

UNIVERSITÀ POLITECNICA DELLE MARCHE Repository ISTITUZIONALE

Influence of viscous dampers ultimate capacity on the seismic reliability of building structures

This is the peer reviewd version of the followng article:

Original

Influence of viscous dampers ultimate capacity on the seismic reliability of building structures / Scozzese, Fabrizio; Gioiella, Laura; Dall'Asta, Andrea; Ragni, Laura; Tubaldi, Enrico. - In: STRUCTURAL SAFETY. - ISSN 0167-4730. - 91:(2021). [10.1016/j.strusafe.2021.102096]

Availability:

This version is available at: 11566/299441 since: 2024-04-27T16:51:45Z

Publisher:

Published DOI:10.1016/j.strusafe.2021.102096

Terms of use:

The terms and conditions for the reuse of this version of the manuscript are specified in the publishing policy. The use of copyrighted works requires the consent of the rights' holder (author or publisher). Works made available under a Creative Commons license or a Publisher's custom-made license can be used according to the terms and conditions contained therein. See editor's website for further information and terms and conditions. This item was downloaded from IRIS Università Politecnica delle Marche (https://iris.univpm.it). When citing, please refer to the published version.

Influence of viscous dampers ultimate capacity on the seismic reliability of building structures

Fabrizio Scozzese ^a, Laura Gioiella ^a, Andrea Dall'Asta ^a, Laura Ragni ^b, Enrico Tubaldi ^c

4

3

5 *a SAAD, University of Camerino, Viale della Rimembranza 3, 63100 Ascoli Piceno, Italy*

6 ^b Department of Civil and Building Engineering and Architecture, Università Politecnica delle Marche, Via Brecce Bianche,

7 60131 Ancona, Italy

8 ^c Department of Civil and Environmental Engineering, University of Strathclyde75 Montrose Street, Glasgow G1 1XJ, UK

9 ABSTRACT

10 Anti-seismic devices should be designed with proper safety margins against their failure, because the reliability of

- 11 the structural system where they are installed is strongly influenced by their reliability. Seismic standards generally
 - prescribe safety factors (reliability factors) amplifying the device responses at the design condition, in order to reach a target safety level. In the case of Fluid Viscous Dampers (FVDs), these factors are applied to the stroke and velocity,
 - 14 and their values are not homogenous among seismic codes.

This paper investigates the influence of the values of the safety factors for FVDs on the reliability of the devices and of the structural systems equipped with them. An advanced FVD model is employed to account for the impact forces arising when the dampers reach the end-stroke and the brittle failure due to the attainment of the maximum force capacity. The effect of damper failure on both the fragility and the seismic risk of the structural system is investigated by performing multiple-stripe analysis and monitoring different global and local demand parameters. In particular, a

20 parametric study has been carried out, considering two case studies consisting of a low-rise and a medium-rise steel 21 building, coupled with a dissipative system with linear and nonlinear properties and studying the consequences of

different values of safety factors for stroke and forces. The study results give evidence to the potential brittle

behaviour of the coupled system and provide information about the relationships between damper safety factors and

24 effective structural reliability. Some preliminary suggestions are given on possible improvements of current design

25 approaches and on the values of the reliability factors to be considered for future code revision.

Keywords: energy dissipation; failure; seismic risk and safety; reliability factors; multiple stripe analysis; nonlinear
 dynamic analysis.

28 1 INTRODUCTION

Fluid viscous dampers (FVDs) are devices widely used for seismic passive protection of both new and existing structures. They are widely employed for reducing displacements and interstorey drift demands in newly-designed structures as well as in existing ones by using both external and internal configurations [1]-[8].

Several approaches are to date available for designing both size and location of viscous dampers 33 within a building frame based on direct procedures [1][9][10][11][12] or optimization methods 34 35 [13][14] (see [15] for a thorough review of design strategies for viscous dampers). These design approaches generally allow to control the seismic performance of buildings under the design seismic 36 37 intensity level. However, the reliability under extreme, low-probability earthquake events may be 38 characterized by low robustness and inadequate safety levels because dampers usually exhibit a brittle 39 collapse behaviour and their failure may trigger the collapse of the whole system. Consequently, the 40 choice of adequate safety factors for the design of the dampers is of paramount importance for 41 obtaining a satisfactory performance under strong actions and controlling the probability of failure.

42 It is noteworthy that the robustness under extreme loadings is usually not a concern for traditional 43 steel and concrete structures, thanks to their redundant static schemes and ductile material properties, 44 able to redistribute the structural damage. Thus, frame structures generally behave well under 45 exceptional actions, provided that details or connections are adequately designed [16][17][18]. Moreover, procedures to make high quality structural components are consolidated as well as safety 46 47 coefficients to be used in the design. As a result, while code conforming traditional solutions are characterized by adequate reliability levels, code conforming structures equipped with fluid viscous 48 49 dampers may show reliability levels below the target suggested by the design codes and the technical 50 literature [19][20][21][22][23].

51 In order to evaluate the probability of collapse of structures equipped with dampers, risk analyses must be performed by using probabilistic approaches [24]-[32]. Recent probabilistic analyses have 52 investigated some specific issues, such as the effect of ground motion variability on the response of 53 54 systems equipped with either linear and nonlinear viscous dampers [27][28][29]; the influence of the 55 degree of nonlinearity of the dampers [28][30], and the effect of the damper parameters variability [30][31][32] stemming from the device manufacturing process, as acknowledged by the main 56 57 international Standards for seismic structural design [19][20][21][22]. However, in these studies the device failure was not explicitly taken into account. Thus, more accurate studies simulating the effect 58 59 of the device failure should be carried out to provide a better evaluation of the structural reliability 60 under strong earthquakes.

61 This paper aims to evaluate the consequences of the dissipative device failure on the seismic performance of two benchmark structural systems, by adopting a model describing the brittle failure 62 63 of the devices due to the attainment of the force capacity, related to the over-velocity or to the achievement of the end-stroke and its influence on the structural reliability. In particular, it is assumed 64 65 that a brittle failure occurs in the dampers once the maximum force is attained, consistently with the viscous damper numerical model proposed in [33]. The problem is analysed by using a probabilistic 66 67 approach and by evaluating the mean annual frequency of exceedance of different values of the multiple response parameters related to the performance of dampers and structure. For this purpose, 68 Multi Stripe Analysis (MSA) [34] is carried out and results are given in terms of fragility curves and 69 70 demand hazard curves for the engineering demand parameters (EDPs) of interest. Fragility analyses 71 of failure of dampers give evidence to the failure sequence and potential lack of robustness of the 72 coupled system.

The two case studies analysed here consist of steel buildings with different dynamic properties, already considered as benchmark cases in previous studies (SAC Phase II Steel Project, [42]). For consistency with the adopted benchmark case studies, the seismic hazard is also assumed equal to the one of [42]. The dissipative system is dimensioned to provide an added damping equal to 30%, using both linear and nonlinear devices, by varying their degree of nonlinearity among three values. The capacity of the dampers (stroke and strength) is evaluated at the design condition, corresponding to a seismic action with Mean Annual Frequency (MAF) of exceeding equal to $2x10^{-3}$.

80 Some preliminary results under increasing harmonic load histories are reported to illustrate the model capabilities and the sequence of failures triggered by the damper failure. Subsequently, 81 82 fragility curves and demand hazard curves are illustrated, where the structural performance is analysed considering MAF of exceeding up to 10⁻⁵ 1/yr. Results obtained by considering different 83 84 amplification factors for the design of damper parameters are evaluated and compared. In particular, 85 the prescriptions of European codes [20][21] and American Standards [19] are considered. Parametric 86 analysis includes both the case of linear viscous dampers and nonlinear viscous dampers with different nonlinear properties. The case without dampers and the one in which the damper failure is 87 88 disregarded are also considered for comparison purposes.

The obtained results shed light on the influence of the damper failure on the global reliability of the system and on the effect of the amplification factors on the MAF of failure.

91 2 FVDS MODELLING AND SEISMIC CODE PROVISIONS

92 2.1 Fluid viscous dampers modelling

93 The constitutive law of a fluid viscous damper (FVD) can be described through the following94 relationship [33][43]:

$$F_d(v) = c|v|^{\alpha} sgn(v) \tag{1}$$

95 where v is the relative velocity between the device ends, F_d is the damper resisting force, |v| is 96 the absolute value of v, sgn is the sign operator, c and α are two constitutive parameters: the former 97 is an amplification factor, while the latter describes the damper nonlinear behaviour.

It is worth noting that viscous dampers can be produced with α values ranging from 0.1 and 2. Devices with $\alpha > 1$ are not dissipative and are used as shock transmitters. Devices with $0.1 \le \alpha \le 1.0$ are all potentially suitable for seismic energy dissipation, among these values, the range $0.3 \le \alpha \le 1.0$ is the most widespread [35][36][37][38].

A fluid viscous damper generally consists of a steel cylinder filled of a silicone fluid, within which 102 103 a steel piston with small orifices on its head can move. In case of seismic events, the fluid is forced 104 to pass through the orifices, moving from one side to the opposite side of the cylinder, thus dissipating 105 into heat the input mechanical energy. The higher is the velocity of the movement, the greater is the 106 dissipated energy. The cylinder is equipped with spherical hinges at its ends to avoid device bending. FVDs are generally connected to the structure by a stiff connection, consisting in a driver brace, 107 108 dimensioned using an over-strength factor with respect to the viscous device. The stiffness of the 109 driver brace is an important feature, because it needs to be sufficiently high to allow the device to be 110 effective in dissipating energy. Further details on the damper components and their behaviour can be 111 found in [33].

112 The failure of a damper is related to the exceedance of its strength capacity and can be attained 113 because of the forces related to the end-stroke impact or can be due to excessive piston velocity. 114 According to the described behaviour, dampers are generally classified and tested with reference to 115 two characteristic parameters: the maximum values of stroke $\Delta_{d,max}$, and the maximum transmissible 116 force $F_{d,max}$.

117 The end-stroke can be attained both in tension (maximum elongation of the device) and in 118 compression (maximum shortening of the device). However, the attainment of an impact does not 119 strictly imply the damper failure because the impact force may be lower than the device strength.

The second mechanism refers to the attainment of the maximum viscous force due to an excessive value of the velocity of the piston (over-velocity with respect to the design value). This extreme value of the force can induce a leak of the fluid or can damage the damper components, resulting in the failure of the device. It is noteworthy that once the maximum capacity ($F_{d max}$) is attained, the resulting failure mechanisms is brittle, thus making the device ineffective, with no residual ability to sustain loads or dissipate energy.

