. The floor vertical loads are assigned to this column at each floor level and a corotational formulation is used to capture the nonlinear geometrical effects. The damping properties inherent to the behaviour of the steel frame within the elastic range are described by using the Rayleigh damping model, whose parameters have been calibrated by assuming a 2% damping factor for the first two vibration modes. Table 1 reports the modal properties of the structure, i.e., the vibration periods, T_i, and the mass participation factors (normalized by the total mass), MPF_i, of the first three modes. The observed vibration periods are in close agreement with those reported in [24, 25].

Table 1 – Modal properties of the 9-storey frames.

mode	$\mathbf{T_i}$	$\mathbf{MPF_i}$
1	2.225	0.828
2	0.836	0.109
3	0.481	0.038

Fig.2 reports the capacity curve of the frame, obtained through pushover analysis by considering a lateral load pattern proportional to the first modal shape. The curve is in good agreement with the corresponding curve reported in [24].

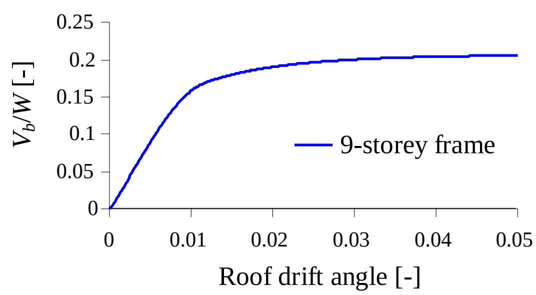

Fig. 2 – Capacity curve: base shear (normalized by the self weight W) vs roof drift angle.

The seismic intensity *IM* is described by the spectral pseudo-acceleration $S_a(T_1,2\%)$ of a linear elastic SDOF system with 2% damping ratio and fundamental vibration period equal to that of the structure T_1 [24].

This *IM* has been chosen because it represents the basis of the current seismic hazard maps and building code practice [21]. The choice of this *IM* is also driven by the aim of this study to evaluate the safety levels achieved by employing a “deterministic approach” for the dampers design consistent with modern seismic codes [18] which employ a response spectrum to define the seismic input.

The hazard curve corresponding to the chosen *IM* is reported in Fig.3a and is taken from [24]. It is in the form v_{IM} $(im)=k_0$ im^{-k1}, where k_0=0.00142 and k_1=3.25. In Fig.3a, the intensity levels corresponding to a probability of exceedance of 2%, 10% and 50% in 50yrs are also highlighted. These are the intensity levels considered for the assessment of the frames according to the codes. The case study is designed by considering the MAF of exceedance (probability of exceedance of 10% in 50 years), associated to the intensity 0.3676g (g is the gravity acceleration).

The record-to-record variability has been described by employing ground motions taken from the set of 60 records used in the SAC project, whose characteristics are reported in [23]. These records are characterized by different seismic intensities, frequency content, and duration. For each *IM* level covered in the multi-stripe analysis [29], the 30 ground motions with the closest *IM* values have been selected and scaled to that *IM* level.

This approach, yielding different ground motion combinations for the different *IM* levels considered, permits to avoid excessive scaling of the records. In Fig.3b, the pseudo-acceleration response spectra of the 30 records representative of the earthquake input at $IM = im_{ref}$ are reported together with the mean response spectrum. In the same figure, the spectral values at the first three vibration periods are also reported by circles.

