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INFLUENCE OF DESIGN PARAMETERS ON THE SEISMIC RELIABILITY OF BASE ISOLATED SYSTEMS

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Abstract

Seismic isolation is considered an effective solution to protect buildings and reduce seismic losses. However, the reliability level achieved by code-conforming base-isolated structures can be not uniform due to the different behavior of the most used isolation bearings (elastomeric or sliding bearings) as well as the different design choices that strongly influence the seismic reliability of base-isolate structures. In this paper High Damping Rubber Bearings (HDRBs) are considered and a parametric analysis is performed to assess the overall reliability of base-isolated buildings designed according to Eurocodes and Italian seismic codes. In particular, the case of ordinary residential buildings is considered, and the following parameters have been considered and varied, within the range of most common values: the bearings design shear deformation and the superstructure design over-strength ratio. Probabilistic analyses are performed using a robust direct simulation approach (Subset Simulation) and a bidirectional stochastic ground motion model. For each case analyzed, results are provided in terms of demand hazard curves of the most relevant response parameters characterizing the isolation system and the superstructure.

Keywords: Seismic isolation, rubber bearings, failure, seismic risk, reliability, probabilistic analysis, Subset Simulation, Intensity Measure

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1 INTRODUCTION

Seismic base isolation is an efficient technique for passive seismic protection of buildings and bridges [1,2], able to drastically reduce damage of structural and non-structural elements even in the case of earthquakes of medium-high intensities, notably shortening the post-event recovery time. Most of the modern codes adopted by earthquake prone countries include prescriptions about the design of base-isolated structures, contributing to the increment of worldwide applications. However, systems designed in accordance with these standards are based on conventional values of the design seismic intensity that can be different form a seismic code to another. Additionally, different choices about design parameters can be made leading to very different performances under extreme events. In other terms, the reliability, measured in terms of the mean annual frequency (MAF) of failure, can result significantly inhomogeneous case by case. For example, according to European standards the target design hazard level for base-isolated buildings without public or strategic structures (importance class 2) is characterized by a MAF of exceedance ν_d = 0.0021 1/year, i.e., a return period equal to T_R=475 years or a probability of exceedance of 10% in 50 years typical of the Ultimate Limit State (ULS). Only for buildings classified in importance classes 3 or 4 (depending on the consequences of collapse for human life, on their importance for public safety and civil protection in the immediate post-earthquake period or on the social and economic consequences of collapse) an important factor (γ_I) larger than one is considered to amplify the reference seismic action. [3]. According to this code the superstructure can be design by adopting a behaviour factor ranging from 1 to 1.5, while design indications about the isolation system are given in the European standard for anti-seismic devices EN15129 [4] by accounting for a reliability factor γ_X (equal to 1.2 for buildings and 1.5 for bridges) to ensure a larger reliability to the isolation system, due to its essential role in the seismic behaviour. Moreover, with reference to High Damping Rubber bearings (HDRBs) a maximum design shear strain up to 250% is allowed.

Similarly, the Italian seismic code [5] allows to design the superstructure by adopting the same approach (assuming a seismic action at the ULS with a probability of exceedance of 10% in the reference period and a behaviour factor ranging from 1 to 1.5), but by assuming minimum requirements of the low ductility class about steel reinforcements. Differently, the isolation system is designed by assuming a seismic action with a lower probability of exceedance (5% in the reference period typical of the Collapse Limit State, CLS) but without using a reliability factor and by considering a lower maximum allowed shear design strain equal to 200%. For ordinary base-isolated buildings (reference period of 50 years) the ULS corresponding to return a period T_R=475 years, while the CLS to a return period of T_R=978 years.

From the side of displacement capacity, the current version of EN15129 [4] prescribes a type test, called "lateral capacity test", carried out up to a shear deformation equal to the design value (at ULS) amplified by the reliability factor γ_X and by a further small amplification factor of 1.15, where, as specified by the Italian national code [5], the design displacement at the ULS amplified by the reliability factor γ_X corresponds to the design displacement at the CLS in the national context. However, in both the cases, the tested displacement is far from the actual collapse condition of the isolation bearing,. Therefore, some code conforming base-isolated structures may show theoretical reliability levels below the target one [6,7] and not uniform, as demonstrated in [8]. The American seismic code [9] also presents similar limits, as recently highlighted in [10] and demonstrated in detail in [10] and [11] for friction isolators, even though the input level considered is different, i.e., the Maximum Considered earthquake (MCE).

In this paper, a systematic study on the role of design parameters choice on the overall reliability is carried out to evaluate the potential variation of the failure probability of base-isolated buildings designed according to Eurocodes [3][4] and NTC 2018[5] respect to required target values. In particular, the case of ordinary residential buildings is considered, and the following parameters have been considered and varied, within the range of most common values: the bearings design shear deformation and the superstructure 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 the aforesaid parameters and for each of them a full probabilistic analysis is performed to evaluate the collapse risk. To reduce the computational effort, 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, also accounting for the coupling between vertical and horizontal response in large displacements [12-14]. 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 of the isolation system (the maximum shear deformation of the isolation system) and the superstructure (the maximum relative displacement).