126 The model, proposed hereinafter, aims to describe the two aforesaid mechanisms using the damper 127 model, depicted in **Fig. 1**. It is composed of three elements: a dashpot, describing the dissipative 128 behaviour; a hook and gap element, set in parallel to the dissipative device, which simulate the impact 129 due to either excessive shortening $(-\Delta_{d,max})$ or elongation $(+\Delta_{d,max})$; and a third element, set in 130 series with the others, simulating the failure due to the attainment of the force capacity. In this paper, 131 the strength capacity is assumed to be the same in traction and in compression and the failure occurs 132 when the modulus of damper force attains the limit value $F_{d,max}$.

The damper model discussed above is implemented in OpenSees [44] using two-node link 133 134 elements simulating each of the three components, while various material properties are used to describe the different behaviours. A "Viscous material" is used for the dissipative element, by 135 assigning the values of the constitutive parameters c and α . An "ElasticMultilinear material" depicts 136 the force-displacement relationship related to impacts occurring both for elongation and shortening. 137 138 Finally, a "MinMax material" is used to simulate the brittle failure, assigning the value of the strength capacity $F_{d,max}$. The stiffness of the "MinMax material" can be used to model the overall 139 deformability of damper, connections, and brace. However, once the strength capacity is reached, the 140

141 element fails and does not provide any more contribution in terms of reaction force.



142 143

Fig. 1. Dissipative device model encompassing the failure mechanisms

144 2.2 International regulatory framework: an overview

145 Modern seismic codes prescribe that anti-seismic devices shall be dimensioned starting from the values of the control parameters evaluated for seismic design actions having an assigned probability 146 147 of exceedance. Then, the capacities of the devices are assigned amplifying these control parameters, 148 which are stroke and force for the FVDs, by means of amplification factors, or reliability factors, in order to ensure a target level of safety. This procedure makes simpler the dimensioning, avoiding an 149 explicit probabilistic analysis considering all the uncertainties of interest. Generally, in the case of 150 151 dampers, Standards suggest reliability factors that account for uncertainties related to damper 152 response, manufacturing tolerances, ageing phenomena and temperature variations, in addition to uncertainties related to seismic action and structure. 153

The amplification factors proposed by Codes are two and aim to control the two failure mechanisms discussed above. The former, here denoted by γ_{Δ} , amplifies the maximum stroke measured at design condition. The amplified stroke must not exceed the damper capacity $\Delta_{d,max}$. The latter, here denoted by γ_{ν} , amplifies the maximum velocity measured at design condition. Damper force is obtained by Eqn. (1) and must not exceed the damper capacity $F_{d,max}$.

159 In this study we refer to the provisions of US and EU Codes. In the US, the standard for the retrofit 160 of existing buildings (ASCE 41-2017) [19] provides clear indications on the values and applicability 161 of safety factors for viscous dampers. It prescribes that all energy dissipation devices shall be capable 162 of sustaining the force and displacement associated with a velocity equal to 130% ($\gamma_{\Delta} = \gamma_{\nu} = 1.3$) 163 or 200% ($\gamma_{\Delta} = \gamma_{\nu} = 2.0$) of the maximum calculated velocity for that device. The two options

depend on the number of devices installed within each storey and each direction of the building and 164 the performance objective assumed [19]. The safety coefficients should be applied to the velocity 165 166 calculated with a seismic action characterized by an exceedance probability of 5% in 50 years for existing buildings or the Risk-Targeted Maximum Considered Earthquake (MCE) [22] for the new 167 168 ones. The value of 200% applies only in the case that less than four energy dissipation devices are 169 installed in a given storey along one principal direction of the building, otherwise the coefficient 170 130% can be used. In the ASCE 41-2017 [19] the property variations of the energy dissipation devices 171 are taken into account through the so called property modification factors (λ factors). These factors 172 define the upper- and lower-bound properties of the devices, accounting for manufacturing tolerances, 173 device characteristics not explicitly considered during testing and environmental effects and aging. 174 The λ factors are not considered in the present work.

In Europe, the reference standards are Eurocode 8 [21] and EN15129 [20], which regulates the 175 176 devices production and integrates the Eurocode prescriptions concerning design and structural reliability. In the section "General design rules" of EN15129, it is specified that, for anti-seismic 177 178 devices (seismic isolators excluded), a reliability factor γ_x equal or greater than 1 shall be applied to 179 the effects of the design seismic action on the devices, while an over-strength factor $\gamma_{Rd} = 1.1$ is recommended for designing the connections with the structure. According to Eurocode 8, the design 180 181 seismic action shall be evaluated considering the Ultimate Limit State (ULS) hazard intensity, 182 characterized by an exceedance probability of 10% in 50 years, corresponding to a MAF of exceedance equal to 2.1x10⁻³. The value of the reliability factor should be provided by the Eurocodes 183 184 (as specified in section 4.1.2, note 2 of EN15129), but this information is lacking in the current 185 version. The same EN15129, in the section dedicated to velocity dependent devices, prescribes that 186 the design velocity shall be amplified by a reliability factor $\gamma_{\nu} = 1.5$. However, it is worth to observe 187 that no amplification factor is specified for the damper stroke, which means that the stroke capacity 188 could be determined by assuming a γ_{Δ} factor equal to 1.0. The Italian standard, NTC 2018 [45], is 189 compatible with European codes but its prescriptions are more demanding, requiring that design 190 velocities are amplified by the same reliability factor γ_v given by the EN15129, but prescribes that the response parameters of the devices are evaluated at the Collapse Limit State (seismic actions with 191 192 exceedance probability of 5% in 50 years). However, similarly to EN15129, no specific indications 193 are given about the damper stroke capacity. Similarly to the US Code [19], also the EN15129 provides 194 tolerance limits (t_d) for velocity dependent devices which are relevant to variations within the supply 195 (statistical variations), as well as variations due to temperature, ageing, etc. These indications 196 regarding the tolerances are also adopted by the Italian NTC 2018 [45] and are not considered in this 197 work.

198 3 PARAMETRIC ANALYSIS

In this section, a parametric analysis is carried out to understand how the failure of dampers affects the seismic response and performance of steel frame structures. Two different steel moment-resisting frames are considered, representative of low-rise and medium-rise building. The buildings are equipped with FVDs with different non-linearity levels, corresponding to values of the damper exponent α of 1.0, 0.6 and 0.3. Different values of the amplification factors are considered for dampers design. The obtained results are also compared to those corresponding to two limit cases: a) without dampers (bare frame), and b) dampers do not fail.

206 3.1 Steel buildings frame structures

The two case studies consist of a 3-storey and 9-storey steel moment-resisting frame buildings, designed as part of the SAC Phase II Steel Project, and located in the Los Angeles area. The buildings were designed for gravity, wind, and seismic loads in order to conform to local code requirements and have been widely used as benchmark structure in several studies concerning structural response control (e.g., [2][42][28][46]). **Fig. 2** illustrates the structural system of the buildings, consisting of 212 perimeter moment-resisting frames and internal gravity frames with shear connections. The numerical

model of the buildings consists only in a two-dimensional frame, representing one half of the structure 213

214 in the north-south direction, which is the short and also the weak direction of the buildings. Fig. 2

also shows the main geometrical details and dimensions of the steel members (wide-flange sections 215

are used for both columns and beams), together with the locations of the fluid viscous dampers, whose 216

217 design is described in section 3.3. Further details concerning the structural geometry and loads can







Fig. 2. Case studies: (a) elevation (red lines highlight FVDs location) and (b) plan (thick lines highlight moment-resisting frames) of 3-storey frame; (c) elevation and (d) plan of 9-storey frame. 220

The finite element models of the systems are developed in OpenSees [44] following the same 221 methodology described in [28] and briefly recalled below. A distributed plasticity approach is adopted 222 [47][48], with nonlinear force-based elements and fibre sections with Steel02 uniaxial material, 223 accounting for the hysteretic behaviour of the members. A corotational approach for the system 224 225 coordinate transformation is used to perform large displacement (small strain) analysis and thus account for the nonlinear geometrical effects, whereas an elastic fictitious P-delta column is 226 introduced to consider the vertical loads carried by the inner gravity frames (not explicitly modelled). 227 228 The strength and deformability of panel zones are neglected. The inherent damping properties are 229 accounted through the Rayleigh model by assigning a 2% damping ratio at the first and second 230 vibration modes. Table 1 reports, for both the bare buildings, the first three estimated vibration 231 periods T_i , together with the related mass participant factors normalized with respect to the total mass, 232 (*MPF_i*).

233

3-st	corey case st	udy	9-storey case study				
Mode	$T_i[\mathbf{s}]$	MPF_i	Mode	$T_i[\mathbf{s}]$	MPF_i		
1	0.995	0.827	1	2.225	0.828		
2	0.325	0.136	2	0.836	0.109		
3	0.173	0.037	3	0.481	0.038		

Table 1. Vibration periods for the bare 3-storey and 9-storey steel moment-resisting frame.

235 3.2 Seismic hazard

236 For consistency with the adopted benchmark case studies, the seismic hazard is based on [42]. The 237 spectral pseudo-acceleration $S_a(T_1)$ of a linear elastic SDOF system with 2% damping ratio and fundamental vibration period equal to that of the structure T_1 is considered as intensity measure (IM). 238 239 Such IM also represents the basis of the current seismic hazard maps and building code practice [49]. 240 Fig. 3 illustrates the hazard curves corresponding to the chosen IM, for the three-storey and the ninestorey building frames. The IM levels at which MSA is performed are 20 (highlighted by circles in 241 Fig. 3), whose corresponding values of spectral accelerations are summarised in Table 2. The IM 242 243 values corresponding to the main limit states suggested by codes are identified by red circles, and 244 they correspond to seismic events with exceedance probability of 50%, 10% and 2% in 50 years.

The record-to-record variability effects are taken into account in the analyses by considering the set of 60 records used in the SAC project [2]. These records are characterized by different seismic intensities, frequency content, and duration. At each intensity level, a subset of 30 ground motions is taken from this set, with *IM* values closest to the considered *IM* level, in order to minimize the scaling procedure operated for making the samples conditional to the *IM*. Further details and features of these records can be found in [2].

For what concerns the FVDs design, this is carried out by considering the set of 30 records corresponding to a MAF of exceedance $v_{design} = v_{IM}(im_{design}) = 0.0021$ 1/yr (probability of exceedance of 10% in 50 years), associated to the intensities $im_{design} = 0.8866$ g for the three-storey frame and $im_{design} = 0.3676$ g for the nine-storey frame (g is the gravity acceleration).



Fig. 3. IM hazard curves for the 3- and 9-storey buildings.