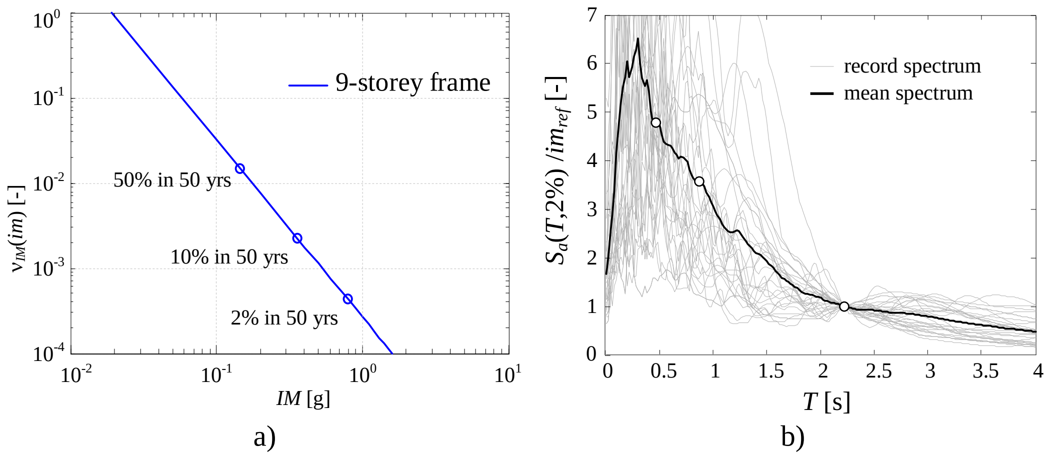

Fig. 3 – a) Hazard curve and b) pseudo-acceleration response spectra.

The viscous dampers design is carried out by assigning the damper properties, i.e., the exponent α and the viscous constant c_{di} at the various storeys. A deterministic performance objective is sought, consistently with modern seismic codes. The design objective corresponds to achieving a target mean value of $IDR_{\max}$, the maximum peak inter-storey drift among the various storeys, for the set of records scaled to the reference intensity level im_{ref}, with relevant exceedance rate $\nu_{IM}(im_{ref})=\nu_{ref}$. The same target mean value of $\mu_{IDR|IM}(im_{ref})$ at $IM=im_{ref}$ has been required to generate the set of solutions with added damping characterized by different nonlinearity levels. In particular, the starting value of $\mu_{IDR|IM}(im_{ref})$ for the bare frame is 2.41%, whereas the target value of $\mu_{IDR|IM}(im_{ref})$ is equal to 1.07%. This target value exactly matches an additional damping ratio of 30% to the first mode in the linear case (α =1). In this example, the same values of c_d have been assumed for the bracing systems at all the storeys. Table 2 reports the set of solutions corresponding to the different α values.

Table 2 – Damper design parameters for different levels of damper nonlinearity.

α [–]	0.15	0.3	0.6	1
c_d [kNs^{α}/m^{α}]	35750	15500	9900	8500

3.2 Probabilistic response and seismic demand hazard curves

Multi-stripe analysis is performed in this section to estimate, for the different levels of the dampers nonlinearity, the samples that define the statistical distribution of the *EDP*s of interest for the performance assessment. In particular, the parameters $\mu_{lnEDP|IM}(im)$ and $\sigma_{lnEDP|IM}(im)$ as well as the response percentiles are evaluated.

Additionally, response hazard curves for the different *EDP*s are derived based on Eq. (1). In this study the inter-storey drift as well as two local *EDP*s monitoring the damper performance (damper force and the stroke) are considered.

Fig.4a-c shows the variability of the response at different intensity levels, as synthetically described by the median, 84th and 16th percentiles of the considered *EDPs*, whereas the corresponding demand hazard curves $\nu_{EDP}(edp)$ are reported in Figure 5a-c, providing the MAF of exceedance of each response parameter. The reported plots refer to the two extreme values of the damper exponents, α =0.15 and α =1.00. The dotted lines are located at the reference seismic intensity im_{ref} and at the corresponding MAF of exceedance ν_{ref}.

It can be observed that the median value of IDR_{max} (design target parameter) is lower in the nonlinear case than in the linear case for low *IM* values, and becomes higher at high *IM* values (Fig.4a). The dispersion for the nonlinear case is greater than that shown by the linear one within the whole range of *IM* (Fig.4a), and this affects also the trend of the MAF of exceedance that is always higher than that of the linear one (Fig.5a), except in the proximity of ν_{ref} where the linear and the nonlinear curves tend to join one to each other.