2 PROBABILISTIC FRAMEWORK

The probabilistic framework used to perform seismic reliability analyses on base-isolated systems is described in detail in [15]. It consists of an efficient probabilistic tool, the Subset Simulation [16], used to estimate accurate demand hazard curves of the main engineering demand parameters (EDPs) up to the target reliability level commonly required by the Codes for structural systems equal to $\nu_{target} = 2 \cdot 10^{-4}$ 1/year [6-7]. Moreover, a stochastic ground motion model for seismic hazard characterization and bidirectional seismic samples generation is adopted [17-19]. This allows generating ground motion samples conditional to the features of a given seismic scenario, specified by two main random variables, the moment magnitude M, and the epicentral distance R. The seismic scenario and the related parameters are assumed as in [15].

Although a direct simulation approach is used in this study to perform seismic reliability analyses, an Intensity Measure (*IM*) has been introduced to carry out the design of all the considered case studies: according to the current concept of partial safety factors, the *IM* is used to quantify the seismic intensity at the design rate of occurrence prescribed by the codes. Many different *IM* can be chosen, but an efficiency evaluation specifically made for isolation systems [24] suggests that the best choice is the $S_{aRotD100}(T,\xi)$ [24-25], combined with the recently proposed strategy of averaging the spectral values over a period range [26-27], rather than computing them at a given single T value. It is worth noting that the use of such average *IM* over a range of periods better allows to manage the variability of the *HDR* bearings dynamic response with the strain amplitude and repeated cycles.

The IM hazard curve obtained for the scenario defined above is depicted in Figure 1, where the curve averaged on 10 independent runs is assumed as IM curve (red solid lines identify the average, grey lighter curves the single runs). The horizontal black dotted line identifies the design hazard level considered (a MAF of exceedance v_d = 0.0021 1/year or a probability of exceedance of 10% in 50 years) corresponding to the design intensity im_d = 0.173g (where g is the gravity acceleration), as highlighted by the yellow dots in the chart. Finally, the blue circle markers added to the plot show both intensities and MAFs of the ground motion samples used to design the isolation systems (each circle corresponds to a pair of ground motion components, being the chosen *IM* direction-independent). A set of100 accelerograms is generated to design the base-isolated systems, so that to have IM_s as close as possible to the target *IM* value (im_d). From the same figure values of im_d obtained by assuming a different design MAF can be evaluated, i.e. im_d of the CLS (T_R=978 year corresponding to ν_d = 0.001 1/year) or im_d of the MCE (T_R=2475 year corresponding to ν_d = 0.0004 1/year)

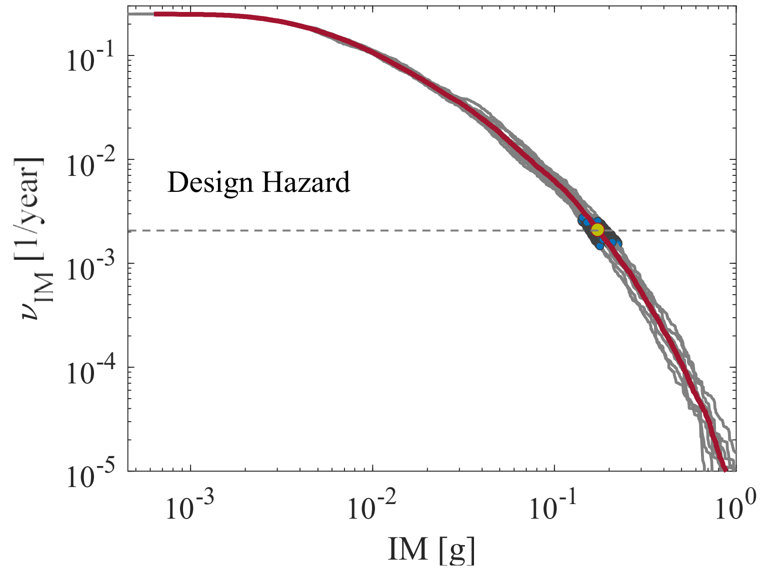

Figure 1: Input hazard curve

3 PARAMETRIC ANALYSIS

3.1 Case study and design parameters

The selected case study (Figure 2) is a four-storey reinforced concrete (r.c.) building (total height 12m) with 1 kNs^2/m^3 distributed mass for each floor (5 floors including the base floor above the isolation system), 2 x 4 spans of 5m each, and 15 columns, for a total mass of 1000 kNs^2/m. The isolation period is Tis=3s, which is a current common value for new base-isolated residential buildings. Regarding the design shear deformation, three values are considered, i.e., γ_d =1, γ_d =1.5 and γ_d =2, which are all lower than the limit of 2 imposed by the Italian national code [5] and 2.5 imposed by the European code on anti-seismic devices [4] and around common values (1.5) currently used by designer in European countries.