256



257 Fig. 4. Ground motion response spectra averaged at every intensity level: (a) 3-storey; (b) 9-storey.



1.48

0.82

258

259

Table 2. Correspondence between IM levels and spectal accelerations (in g).

1.68

1.00

1.99

1.30

2.35

1.68

2.78

2.19

3.28

2.85

3.89

3.70

4.00

4.00

260 3.3 Damping systems

 $\overline{3}$ -storey S_a(T₁=1.0)

9-storey $S_a(T_1=2.2)$

1.01

0.45

1.20

0.59

261 The design of the FVDs is carried out to enhance the buildings performance under a seismic 262 scenario with a 10% probability of exceedance in 50 years (ULS scenario according to Eurocode 8). 263 To this aim, a target value $\xi_{add} = 30\%$ has been chosen for supplemental damping. It is worth noting 264 that a damping ratio higher than 30% is usually not recommended because it may lead to a too significant modification of the natural dynamic properties of the building, with potentially detrimental 265 266 effects in terms of absolute accelerations ([2][15][50]). For this reason, the value of 30% is assumed 267 to investigate an upper bound of the retrofitting scenarios with passive seismic protection strategies. This value is expected to lead to the worst consequences in case of dampers failure. 268

Dampers design is initiated under the hypothesis of linear viscous behaviour ($\alpha = 1.0$); constants 269 270 c_i required to achieve the target damping ratio ξ_{add} are thus calculated for each building storey using the general formula proposed by the ASCE/SEI-41 [19]: 271

$$\xi_{add} = \frac{T \sum_{j} c_j f_j^2 \phi_{rj}^2}{4\pi \sum_{i} m_i \phi_i^2} \tag{2}$$

272 where the index j = 1, ..., n denotes the *j*-th device, T is the period of the first vibration mode of 273 the building; f_i is a magnification factor related to the installation scheme of dampers; ϕ_{ri} the first 274 modal relative displacement between the ends of the damper j in the horizontal direction; m_i is the 275 mass of the *i*-th storey and ϕ_i is the horizontal first modal displacements of the *i*-th storey.

276 In the present study, the dampers are installed in a diagonal arrangement, therefore $f_i = \cos\theta$, where θ is the angle between the horizontal direction and the *j*-th diagonal brace. Moreover, the damping 277 coefficients of the linear devices have been distributed proportionally to the storey shear force of the 278 279 first mode of the bare frame. As suggested in [9], the relation between the damping coefficient of a 280 single storey, c_i , and the total damping of the building, $\sum_i c_i$ can be expressed as:

$$c_j = \left(V_j / \sum_i V_i \right) \sum_i c_i \tag{3}$$

where V_j is the shear force of the *j*-th storey. By substituting Eqn. (3) into Eqn. (2), it is possible to achieve the total supplemental damping ξ_{add} as:

$$\xi_{add} = \frac{T \sum_{j} \left[V_j(\sum_{i} c_i) \left(\cos \theta_j \phi_{rj} \right)^2 \right]}{4\pi (\sum_{i} m_i \phi_i^2) (\sum_{i} V_i)}$$
(4)

Eqn. (4) can be rearranged to find the total damping coefficient of the structure, $\sum_i c_i$, and the damping coefficient at the *j*-th storey can be finally expressed as:

$$c_j = \frac{4\pi\xi_{add}V_j\sum_i m_i\phi_i^2}{T\sum_i V_i (\cos\theta_i\phi_{ri})^2}$$
(5)

285 Having determined the damping coefficients of the devices for the linear case, the viscous 286 coefficients for the nonlinear FVD corresponding to given value of the exponent α , are evaluated 287 following the approach outlined in [9], [51], [52] and based on the equivalence of the energies dissipated by the linear and nonlinear FVDs. For this purpose, seismic analyses of the system with 288 linear devices are carried out under a set of 30 recorded ground motions, scaled to the design intensity 289 290 level (i.e., exceedance probability of 10% in 50 years) as discussed in the previous chapter. The mean 291 response in terms of roof displacement of the building, A, is then used to determine the equivalent 292 nonlinear damping coefficients through the following general expression:

$$\xi_{add} = \frac{T^{2-\alpha} \sum_j c_j \lambda f_j^{1+\alpha} \phi_{rj}^{1+\alpha}}{(2\pi)^{3-\alpha} A^{1-\alpha} \sum_i m_i \phi_i^2}$$
(6)

293 where ϕ_i is the modal displacement shape normalised to a unit value at the roof and λ is given by 294 the following expression:

$$\lambda = 2^{2+\alpha} \frac{\Gamma^2 (1+\alpha/2)}{\Gamma(2+\alpha)} \tag{7}$$

295 in which Γ is the gamma function.

Eqn. (6) can be specialized to the case of dampers with viscous constant distributed proportionally to the storey shear force of the first mode of the bare frame, installed in a diagonal arrangement. It can be then rearranged to obtain the nonlinear damping coefficient c_i at each elevation, as:

$$c_j = \frac{\xi_{add} (2\pi)^{3-\alpha} A^{1-\alpha} V_j \sum_i m_i \phi_i^2}{T^{2-\alpha} \sum_i V_i \lambda \cos \theta_i^{1+\alpha} \phi_{ri}^{1+\alpha}}$$
(8)

Table 3 and **Table 4** report the properties of the dissipative devices, c_j and α , for the 3-storey and 9-storey buildings, respectively, for the various levels of dampers nonlinearity considered. It is noteworthy that the maximum interstorey drift along the building height, averaged over the 30 records considered, is equal to 3% and 2.1% respectively for the three-storey and nine-storey bare frames. With the addition of the dampers, they become respectively 1.2% and 1.0%.

Table 5 and Table 6 report the values of mean displacement $\Delta_{d,j}$, force $F_{d,j}$ and velocity v_j demand for the dampers, evaluated at the design condition. These values result in a probability of failure of the dampers of about 50% under the design earthquake level, if no amplification factors are considered for the damper response parameters.

- 308
- 309
- 310

Table 3. 3-storey building damping properties for different levels of damper nonlinearity.

		Floor 1	Floor 2	Floor 3
Case study	α	<i>C</i> ₁	<i>C</i> ₂	<i>C</i> ₃
		[<i>k</i>	xNs ^a /m	α]
3-storey	1	13,780	11,914	7428
	0.6	7477	6465	4031
	0.3	4669	4037	2517

Table 4. 9-storey building damping properties for different levels of damper nonlinearity.

		Floor 1	Floor 2	Floor 3	Floor 4	Floor 5	Floor 6	Floor 7	Floor 8	Floor 9
Case study	α	<i>C</i> ₁	<i>C</i> ₂	<i>C</i> ₃	C_4	C_5	<i>C</i> ₆	<i>C</i> ₇	<i>C</i> ₈	C9
	$[kNs^{\alpha}/m^{\alpha}]$									
9-storey	1	48,103	46,834	44,578	41,282	36,918	31,534	25,199	17,903	9675
	0.6	17,506	17,044	16,233	15,024	13,435	11,476	9171	6515	3521
	0.3	8133	7899	7518	6962	6226	5318	4250	3019	1632

 Table 5. 3-storey building damper design parameters at the design condition.

3-storey building									
	Floor 1	Floor 2	Floor 3	Floor 1	Floor 2	Floor 3	Floor 1	Floor 2	Floor 3
α	$\Delta_{d,1}$ [mm]	$\Delta_{d,2}$ [mm]	$\Delta_{d,3}$ [mm]	F _{d,1} [kN]	F _{d,2} [kN]	F _{d,3} [kN]	v_1 [m/s]	v ₂ [m/s]	v ₃ [m/s]
1.0	35.4	44.5	37.1	3109	3336	1956	0.23	0.28	0.26
0.6	32.4	41.7	35.6	3090	3050	1824	0.23	0.29	0.27
0.3	29.6	39.7	35.7	3044	2796	1712	0.24	0.29	0.28

 Table 6. 9-storey building building damper design parameters at the design condition.

		9-storey building								
		Floor 1	Floor 2	Floor 3	Floor 4	Floor 5	Floor 6	Floor 7	Floor 8	Floor 9
<i>α</i> =1	$\Delta_{d,j}[mm]$	38.4	32.7	31.9	31.9	29.5	27.8	28.7	29.9	24.3
	$F_{d,j} [kN]$	7781	6060	5416	5075	4379	3930	3559	3011	1614
	$v_j [m/s]$	0.16	0.13	0.12	0.12	0.12	0.12	0.14	0.17	0.17
	$\Delta_{d,j} [mm]$	38.4	32.3	31.0	31.3	28.3	26.6	28.9	30.2	24.4
<i>α</i> =0.6	$F_{d,j} [kN]$	6580	5461	4900	4571	3914	3385	3016	2370	1307
	$v_j [m/s]$	0.20	0.15	0.14	0.14	0.13	0.13	0.16	0.19	0.19
<i>α</i> =0.3	$\Delta_{d,j} [mm]$	38.7	32.4	31.3	30.8	28.7	26.1	29.1	31.0	25.8
	$F_{d,j} [kN]$	5244	4688	4345	4014	3546	2952	2507	1920	1062
	$v_j [m/s]$	0.23	0.18	0.16	0.16	0.15	0.14	0.17	0.22	0.24

320 3.4 Amplification factors

321 Per each value of the constitutive parameter α and per each case-study, five combinations of 322 amplification factors relevant to damper stroke and strength are considered. It is worth to recall that 323 the probability of exceedance of the seismic action suggested by the standards for the dampers design 324 is not homogeneous (10% in 50 years for the European codes and 5% in in 50 years for the existing 325 buildings or the MCE for the new ones in the American code). In order to compare results, the same 326 design action has been considered in the parametric analysis. More precisely, the design action has 327 an annual probability of exceedance equal to 2.1×10^{-3} and it coincides with the action suggested by 328 European Standards.

In detail, γ_{v} and γ_{Δ} denote the amplification factors relevant to velocity and stroke, respectively. 329 The first case analysed, (case $\gamma_v = \gamma_{\Delta} = 1.0$), considers the response parameters reported in **Table 5** 330 331 and Table 6 for the design, without applying any amplification through safety factors. Three more cases are analysed: " $\gamma_{\nu} = 1.5$ and $\gamma_{\Delta} = 1.0$ " where the displacement is not amplified, while the force 332 333 is associated with a velocity equal to $\gamma_v = 1.5$ times the maximum one; " $\gamma_v = \gamma_{\Delta} = 1.5$ " where the 334 displacement is amplified with a coefficient equal to $\gamma_{\Delta} = 1.5$, while the force is associated with a velocity equal to $\gamma_v = 1.5$ times the maximum one; " $\gamma_v = \gamma_{\Delta} = 2.0$ " where the displacement is 335 amplified with a coefficient equal to $\gamma_{\Delta} = 2.0$, while the force is associated with a velocity equal to 336 337 $\gamma_v = 2.0$ times the maximum one.