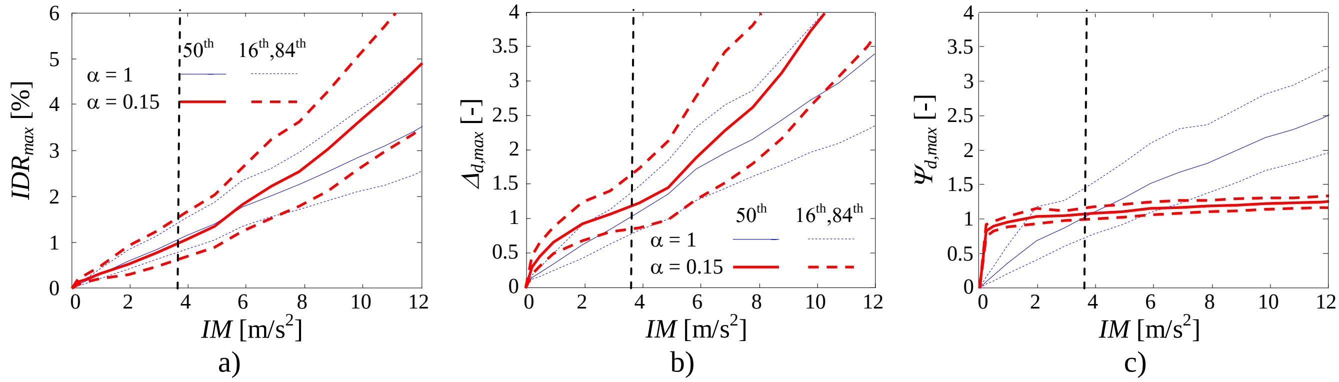

Fig. 4 – Response statistics for the nonlinearity levels α=1 and α=0.15: 50th, 16th and 84th percentiles vs. *IM* .

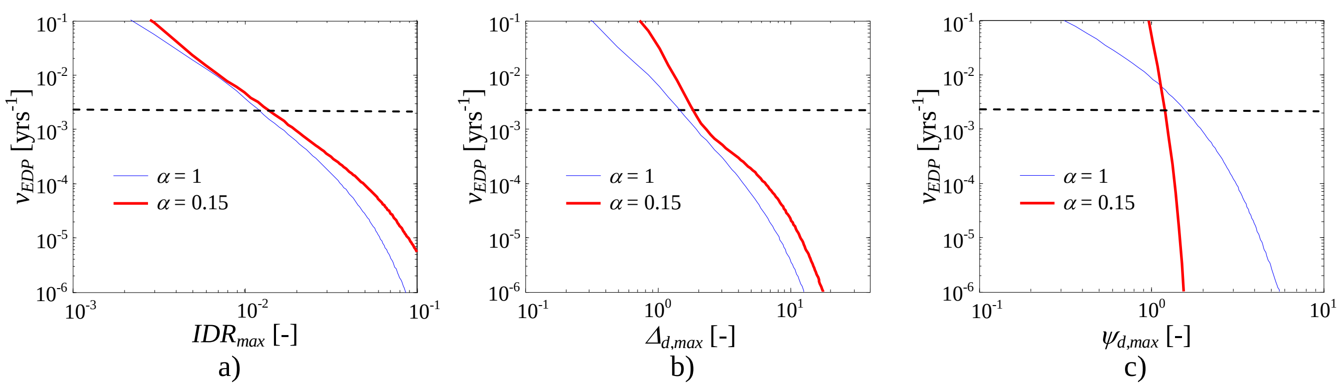

Fig. 5 – Response hazard curves corresponding to the nonlinearity levels α=1 and α=0.15.

Differently from IDR_{max}, the strokes $D_{d,i}$ and forces $F_{d,i}$ of the dampers are local parameters exhibiting statistics which usually differ from storey to storey and they show a different sensitivity to the seismic input variability [33, 34]. In order to compare results coming from different dampers it is necessary to normalize their *EDP*s. In the following the reported values of the strokes and of the forces are normalized storey by storey by dividing them by the deterministic demand value at the reference seismic intensity. The normalized strokes and forces are denoted as $\Delta_{d,i}$ and $\psi_{d,i}$, respectively. The damper response at the different storeys will be discussed in detail in the last section whereas in Fig.4b-c and Fig.5b-c only a synthetic information is reported by plotting the statistics and the demand hazard curves of the maximum $\Delta_{d,max}$ and $\psi_{d,max}$ values.

