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## Facial Muscle Activity: High-Sensitivity Noncontact Measurement Using Laser Doppler Vibrometry

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(Article begins on next page)

# SEISMIC RELIABILITY OF BASE ISOLATED SYSTEMS: SENSITIVITY TO DESIGN CHOICES

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**Abstract.** Seismic isolation is considered an effective solution to protect buildings and related content from earthquakes, and consequently reduce seismic losses. However, the overall reliability levels achieved on these systems by following the design rules suggested by codes are not uniform and they may be strongly influenced by some choices made in the structural design. This study aims to investigate the seismic reliability of structural systems equipped with high-damping rubber bearings, which is a widely used class of isolators. An extensive parametric analysis is performed to assess the influence of design choices on the failure probability, considering design parameters concerning both the isolation system and the superstructure, such as: isolation period; bearings shear strain; percentage of flat sliders (i.e., bearing shape factors); superstructure overstrength ratio. A set of case studies have been configured by varying and combining all the aforesaid parameters. A stochastic model is used for the bidirectional seismic input and the generation of horizontal ground motion components, whereas full probabilistic analyses are performed via Subset Simulation to achieve accurate estimates of the demand hazard curves up to very small failure probabilities. To reduce the computational effort, a 3D-model with a reduced number of DOFs (Degrees of Freedoms) is adopted for each case study. It consists of an uncoupled bidirectional elastoplastic model of the superstructure, and an advanced nonlinear 3D model of the rubber isolators, accounting for the coupling between vertical and horizontal response in large displacements. For each case analysed, demand hazard curves are evaluated to illustrate the probabilistic properties of the seismic response for both isolation system and superstructure. Results show a noticeable sensitivity of the system reliability with respect to the examined design choices and in some cases the achieved structural performance can be far from the safety levels required by the Codes.

**Keywords:** seismic isolation design, rubber bearings, seismic reliability, Subset Simulation, stochastic model, overstrength factors.

## 1 Introduction

This paper focuses on seismic base isolation with High Damping Rubber (*HDR*) bearings, which is an efficient and widely used technique for passive seismic protection of buildings and related content [1][2]. In general, seismic isolation drastically reduces the structural and non-structural damage even in the case of earthquakes of medium-high intensities, notably shortening the post-event recovery time and enhancing structures resilience. Most of the modern codes adopted by earthquake prone countries include prescriptions about the design of isolated structures; however, systems designed in accordance with these standards, based on deterministic and conventional design value of the seismic intensity, can show very different performances under extreme events. Consequently, the reliability, measured in terms of the mean annual frequency (*MAF*) of failure, can result significantly inhomogeneous case by case.

Isolated buildings can be seen as in-series systems in which two main structural components are involved: the isolation system and the superstructure. Both these components may exhibit anticipated failure potentially lowering the overall robustness of the system (i.e., the failure of a single component can lead to the collapse of the whole structure). Moreover, even if the isolation system does not fail, the stiffening behaviour at large displacements of *HDR* bearings may cause an increase of the base shear leading to a brittle collapse of the superstructure, as it is not designed for a large ductile behaviour.

Recent studies [3] highlighted that current code prescriptions for isolator production or qualifications may be sometimes inadequate to guarantee that bearings are able to face events significantly larger than the design one. Thus, the calibration of adequate safety factors is essential to achieving satisfactory safety levels. For example, the actual displacement capacity of *HDR* bearings under horizontal and vertical loads is a key parameter controlling the failure, but the current version of EN15129 [4] (i.e., the European standard for seismic isolation devices) prescribes a strength test (called “lateral capacity test”) up to a shear deformation only a little greater than the reference value

used in the design. Consequently, in the European context the collapse deformation is not actually known by the manufacturer neither by the structural designer. The American seismic code [5] also presents similar limits, as recently highlighted in [3] and demonstrated in detail by Kitayama and Constantinou [6] and Shao et al. [7] for friction isolators. Furthermore, prescriptions for superstructure strength are not consolidated and are still matter of discussion [6][7][8][9][10]. Therefore, while code conforming traditional solutions are characterized by adequate reliability levels (procedures to make high quality structural components are consolidated as well as safety coefficients to be used in the design), code conforming base-isolated structures may show reliability levels below the target suggested by the design codes [11][12]. At this regard, American code [5] prescriptions for seismic design requires an “absolute” collapse probability lower than 1% in 50 years, and this limit value is going to be implemented in the future revisions of Eurocodes too, as illustrated in [11].

In order to assess whether the probability of structural collapse is under the target reliability level, seismic reliability analyses must be carried out by using proper probabilistic approaches, as recently carried out for structures equipped with dissipation devices which suffer similar issues [13][14][15][16][17], and for base-isolated structures equipped with different kinds of isolators [6][18][19].

However, most of the previous studies and relevant conclusions about the system reliability are based on simplified mechanical behaviours of *HDR* bearings (e.g., equivalent elastic or elastoplastic) [1][19] and/or deduced from a planar seismic analysis (neglecting the effects related to the two-directional behaviour of isolators and structures [6][7][18][20]). Recently, some code-conforming case studies have been analysed [21][22][23][24] by considering a bi-directional input [25] and advanced 3D nonlinear models for *HDR* bearings and unsatisfactory failure rates have been observed in these studies too. Nevertheless, a deeper analysis is required to consolidate these observations, especially because most of the previous studies are focused on specific single case studies instead of looking at a wide range of possible case studies resulting from the design process.

For this reason, in this paper, a systematic study on the role of design parameters choice on the overall reliability is investigated to evaluate the potential variation of the failure probability respect to required target values. In particular, the following parameters have been considered and varied within the range of most common values: isolation periods; bearings design shear deformation; percentage of flat sliders (i.e., the number of *HDR* bearings and flat sliders, if any, respect to the overall number of bearings); design overstrength ratio (i.e., the ratio between the actual superstructure base shear strength and the superstructure base shear demand at the design condition). A set of case studies are configured by varying and combining all the aforesaid parameters.

Probabilistic analyses are performed via Subset Simulation [26][27], which is an efficient and robust tool able to provide accurate estimates of the demand hazard curves up to very small failure probabilities, which is essential especially in the case of strategic structures in which the reliability level must be higher than standard structure [28][29][30]. A stochastic model is used for the bidirectional seismic input characterization, whose parameters have been calibrated to be representative of Italian high seismicity zones. Moreover, to reduce the computational effort of analyses, a 3D-model with a reduced number of DOFs is adopted for each case study. It consists of an uncoupled bidirectional elastoplastic model of the superstructure, and an advanced 3D nonlinear model of the *HDR* bearings, accounting for the coupling between vertical and horizontal response in large displacements [31][32][33]. The choice of this simplified 3D-model allows considering the characteristic bidirectional behaviour of isolation system keeping as low as possible the computational effort [13] and enabling the use of a full probabilistic approach.

The influence of the above parameters on the seismic response of the system is evaluated by providing a comparison in terms of demand hazard curves for the two main demand parameters: the maximum relative displacement of the superstructure and the maximum shear deformation of the isolation system. Results are discussed and useful insights are provided about the safety margins needed to obtain adequate reliability levels of base-isolated systems. In details, the paper is structured

as follows: first the probabilistic framework is introduced, by presenting both the reliability analysis tool and the stochastic hazard model; then the case studies and their design are presented, along with the relevant modelling strategy; finally, the outcomes of the parametric investigation are discussed, and conclusions are provided.

## 2 Probabilistic method

This section describes the probabilistic framework used to perform seismic reliability analyses on base-isolated systems. The framework consists of an efficient probabilistic tool, Subset Simulation [26], and a stochastic ground motion model for seismic hazard characterization and bidirectional seismic samples generation.

### 2.1 Reliability analysis

Seismic reliability analysis aims to assess the probability of a structural system attaining an unsatisfactory performance at least once within a reference time frame. The system response subjected to the seismic hazard is described by the random variable  $D$ , whose recurrence properties over time are expressed by the mean annual frequency (*MAF*) of exceedance of a threshold  $d$ :

$$\nu_D(d) = \bar{\nu} G_D(d) \quad (1)$$

with  $\bar{\nu}$  denoting the *MAF* of occurrence of at least one event within the range of intensities of interest, which is a function of the seismic scenario (location of seismic source and recurrence properties of seismic events), and  $G_D(d) = P[D > d]$  characterizing the probability of exceedance of a threshold  $d$  of the demand parameter  $D$ , given the occurrence of any earthquake of intensity higher than the minimum expected from the source (i.e., consistent with  $\bar{\nu}$ ). To perform a reliability analysis, the function  $\nu_D(d)$  must be estimated over a wide range of threshold values to characterise the probabilistic response of the system from the highest up to the lowest probabilities of exceedance. Being the *MAF*  $\nu_{target} = 2 \cdot 10^{-4}$  1/year the target reliability level commonly required by the Codes

125 for structural systems [11][12], the systems' reliability and failure conditions should be assessed at  
 126 least up to this *MAF* value.

127 To achieve this aim, different probabilistic approaches could be used, such as (direct)  
 128 simulation-based methods or conditional approaches. The first class of methods consists of tools  
 129 based on the observation of the system response to samples drawn from the probability distribution  
 130 of the random inputs (e.g., earthquake characteristics, structural model) and encompasses methods  
 131 like Monte Carlo simulation [34] and the more efficient variance reduction techniques, such as  
 132 Importance Sampling [35] and Subset Simulation [26]. The methods belonging to the second class  
 133 have been developed in the last 20 years, since the seminal works of Cornell et al. [36], with the main  
 134 purpose of making seismic reliability and risk estimation more practice-oriented and computationally  
 135 affordable. The latter methods are widely adopted within the performance-based earthquake  
 136 engineering (PBEE) approach [37] proposed by the Pacific Earthquake Engineering Research Center  
 137 (PEER) [38][39].

138 In this study, the robust Subset Simulation [26] is used for estimating accurate demand hazard  
 139 curves within the range of *MAFs* from  $10^{-1}$  to  $10^{-5}$  1/year. The basic idea behind this advanced  
 140 simulation technique is to express the rare-event probability  $G_D(d_i)$  in terms of the product of larger  
 141 conditional probabilities, by introducing intermediate exceedance events corresponding to lower  
 142 threshold values  $d_1 < d_2 < \dots < d_i$ . In the analyses, the original implementation [27][40] of the method is  
 143 employed. This relies on the Markov Chain Monte Carlo algorithm and the Metropolis–Hastings  
 144 sampler to generate samples conditional on the intermediate failure regions and thus gradually  
 145 populate from the frequent to rare event region in an efficient way. Assuming a fixed value  $p_0$  for the  
 146 conditional probabilities of exceedance of the various thresholds, each time a set of  $n_{sim}$  samples is  
 147 generated through the Metropolis-Hastings algorithm (standard Monte Carlo simulation for the first  
 148 threshold), and the corresponding demand threshold  $d_i$  is simply evaluated as the  $(1-p_0)n_{sim}$ -th largest  
 149 value. The exceedance probability of the  $i$ -th threshold, computed by carrying out  $i$ -times the product

150 of the same probability  $p_0$ , is  $p_0^i$ , for  $i=1, 2, \dots, l$ , and the lowest obtained value of the failure  
 151 probability is  $p_0^l$ .