Moreover, one more case is considered that accounts for larger amplification factors: " $\gamma_v = \gamma_\Delta =$ 339 3.0" where the displacement is amplified with a coefficient equal to $\gamma_\Delta = 3.0$, while the force is associated with a velocity equal to $\gamma_v = 3.0$ times the maximum one. Finally, for comparison purposes, two more limit cases are considered: "No Failure" that is the case where no dampers' failures are permitted, and "Bare Model", which represents the frame without FVDs.

343 3.5 Probabilistic framework

344 A conditional probabilistic approach is used to estimate, for each case study, the demand hazard 345 functions $v_D(d)$ of the random variable D describing the main parameters characterizing the seismic 346 response of the structural systems. The stages needed to estimate $v_D(d)$ by a conditional probabilistic 347 approach are: i) evaluation of the hazard function $v_{IM}(im)$, i.e., the MAF of exceeding the value im of 348 the intensity measure IM; ii) construction of a probabilistic demand model, expressed by the function 349 $G_{D|IM}(d|im)$, linking the generic demand D with the IM and expressing the probability of exceeding the demand value d conditional to the seismic intensity level im; iii) estimation of the mean annual 350 rate of exceedance $v_D(d)$ by solving the following convolution integral between the seismic hazard 351 352 function v_{IM} and the conditional demand G_{DIIM} .

$$v_D(d) = \int_{IM} G_{D|IM}(d|im) |dv_{IM}|$$
(9)

In this study, the standard trapezoidal rule is used to solve the integral of Eq. (9), while Multy-353 354 Stripes Analysis (MSA) is employed to build the $G_{D|IM}$ function, which requires performing a number (n_{sim}) of nonlinear dynamic structural analyses at discrete IM levels (n_{IM}) . In order to achieve accurate 355 356 risk estimations, the number of IM levels used to perform MSA is set equal to 20, and at each IM 357 level the 30 ground motions with the closest IM values are selected and scaled to that IM level. This approach, yielding different ground motion combinations for the different IM levels considered, 358 359 permits to avoid excessive scaling of the records. The choice of the values of n_{sim} (30) and n_{IM} (20) is based on the results of a recently proposed study [34], in which an extensive parametric analysis was 360 361 performed to assess the influence of the main parameters governing MSA on the accuracy of the risk 362 estimates.

363 4 EFFECTS OF FVDS FAILURE ON THE BENCHMARK STRUCTURES RESPONSE

Before illustrating the results of the probabilistic analyses in detail, it is useful to provide a first insight on the dynamic behaviour following the damper failures. FVDs failures are explicitly modelled based on section 2.1.

In the following, the results obtained for a sinusoidal ground motion of increasing intensity striking the three-storey building are presented first. Successively, the seismic response is discussed, considering some ground motions selected from the MSA analysis. Finally, a preliminary and qualitative evaluation of the overall probabilistic response of the three-storey case-study is proposed.

Analysis results highlight some typical issues related to the damper failures, such as the domino effect on dampers at different storeys, acceleration peaks due to end-stroke impacts, and overall brittle behaviour of the system.

374 4.1 System response under an increasing sinusoidal input

In this subsection, the results obtained for a sinusoidal ground motion of increasing intensity striking the 3-storey building are presented. The FVDs response parameters at the design condition refer to linear devices ($\alpha = 1$) and to the case " $\gamma_{\nu} = \gamma_{\Delta} = 1.0$ ". The choice of an increasing harmonic input motion is motivated by the fact that it allows to easily identify the attainment of the damper strength capacity through one of the two mechanisms, impact and over-velocity and the related consequences on the frame undergoing a more general time-history input motion.

Fig. 5 shows the sinusoidal input having a period of 0.9 seconds and an initial magnitude of $1 m/s^2$. The amplitude of the motion is constant for five cycles, after that it is increased with a coefficient equal to 1.5 and remains again constant for five cycles. The magnification of the motion amplitude is repeated four times, resulting in a motion that has five different amplitudes, with a maximum equal to $5 m/s^2$, and that lasts 22.5 seconds. At the end of the input, there are few seconds, which are useful to understand how the case study restores its rest condition.





Fig. 5. Sinusoidal incremental dynamic input

Fig. 6 a) - Fig. 6 h) show the response in terms of damper forces and strokes, $F_{d,i}$ and $\Delta_{d,i}$. floor 389 displacements u_i , floor relative velocities $v_{r,i}$ and floor absolute accelerations A_i . In particular, Fig. 390 6 a) and b) illustrate the time-history of the damper forces $F_{d,i}$ recorded along the height of the 391 392 building. The black solid line refers to the device installed between the ground and the first floor, the 393 red one refers to the intermediate device at the second storey, while the blue one represents the damper 394 at the top storey. At the beginning of the third increment of the sinusoidal input (between 14.4 and 395 14.5 seconds), graphs show some small ripples, which are more evident for the intermediate and top-396 storey devices (Fig. 6 b), and they are caused by small impacts due to the end-stroke attainment. In 397 this case, the impact occurs but it does not lead to the attainment of the damper strength capacity. At 398 the time instant 14.6 s the damper placed at the first level reaches its force capacity due to over-399 velocity and its force drops to zero. Few instants later, also the other devices fail for over-velocity. 400 These effects can be deeper investigated through Fig. 6 c) - e), where the stroke-force relationship of 401 each device is shown. In particular, by observing the stroke-force relationships of Fig. 5 c) - e) it is 402 evident that at the beginning of the third increment of the input motion, all the three dampers 403 experience the end-stroke attainment without failure, with impacts that are more evident for the intermediate and top-storey devices. After these impacts, occurred without consequences, the FVDs 404 restore their behaviour as pure dissipative devices. Few instants later, suddenly, the damper located 405 406 at the ground floor fails due to over-velocity, triggering the sequence of damper failures at the upper 407 elevations. The sequence is highlighted by a series of ripples in the stroke-force relationship of the intermediate and especially top-storey device. The ripples begin when the first device fails and last 408 409 until all the devices fail for over-velocity.

Fig. 6 f) - h) shows the time-histories of the parameters strictly related to the frame response, highlighting the consequences of the damper failures on the frame itself. Generally, the responses in terms of displacements, relative velocities and absolute accelerations are significantly amplified by the impacts occurring in the dampers and by their failure. The absolute accelerations are more affected than the displacements. It is worth to note that the peaks in terms of absolute accelerations, recorded between 14 and 15 seconds, are mainly related to the impacts experimented by the devices before their failure.





Fig. 6. Dampers response and time histories of different local and global EDPs under harmonic
 increemental dynamic input

419 4.2 Three-storey building and nine-storey building seismic response overview

420 In this subsection, few selected information from MSA analysis are shown to illustrate overall the 421 problem of damper failure and related effects on the structural performance of the three-storey and 422 nine-storey building case studies.

423 To shed further light on the consequences of FVDs failure, the time-histories of the interstorey-424 drift ratio (IDR) response of the bare model and of the system with (linear and nonlinear) dampers 425 designed without amplification factors ($\gamma_{\nu} = \gamma_{\Delta} = 1.0$) are compared in Fig. 7. Comparison is performed at both low (IM level 5) and high (IM level 15) seismic intensities and for a single time-426 427 history (TH) analysis (TH 17). For sake of brevity and given the high similarity of the response at all 428 floors the response in terms of IDR at floor 1 is only discussed. It is confirmed that at lower seismic 429 intensities FVDs are effective in damping the response (by also reducing residual drift) and that the 430 beneficial response mitigation provided by the dampers vanishes at higher IMs, due to the device failure. More specifically, it can be observed that dampers fail at around 4.0 seconds since the 431 432 beginning of the time-history of IM level 15. This is detailed in the inset of Fig. 7 (b) (close-up plot 433 between 2 and 6 seconds), showing that the IDR response (with both linear and nonlinear devices) is 434 damped until the 4.0 s and then tends towards the bare-frame response; on the contrary, at *IM* level 5 435 the response of the frame with FVDs is damped over the whole earthquake duration, since no device 436 failure is observed at this intensity level.

For sake of completeness, the dampers force-stroke cyclic responses corresponding to the 437 aforesaid cases (plotted in Fig. 7) are shown in Fig. 8 (IM level 5) and Fig. 9 (IM level 15). In each 438 figure a comparison is made between the responses of the linear (red dashed line) and nonlinear 439 dampers (blue dotted line) at the first (figure a) and third storey (figure b). The attainment of the end-440 stroke (impact) is characterised by a sudden rise in force (with no increase of displacement) while the 441 442 attainment of the maximum force capacity (hence the failure) can be identified because the force 443 suddenly becomes null and the hysteretic cycle is interrupted. It can be noted that at IM level 5 failure is never attained, and thus complete cycles can be observed in Fig. 8. 444

On the contrary, at *IM* level 15 failure occurs on the dampers of both storeys 1 (Fig. 9a) and 3
(Fig. 9b), corresponding to the cycle's sudden interruption. More in detail, in Fig. 9a (floor 1) the
failure is achieved with no sign of impact (for both linear and nonlinear dampers); differently, in Fig.
9b (floor 3), the force of the linear damper (red line) increases abruptly and immediately after drops
to zero, meaning that the impact is responsible for the failure.



451 Fig. 7. Time histories of the IDR at two different *IM* levels (a) *IM* n. 5 and (b) 15. Comparison
452 between the bare model and the model with linear and nonlinear dampers (withouth amplification
453 factors).



454 Fig. 8. Damper response at *IM* levels 5. Comparison between linear and nonlinear dampers
455 (withouth amplification factors) at (a) first and (b) third floor.





In **Fig. 10**, time histories (selected from *IM* level 15) of the force on dampers at different floors are compared. Here dampers failure occurs at 4.0 s, when the forces suddenly drop to zero and the dampers become ineffective. It can be also observed that failure involves devices at all the storeys quite simultaneously.



462 463

Fig. 10. Failure time-lag among dampers at different floors.