The global trend of the maximum (normalized) stroke (Fig.4b and Fig.5b) is similar to the trend of IDR_{max}, even if the two quantities may attain their maxima at different storeys. The maximum normalized damper forces $\psi_{d,max}$ (Fig.4c) show very different values and trends in the linear and nonlinear case. In the nonlinear case, the median values $\psi_{d,max}$ exhibit limited variations with the seismic intensity and the dispersion is low for all the *IM* levels. In the linear case, the median value increases almost linearly with *IM* and the dispersion remains almost constant. In the nonlinear case the force demand hazard increases only slightly by reducing the MAF of exceedance while very larger variations occur in the linear case (Fig.5c). The two hazard curves intersect one to each other, and the values of $\psi_{d,max}$ are lower in the linear case than in the nonlinear case for high ν values (i.e., more probable events), and higher for low ν values. Both the linear and nonlinear hazard curves of the normalized damper forces are steeper than the corresponding stroke curves, so the normalized force demand is lower than the normalized stroke demand for MAF of exceedance larger than the reference one.

3.3 Deterministic vs. probabilistic performance assessment

In order to compare and quantify the differences between the deterministic approach and the probabilistic approach at the reference condition considered for the design, the demand ratios $R_{EDP}(\nu_{ref})$ are numerically evaluated on the basis of the following definition:

$$R_{EDP}\left(\nu_{ref}\right)=\frac{edp\left(\nu_{ref}\right)}{\mu_{EDP|IM}\left(im_{ref}\right)} \tag{3}$$

The ratios $R_{EDP}(\nu_{ref})$ provide a comparison between the demand evaluated through the probabilistic approach for a MAF of exceedance ν_{ref} and the demand $\mu_{EDP|IM}(im_{ref})$ resulting from the deterministic approach.

It is noteworthy that the smaller the response dispersion, the closer the ratio $R_{EDP}(\nu_{ref})$ gets to 1. The observed $R_{EDP}(\nu_{ref})$ values are reported in Table 3 and they are higher than 1 for all the *EDP*s considered. Thus, the actual demand value with an exceedance rate of ν_{ref} is always larger than the corresponding mean demand value employed in the deterministic performance assessment. It is also worth to observe that the $R_{EDP}(\nu_{ref})$ values differ significantly for the various response parameters and they also change significantly by varying α.

Table 3 – Ratios R_{EDP} (ν_{ref}) for the different EDPs.

α	**0.15**	**0.3**	**0.6**	**1**
IDR_{max}	1.327	1.289	1.225	1.138
$\Delta_{d,max}$	1.275	1.202	1.224	1.197
$\Psi_{d,max}$	1.114	1.208	1.373	1.451

The ratio R_{EDP} might be calculated also by considering a value of ν different from ν_{ref}. For example, in Table 4 the values of the ratio $R_{EDP}(\nu)$ are reported for a target value of ν equal to 10^{-4}, as suggested by the probabilistic model code in [35] for ultimate limit state. These $R_{EDP}(\nu)$ values may be interpreted as amplification factors to be applied to the deterministic demand in order to obtain the desired safety level ν. Also in this case, as expected, the obtained values differ significantly for the various response parameters and they also change significantly by varying α.

Table 4 – Ratios R_{EDP} ($\nu=10^{-4}$) for the different EDPs.

α	**0.15**	**0.3**	**0.6**	**1**
IDR_{max}	4.552	4.511	3.853	3.320
$\Delta_{d,max}$	4.081	4029	3.739	3.371
$\Psi_{d,max}$	1.270	1.612	2.203	2.811

4. Reliability levels provided by simplified formulas

In this section, the MAF of exceedance of the demand provided by current code prescriptions for the damper design is evaluated by employing the hazard curves and the relationships between demand and MAFs of exceedance discussed in the previous sections. In particular, the seismic standards [2] suggest that velocity-dependent dissipation devices shall be capable of sustaining displacements equal to 200% of the maximum displacement calculated for earthquakes intensities with a 2% probability of being exceeded in 50 years.