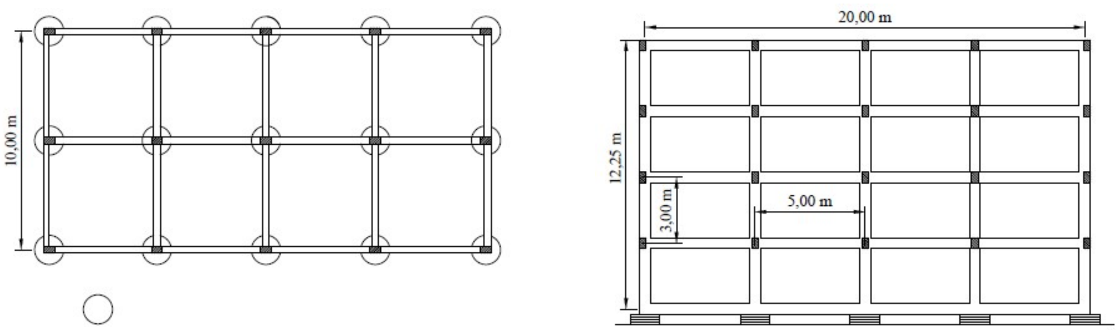

Figure 2: Plan view of the bearing configuration and section view of the case study

Another design parameter considered in the parametric study is the superstructure strength, which depends on design prescriptions and safety factors. At this regard, for isolated structures, the Eurocode [3] allows designing with a reduced value of the seismic lateral force by adopting a behavior factor q ranging from 1 to 1.5, similarly, ASCE 7 [9] prescribes q values in the range [1.0, 2.0]. However, the ratio between the design base shear and the actual yielding force of the system, generally defined over-strength factor, Ω, may be notably higher than the behavior factor q, especially for isolated structures, up to a value of 2.5 [28]. This is due to safety factors applied to material strengths and the minimum structure redundancy as well as other superstructure strength sources stemming from non-structural elements (e.g., strong infill panels) and non-seismic actions (gravity and wind loads). It is therefore useful to define an overstrength ratio Ω/q which directly expresses the ratio between the actual strength capacity and the seismic demand. Considering the limit values for both q and Ω, the two limit cases of Ω/q=1.0 and Ω/q=2.5 have been analyzed in this work, according to q-Ω pairs equal to 1.5-1.5 and 1.0-2.5 respectively. The fixed-base fundamental period of the superstructure is set as constant parameter, equal to Ts = 0.5s regardless of the overstrength ratio Ω/q.

Case	T_{is} [s]	γ_d [-]	N_{is} [-]	Ω/q [-]
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

Table 1: Design parameters of the considered case studies

3.2 Numerical model

For each case study a numerical model is developed. With the aim of reducing as much as possible the computational effort of probabilistic a of the two-degree of freedom (2-DOF) model originally proposed by Kelly [1] has been adopted. According to this model, the dynamic response of a multi-degree of freedom model (M-DOF) can be efficiently assessed using an equivalent 2-mass model able to account for the first two modal contributions of the base-isolated building. In this study, this concept has been extended to a bidirectional seismic input (6 degrees of freedom), considering the complex nonlinear behavior of the rubber.

More in detail, the model consists of two masses, m_b and m_s, related to the mass of the base slab and the deformable super-structure respectively. In particular, the mass 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). In this paper, 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, i.e., m_s =0.6M and m_b=0.4M.

Regarding the mechanical behavior of the isolation bearings, the Kikuchi bearing element [29] available in the Opensees software [30] has been used. It is a fully coupled three-dimensional model able to capture the buckling and post-buckling behavior, i.e., the interaction between the coupled bi-dimensional horizontal behavior and the axial force. Figure 6 shows the cyclic behavior of one of the bearings used in the design cases subjected to horizontal and vertical loads under the hypothesis of no shear failure. The elastomeric compound used s the rubber X0.4S [31] Since the Kikuchi Bearing Element is a geometry-influenced model, i.e., describes the response of a single bearing and it explicitly depends on the real diameter and height of the isolator, some assumptions have introduced to describe the response of the whole base-isolation system by a single element: the rubber bearings have all the same geometry, they support almost the same amount of vertical load and the base slab of the structure behaves as a diaphragm constraint. With these assumptions the model provides a satisfactory approximation of the real behaviour of base-isolated building, except for the overturning effect leading to a variable axial load during the seismic event. Regarding flat sliders,where present, the assumption of negligible friction and an adeguate displacement capacity is assumed, thus their contribution is neglected in the collapse risk assesment.