For what concerns the seismic response of the nine-storey building, the differences between the 464 seismic response of this structural system and the previous low-rise building are highlighted in the 465 following. Fig. 11 a) shows the time-history of the forces at the various levels under record #17 466 scaled to the IM=15 (with intensity 2.0 g, 2.26 times higher than the design seismic intensity). 467 Although damper failure initiates at the bottom storey, it propagates quite rapidly to the devices 468 placed at the higher levels. However, damper failure can also propagate from the top to the bottom 469 470 of the building, as observed by the response shown in Fig. 11 b), related to the same record scaled 471 to IM=10 (design seismic intensity). In general, it is observed that when one device fails, all the other devices fail too, even though at different times. 472 473



474 Fig. 11. Failure time-lag among dampers at different floors: *IM* level n. 15 (a) and 10 (b) *IM*.

475 4.3 Qualitative evaluation of the overall probabilistic response

In this section a preliminary evaluation of the overall probabilistic response of the three-storey building is provided. **Fig. 12** shows the building response in terms of IDR and acceleration at storeys (A) at different seismic intensities for the case of linear dampers ($\alpha = 1.0$). For each *IM* level, the median response values are shown by using continuous lines with circle markers, and different colours are used to compare the following three cases: 1) bare model (black); 2) building with dampers designed without amplification factors ($\gamma_v = \gamma_{\Delta} = 1$) (blue); 3) building with dampers with neither impact nor failure model ($\gamma_v = \gamma_{\Delta} = \infty$) (red). Moreover, the 16th and 84th percentiles are plotted by dotted lines by using the same colours described above.

484 The following observations can be made:

485

486

489

490 491

492

- FVDs without failure significantly reduce the IDR of the building up to the highest seismic intensities, with a lower beneficial effect in terms of acceleration mitigation;
- 487
 488
 488 If the device failure is taken into account, the response mitigation provided by the dampers vanishes for *IM* levels higher than 10, corresponding to design condition (0.8866 g);
 - Once failed, devices are no longer effective and the IDR response of the damped systems tends to be almost that of the bare building, while the response in acceleration shows peaks higher than the undamped frame system, due to the impacts induced by the devices end-stroke attainment.

493 The observations above also apply to the case with nonlinear dampers (not shown due to space 494 constraints).



495 **Fig. 12.** Building response at different *IM* levels for the case with linear dampers (α =1.0) in terms 496 of (a) IDR and (b) A. Comparison between damped (with and without failure) and bare model.

497 5 PROBABILISTIC ANALYSIS RESULTS: THREE-STOREY BUILDING

The performance of the case studies is evaluated by monitoring a wide set of EDPs. To provide information on the damage level of the main structural system, the following global EDPs are considered: the maximum interstorey drift among the various storeys (IDR), the maximum roof drift (RDR), the maximum residual interstorey drift among the storeys (IDR_{res}), and the maximum absolute acceleration at storeys (A). The dampers performance is monitored by considering the following two local EDPs, accounting for the cost, the size and the failure of the devices: the maximum absolute force of the dampers (F_{di}) and the maximum stroke (Δ_{di}).

505 5.1 Demand hazard curves

This subsection shows the demand hazard curves of all the monitored EDPs, with respect to the 506 mean annual rate of exceedance v_D , for each damper typology ($\alpha = 1.0, \alpha = 0.6, \alpha = 0.3$). 507 508 Comparisons are made among the various analysed cases, namely: dampers without amplification 509 factors ($\gamma_{\nu} = \gamma_{\Delta} = 1.0$) (blue solid line) and dampers designed with different γ factors, that is $\gamma_{\nu} =$ 1.5 and $\gamma_{\Delta} = 1.0$ (brown solid line); $\gamma_{\nu} = \gamma_{\Delta} = 1.5$ (yellow solid line); $\gamma_{\nu} = \gamma_{\Delta} = 2.0$ (violet solid 510 line); $\gamma_v = \gamma_{\Delta} = 3.0$ (green solid line). Moreover, the demand hazard curve of the following two cases 511 512 are added for comparison purposes: bare frame model (black dashed line) and damped model without damper failure (i.e., with $\gamma_{\Delta} = \gamma_{\nu} = \infty$) (red solid line). Also, two horizontal dotted lines are depicted 513 in the charts, one identifying the design hazard level 0.0021 yr⁻¹ (black dotted line) and the other (red 514 515 dotted line) denoting the target risk level desired for the structural systems $(2x10^{-4} \text{ yr}^{-1})$ [23].

816 Results concerning the linearly damped building are first presented. The demand hazard curves of 817 the main global EDPs (IDR, RDR, IDR_{res}, A) are illustrated in **Fig. 13**, whereas those concerning the 818 damper response (F_{di} and Δ_{di}) are illustrated in **Fig. 16**. Only the curves of the dampers at floor 1 are 819 shown, given the similarity of the results among the storeys.

Based on Fig. 13 the following comments can be made:

- For all the cases with dampers and amplification factors larger than 1.0, the rate of exceeding of the target drift performance (IDR=0.012) is around 0.0021 yr⁻¹, the hazard level of the design action, represented by the horizontal black dotted line, with some slight deviations that can be justified by the probabilistic nature of the analysis (contribution to the exceedance probability from *IM* levels different from the reference one [28][29]).
- If no amplification is considered ($\gamma_v = \gamma_{\Delta} = 1$, blue curve), the rate of exceeding of the target drift performance, highlighted by the red dotted line (IDR=0.012), is notably higher than the expected one, due to the failures experienced by the dampers at intensity levels lower than the design one (i.e., IM = 0.89 g) (see Section 5.3 for further details about this point).
 - Once damper rupture is attained, the building response in terms of maximum and residual drift tends to that of the bare model (black dashed line) and the magnitude of the amplification factors governs the "rapidity" of the transition from the damped to the bare frame curve.
 - In particular, the IDR, RDR and IDR_{res} approach the bare frame model quite perfectly, conversely, the absolute accelerations, which are lower than those of the bare frame until the dampers are effective, become even higher due to end-strokes impacts experienced by the dampers, before their failures.
- The hazard curves of RDR and IDR are very similar and both of them tend to overlap those of
 the bare model once the dampers fail, meaning that, in this case, the drift demand is uniform
 along the building height (no soft storey mechanisms have been observed).
- 540

520

530

531

532 533

534

535



541 Fig. 13. Demand hazard curves of the main global EDPs (a) IDR, (b) RDR, (c) IDR_{res}, (d) A for 542 different damper amplification factors. Case of building with linear dampers ($\alpha = 1.0$).

543 Fig. 14 shows the IDR demand hazard curves for the cases with nonlinear dampers (α =0.6 and 544 α =0.3). The trends are similar to those observed with linear dampers, although there are some 545 differences worth to be stressed:

- The curves of nonlinear dampers have a lower slope, which lead the system to show, for a given demand value, higher exceedance annual rates. This is consistent with previous studies comparing the performance of linear and nonlinear FVDs [28].
- The MAF levels corresponding to the transition from the curve of the damped system to that of 550 the undamped one are higher for nonlinear dampers compared to the linear ones, and the slope of 551 such transition increases with the degree of nonlinearity of dampers.





Fig. 14. Demand hazard curves of the IDR parameter for different damper's amplification factors. Case of building with nonlinear dampers: (a) α =0.6; (b) α =0.3.

Finally, a deeper discussion is due on the influence of the amplification factors on the structure 554 reliability (Fig. 15). For this purpose, the response corresponding to the reference MAF of $2x10^{-4}$ is 555 selected. This value is generally considered as a satisfactory target for the MAF of collapse, as 556 illustrated in [23][53]. The response corresponding to the reference MAF, in terms of IDR, achieved 557 558 for the case where no damper failure is permitted ("No Failure") is assumed as the target response and identified as IDR₀. This result is then compared, through the ratio IDR/IDR₀, with the values of 559 IDR achieved with four different values of the γ -factors. The analysed cases are $\gamma_{\nu} = \gamma_{\Delta} = 1.0$, that 560 is dampers without amplification factors and three more cases in which the displacements and the 561 forces associated with velocities are amplified, that is $\gamma_{\nu} = \gamma_{\Delta} = 1.5$, $\gamma_{\nu} = \gamma_{\Delta} = 2.0$ and $\gamma_{\nu} = \gamma_{\Delta} = 2.0$ 562 563 3.0.

Fig. 15 shows the variation with γ of the ratio IDR/IDR₀ highlighting that in the case of linear 564 565 dampers, the use of a γ -factor equal to 3 permits to obtain the same IDR of the "No failure" case, whereas in the case of nonlinear devices a value just larger than 1 is reached, ensuring similar 566 567 performance in both the linear and nonlinear case. Differently, with lower values of y-factors, significantly larger values of the ratio IDR/IDR₀ are obtained, meaning that the response achieved 568 569 when accounting for the devices failure is far from the reference one (IDR₀). The trend of the ratio 570 achieved with linear dampers seems to be more sensible to the variation of the γ -factors, as highlighted by a change of the slope when γ are comprised between 1.5 and 2. Differently, with 571 nonlinear devices the trend has a slighter slope, highlighting a value of the ratio IDR/IDR₀ closer to 572 1 for higher values of the γ -factors. 573



575 **Fig. 15**. Ratios IDR/IDR₀ for different damper amplification factors. Case of building with linear 576 dampers ($\alpha = 1.0$) and nonlinear dampers ($\alpha = 0.3$).

577 5.2 Dampers failure rates

578 This sub-section examines the demand hazard curves of the EDPs related to the dampers, i.e., the 579 maximum force and the maximum stroke. **Fig. 16** illustrates the curves of the maximum force (**Fig.** 580 **16**a) and the maximum stroke (**Fig. 16**b) for the linear damper ($\alpha = 1.0$) at floor n. 1. These are 581 representative of the outcomes observed at all the floors and the trends observed are the same for all 582 the types of dampers ($\alpha = 0.6$, $\alpha = 0.3$). Some general comments on **Fig. 16** follow, which also 583 apply to all the other cases not displayed in the plots:

- Dampers designed with $\gamma_v = \gamma_{\Delta} = 1.0$ (blue curves) fail at a MAF of exceedance higher than the design hazard level 0.0021 yr⁻¹ (black dotted lines), mainly because of over-velocity phenomena which lead the dampers to attain the ultimate force capacity.
- Despite the ultimate force capacity is the same, the annual rate of failure for the case $\gamma_v = 1.5$ and $\gamma_{\Delta} = 1.0$ (brown curves) is higher than the case $\gamma_v = \gamma_{\Delta} = 1.5$ (yellow curves) due to the higher number of collapses induced by the end-stroke attainment.
- All the curves follow the trend of the case with dampers with unlimited capacity (red curves)
 until the collapse is attained, then the curves show a sudden vertical drop due to the impossibility to exceed the ultimate capacity values.