152 In this study, demand hazard curves are estimated by performing, for every case of analysis,  
 153 a set of 10 independent runs of Subset Simulation and by taking their average. In this way, the  
 154 obtained results have a level of accuracy comparable to that of a robust direct Monte Carlo analysis  
 155 performed with millions of simulations [27][40].

## 156 2.2 Stochastic ground motion model

157 A direct simulation approach such as Subset Simulation requires a reliable stochastic representation  
 158 of the bidirectional seismic input to achieve an accurate estimate of small failure probability. In this  
 159 paper, the flexible and widely used stochastic point source simulation method of Boore [41] is  
 160 employed in conjunction with the Atkinson-Silva [42] source-based ground motion model. These  
 161 allow generating ground motion samples conditional to the features of a given seismic scenario,  
 162 specified by two main random variables, the moment magnitude  $M$ , and the epicentral distance  $R$ .  
 163 The moment magnitude is assumed to follow the Gutenberg-Richter recurrence law [43][44],  
 164  $\nu_M(m) = 10^{(a-bm)}$ , with parameters  $a$  and  $b$  characterising the seismic source (mean number of  
 165 earthquakes expected) and the regional seismicity (factor governing the proportion of small to large  
 166 earthquakes), respectively. Given an earthquake event, the aforesaid recurrence law bounded within  
 167 the range of magnitudes of interest  $[m_0, m_{max}]$  leads to the following probability density function of  
 168  $M$  (with  $\beta = b \cdot \log_e(10)$ ).

$$f_M(m) = \beta \frac{e^{-\beta(m-m_0)}}{1 - e^{-\beta(m_{max}-m_0)}} \quad (2)$$

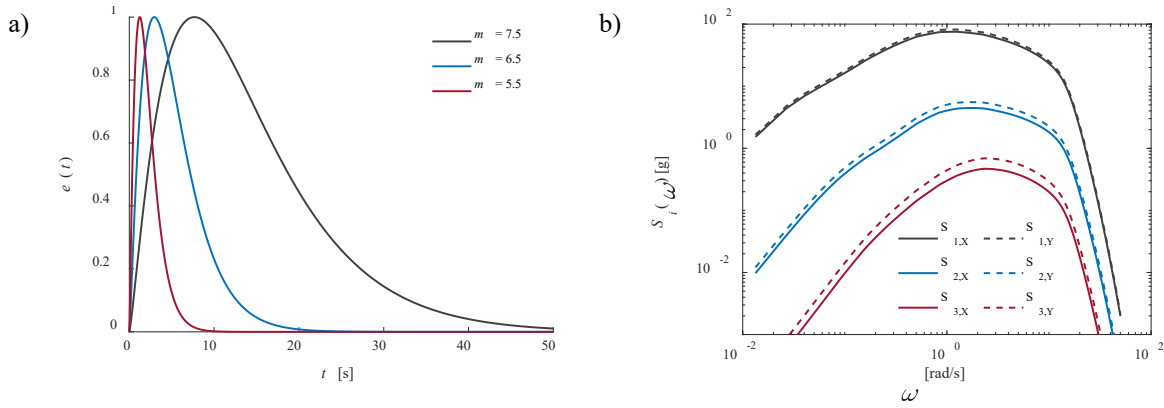
169 The probability density function of  $R$  is obtained under the hypothesis that the source produces  
 170 random earthquakes with equal likelihood anywhere within a distance from the site  $r_{max}$ , beyond  
 171 which the seismic effects are assumed to become negligible.

172 The procedure for the simulation of two horizontal ground motion components is summarised  
 173 below and follows from [41][42] as modified by [45]. A pair of seismic acceleration time series is  
 174 obtained by modulating in time two white noise signals,  $w_i(t)$  (with  $t$  denoting time and  $i = 1, 2$ ), by  
 175 means of the shape function  $e(t)$ ; the Fourier transform  $\bar{Z}_i(\omega)$  of the resulting time-functions  $z_i(t) =$   
 176  $e(t)w_i(t)$  (normalized to have unitary mean square amplitude) are then multiplied by the target  
 177 radiation spectra  $S_i(\omega) = \varepsilon_i A(\omega)$ , where  $A(\omega)$  is a deterministic function of the angular frequency  $\omega$   
 178 while  $\varepsilon_i$  are random scaling factors accounting for the spectral amplitude variability; the desired  
 179 ground motion acceleration time series  $a_i(t)$  can be finally obtained by the inverse Fourier transform  
 180 of the function  $\varepsilon_i A(\omega) \bar{Z}_i(\omega)$ . It is worth noting that both the time modulating function and the radiation  
 181 spectrum depend (also) on seismic scenario and site-related parameters (i.e., the moment magnitude,  
 182 the epicentral distance and the local soil conditions) although (for simplicity of notation) such  
 183 dependency was not made explicit in the description provided above. For sake of clarity, Fig. 1  
 184 illustrates the variability with the magnitude (at fixed epicentral distance 20 km) of both the radiation  
 185 spectra and the time-envelope functions.

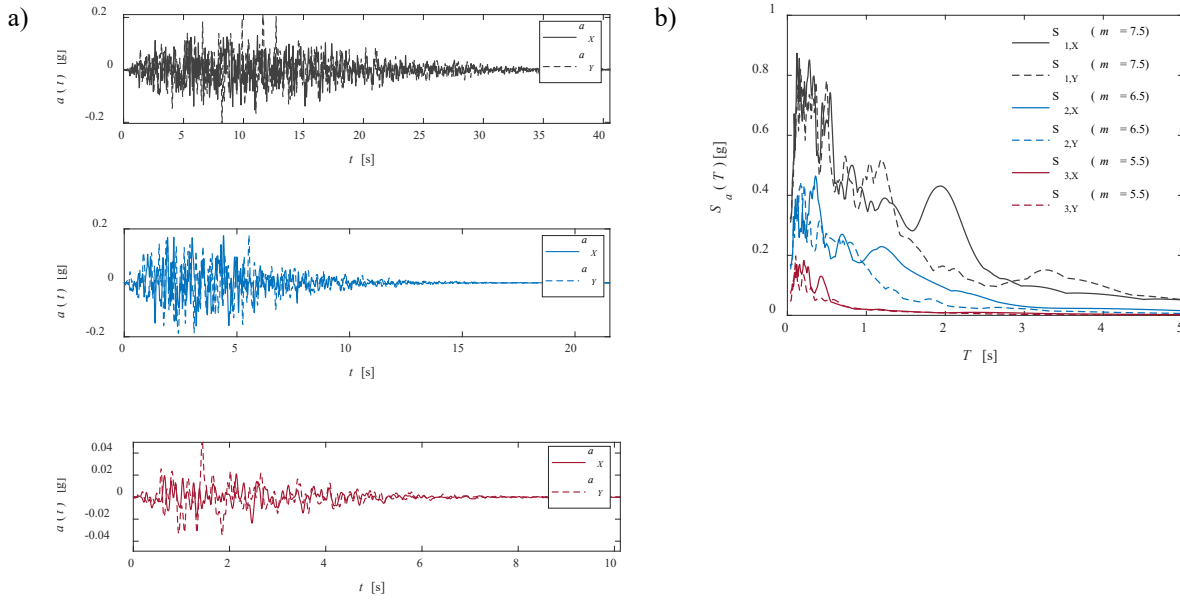
186 The two scaling parameters,  $\varepsilon_1$  and  $\varepsilon_2$  (also called random scaling disturbance), are modelled  
 187 as lognormal random variable having unit median, standard deviation  $\sigma_{\ln \varepsilon} = 0.523$  (similarly to what  
 188 suggested by [40] for unidirectional seismic actions) and correlation  $\rho = 0.8$  [46]. The random scaling  
 189 disturbance ( $\varepsilon_1, \varepsilon_2$ ), together with the Gaussian white noise process, ensure that the ground motions  
 190 record-to-record variability is accounted for, in terms of energy content variability within both the  
 191 time and frequency domain. The resulting overall variability provided by the model is shown through  
 192 the plots of Fig. 2a and Fig. 2b, which depict the response spectra of pairs of horizontal components  
 193 for different values of magnitudes and the corresponding acceleration time series, respectively.

194 To summarise this section, the random properties of the seismic ground motions are described  
 195 by the set of variables  $\{M, R, \varepsilon, \mathbf{W}\}$ , being  $\mathbf{W}$  the  $2 \times K$  matrix collecting the stochastic white-noise  
 196 processes  $w_1(t)$  and  $w_2(t)$ , each of which is modelled through an independent  $K$ -dimensional Standard

197 Gaussian vector  $\mathbf{W}_i$  ( $i = 1, 2$ ) with elements  $w_{i,k}$  ( $k = 1, 2, \dots, K$ ) evaluated at discrete time instants  $t_k$   
 198  $= k\Delta t$ , consistently with the finite time interval  $\Delta t$  adopted to perform the numerical integration. The  
 199 rest of the scenario's parameters (e.g.,  $a$  and  $b$  related to the Gutenberg-Richter law, the shear-wave  
 200 velocity  $V_{S30}$  characterising the seismic response amplification of the soil, etc.) are fixed parameters.



201 Fig. 1. Time-envelope functions (a) and Target Fourier spectra (b) of pairs of horizontal components for  $r = 20$  km and  
 202 different  $M$  values.



203 Fig. 2. Acceleration time series (a) and Response spectra (b) for three pairs of horizontal seismic components  
 204 corresponding to different magnitudes (at  $r = 20$  km).

## 205 2.3 Seismic hazard

206 A seismic hazard representative of Italian high seismicity zones is adopted in this study. The  
 207 following set of parameters governing the stochastic hazard model is selected, also according to the  
 208 existing literature [43][47][48]:  $m_0 = 5.5$ ,  $m_{max} = 8$ ,  $a = 4.35$  and  $b=0.9$ ,  $r_{max} = 50$  km. A shear wave  
 209 velocity  $V_{S30}$  equal to 255 m/s has been chosen as representative of deformable soil conditions at the  
 210 site [49].