Velocity-dependent devices should also sustain the forces associated to velocities calculated for the same earthquake intensity and amplified by the same factor. The 200% factor reduces to 130% if at least four devices are provided at each storey. The aim of this type of provisions is to extrapolate a conventional value of the demand with a MAF of exceedance suitable for a reliability assessment by starting from the seismic demand obtained from the structural analysis and referred to a lower MAF of exceedance. Table 5 and Table 6 show the results of this assessment for the case of linear dampers (α =1) and nonlinear dampers (α = 0.15).

Table 5 – Code design values and MAF of exceedance for stroke and force of the dampers (amp. factor 130%).

	α = 1.00						**α = 0.15**					
Storey	**D_d [m]**	**v_D [1/yrs]**	**r_D [-]**	**F_d [kN]**	**v_F [1/yrs]**	**r_F [-]**	**D_d [m]**	**v_D [1/yrs]**	**r_D [-]**	**F_d [kN]**	**v_F [1/yrs]**	**r_F [-]**
1	0.131	1.55E-04	0.42	17623	2.36E-04	0.31	0.156	2.15E-04	0.48	8043	7.04E-04	0.34
2	0.113	1.62E-04	0.45	13675	1.53E-04	0.29	0.131	2.29E-04	0.53	7710	7.51E-04	0.50
3	0.106	1.57E-04	0.44	12331	1.11E-04	0.25	0.114	2.39E-04	0.54	7519	8.85E-04	0.47
4	0.096	1.52E-04	0.42	11662	9.91E-05	0.22	0.093	2.47E-04	0.52	7397	9.89E-04	0.56
5	0.078	1.53E-04	0.40	10408	1.01E-04	0.21	0.061	2.38E-04	0.46	7035	8.26E-04	0.52
6	0.063	1.54E-04	0.38	9223	1.23E-04	0.23	0.04	2.04E-04	0.35	6722	5.45E-04	0.59
7	0.053	1.62E-04	0.36	8280	1.64E-04	0.25	0.029	1.65E-04	0.14	6725	1.58E-04	0.52
8	0.043	1.73E-04	0.36	6979	1.53E-04	0.25	0.026	3.81E-04	0.09	6817	2.73E-05	0.50
9	0.027	1.96E-04	0.40	4854	4.33E-05	0.19	0.024	5.20E-04	0.06	6864	1.90E-10	0.35
max			**0.45**			**0.31**			**0.54**			**0.59**

Table 6 – Code design values and MAF of exceedance for stroke and force of the dampers (amp. factor 200%).

	α = 1.00						α = 0.15					
Storey	**D_d [m]**	**v_D [1/yrs]**	**r_D [-]**	**F_d [kN]**	**v_F [1/yrs]**	**r_F [-]**	**D_d [m]**	**v_D [1/yrs]**	**r_D [-]**	**F_d [kN]**	**v_F [1/yrs]**	**r_F [-]**
1	0.201	3.04E-05	0.081	27113	1.92E-05	0.025	0.239	6.01E-05	0.133	8579	1.22E-04	0.060
2	0.174	3.30E-05	0.091	21038	7.62E-06	0.015	0.202	6.67E-05	0.156	8225	1.74E-04	0.115
3	0.163	3.15E-05	0.088	18971	2.63E-06	0.006	0.176	6.98E-05	0.159	8023	2.36E-04	0.125
4	0.148	2.95E-05	0.081	17941	1.90E-06	0.004	0.143	7.08E-05	0.150	7889	3.05E-04	0.172
5	0.12	2.86E-05	0.074	16013	2.36E-06	0.005	0.093	4.88E-05	0.094	7507	2.46E-04	0.155
6	0.097	2.80E-05	0.068	14189	4.12E-06	0.008	0.061	2.73E-05	0.047	7170	1.61E-04	0.174
7	0.081	2.79E-05	0.062	12738	7.92E-06	0.012	0.045	6.34E-06	0.005	7173	3.38E-05	0.111
8	0.066	3.27E-05	0.068	10737	8.29E-06	0.014	0.04	3.12E-06	0.001	7268	4.02E-06	0.073
9	0.042	4.42E-05	0.091	7467	1.70E-06	0.007	0.037	2.56E-06	0.000	7318	2.48E-12	0.005
max			**0.091**			**0.025**			**0.159**			**0.174**