593 **Table 7** to **Table 9** summarise the damper failure rates (v_{fail}) for all the cases analysed, by also providing the values of $v_{\text{fail}}/v_{\text{target}}$, i.e., the ratios between the actual failure rates and the target risk 594 levels desired for the structural systems $(2x10^{-4} \text{ yr}^{-1})$ [23]. Ratios higher than one identify cases in 595 which the target reliability level is not attained, ratios equal or lower than one identify cases in which 596 597 the requirement is fulfilled (such values are highlighted by bold font in the tables). It can be observed 598 that without amplification factors the failure is always attained with a probability higher than the 599 target one. If the amplification factors are used, the higher the amplification factors, the lower the 600 $v_{\text{fail}}/v_{\text{target}}$ ratios are. When the amplification factor $\gamma_{\nu} = \gamma_{\Delta} = 3.0$ is applied, the ratios are always 601 lower than one, except for the nonlinear dampers with $\alpha = 0.3$ at the first and last elevation. 602



603 **Fig. 16**. Demand hazard curves of the main local EDPs (a) dampers force F_{di} and (b) stroke Δ_{di} for 604 different damper amplification factors. Case of building with linear dampers ($\alpha = 1.0$).

- 605
- 606
- 607
- 608
- 609
- 610
- 611

612

~ 10									
		ν_{fail}	v_{fail}/v_{target}						
Case of analysis		[1/yr]	[-]						
	Floor 1	Floor 2	Floor 3	Floor 1	Floor 2	Floor 3			
$\gamma_{\Delta}\!=\gamma_{\rm v}\!=1.0$	3.48E-03	3.93E-03	5.45E-03	17.42	19.65	27.26			
$\gamma_{\Delta} = 1.0 \& \gamma_v = 1.5$	1.28E-03	1.21E-03	1.54E-03	6.38	6.05	7.70			
$\gamma_{\Delta}\!=\!\gamma_{v}\!=1.5$	8.80E-04	7.43E-04	1.01E-03	4.40	3.72	5.05			
$\gamma_{\Delta}\!=\gamma_{\rm v}\!=2.0$	2.99E-04	2.57E-04	2.41E-04	1.50	1.29	1.20			
$\gamma_{\Delta}\!=\gamma_{\rm v}\!=3.0$	8.20E-05	3.53E-05	5.41E-05	0.41	0.18	0.27			

Table 7. Damper failure rates (v_{fail}) and $v_{\text{fail}}/v_{\text{target}}$ ratios of the 3-storey building ($\alpha = 1.0$). $\alpha = 1.0$

613614

Table 8. Damper failure rates (v_{fail}) and $v_{\text{fail}}/v_{\text{target}}$ ratios of the 3-storey building ($\alpha = 0.6$).

$\alpha = 0.6$									
		ν_{fail}		v_{fail}/v_{target}					
Case of analysis		[1/yr]		[-]					
	Floor 1	Floor 2	Floor 3	Floor 1	Floor 2	Floor 3			
$\gamma_{\Delta}\!=\gamma_v\!=1.0$	4.08E-03	3.69E-03	4.97E-03	20.39	18.44	24.87			
$\gamma_{\Delta} = 1.0 \& \gamma_v = 1.5$	1.68E-03	1.89E-03	1.71E-03	8.38	9.45	8.55			
$\gamma_{\Delta}\!=\!\gamma_{v}\!=1.5$	1.26E-03	1.33E-03	1.16E-03	6.30	6.65	5.82			
$\gamma_{\Delta}\!=\gamma_{v}\!=2.0$	4.42E-04	4.46E-04	4.28E-04	2.21	2.23	2.14			
$\gamma_{\Delta}\!=\gamma_{\rm v}\!=3.0$	1.54E-04	1.33E-04	1.32E-04	0.77	0.66	0.66			

615

616

Table 9. Damper failure rates (v_{fail}) and $v_{\text{fail}}/v_{\text{target}}$ ratios of the 3-storey building ($\alpha = 0.3$).

u – 0.5									
	ν_{fail}	v_{fail}/v_{target}							
	[1/yr]	[-]							
Floor 1	Floor 2	Floor 3	Floor 1	Floor 2	Floor 3				
4.86E-03	5.21E-03	7.72E-03	24.30	26.07	38.59				
2.06E-03	2.04E-03	2.50E-03	10.60	10.22	12.49				
1.61E-03	1.60E-03	1.94E-03	8.07	8.02	9.70				
6.27E-04	7.58E-04	8.24E-04	3.14	3.79	4.12				
2.48E-04	1.81E-04	2.25E-04	1.24	0.91	1.12				
	Floor 1 4.86E-03 2.06E-03 1.61E-03 6.27E-04 2.48E-04	Vfail [1/yr] Floor 1 Floor 2 4.86E-03 5.21E-03 2.06E-03 2.04E-03 1.61E-03 1.60E-03 6.27E-04 7.58E-04 2.48E-04 1.81E-04	Vfail [1/yr] Floor 1 Floor 2 Floor 3 4.86E-03 5.21E-03 7.72E-03 2.06E-03 2.04E-03 2.50E-03 1.61E-03 1.60E-03 1.94E-03 6.27E-04 7.58E-04 8.24E-04 2.48E-04 1.81E-04 2.25E-04	Vfail Image: Floor 1 Floor 2 Floor 3 Floor 1 4.86E-03 5.21E-03 7.72E-03 24.30 2.06E-03 2.04E-03 2.50E-03 10.60 1.61E-03 1.60E-03 1.94E-03 8.07 6.27E-04 7.58E-04 8.24E-04 3.14 2.48E-04 1.81E-04 2.25E-04 1.24	Vfail Vfail/Vtarget [1/yr] [-] Floor 1 Floor 2 Floor 3 Floor 1 Floor 2 4.86E-03 5.21E-03 7.72E-03 24.30 26.07 2.06E-03 2.04E-03 2.50E-03 10.60 10.22 1.61E-03 1.60E-03 1.94E-03 8.07 8.02 6.27E-04 7.58E-04 8.24E-04 3.14 3.79 2.48E-04 1.81E-04 2.25E-04 1.24 0.91				

617

Finally, further light is shed regarding the effect of the amplification factors on the sequence of 618 dampers failure among different storeys. For this purpose, it can be useful to refer to Fig. 17, where 619 the average trends of the $F_d/F_{d,max}$ ratios are depicted (together with the 16th and 84th response 620 621 percentiles), for different intensity levels and for all the building storeys. Being all the curves almost 622 perfectly overlapped, it means that there are not cases in which some devices remain active while 623 others fail. The only exception to this general result is represented by the case in which high γ -factors $(\gamma_{\nu} = \gamma_{\Delta} = 3)$ are used. Indeed, beside the curve shifting towards higher *IMs*, curves of dampers 624 belonging to different floors slightly deviate at the highest seismic intensities, by testifying the 625 presence of few cases in which the dampers at the higher floors do not fail together with the other 626 located at the lower floors. This aspect will be further discussed for the case of the 9-strey building, 627 628 which shows a higher sensitivity to the γ -factor values. The results obtained for nonlinear dampers 629 are similar to the ones presented here and are not reported due to space constraints.



630 **Fig. 17**. Average and 16^{th} and 84^{th} response percentiles of the $F_d/F_{d,max}$ – IM trends at different 631 floors for γ factors (a) $\gamma_{\nu} = \gamma_{\Delta} = 1$ and (b) $\gamma_{\nu} = \gamma_{\Delta} = 3$.

632 5.3 Dampers collapse fragility functions

633 In this section, the problem of damper failure is analysed in terms of fragility functions $G_{fail|IM}$, 634 providing information about the dependency of the probability of failure with the seismic intensity.

Fig. 18 shows the fragility curves of the linear dampers placed at the first and third floor, for all the different γ -factors analysed.

637 Based on these results, the following observations can be made:

- 638 The absence of amplification factors leads to high damper failure probabilities (>50%) at 639 seismic intensities lower than the design level (i.e., IM = 0.89 g), and from IM = 1.5 g a 640 100% probability of damper failure is obtained.
 - The beneficial effect of γ -factors larger than 1 is testified by the shifting of the fragility curves towards higher seismic intensities.
 - Failure probabilities also reduce by moving from floor 1 to floor 3, as can be observed by comparing the curve of **Fig. 18**a and **Fig. 18**b. However, no differences are observed among the floors for the case without amplification ($\gamma_{\nu} = \gamma_{\Delta} = 1$).





⁶⁴⁹ Comments above also apply to the case with nonlinear dampers ($\alpha = 0.3$, shown in Fig. 19), with 650 the main exception given by the slightly higher failure probabilities observed in this latter case.

651

641

642

643

644 645



Fig. 19. Damper collapse fragility at (a) floor n. 1 and (b) floor 3 with different amplification
 factors. Case of building with nonlinear dampers (α=0.3).

654 6 PROBABILISTIC ANALYSIS RESULTS: NINE-STOREY BUILDING

This section shows the results concerning the 9-storey building. Due to space constraints, only selected results are presented. The differences between the seismic response of this structural system and the previous low-rise building are highlighted, with particular focus on the effect of the amplification factors on the sequence of dampers failure along the storeys, and the levels of seismic reliability that are achieved.

660 6.1 Demand hazard curves and failure probabilities

Fig. 20 and **Fig. 21** show the demand hazard curves of the 9-storey building equipped respectively with linear and nonlinear dampers. In general, the curves follow the same trends observed for the low-rise system. However, in this case the MAF levels at which the curves start diverging due to damper failure are notably higher. For instance, the case without amplification factors (blue curve) deviates from the "no failure" case (red curve) at $v = 10^{-2}$ yr⁻¹. This is due to the fact that the damper design is carried out based on the first mode response approximation, which is less accurate for the medium and high-rise buildings, whose response is significantly influenced by higher-order modes.

Moreover, by comparing Fig. 20 a) and Fig. 21 a) it is worth noting that the efficiency of the added 668 669 dampers reduces for decreasing MAF of exceedances. In fact, higher reductions of drifts are observed 670 for higher MAF of exceedances than for lower ones, for both the cases of linear and nonlinear dampers. In this regard, the nonlinear behaviour of the frame (and consequent period-elongation) has 671 672 a significant contribution and affects the dampers performance and their efficiency. It is also observed that the beneficial effect in terms of IDR reduction reduces for increasing levels of nonlinearity of the 673 674 dampers (i.e. lower alpha values). In fact, as already highlighted in previous works carried out by the 675 authors ([28]-[30]), the nonlinear devices are more effective with respect to the linear ones in controlling the viscous forces, while this efficiency is paid in terms of higher displacements, 676 particularly for less probable events (lower MAF of exceedance). 677

678 The use of amplification factors improves the response by shifting the curves towards lower failure 679 probabilities, as already shown previously for the low-rise building. However, results are worse in 680 terms of system reliability levels achieved with respect to the 3-stroey building.