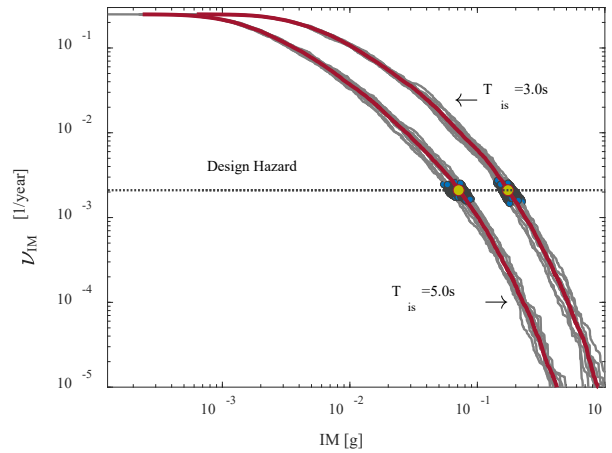
211 Although a direct simulation approach is used in this study to perform seismic reliability  
 212 analyses, an Intensity Measure ( $IM$ ) has been introduced to carry out the design of all the considered  
 213 case studies: according to the current concept of partial safety factors, the  $IM$  is used to quantify the  
 214 seismic intensity at the design rate of occurrence prescribed by the codes. Many different  $IM$  can be  
 215 chosen, but an efficiency evaluation specifically made for isolation systems [50] suggests that the  
 216 best choice is the  $S_{aRotD100}(T, \xi)$  [50][51], combined with the recently proposed strategy of averaging  
 217 the spectral values over a period range ([52][53]), rather than computing them at a given single  $T$   
 218 value. The resulting  $IM$ , denoted as  $AvgS_{aRotD100}$ , is expressed as follows:

$$AvgS_{aRotD100} = \exp \left\{ \frac{1}{N_T} \sum_{i=1}^{N_T} \ln[S_{aRotD100}(T_i)] \right\} \quad (3)$$

219 being  $N_T$  the number of periods in which the considered range  $[T_1, T_{N_T}]$  is discretised; the  
 220 inherent damping rate  $\xi$ , implicit in the above expression, is assumed equal to 5.0%. It is worth noting  
 221 that the use of an average  $IM$  over a range of periods better allows to cope with the variability of the  
 222  $HDR$  bearings dynamic response with the strain amplitude and repeated cycles [54].

223 The isolation systems analysed in the following are characterized by two different isolation  
 224 periods,  $T_{is}=3.0$  s and  $T_{is}=5.0$  s, hence two  $IM_s$  ( $IM_{3s}$ ,  $IM_{5s}$ ) with two different ranges of periods have  
 225 been adopted: from 2.0 s to 4.0 s for the  $T_{is}=3.0$ s isolation system, from 4.0 s to 6.0 s for the  $T_{is}=5.0$ s  
 226 one. Both the intervals have been discretized by steps of 0.1 s.

227 The *IM* hazard curves obtained for the scenario defined above via Subset Simulation  
 228 (according to the method of [55]) are depicted in Fig. 3, along with some further information provided  
 229 to ease the understanding of the design strategy discussed later. The curves averaged on 10  
 230 independent runs ([27][40]) are assumed as *IM* curves (red solid lines identify the average, grey  
 231 lighter curves the single runs); the horizontal black dotted line identifies the design hazard level  
 232 considered by the European codes [56], represented by a *MAF* of exceedance  $\nu_d = 0.0021$  1/year  
 233 (probability of exceedance of 10% in 50 years), which corresponds to the intensities  $im_d = 0.173$  g  
 234 for the  $T_{is}=3.0$ s isolation system and  $im_d = 0.071$  g for the  $T_{is}=5.0$ s one ( $g$  is the gravity acceleration),  
 235 as highlighted by the yellow dots in the chart. Finally, the blue circle markers added to the plot show  
 236 both intensities and *MAFs* of the ground motion samples used to design the isolation systems (each  
 237 circle corresponds to a pair of ground motion components, being the chosen *IM* direction-  
 238 independent). As better described in Section 3.3, 100 accelerograms are generated to design the base-  
 239 isolated systems, so that to have *IMs* as close as possible to the target *IM* values ( $im_d$ ).



240  
 241 Fig. 3. *IM* hazard curves and design conditions for two isolation periods: 3.0s, 5.0s.

## 242 3 Parametric analysis

### 243 3.1 Case studies and parameters investigated

244 The design process of isolated structures involves a series of design choices. To assess their  
 245 effect on the structural reliability, an extensive parametric analysis has been performed considering  
 246 one archetype building and by varying the following set of design parameters: isolation periods  $T_{is}$ ;  
 247 bearings design shear deformation  $\gamma_d$ ; percentage of flat sliders; design overstrength ratio (i.e., the  
 248 ratio between the superstructure base shear capacity and the superstructure base shear demand at the  
 249 design condition). More in detail, the following archetype building is selected as case study (Fig. 4):  
 250 a four-storey reinforced concrete (r.c.) building (total height 12m) with 1 kNs<sup>2</sup>/m<sup>3</sup> distributed mass  
 251 for each floor (5 floors including the base floor above the isolation system), 2 x 4 spans of 5m each,  
 252 and 15 columns, for a total mass of 1000 kNs<sup>2</sup>/m.

253 For what concerns the variable design parameters, two isolation periods, equal to  $T_{is}=3$ s and  
 254  $T_{is}=5$ s, have been considered: the former is somehow a current common value for new isolated  
 255 buildings whereas the latter is an upper limit value for residential buildings. Regarding the design  
 256 shear deformation, three values are considered, i.e.,  $\gamma_d=1$ ,  $\gamma_d=1.5$  and  $\gamma_d=2$ , which are all lower than  
 257 the limit of 2.5 imposed by the European code on anti-seismic devices [4] and around common values  
 258 (1.5) currently used by designer in European countries.

259 Two different device configurations have been investigated by varying the number of rubber  
 260 bearings ( $N_{is}$ ): with only rubber bearings (a total of 15 *HDR* bearings, one under each column) and  
 261 with a combination of rubber bearings and flat sliders (8 *HDRs* and 7 flat sliders, placed according to  
 262 the configuration of Fig. 4a).

263 Another design parameter considered in the parametric study is the superstructure strength,  
 264 which depends on the seismic demand at the design condition as well as on the design prescriptions  
 265 and safety factors. At this regard, for isolated structures, the Eurocode [56] allows designing with a

266 reduced value of the seismic lateral force by adopting a behaviour factor  $q$  falling in the range [1,  
 267 1.5]; similarly, ASCE 7 [5] prescribes  $q$  values in the range [1.0, 2.0]. These Codes' indications are  
 268 based on the hypothesis that the minimum superstructure yielding strength is higher than the design  
 269 value magnified by  $q$ , due to safety factors applied to material strengths and a minimum structure  
 270 redundancy [57]. The ratio between the design base shear and the actual yielding force of the system  
 271 is generally defined over-strength factor,  $\Omega$ , which may be notably higher than the behaviour factor  
 272  $q$ , especially for isolated structures (up to a value of 2.5 [22]), because of other superstructure strength  
 273 sources stemming from non-structural elements (e.g., strong infill panels) and non-seismic actions  
 274 (gravity and wind loads). It is therefore useful to define an overstrength ratio  $\Omega/q$  which directly  
 275 expresses the ratio between the actual strength capacity and the seismic demand. Considering the  
 276 limit values for both  $q$  and  $\Omega$ , the two limit cases of  $\Omega/q=1.0$  and  $\Omega/q=2.5$  have been analysed in this  
 277 work, according to  $q$ - $\Omega$  pairs equal to 1.5-1.5 and 1.0-2.5 respectively.

278         Given the already high number of variable parameters (as detailed above), the fixed-base  
 279 fundamental period of the superstructure is set as constant parameter, equal to  $T_s = 0.5$ s regardless of  
 280 the overstrength ratio  $\Omega/q$ .

281         A total of 12 case studies have been configured by varying and combining all the aforesaid  
 282 parameters, as summarised in Table 1.

283         The last case study (case 12) has the specificity of having the same superstructure yielding  
 284 strength of the case 4 ( $T_{is}=3$ s and  $\Omega/q=2.5$ ) and an isolation system designed with  $T_{is}=5$ s. The resulting  
 285 value  $\Omega/q$  is equal to 4.75 due to the lower design base shear of  $T_{is}=5$ s. Indeed, this case has been  
 286 considered with the aim of showing how a higher isolation period can improve the structural seismic  
 287 performances without gravitating too much on the costs of the superstructure.

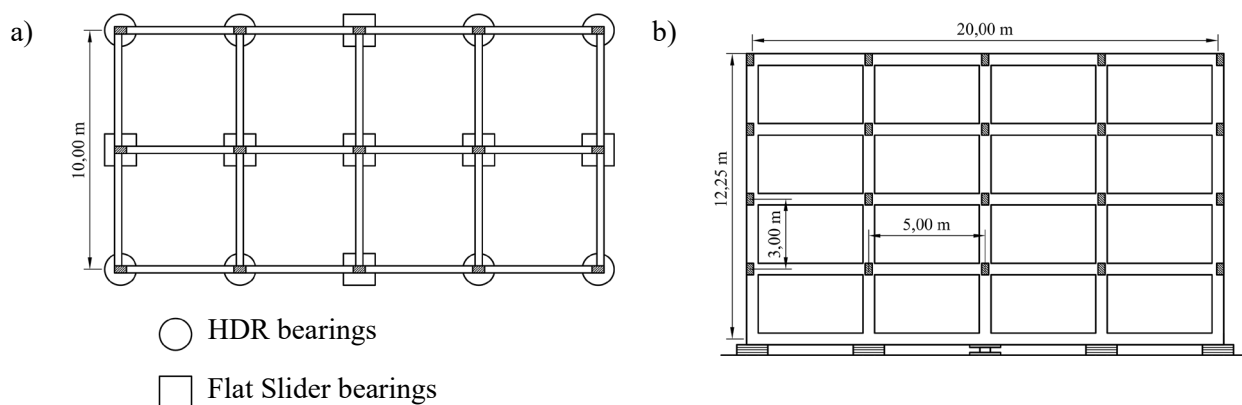


Fig. 4. Plan view of the second bearing configuration (a) and section views of the case study (b) (configuration with 8 rubber bearings and 7 flat slider)

Table 1. Case studies considered in parametric analysis.