In particular, they report the code design values of the damper stroke D_d and forces F_d at the storeys, obtained by amplifying strokes and damper velocities with a factor of 130% (Table 5) and of 200% (Table 6). The MAFs of exceedance of the design values (including the amplification factor) deduced from the hazard curves and the ratios r_D and r_F between these MAFs of exceedance and the MAFs of exceedance corresponding to the reference (not amplified) values of strokes D_0 and forces F_0 are also reported. For each case also the maximum values of the ratios r_D and r_F are reported. It is worth to recall that, the more the approximated approach based on amplification factors is effective, the more the ratio values of r_D and r_F are similar.

In Table 5, it can be observed that in the linear case, despite the ratios r_D and r_F are quite different throughout the storeys, the force-related ratios are significantly lower than the values observed for the strokes. The MAFs of exceedance of the magnified strokes D_d are equal or lower than 45% of the MAFs of exceedance of the reference value D_0 while the values concerning the damper forces reduce to 31% or more. By passing from the linear to the nonlinear case, the design values of strokes and forces change differently storeys by storeys (as well as the MAFs) and some trends can be observed for what concerns the r_D and r_F variability. In fact, the r_F values at storeys (and the maximum value too) became notably larger, while the strokes, even having a similar rising trend, show some exception for the upper levels storey, where smaller ratios are observed. Unlike the linear case, the maximum r_F is a bit greater than the maximum $r_{D.}$

The discussed trends and the differences between the linear and nonlinear case are confirmed in the Table 6, corresponding to the stronger extrapolation inherent to the factor 200%. The only difference respect to the previous case lies into the notably lower values of MAFs of exceedance, as well as the ratios r_D and r_F.

4. Conclusions

This paper analyzes the influence of damper nonlinearity level on the probabilistic seismic performance of building frames equipped with viscous dampers subjected to an uncertain seismic input (set of ground motions scaled to different intensity levels). In particular, a probabilistic methodology is adopted to evaluate how the viscous damper exponent α affects the statistics of different response parameters and the seismic performance as measured in terms of demand hazard curves. A comparison between the deterministic and probabilistic estimates of the seismic demand is carried out by evaluating the performance of a family of case studies consisting of a 9-storey frame equipped with dampers characterized by different α, designed to ensure the same deterministic performance objective. Global (maximum inter-storey drift) and local (strokes and forces at dampers) demand parameters are reported and discussed. Results show that the response statistics (*edp* vs *im*) are notably influenced by the nonlinear exponent α. This influence is different for the various parameters considered and also changes with the seismic intensity level. These differences in the response reflect on the demand hazard (ν vs *edp*) and quite similar qualitative trends have been observed for the case studies and seismic scenarios considered. Moreover, the probabilistic seismic demand corresponding to the reference value of MAF of exceedance ν_{ref} is higher than the deterministic design demand for all the observed parameters. The difference between the two values reflects the dispersion of the response due to record-to-record variability, which is different for the different *EDP*s and α levels considered. In the final part of the paper, the analysis results are employed to evaluate the reliability of simplified approaches usually adopted in codes of practice for the damper design. These approaches, which generally magnify the deterministic results coming from the structural analysis to estimate values of the demand to be used for the reliability assessment, lead to not uniform differences in the MAF of exceedance of the linear and nonlinear case. Based on these results, it is concluded that the magnification factors should depend on the nonlinear parameter α and a more extended investigation is necessary to provide reliable approximated formulas for their estimation.

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