682 **Fig. 20**. Demand hazard curves of the main global EDPs (a) IDR, (b) IDR_{res} for different damper 683 amplification factors. Case of building with linear dampers ($\alpha = 1.0$).



684 **Fig. 21**. Demand hazard curves of the main global EDPs (a) IDR, (b) IDR_{res} for different damper 685 amplification factors. Case of building with nonlinear dampers ($\alpha = 0.3$).

As already done for the 3-storey building, with the aim to provide an insight on the influence of 686 the response amplification factors on the damper failure probability, the IDR response corresponding 687 to the target MAF of exceedance of $2x10^{-4}$ is evaluated for different values of γ -factors, and 688 normalized with respect to the response obtained with dampers that do not suffer failure (IDR₀). Fig. 689 690 22 shows the results obtained with γ -factors ranging from 1 to 3 for linear devices (α =1.0) and nonlinear ones (α =0.3). It can be observed that, differently from the 3-storey building, the trends 691 obtained with linear and nonlinear devices are significantly different among them. With linear 692 693 dampers, indeed, the use of γ -factors equals to 3 leads nearly to the achievement of the desired response (IDR₀), ensuring a ratio IDR/IDR₀ slightly higher than one, while lower values of γ -factors 694 correspond to higher values of the ratio. Differently, the response achieved with nonlinear dampers 695 696 seems to be insensitive to change of the γ -factors, with a ratio IDR/IDR₀ that always remains 697 comprised between 1.56 and 1.43.





699 **Fig. 22**. Ratios IDR/IDR₀ for different damper's amplification factors. Case of building with linear 700 dampers ($\alpha = 1.0$) and nonlinear dampers ($\alpha = 0.3$).

701 Regarding the effect of the amplification factors on the sequence of dampers failure, some further 702 details are provided in Fig. 23, as already done for the 3-storey building. In general, it is observed 703 that when one device fails, all the other devices fail too, even though at different times. However, this 704 not always true, and in order to analyse this issue it can be useful to refer to Fig. 23, where the average 705 trends of the $F_d/F_{d,max}$ ratios are depicted, for different intensity levels and for different building storeys (i.e., floors n. 1, 3, 6, 9). When no amplification factors are used ($\gamma_{\nu} = \gamma_{\Delta} = 1$), the curves 706 707 are almost perfectly overlapped (Fig. 23 a), thus there are not cases in which some devices remain 708 active while others fail. The response changes if higher γ -factors are used, as shown in Fig. 23 b $(\gamma_{\nu} = \gamma_{\Delta} = 3)$. Indeed, beside the curve shifting towards higher *IMs*, curves of dampers belonging to 709 710 the upper floors slightly deviate at the highest seismic intensities, by testifying a lower average rate 711 of failures, and thus a concentration of collapse cases in the dampers at the lower floors. It is noteworthy that the damage concentration on the structural elements at the storeys with failed 712 713 (inactive) dampers results in a building performance worse than the one of the bare frame case, this 714 result can be also related to the design method used for the FVDs, which disregards higher order 715 modes. 716



Fig. 23. Average and 16th and 84th response percentiles of the $F_d/F_{d,max}$ – IM trends at different floors for linear dampers with (a) $\gamma_v = \gamma_\Delta = 1$ and (b) $\gamma_v = \gamma_\Delta = 3$.

The damper reliability is also analysed by showing in **Fig. 24** and **Fig. 25** the fragility functions for, respectively, the linear and nonlinear dampers placed at different floors, for the different γ -factors analysed. The results shown in these figures are very similar to those obtained for the 3-floors building (**Fig. 18** and **Fig. 19**). However, in this case the differences between the fragilities of dampers placed at different floors is more evident, thus confirming that a more specialized design method for the
 FVDs or different amplification factors at different floors should be used in order to obtain a uniform
 failure among dampers of different storeys, as already observed previously.



Fig. 24. Damper collapse fragility at (a) floor 1 and (b) 9, with and withouth amplification factors. Case of building with linear dampers ($\alpha = 1.0$).



Fig. 25. Damper collapse fragility at (a) floor 1 and (b) 9, with and withouth amplification factors. Case of building with nonlinear dampers ($\alpha = 0.3$).

731 7 CONCLUSIONS

732 The seismic design of Fluid Viscous Dampers (FVDs) for enhancing the performance of buildings 733 should ensure proper safety margins against their collapse, and the reliability of the whole structural system is strongly influenced by the reliability of these devices. Seismic standards generally prescribe 734 735 that that the FVDs must be designed based on values of the response parameters (i.e., stroke and 736 velocity) evaluated at the design condition and amplified by safety factors (reliability factors), in order to reach a target level of safety. However, the values of these reliability factors are not 737 homogenous among the various codes and the level of safety attainable through their use has not been 738 739 sufficiently investigated.

740 The present paper investigates the issue through the analysis of two benchmark case studies 741 consisting of a low-rise and a medium-rise steel building equipped with FVDs. A wide range of safety 742 factor values is considered for the damper design, considering suggestions from international seismic 743 codes (EN15129 and ASCE-41). A wide parametric investigation is carried out to explore the influence of these safety factors on both the fragility and the seismic risk of the whole structural 744 745 system. The effect of damper nonlinearity is also taken into account analysing damper velocity exponents ranging from 0.3 to 1.0. The damper shows a brittle failure when its internal force attains 746 the device strength and this may occur for two reasons: impact when end-stroke is attained, or 747 748 attainment of excessive velocity. Both these failure modalities are described by the structural model 749 and considered in the analyses.

As a general result, it is observed that combined effects of impacts and extreme velocities may induce a global brittle behaviour that cannot be perceived by models neglecting these phenomena. More specifically, based on the outcomes of the present study, the following conclusions can be drawn:

- The consequences of the damper failure on the performance of the whole structural system
 depend on the number of dampers remained active: if all dampers fail together, then the
 system response tends to that of the bare building, however absolute accelerations may be
 higher as a consequence of impacts and dissipation concentrated at some storeys only may
 leads to a worse global response.
- The likelihood of the damper failure as well as the "rapidity" of the response transition from damped to bare (or partially damped) structural system are governed by the magnitude of the two amplification factors ($\gamma_{\Delta}, \gamma_{\nu}$) adopted for damper stroke and velocity.
- If no amplification is provided ($\gamma_v = \gamma_{\Delta} = 1.0$), the dampers probability of failure is higher than the design hazard level (assumed equal to 0.0021 yr⁻¹ in this work), thus, dampers experience failure at intensity levels lower than the design one.
- The use of amplification factors higher than 1.0 allows attaining lower failure probabilities,
 and this beneficial effect is more significant for larger γ–factors.
- Nonlinear dampers (α=0.3) exhibit higher failure probabilities (about two times) than the linear ones; moreover, the transition from the damped response (active devices) to that of the undamped one (failed devices) increases at a faster rate increasing the degree of damper nonlinearity.
 In tall buildings where a design method disregarding higher order modes is used for FVDs,
 - In tall buildings where a design method disregarding higher order modes is used for FVDs, non-uniform failures among dampers of different storeys may occur.

773 Based on the study results, some suggestions can be proposed for further improvements of the 774 design prescriptions of the main international seismic codes. First of all, it should be observed that γ -factors equal to 3, both for stroke and velocity, generally ensure that the target failure probability 775 $2x10^{-4}$ yr⁻¹ is achieved, despite they might result inadequate in case of dampers with strong nonlinear 776 777 behaviour (i.e., $\alpha = 0.3$ or lower). Such result, also observed in the 9-storey building, confirms the 778 need of extending the study to γ -factors higher than 3.0. Additionally, the study outcomes suggest 779 that in the case of medium and high-rise buildings, different γ -factors should be employed at the 780 various storeys, and they should be tailored to the specific damper properties present at each storey. 781 It might be also worth to investigate the problem of γ -factors by analysing more closely the 782 damage/plasticity evolution and distribution over the structural elements when devices fail. 783 Moreover, it should be observed that the choice of γ -factors depends on the ratio between the MAF 784 of exceedance chosen for the seismic design action and the target MAF of failure. For example, ASCE 785 code suggests lower MAF for seismic design actions and relevant γ -factors seem to be in line with 786 suggested target value of MAF of failure.

It is also worth to note that the amplification of the damper velocity only, without a corresponding amplification of the damper stroke (i.e., $\gamma_{\Delta} = 1.0$ and $\gamma_{\nu} > 1.0$), as allowed by the European code EN15129, does not provide significant beneficial effects because the impacts due to the end-stroke attainment makes the effect of γ_{ν} useless. Thus, homogeneous amplification factors (i.e., $\gamma_{\Delta} = \gamma_{\nu}$) should be used to achieve a reliable and effective design of FVDs.

Given the relevance of these aspects, the extension of the study to a wider range of buildingstypologies and design methods will be considered in future works.

794 REFERENCES

- Tubaldi, E., Gioiella, L., Scozzese, F., Ragni, L., & Dall'Asta, A. (2020). A design method for viscous dampers connecting adjacent structures. *Frontiers in Built Environment*, *6*, 25.
- [2] Barroso LR. Performance evaluation of vibration controlled steel structures under seismic loads. PhD thesis, Stanford University, California, US, 1999.