| Case | $T_{is}$<br>[s] | $\gamma_d$<br>[-] | $N_{is}$<br>[-] | $\Omega/q$<br>[-] |
|------|-----------------|-------------------|-----------------|-------------------|
| 1    | 3.0             | 2.0               | 15              | 1.0               |
| 2    | 3.0             | 1.5               | 15              | 1.0               |
| 3    | 3.0             | 1.0               | 15              | 1.0               |
| 4    | 3.0             | 2.0               | 15              | 2.5               |
| 5    | 3.0             | 1.5               | 15              | 2.5               |
| 6    | 3.0             | 1.0               | 15              | 2.5               |
| 7    | 3.0             | 2.0               | 8               | 1.0               |
| 8    | 3.0             | 1.5               | 8               | 1.0               |
| 9    | 3.0             | 1.0               | 8               | 1.0               |
| 10   | 5.0             | 2.0               | 8               | 1.0               |
| 11   | 5.0             | 2.0               | 8               | 2.5               |
| 12   | 5.0             | 2.0               | 8               | 4.75              |

### 3.2 Numerical model

A numerical model is developed for each case study with the aim of reducing as much as possible the computational cost of probabilistic analyses (i.e., high number of simulations, each requiring the solution of a nonlinear-time history analysis of the base-isolated structure), without

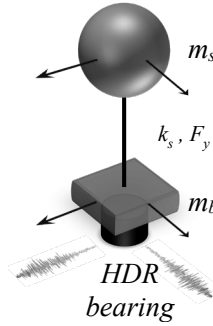
losing the accuracy in terms of the complex *HDR* bearing behaviour. Indeed, differently from past studies, where simplified mechanical behaviours of *HDR* bearings (e.g., equivalent elastic or elastoplastic) [1][18] and/or planar models were considered ([6][7][18][20]), the present work exploits a refined modelling approach, which takes origin from the two-degree of freedom (2-DOF) model proposed by Kelly [1][58]. According to [1], the dynamic response of a multi-degree of freedom model (M-DOF) can be efficiently assessed using an equivalent 2-mass model (Fig. 5), able to account for both the isolation and the first fixed base modal contributions to the dynamic response. In the current study, this concept has been extended to a bidirectional seismic input (6 degrees of freedom), considering the complex nonlinear behaviour of the rubber.

More in detail, the model consists of two masses,  $m_b$  and  $m_s$ , both related (but not equal) to the mass of the base slab and the deformable super-structure. To guarantee the dynamic equivalence of the response between the two mass model and a full M-DOF model, the two masses  $m_b$  and  $m_s$  should be chosen in such a way that  $m_s$  is equal to the effective mass of the first fixed base superstructure mode while  $m_s + m_b$  is equal to the total mass of the building  $M$  (including the base slab mass). Given the features of the building, the ratio between the effective mass of the first fixed base superstructure mode and the total mass of the system has been assumed equal to 0.6 [1], i.e.,  $m_s = 0.6M$  and  $m_b = 0.4M$ .

The motion of  $m_b$  with respect to the ground is described by the vector  $\mathbf{u}_b = [u_{bx}, u_{by}, u_{bz}]$  collecting the motion component along two horizontal directions and the vertical direction. The motion of the mass  $m_s$  relative to  $m_b$  is described by the vector  $\mathbf{u}_s = [u_{sx}, u_{sy}]$ , neglecting the vertical relative motion of the two masses. The dynamic balance equations can thus be formulated as follows:

$$\begin{aligned}
 -\mathbf{f}_s(\mathbf{u}_s) &= m_s(\ddot{\mathbf{u}}_s + \ddot{\mathbf{u}}_b + \ddot{\mathbf{u}}_g) \\
 \mathbf{f}_s(\mathbf{u}_s) - \mathbf{f}_b(\mathbf{u}_b) &= m_b(\ddot{\mathbf{u}}_b + \ddot{\mathbf{u}}_g)
 \end{aligned} \tag{4}$$

317 where  $\mathbf{u}_g$  is the ground motion and  $\mathbf{f}_s$ ,  $\mathbf{f}_b$  describe the response forces due to the super-  
 318 structure and the isolation system. As for the displacement vectors,  $\mathbf{f}_b = [f_{bx}, f_{by}, f_{bz}]$  is a three  
 319 component vector deriving from the Kikuchi bearing element [32] used in the model while  
 320  $\mathbf{f}_s = [f_{sx}, f_{sy}]$  is the two component force representing the superstructure base reaction.

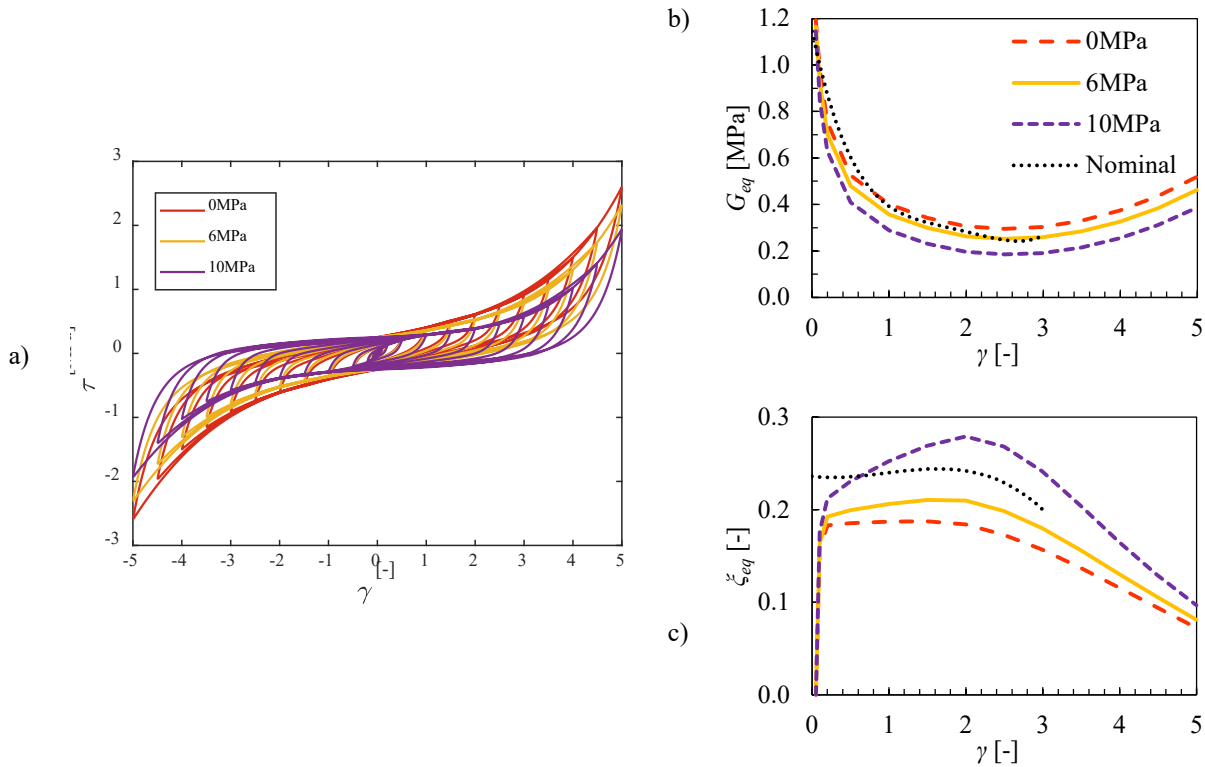


321  
 322 Fig. 5. Scheme of the equivalent 2-mass model adopted for the isolated structures

323 Regarding the mechanical behaviour of the isolation bearings, the Kikuchi bearing element  
 324 [32] available in the Opensees software [59] has been used. It is a fully coupled three-dimensional  
 325 model able to capture the buckling and post-buckling behaviour (i.e. the interaction between the  
 326 coupled bi-dimensional horizontal behaviour and the axial force). More in detail, the model is  
 327 composed by two sets of axial springs (one at the top and the other at the bottom of the element) and  
 328 a radial distribution of shear springs at the mid-height of the element [31]. The large displacement  
 329 formulation permits to reproduce nonlinear geometric effects, i.e. the horizontal and vertical stiffness  
 330 reduction due to the rise of vertical loads and horizontal displacements, as originally proposed by  
 331 Kelly [1][58]. This capability appears very important especially for medium-high level of vertical  
 332 pressure and high displacements, where the buckling load could be attained (usually assumed as the  
 333 condition with zero tangent stiffness of the horizontal response) and the post-buckling behaviour  
 334 could take place up to the theoretical loss of vertical load capacity. This is also influenced by the

335 hardening behaviour at large shear strains, which strongly depends on the rubber compound used for  
 336 the bearings. In particular, the Kikuchi bearing element requires a model for the nonlinear behaviour  
 337 of the shear springs, and in this study the *KikuchiAikenHDR* material has been used (code X0.40MPa).  
 338 The nominal equivalent elastic parameters (i.e., the shear modulus  $G_{eq}$  and the damping ratio  $\xi_{eq}$ ) [4]  
 339 of high damping rubber bearings belong to the Bridgestone rubber compound X0.4S [60] and are  
 340 illustrated in Fig. 6b and Fig. 6c (black dotted line) as a function of the shear strain  $\gamma$ . The other curves  
 341 represent the equivalent elastic parameters coming from the numerical cycles reported in Fig. 6a, with  
 342 reference to a bearing used in the design, as better specified later. It is worth to note that in the  
 343 considered case, the reduction of stiffness due to the increment of vertical load is evident although  
 344 the hardening behaviour at large strain is prevalent with respect to the softening behaviour due to the  
 345 vertical stress.

346 The Kikuchi Bearing Element is also a geometry-influenced model, i.e., describes the  
 347 response of a single bearing and it explicitly depends on the real diameter and height of the isolator.



348 Fig. 6. Behaviour of the *HDR* bearing with 427/158/3  $D_{is}=427$ mm,  $h_{is}=158$ mm at different pressures (a); Nominal and  
 349 numerical equivalent linear parameters of the rubber X0.4S [60] (b, c).

350 To describe the response of the whole base-isolation system by a single element, as assumed  
 351 in this study, it is necessary to introduce the following assumptions: 1) the rubber bearings have all  
 352 the same geometry, 2) they support almost the same amount of vertical load (computed during the  
 353 design process) and 3) the base slab of the structure is stiff enough to be well approximated by a  
 354 diaphragm constraint. With these assumptions the model provides a satisfactory approximation of the  
 355 real behaviour of base-isolated building, except for the overturning effect leading to a variable axial  
 356 load during the seismic event. However, this effect is negligible for low-rise buildings [61] as the  
 357 archetype building considered in this paper. Regarding flat sliders, under the assumption of negligible  
 358 friction coefficients and very high displacement capacity, their contribution is neglected within and  
 359 for the purposes of the present study, although slight response modifications might arise from their  
 360 explicit consideration in the model [20][22][23].

361 As already mentioned, the superstructure has been modelled by two uncoupled elastoplastic  
 362 springs with stiffness  $k_s$  and yielding force  $F_y$ , describing the behaviour of the superstructure frame  
 363 along the two main horizontal directions. The choice of uncoupled springs instead of a coupled  
 364 (isotropic) behaviour for the elastoplastic response is driven by the observation that frames and infill  
 365 panels, which provide stiffness and strength to the superstructure, are usually aligned along the  
 366 x-direction and y-direction with reduced interaction between them. Finally, only tangent-stiffness  
 367 proportional damping is provided to the superstructure to avoid overdamping issues [62][63][64][65],  
 368 with damping rate equal to 2%, typical value for reinforced concrete structures equipped with seismic  
 369 isolators [63][64].

### 370 **3.3 Design of the base isolated system**

371 As already mentioned in Section 2.3, the design of the isolation bearing has been carried out  
 372 using a set of 100 accelerograms consistent with the target design hazard level, represented by a *MAF*  
 373 of exceedance  $\nu_d = 0.0021$  1/year (probability of exceedance of 10% in 50 years), typical of the

374 Ultimate Limit State (ULS) according to European standards [4][56]. The aforesaid design hazard  
 375 level corresponds to the seismic intensity values  $im_d = 0.173\text{ g}$  for the  $T_{is}=3.0\text{s}$  isolation system and  
 376  $im_d = 0.071\text{ g}$  for the  $T_{is}=5.0\text{s}$ . The 100 design accelerograms, generated using the same probabilistic  
 377 hazard framework previously described, are selected to have  $IM$ s as close as possible to these target  
 378 values ( $im_d$ ). In particular, the total rubber thickness and the diameter of the isolation bearing ( $h_{is}$  and  
 379  $D_{is}$  respectively) as well as the superstructure yielding force ( $F_y$ ) have been determined by iteratively  
 380 performing nonlinear time history analysis on each case study with the 100 design accelerograms till  
 381 attaining an average maximum shear strain value equal to the design one  $\gamma_d$ , and a superstructure  
 382 average maximum displacement coherent with the overstrength ratio. This procedure, despite  
 383 cumbersome, has been adopted because of the complex dynamic behaviour of *HDR* bearings, whose  
 384 equivalent linear properties strongly depends on the shear strain amplitude, as shown in Fig. 6. In fact,  
 385 despite a more efficient  $IM$  has been assumed in this work (as described in Section 2.3), design  
 386 procedures based on simplified linear approaches would lead to a seismic response at the design  
 387 condition different from the assumed one, due to the record-to record variability of the strongly  
 388 nonlinear response of *HDR* bearings.

389 Results of the design are summarized in Table 2, Table 3 and Table 4, where the thickness of  
 390 the single rubber layer  $t_r$  and the compression stress  $\sigma$  are also reported. For completeness, also the  
 391 normalised yielding force values  $F_y / Mg$  (where  $g$  is the acceleration of gravity) of the  
 392 superstructure are reported. As expected, the design features of the isolation systems are the same for  
 393 the two overstrength ratios  $\Omega/q=1$  and  $\Omega/q=2.5$ , thus confirming that the strength of the  
 394 superstructure doesn't affect the isolation design.

395 Regarding the two configuration chosen, the cases with  $T_{is}=3\text{s}$  can be designed by using both  
 396 15 and 8 rubber bearings (Table 2 and Table 3), whereas the cases  $T_{is}=5\text{s}$  can be designed only by  
 397 adopting the configuration with 8 rubber bearings and 7 flat sliders (Table 4). A further important  
 398 remark is about the compression stress ( $\sigma$ ) values, which are notably lower than the corresponding

critical pressure values at zero displacement  $\sigma_{cr}$  (especially in the cases with  $T_{is}=3s$  and 8 bearings), as reported in the tables. Consequently, P- $\Delta$  effects due to large displacements are limited, as can be observed in Fig. 6 where numerical cycles are reported for different compression levels, with reference to the bearing with  $D_{is}=427mm$  and  $h_{is}=158mm$ . Only for  $\sigma=10$  MPa differences are more evident. However, the behaviour at large deformations (for shear strains larger than 3) is always characterized by a significant hardening, that prevent the buckling (zero tangent stiffness) and post-buckling behaviour. As expected, increasing the vertical pressure,  $G_{eq}$  decreases in the whole range of shear deformations while  $\xi_{eq}$  increases (as can be seen in Fig. 6 b and c respectively). Other bearings with different dimensions (not shown in these figures) are characterized by similar results, due to the limits defined by the codes.

As final remark, it is worth to note that only nominal properties of bearings are considered in designing the isolation systems, neglecting the variability related to the bearings production or ambient conditions. The aim of this work is in fact to focus only on the effect of the considered design parameters on the final reliability of the system, without introducing other source of uncertainties. For the same reason the probabilistic framework illustrated in the previous section account for only the record-to-record variability, whereas other uncertainties (such as the variability of bearing properties or the variability of their shear deformation capacity) are disregarded in this work.

Table 2. Dimensions of the isolation bearings and superstructure yielding force ( $T_{is} = 3s$ , 15 HDR bearings)

| <i>Case</i> | $\gamma_d$ | $D_{is}$ | $h_{is}$ | $t_r$ | $\sigma$ | $\sigma_{cr}$ | $F_y/Mg$ |
|-------------|------------|----------|----------|-------|----------|---------------|----------|
|             | [-]        | [mm]     | [mm]     | [mm]  | [MPa]    | [MPa]         | [-]      |
| <b>1</b>    | 2          | 393      | 117      | 2.8   | -5.40    | -30.01        | 0.114    |
| <b>2</b>    | 1.5        | 427      | 158      | 3.0   | -4.56    | -23.13        | 0.111    |

| <i>Case</i> | $\gamma_d$ | $D_{is}$ | $h_{is}$ | $t_r$ | $\sigma$ | $\sigma_{cr}$ | $F_y/Mg$ |
|-------------|------------|----------|----------|-------|----------|---------------|----------|
|             | [-]        | [mm]     | [mm]     | [mm]  | [MPa]    | [MPa]         | [-]      |
| <b>3</b>    | 1          | 476      | 239      | 3.4   | -3.67    | -16.03        | 0.109    |
| <b>4</b>    | 2          | 393      | 117      | 2.8   | -5.40    | -30.01        | 0.258    |
| <b>5</b>    | 1.5        | 427      | 158      | 3.0   | -4.56    | -23.13        | 0.245    |
| <b>6</b>    | 1          | 476      | 239      | 3.4   | -3.67    | -16.03        | 0.241    |

Table 3. Dimensions of the isolation bearings and superstructure yielding force ( $T_{is} = 3s$ , 8 HDR bearings)

| <i>Case</i> | $\gamma_d$ | $D_{is}$ | $h_{is}$ | $t_r$ | $\sigma$ | $\sigma_{cr}$ | $F_y/Mg$ |
|-------------|------------|----------|----------|-------|----------|---------------|----------|
|             | [-]        | [mm]     | [mm]     | [mm]  | [MPa]    | [MPa]         | [-]      |
| <b>7</b>    | 2          | 543      | 119      | 3.9   | -1.99    | -44.78        | 0.122    |
| <b>8</b>    | 1.5        | 590      | 160      | 4.2   | -1.68    | -23.13        | 0.118    |
| <b>9</b>    | 1          | 659      | 244      | 4.7   | -1.35    | -23.06        | 0.114    |

Table 4. Dimensions of the isolation bearings and superstructure yielding force ( $T_{is} = 5s$ , 8 HDR bearings)

| <i>Case</i> | $\gamma_d$ | $D_{is}$ | $h_{is}$ | $t_r$ | $\sigma$ | $\sigma_{cr}$ | $F_y/Mg$ |
|-------------|------------|----------|----------|-------|----------|---------------|----------|
|             | [-]        | [mm]     | [mm]     | [mm]  | [MPa]    | [MPa]         | [-]      |
| <b>10</b>   | 2          | 383      | 166      | 2.8   | -3.98    | -19.41        | 0.064    |
| <b>11</b>   | 2          | 383      | 166      | 2.8   | -3.98    | -19.41        | 0.122    |
| <b>12</b>   | 2          | 383      | 166      | 2.8   | -3.98    | -19.41        | 0.258    |

#### 4 Results: demand hazard curves

This chapter illustrates the results of the parametric probabilistic analysis carried out on the set of 12 case studies previously presented. The response of the various systems is investigated by observing two main parameters: (i) the maximum superstructure's relative displacement among the

425 x and y directions,  $u_s = \max_t(|u_{sx}(t)|, |u_{sy}(t)|)$ ; (ii) the maximum bearing's shear strain  $\gamma_{is} =$   
 426  $\max_t \sqrt{u_{bx}(t)^2 + u_{by}(t)^2} / h_{is}$ .

427 The outcomes presented in the next subsections are in the form of demand hazard curves. In  
 428 each figure, two horizontal lines are added: a grey dotted line identifying the design hazard  $MAF v_d$   
 429  $= 0.0021$  1/year and a green dashed line representing the target reliability level  $v_{target} = 2 \cdot 10^{-4}$  1/year,  
 430 consistent with Codes [11][12]. To better understand the influence of the parameters varied within  
 431 the analysis, two design parameters (e.g.,  $\Omega/q$  and  $N_{is}$ ) out of three are kept fixed in every chart.

432 First, the case studies with  $T_{is}=3s$  are examined, by discussing the superstructure response  
 433 (Section 4.1) and the isolation system response (Section 4.2). In Section 4.3, the influence of the  
 434 isolation period is addressed and results from systems with  $T_{is}=3s$  and  $T_{is}=5s$  are compared. Finally,  
 435 in Section 4.4, results are commented from the point of view of the seismic reliability: safer and less  
 436 safe cases (sets of design parameters) are highlighted, providing a preliminary quantification of the  
 437 safety factors required to satisfy the target reliability levels.

#### 438 4.1 Superstructure response

439 Fig. 7 illustrates the demand hazard curves of the superstructure relative displacement  $u_s$  for  
 440 the cases  $T_{is}=3s$ . Charts of Fig. 7 (a, b, and c) compare the curves relating to different values of the  
 441 design shear strain  $\gamma_d$  (1, 1.5 and 2 respectively), from case 1 to case 9.

442 In all the figures the initial branches of the curves, representing the elastic range of the  
 443 superstructure response, are overlapped. This suggests that the elastic behaviour of the superstructure  
 444 is not affected by the design shear strain  $\gamma_d$ . In other words, the superstructure response is the same  
 445 despite the isolation stiffness vary in the three cases for shear deformations lower than the design one  
 446 (due to the different behaviour of the rubber, as depicted in Fig. 6). This can be explained considering  
 447 two phenomena: first, high  $T_{is}$  values usually fall within the range of almost constant displacement  
 448 spectrum, and second, the high isolation ratio at the design condition ( $T_{is}/T_s$ ) ensures that almost only

the isolation modal component contributes to the superstructure response [1]. The result is a superstructure response which is proportional to the spectral displacement at the isolation period.