- Pavlou, E, Constantinou, M.C., 2006. Response of Nonstructural Components in Structures with Damping Systems, Journal of Structural Engineering, 132(7), 1108-1117.
- [4] Lavan, O, Dargush, G.F., 2009. Multi-Objective Evolutionary Seismic Design with Passive Energy Dissipation
 Systems, *Journal of Earthquake Engineering*, 13(6), 758-790.
- [5] Gioiella, L., Tubaldi, E., Gara, F., Dezi, L., Dall'Asta, A. (2018). Modal properties and seismic behaviour of buildings
 equipped with external dissipative pinned rocking braced frames. *Engineering Structures*, 172.
 https://doi.org/10.1016/j.engstruct.2018.06.043
- Pavia, A., Scozzese, F., Petrucci, E., & Zona, A. (2021). Seismic Upgrading of a Historical Masonry Bell Tower through an Internal Dissipative Steel Structure. Buildings, 11(1), 24.
- [7] Gioiella, L., Tubaldi, E., Gara, F., Dezi, L., Dall'Asta, A. (2018). Stochastic Seismic Analysis and Comparison of Alternative External Dissipative Systems. *Shock and Vibrations*, 47. https://doi.org/10.1155/2018/5403737
- [8] Tubaldi, E. (2015). Dynamic behavior of adjacent buildings connected by linear viscous/viscoelastic dampers. *Structural Control and Health Monitoring*, 22(8), 1086-1102.
- [9] Hwang, JS., Lin, WC., Wu, NJ. (2010). Comparison of distribution methods for viscous damping coefficients to building. *Structure and Infrastructure Engineering: Maintenance, Management, Life-Cycle Design and Performance, 9*(1), 28-41. http://dx.doi.org/10.1080/15732479.2010.513713
- [10] Silvestri, S., Palermo, M., & Trombetti, T. (2018). A direct procedure for the seismic design of frame structures with
 added viscous dampers. Seismic Resistant Structures, 37.
- [11] Tubaldi, E., Barbato, M., Dall'Asta, A., 2015. Efficient approach for the reliability-based design of linear damping devices for seismic protection of buildings. ASCE-ASME Journal of Risk and Uncertainty in Engineering Systems, Part A: Civil Engineering, 2(2), C4015009. DOI: 10.1061/AJRUA6.0000858.
- [12] Palermo, M., Silvestri, S., Landi, L., Gasparini, G., & Trombetti, T. (2018). A "direct five-step procedure" for the
 preliminary seismic design of buildings with added viscous dampers. *Engineering Structures*, *173*, 933-950.
- [13] Altieri, D., Tubaldi, E., De Angelis, M., Patelli, E., & Dall'Asta, A. (2018). Reliability-based optimal design of
 nonlinear viscous dampers for the seismic protection of structural systems. *Bulletin of Earthquake Engineering*, 16(2), 963-982.
- [14] Pollini, N., Lavan, O., & Amir, O. (2018). Optimization-based minimum-cost seismic retrofitting of hysteretic frames
 with nonlinear fluid viscous dampers. *Earthquake Engineering & Structural Dynamics*, 47(15), 2985-3005.
- [15] De Domenico, D., Ricciardi, G., & Takewaki, I. (2019). Design strategies of viscous dampers for seismic protection
 of building structures: a review. *Soil Dynamics and Earthquake Engineering*, *118*, 144-165.
- [16] Victorsson, V.K. (2011). The reliability of capacity-designed components in seismic resistant systems. *PhD thesis*,
 Stanford University, California, US.
- [17] Tirca, L. and Chen, L. (2014). Numerical simulation of inelastic cyclic response of HSS braces upon fracture.
 Advanced Steel Construction, 10(4), 442-462.
- [18] Matsui, R., Takeuchi, T., Urui, S., & Tokuno, M. (2018). Collapse Analysis of Steel Frames Considering Fracture of
 Braces and End of Beams. In Key Engineering Materials (Vol. 763, pp. 686-693). Trans Tech Publications.
- [19] American Society of Civil Engineers. Seismic Evaluation and Retrofit of Existing Buildings: ASCE Standard
 ASCE/SEI 41-17. American Society of Civil Engineers, 2017.
- 837 [20] European Committee for Standardization. EN 15129:2010 Antiseismic devices, Brussels, Belgium, 2010
- [21] European Committee for Standardization. Eurocode 8-Design of Structures for Earthquake Resistance. Part 1:
 General Rules, Seismic Actions and Rules for Buildings, Brussels, Belgium, 2004.
- [22] ASCE (American Society of Civil Engineers). (2010). Minimum design loads for buildings and other structures. *Standard ASCE/SEI 7-10*.
- [23] Fajfar, P. (2018). Analysis in seismic provisions for buildings: past, present and future. *Bulletin of Earthquake Engineering*, 16, 2567-2608. DOI:°10.1007/s10518-017-0290-8
- [24] Seo, C.Y., Karavasilis, T.L., Ricles, J.M, Sause, R., 2014. Seismic performance and probabilistic collapse resistance
 assessment of steel moment resisting frames with fluid viscous dampers, Earthquake Engineering and Structural
 Dynamics, 43(14), 2135-2154.
- [25] Gidaris, I., Taflanidis, A.A., 2015. Performance assessment and optimization of fluid viscous dampers through life cycle cost criteria and comparison to alternative design approaches. Bulletin of Earthquake Engineering, 13, 1003 1028.
- [26] Tubaldi, E., Barbato, M., Dall'Asta, A., 2014. Performance-based seismic risk assessment for buildings equipped with linear and nonlinear viscous dampers. Engineering Structures, 78, 90-99.
- [27] Tubaldi, E., Kougioumtzoglou, I.A., 2015. Nonstationary stochastic response of structural systems equipped with
 nonlinear viscous dampers under seismic excitation. Earthquake Engineering and Structural Dynamics, 44(1): 121–
 138.
- [28] Dall'Asta, A., Tubaldi, E., Ragni, L., 2016. Influence of the nonlinear behaviour of viscous dampers on the seismic demand hazard of building frames. Earthquake Engineering and Structural Dynamics, 45(1), 149-169.
- [29] Tubaldi, E., Ragni, L., Dall'Asta, A., 2015. Probabilistic seismic response assessment of linear systems equipped with nonlinear viscous dampers. Earthquake Engineering & Structural Dynamics, 44 (1), 101-120.
 DOI: 10.1002/eqe.2461

- [30] Dall'Asta, A., Scozzese, F., Ragni, L., Tubaldi, E., 2017. Effect of the damper property variability on the seismic reliability of linear systems equipped with viscous dampers. Bulletin of Earthquake Engineering, 15(11), 5025-5053. DOI 10.1007/s10518-017-0169-8
- [31] Lavan, O., Avishur, M., 2013. Seismic behavior of viscously damped yielding frames under structural and damping
 uncertainties. Bulletin of Earthquake Engineering, 11(6), 2309–2332.
- [32] Scozzese, F., Dall'Asta, A., Tubaldi, E., 2019. Seismic risk sensitivity of structures equipped with anti-seismic devices with uncertain properties, Structural Safety, 77, 30-47.
- [33] Miyamoto, H. K., Gilani, A. S., Wada, A., & Ariyaratana, C. (2010). Limit states and failure mechanisms of viscous dampers and the implications for large earthquakes. *Earthquake Engineering & Structural Dynamics*, 39, 1279-1297.
- [34] Miyamoto, H. K., Gilani, A. S., Wada, A., & Ariyaratana, C. (2010). Collapse risk of tall steel moment frame
 buildings with viscous dampers subjected to large earthquakes: PART I: DAMPER LIMIT STATES AND FAILURE
 MODES OF 10-STOREY ARCHETYPES. The Structural Design of Tall and Special Buildings, 19(4), 421-438.
- [35] Lee, D., & Taylor, D. P. (2001). Viscous damper development and future trends. The Structural Design of Tall
 Buildings, 10(5), 311-320.
- [36] Agrawal, A. K., & Amjadian, M. (2016). Seismic component devices. In Innovative bridge design handbook (pp. 531-553). Butterworth-Heinemann.
- 876 [37] Filiatrault, A., & Christopoulos, C. (2006). Principles of passive supplemental damping and seismic isolation.
- [38] Impollonia, N., & Palmeri, A. (2018). Seismic performance of buildings retrofitted with nonlinear viscous dampers
 and adjacent reaction towers. Earthquake Engineering & Structural Dynamics, 47(5), 1329-1351.
- [39] Miyamoto, H. K., Gilani, A. S., Wada, A., & Ariyaratana, C. (2011). Identifying the collapse hazard of steel special
 moment-frame buildings with viscous dampers using the FEMA P695 methodology. Earthquake Spectra, 27(4),
 1147-1168.
- [40] Vamvatsikos, D., & Cornell, C. A. (2002). Incremental dynamic analysis. *Earthquake Engineering & Structural Dynamics*, 31(3), 491-514.
- [41] Scozzese, F., Tubaldi, E., & Dall'Asta, A. (2020). Assessment of the effectiveness of Multiple-Stripe Analysis by using a stochastic earthquake input model. *Bulletin of Earthquake Engineering*, 1-37.
- [42] Barroso LR, Winterstein S. Probabilistic seismic demand analysis of controlled steel moment-resisting frame structures. *Earthquake Engineering and Structural Dynamics* 2002; 31(12):2049–2066.
- [43] Constantinou, M. C., & Symans, M. D. (1992). Experimental and analytical investigation of seismic response of structures with supplemental fluid viscous dampers. Buffalo, NY: National Center for earthquake engineering research.
- [44] McKenna, F. (2011). OpenSees: a framework for earthquake engineering simulation. *Computing in Science & Engineering*, 13(4), 58-66.
- [45] NTC 2018 (2018). Aggiornamento delle "Norme Tecniche per le costruzioni" (in italian). D:M: 17/01/2018 Ministry of Infrastructure and Transport.
- [46] Ohtori, Y., Christenson, R.E., Spencer Jr., B.F. and Dyke, S.J. (2004). Benchmark Control Problems for Seismically
 Excited Nonlinear Buildings. *Journal of Engineering Mechanics*, 130, 366-385.
- [47] Scozzese F, Terracciano G, Zona A, Della Corte G, Dall'Asta A, Landolfo R. Analysis of seismic non-structural
 damage in single-storey industrial steel buildings. Soil Dyn Earthq Eng 2018; 114:505–19.
 doi:10.1016/j.soildyn.2018.07.047.
- [48] Scozzese, F., Terracciano, G., Zona, A., Corte, G. D., Dall'Asta, A., & Landolfo, R. (2018). Modeling and seismic
 response analysis of Italian code-conforming single-storey steel buildings. Journal of Earthquake
 Engineering, 22(sup2), 2104-2133.
- [49] Bradley BA. A comparison of intensity-based demand distributions and the seismic demand hazard for seismic performance assessment. Earthquake Engineering and Structural Dynamics 2013; 42(15):2235–2253.
- [50] Occhiuzzi A. Additional viscous dampers for civil structures: analysis of design methods based on effective evaluation of modal damping ratios. Eng Struct 2009;31(5):1093–101.
- [51] Seleemah, A.A. and Constantinou, M.C. (1997). Investigation of seismic response of buildings with linear and nonlinear fluid viscous dampers. Report No. NCEER-97-0004. New York: National Center for Earthquake Engineering Research, State University of New York at Buffalo.
- 910 [52] Hanson, R.D. and Song, T.T. (2001). Seismic design with supplemental energy dissipation devices. Oakland, 911 California: Earthquake Engineering Research Institute.
- [53] Gkimprixis, A., Tubaldi, E. & Douglas, J. Comparison of methods to develop risk-targeted seismic design maps. Bull
 Earthquake Eng 17, 3727–3752 (2019). https://doi.org/10.1007/s10518-019-00629-w