In the case  $\Omega/q=1$ , (Fig. 7 a and c), the superstructure attains the yielding limit at  $v_d$  as expected, because no safety margin is taken on the superstructure capacity (base shear strength) at the design stage; once the superstructure attains the yielding condition, the curves reduce its slope and large increment of displacements are observed in conjunction with small reduction of *MAFs*, leading to a fast increase of the superstructure displacement demand related to the plastic response. This confirms that the ductility demand of isolated structure can be very high if the superstructure exceeds its elastic limit [8]. The same behaviour is recognized in the case  $\Omega/q=2.5$ , but it is postponed (lower *MAF*) due to the larger yielding strength available on the superstructure.

The after-yielding tails of the curves, unlike the elastic branches, have a higher sensitivity to the design shear deformation: curves with higher  $\gamma_d$  values show higher *MAFs* because the hardening of the rubber is attained earlier. The reason is related to the rubber stiffening behaviour that reduces the isolation period, increasing both the base forces and superstructure displacements (which rise faster being the response no longer governed by the spectrum range of constant displacements). Moreover, once the superstructure yields, the  $T_s$  value increases (thus the  $T_{is}/T_s$  ratio reduces) and the hypothesis that only the isolation period contributes to the superstructure response falls.

Finally, there are no substantial differences between the case of 15 *HDR* bearings (Fig. 7 a) and the case with 8 *HDR* bearings (Fig. 7 c) because, as also shown in Fig. 6 a, vertical pressures lower than 6MPa (see Table 2 to Table 4) only slightly affect the cyclic response of the bearings (i.e. P- $\Delta$  effects are not significant given the design limits).

To better highlight the influence of  $\Omega/q$  on  $u_s$ , the curves of Fig. 7 (a and b) are rearranged in Fig. 7 (d, e and f), comparing the cases with  $\Omega/q = 1$  and the cases with  $\Omega/q = 2.5$ , namely cases from 1 to 6 (the other two parameters,  $\gamma_d$  and  $N_{is}$ , are kept fixed in each chart). Again, the two curves are overlapped in their first branch until the case  $\Omega/q = 1$  yields. After this point the deformation demand

strongly increase for  $\Omega/q = 1$  while the curve of  $\Omega/q = 2.5$  continues with the previous slope. To complete this results discussion, an average inter storey drift of 2% is assumed as the threshold value beyond which the superstructure stability could be strongly compromised, i.e., the collapse performance level according to [66]; this threshold is reported in the charts of Fig. 7 (vertical dotted line) in terms of equivalent relative displacement, 0.16m. It can be noted that only the case  $\Omega/q = 2.5$  fulfils the reliability target (cases 5 and 6), i.e., the collapse condition is attained with a *MAF* lower than  $v_{target} = 2 \cdot 10^{-4}$  1/year (the case 4 is not in compliance but close to it). This concept will be further discussed in the Subsection 4.4 on safety factors.

## 4.2 Isolation system response

Regarding the rubber shear deformation  $\gamma_{is}$  of the bearings, the demand hazard curves for  $T_{is}=3s$  are reported in Fig. 8 (a, b, and c) comparing results obtained by adopting different design shear strains.

Unlike the curves previously showed (related to the superstructure), the slope of shear deformation curves decreases monotonically with the *MAF*, suggesting a controlled increase of the bearing response due to the rubber stiffening behaviour (see Fig. 6 a), that limits the growth of the shear deformation with increasing seismic actions.

The response values at  $v_d$  are in all the cases very close to the design values  $\gamma_d$ , thus confirming the effectiveness of the design procedure described in section 3.3.

To better highlight the influence of  $\Omega/q$  on  $\gamma_{is}$ , the curves are rearranged in Fig. 8 (d, e, and f), comparing the cases with  $\Omega/q = 1$  and the cases with  $\Omega/q = 2.5$  ( $\gamma_d$  and  $N_{is}$  are kept fixed in each chart). In all the cases, the curves are almost overlapped in the whole range of *MAFs*, with only slight discrepancies observed for shear strain values higher than the design ones: this confirms the negligible influence of  $\Omega/q$  on the isolation response, as also proven in [8]. This can be explained considering that the isolated structure maintains its initial frequency and the predominant isolation mode response

498 even after the superstructure yielding, i.e., the elongation of the superstructure period is not related to  
499 an elongation of the isolated period.

500 To complete this results discussion, thresholds related to the *HDR* bearing capacity are  
501 reported in the charts of Fig. 8 d-f (vertical dotted line). Based on the available technical literature  
502 [67] a value of 350% of shear strain has been assume as collapse condition of bearings. It can be  
503 noted that only the cases with  $\gamma_d=1$  fulfils the reliability target with a collapse *MAF* lower than the  
504 reliability target, whereas the cases with  $\gamma_d=1.5$  and  $\gamma_d=2$  are not in compliance with it (even though  
505 the case  $\gamma_d=1.5$  is very close to it). This concept will be further discussed in the Subsection 4.4 from  
506 the point of view of safety factors.

507

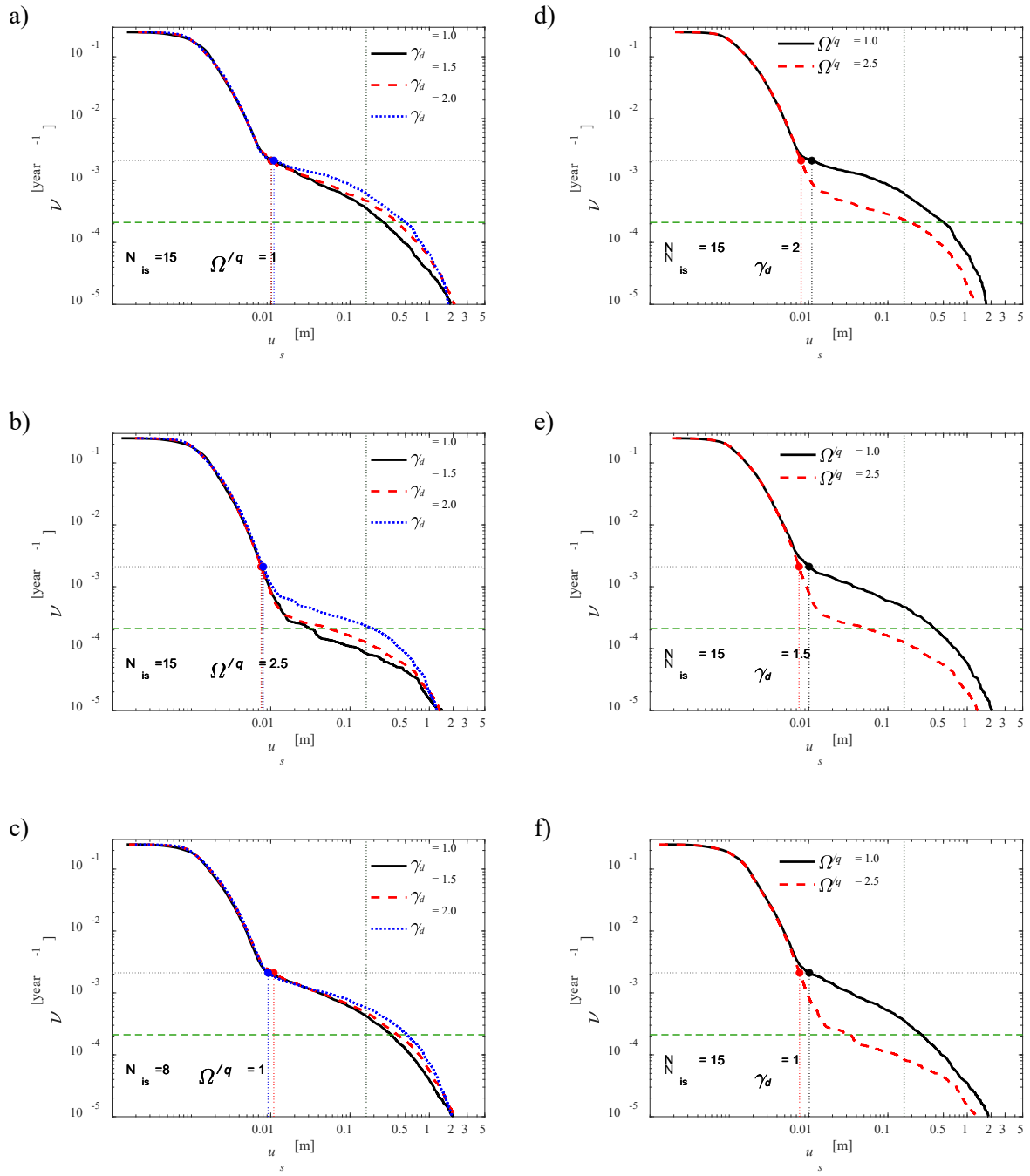


Fig. 7. Demand hazard curves of  $u_s$  for  $T_{is}=3s$  and varying  $\gamma_d$  (a,b,c) or varying  $\Omega'/q$  ratios (d, e, f).

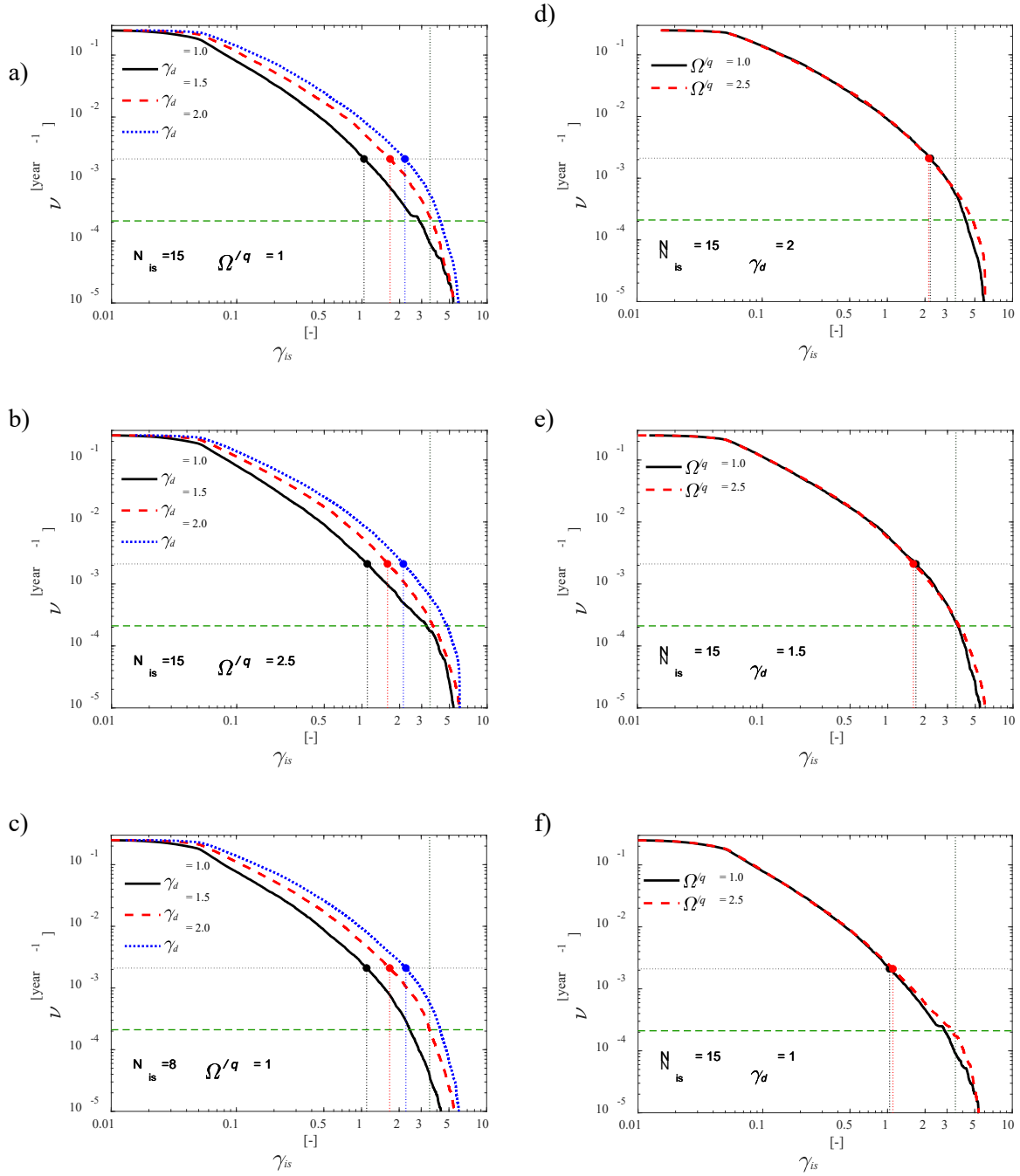


Fig. 8. Demand hazard curves of  $\gamma_{is}$  for  $T_{is}=3s$  and varying  $\gamma_d$  (a,b,c) or varying  $\Omega/q$  ratios (d, e, f).

### 4.3 Influence of the isolation period

The isolation period is another free parameter which can be defined at the design stage. A direct comparison between the case  $T_{is}=5s$  with the previous  $T_{is}=3s$  is made in Fig. 9, for the design shear strain  $\gamma_d=2$ . Regarding the superstructure (Fig. 9 a, c), the first branch of the curves shows a lower displacement demand in the elastic range. This is expected as the cases compared share the

516 same superstructure fixed base period and consequently the same stiffness, but for  $T_{is}=5s$  (the red  
 517 dashed curve) the seismic input filtered by the isolation is lower [1]. On the contrary, after the yielding  
 518 of the superstructure, the  $T_{is}=5s$  curve crosses the case  $T_{is}=3s$ , leading to higher displacement demands  
 519 in the plastic range, confirming that the ductility demand increases with the isolation period [8].  
 520 Conversely, the isolation shear deformation demand does not vary sensibly from the case  $T_{is}=3s$  to  
 521 the case  $T_{is}=5s$  (Fig. 9 b, d).

522 However, it should be noted that the comparison presented here between  $T_{is}=3s$  and  $T_{is}=5s$ ,  
 523 although coherent in terms of design procedure, is made on two superstructures with strong  
 524 differences in terms of actual strength; indeed, the superstructures corresponding to  $T_{is}=5s$  (cases 10  
 525 and 11) are characterised by a normalised yielding force (see  $F_y/Mg$  in Table 2 and Table 4) almost  
 526 half the values of the corresponding cases (4 and 5) with  $T_{is}=3s$ . Consequently, the superstructure  
 527 design for isolation systems with  $T_{is}=5s$  is mostly expected to be governed by minimum code  
 528 requirements or non-seismic actions (gravity and wind loads); moreover, in these cases, the  
 529 contribution provided by non-structural elements to the actual strength of the system is relatively  
 530 higher than the cases with  $T_{is}=3s$ . The case study 12 ( $T_{is}=5s$  and  $\Omega/q=4.75$ , blue dotted line) shown  
 531 in Fig. 9 c and characterised by the same superstructure of case 4 (the higher  $\Omega/q$  ratio stems from  
 532 the lower design seismic demand), is added to represent this specific condition. This last case shows  
 533 a significant reduction of  $u_s$  with respect to both the case 11 ( $T_{is}=5s$  and  $\Omega/q=2.5$  red dashed line) and  
 534 the case 4 ( $T_{is}=3s$  and  $\Omega/q=2.5$  black solid line), hence confirming that, with the same superstructure,  
 535 an enhancement of the structural performance can be achieved increasing the isolation period.  
 536 Moreover, since the superstructures are the same but the total volume of *HDR* bearings in case 12 is  
 537 lower than case 4, also a cost reduction could be pursued by increasing the isolation period  $T_{is}$ .

538 Regarding  $\gamma_{is}$  (Fig. 9 d), no variations are observed, confirming that nor the isolation period  
 539 neither the overstrength ratio affect the isolation response.

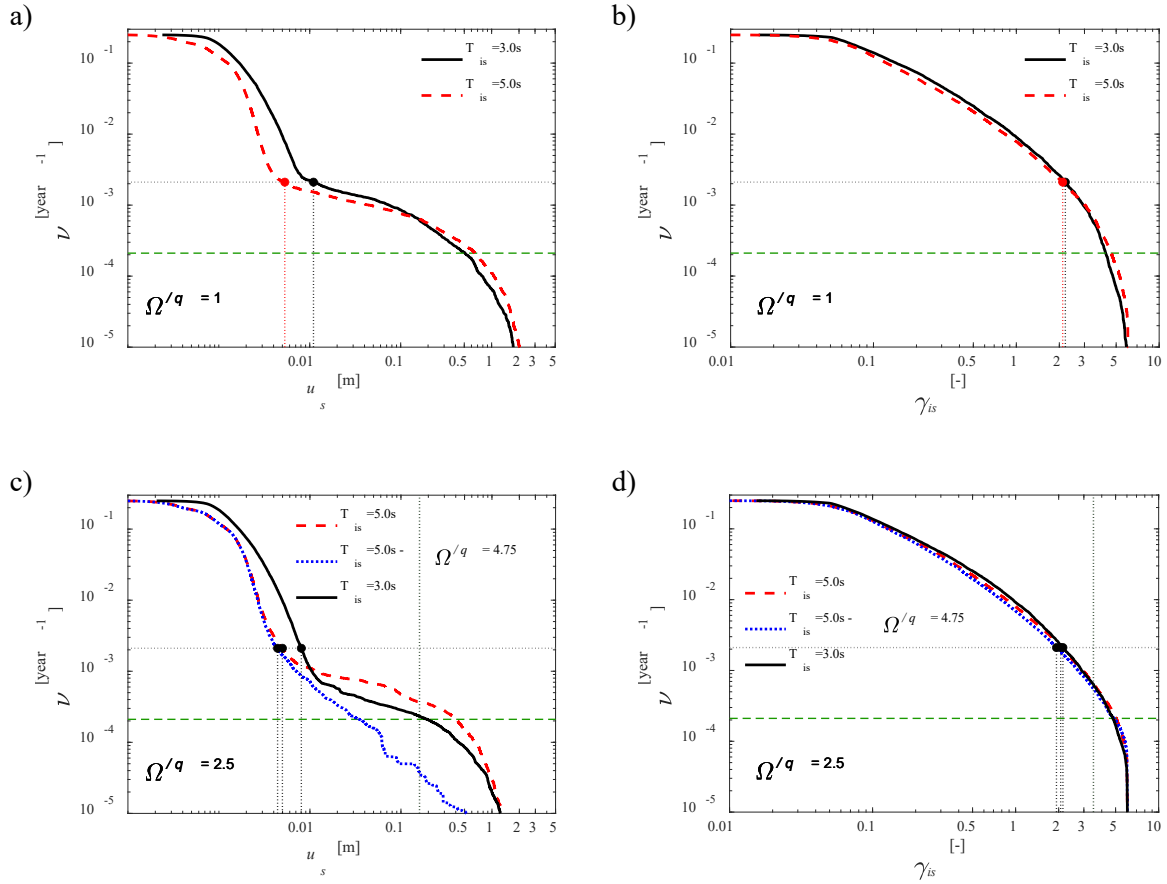


Fig. 9. Demand hazard curves of  $u_s$  (a, c) and  $\gamma_{is}$  (b, d) for  $T_{is}=5$  s and  $T_{is}=3$  s.  $\gamma_d=2$ .

#### 4.4 Safety Factors

Table 5 summarises the main results obtained from the probabilistic analysis. Indeed, the following information are collected for each response parameter ( $u_s$  and  $\gamma_{is}$ ) and for every case study examined (from case 1 to case 12): case study number; principal design parameters; target design response values  $u_d$  and  $\gamma_d$ ; achieved response values at the reference design *MAF* level  $\nu_d$  ( $u_s(\nu_d)$  and  $\gamma_{is}(\nu_d)$ ) and at the target reliability level  $\nu_{target}$  ( $u_s(\nu_{target})$  and  $\gamma_{is}(\nu_{target})$ ); estimates of the required reliability factors  $R_Y$  (explained hereafter) for the isolation shear strain.

The  $R_Y$ -factor is defined as the ratio between the response parameter  $\gamma_{is}$  evaluated at a given *MAF* level,  $\nu$ , and the corresponding design value  $\gamma_d$ :

$$\mathbf{R}_\gamma(v, \gamma_{is}) = \frac{\gamma_{is}(v)}{\gamma_d} \quad (5)$$

550 It is worth to note that the  $\mathbf{R}_\gamma$ -factor can be interpreted as reliability factor, which applied to  
 551 the design value of the parameter provides a demand value consistent to the specified target *MAF*  
 552 ( $v_{target} = 2.1 \cdot 10^{-4}$  1/year in this study). On the other hand,  $\mathbf{R}_\gamma$  can be used as reliability factor to  
 553 define the minimum isolator's shear capacity to be tested (given both  $v_d$  and a  $v_{target}$ ).

554 Examining the values of Table 5 some important comments can be made. For what concerns  
 555 the isolation system, it is possible to observe how the response values estimated at the reference *MAF*  
 556  $v_d$  ( $2.1 \cdot 10^{-3}$  1/year),  $\gamma_{is}(v_d)$ , are very close to the target design ones  $\gamma_d$  (1, 1.5 and 2), due to the  
 557 advanced design procedure adopted. Furthermore, by looking at the shear strain values  $\gamma_{is}(v_{target})$ ,  
 558 one can observe that the demand values are significantly larger than the minimum shear strain  
 559 capacity values required by the European code on anti-seismic devices [4]. In fact, the code  
 560 prescription for rubber bearings is to carry out a ramp test up to a displacement equal to the design  
 561 displacement  $\gamma_d$  amplified by two factors: a magnification factor of 1.2 aimed at increasing the  
 562 reliability of the structural system and a further partial factor of 1.15. Applied to the current cases  
 563 (i.e.,  $\gamma_d = 1, 1.5, 2$ ), the Code requirements would lead to values of maximum shear deformation  
 564 capacity equal to 1.38, 2.07 and 2.76 respectively, which are all significantly lower than the related  
 565 demand values  $\gamma_{is}(v_{target})$  in Table 5. This result suggests that larger amplification factors should be  
 566 applied to  $\gamma_d$  to attain reliability levels consistent with the Codes requirements. Moreover,  $\mathbf{R}_\gamma$  factors  
 567 decrease as the design shear strain increases due to the more pronounced hardening behaviour of the  
 568 isolation bearings, leading to the conclusion that different reliability factors should be tailored for  
 569 different behaviours of the isolation devices. A similar result was achieved in another recent work  
 570 [3], where, however, a different design *MAF* ( $v_d$ ) was used (coherent with American standards [5]  
 571 prescriptions), thus a direct comparison between values stemming from the present work and the one  
 572 in [3] cannot be made.

573 Table 5. Seismic demand values attained at design ( $v_d = 2.1 \cdot 10^{-3}$  1/year) and target ( $v_{target} = 2.1 \cdot 10^{-4}$  1/year) *MAF* levels,  
 574 and reliability factor  $R_\gamma$

| <i>Case</i> | <i>Design features</i>                               | $\gamma_d$<br>[-] | $u_d$<br>[mm] | $\gamma_{is}(v_d)$<br>[-] | $\gamma_{is}(v_{target})$<br>[-] | $R_\gamma$<br>[-] | $u_s(v_d)$<br>[mm] | $u_s(v_{target})$<br>[mm] |
|-------------|------------------------------------------------------|-------------------|---------------|---------------------------|----------------------------------|-------------------|--------------------|---------------------------|
| 1           | $T_{is} = 3.0s$                                      | 2                 | 7.1           | 2.20                      | 4.26                             | <b>2.13</b>       | 10.99              | 529.10                    |
| 2           | $N_{is}=15$                                          | 1.5               | 6.9           | 1.68                      | 3.67                             | <b>2.45</b>       | 10.14              | 409.12                    |
| 3           | $\Omega/q = 1.0$                                     | 1                 | 6.8           | 1.04                      | 2.92                             | <b>2.92</b>       | 10.21              | 282.86                    |
| 4           | $T_{is} = 3.0s$                                      | 2                 | 6.4           | 2.14                      | 4.85                             | <b>2.42</b>       | 8.05               | 209.95                    |
| 5           | $N_{is}=15$                                          | 1.5               | 6.1           | 1.60                      | 3.77                             | <b>2.51</b>       | 7.58               | 60.68                     |
| 6           | $\Omega/q = 2.5$                                     | 1                 | 6.0           | 1.10                      | 3.32                             | <b>3.32</b>       | 7.70               | 35.06                     |
| 7           | $T_{is} = 3.0s$                                      | 2                 | 7.6           | 2.26                      | 4.28                             | <b>2.14</b>       | 9.26               | 542.63                    |
| 8           | $N_{is} = 8$                                         | 1.5               | 7.3           | 1.68                      | 3.53                             | <b>2.35</b>       | 10.93              | 416.05                    |
| 9           | $\Omega/q = 1.0$                                     | 1                 | 7.1           | 1.10                      | 2.47                             | <b>2.47</b>       | 9.45               | 340.81                    |
| 10          | $T_{is} = 5.0s$<br>$N_{is} = 8$<br>$\Omega/q = 1.0$  | 2                 | 4.0           | 2.11                      | 4.67                             | <b>2.34</b>       | 5.31               | 676.70                    |
| 11          | $T_{is} = 5.0s$<br>$N_{is} = 8$<br>$\Omega/q = 2.5$  | 2                 | 3.2           | 2.06                      | 5.10                             | <b>2.55</b>       | 4.98               | 416.37                    |
| 12          | $T_{is} = 5.0s$<br>$N_{is} = 8$<br>$\Omega/q = 4.75$ | 2                 | 3.2           | 1.93                      | 4.98                             | <b>2.49</b>       | 4.41               | 37.87                     |

For what concerns the superstructure, most of the comments made for the isolation response still apply. The response values at  $v_{target}$ ,  $u_s(v_{target})$ , are always higher than the threshold values of 160mm previously introduced (i.e., corresponding to 2% inter-storey drift and denoting the collapse conditions according to [66]), except for three cases out of 12 (namely cases 5, 6 and 12), confirming that the current seismic code prescriptions may not ensure the achievement of the reliability target. As expected, the case 12 (same superstructure strength of case 4 but isolation at  $T_{is} = 5.0s$ ) has the best performance in terms of  $u_s$ .

Finally, it is important to remember that the systems' probabilistic response (thus the seismic risk) associated to the design procedures is hazard- (and thus site-) dependent [21][23]. Consequently, the results presented in this paper (concerning both the isolation system and the superstructure) could vary in absolute terms by changing the hazard, although the observed trends would remain of general validity.

## 5 Conclusions

In this study the seismic reliability of structural systems isolated with *HDR* bearings has been investigated, using a robust probabilistic framework combined with advanced numerical models for the isolation system, accounting for both the hardening and the P- $\Delta$  effect. A parametric study has been carried out by varying the design parameters of both the isolation system and the superstructure to assess their influence on the final seismic reliability of the isolated structure. For each varied design condition, the demand hazard curves of the monitored response parameters, related to both superstructure (relative displacement  $u_s$ ) and isolation system (rubber shear strain  $\gamma_{is}$ ), have been computed. The following conclusions can be drawn from the obtained results.

- Among the design parameters investigated, design shear strain of the rubber bearings ( $\gamma_d$ ), superstructure overstrength ratio ( $\Omega/q$ ) and isolation period ( $T_{is}$ ) strongly influence the final reliability of the system in terms of superstructure relative displacement  $u_s$ , also showing a strong

interdependency, especially between  $\Omega/q$  and  $T_{is}$ . The design shear strain  $\gamma_d$  is the only relevant parameter influencing the response of the isolation system (i.e., affecting the  $\gamma_{is}$  hazard curve). The effects of the percentage of flat sliders (i.e., bearing shape factors) is instead negligible on both  $u_s$  and  $\gamma_{is}$  as the P- $\Delta$  effect is not significant.

- For what concerns the superstructure response, increments of the overstrength ratio  $\Omega/q$  notably affect the post-elastic branches of the  $u_s$  hazard curves, where a strong reduction of the *MAF* of exceedance can be observed. A value  $\Omega/q = 2.5$  is needed for the case  $T_{is} = 3.0s$  to achieve a *MAF* at the collapse threshold (average inter storey drift of 2%) lower than the target reliability. Moreover, a further slight reduction of the superstructure failure probability is observed for lower design shear strains  $\gamma_d$  due to the smaller stiffening behaviour of the rubber during the plastic stage of the superstructure. The value of  $\Omega/q$  should be further increased (beyond 2.5) in the case of  $T_{is} = 5.0s$  to have a *MAF* lower than the target reliability level.
- Regarding the isolation system, the seismic demand  $\gamma_{is}$  increases with the design shear strain  $\gamma_d$ . If a bearing shear deformation capacity of 350% is assumed, only the cases with  $\gamma_d = 1$  show a *MAF* of collapse lower than the reliability target. All the other design parameters have a negligible influence on the isolation response.
- Increasing the isolation period up to  $T_{is} = 5.0s$ , and keeping fixed all the other design parameters, both the isolation bearing diameter and the superstructure strength reduces significantly, while the total rubber thickness slightly increases. The  $u_s$  hazard curve of  $T_{is} = 5.0s$  shows a lower displacement demand in the elastic range with respect to the case  $T_{is} = 3.0s$ ; conversely, once yielded, the  $T_{is}=5s$  hazard curve crosses the  $T_{is}=3s$  one, leading to a higher displacement demand in the plastic range, which confirms that the ductility demand increases with the isolation period for a given  $\Omega/q$  value.
- The case  $T_{is} = 5.0s$  and  $\Omega/q = 4.75$  represents the most reliable solution in terms of  $u_s$ . It is worth to note that this solution is not expensive as might appear, being the superstructure the same

considered in the case  $T_{is} = 3.0\text{s}$  and  $\Omega/q = 2.5$ . Considering this result, the isolation period represents a design parameter that should be maximised to obtain the twofold aim of higher structural performance and total cost reduction.

- The European code on seismic devices (EN 15129) prescribes to carry out a shear capacity test at a shear strain equal to  $1.2 \cdot 1.15$  times the design value  $\gamma_d$ ; according to the outcomes of the present analysis, the shear strain values stemming from the application of such Code requirement do not allow achieving satisfactory reliability levels (i.e., the corresponding *MAFs* are much higher than the reliability target).
- Values of the reliability factors for  $\gamma_d$  are provided, which can be also interpreted as amplification coefficients of the design values to achieve the required reliability level and thus to define the minimum shear capacity to be tested;
- The obtained reliability factors depend on the specific design values chosen for the *HDR* bearing shear strain, showing that the current single reliability factor does not ensure a satisfactory reliability level and that this approach is not actually easy to use in the current codes;

It is important to remember that all these results are strongly related to the hazard selected for the analyses, thus wider seismic reliability assessment procedures considering different sites should be adopted and/or risk-based design approaches should be investigated. Moreover, for a revision/improvement of the current Codes the results should also be confirmed by considering more complex structural systems (also explicitly including the presence of non-structural components).

Finally, other sources of uncertainties (such those related to the superstructure and bearings properties variation within tolerance limits imposed by the codes) should be accounted for both in the design procedure and in the probabilistic framework in order to assess the effectiveness of the approaches suggested by the codes (i.e. upper/lower bound analysis). The uncertainty about the shear capacity of bearings could be also introduced, if sufficient experimental data are available.

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