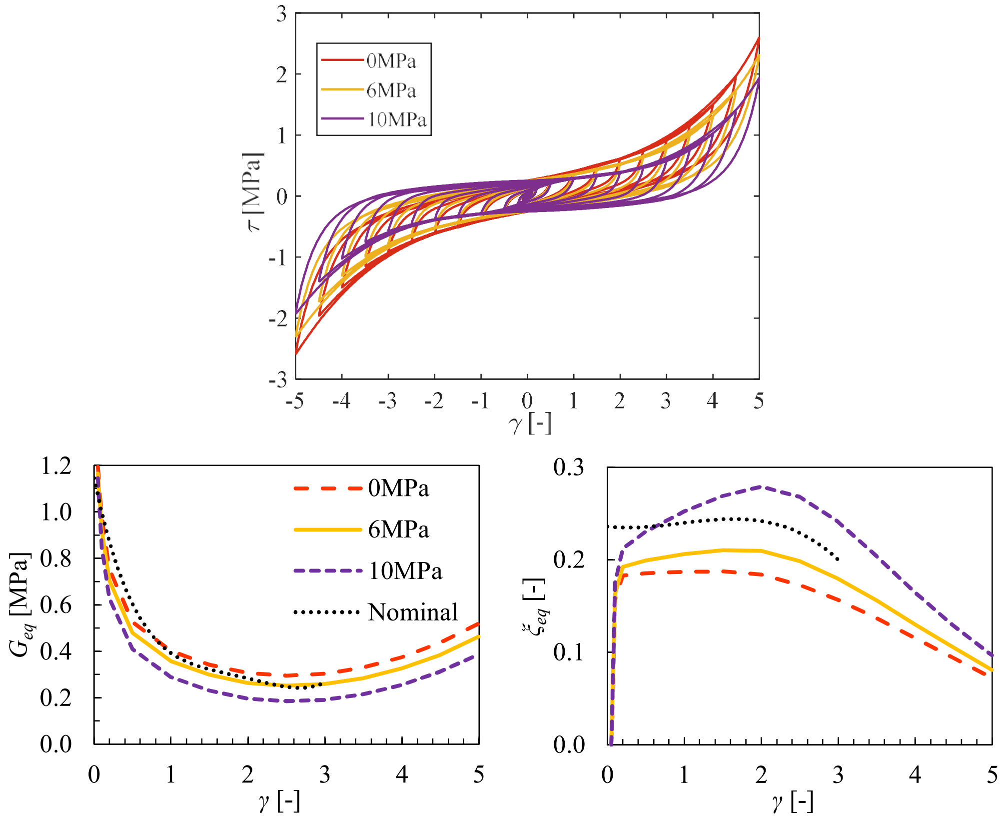

Figure 3: Behavior of the HDR bearing 427/158/3 (D_{is}=427mm, h_{is}=158mm) and equivalent linear parameters (nominal and numerical) of the adopted rubber at different pressures.

Finally the superstructure has been modelled by two uncoupled elastoplastic springs with stiffness k_s and yielding force F_y, describing the behaviour of the superstructure frame along the two main horizontal directions (since r.c. frames and infill panelsare usually aligned along the x direction and y direction with reduced interaction between them). Finally, only tangent-stiffness proportional damping is provided to the superstructure with damping rate equal to 2%, typical value of base-isolated reinforced concrete structures.

3.3 Design of the base isolated systems

The total rubber thickness and the diameter of the isolation bearing (h_{is} and D_{is} respectively) as well as the superstructure yielding force (F_y) have been determined by iteratively performing nonlinear time history analysis on each case study with the set of 100 accelerograms consistent with the target design hazard level, as illustrated in Figure 1 until attaining an average maximum shear strain value equal to the design one γ_d, and a superstructure average maximum displacement coherent with the overstrength ratio. This procedure, despite cumbersome, has been adopted because design procedures based on simplified linear approaches would lead to a seismic response at the design condition different from the assumed one, due to the record-to record variability of the strongly nonlinear response of HDR bearings. Results of the design are summarized in Table 2, where the thickness of the single rubber layer t_r and the compression stress σ are also reported as well as the normalized yielding force values F_y / Mg of the superstructure.

Case	γ_d [-]	D_{is} [mm]	h_{is} [mm]	t_r [mm]	σ [MPa]	σ_{cr} [MPa]	F_y/Mg [-]
1	2	393	117	2.8	-5.40	-30.01	0.114
2	1.5	427	158	3.0	-4.56	-23.13	0.111
3	1	476	239	3.4	-3.67	-16.03	0.109
4	2	393	117	2.8	-5.40	-30.01	0.258
5	1.5	427	158	3.0	-4.56	-23.13	0.245
6	1	476	239	3.4	-3.67	-16.03	0.241

Table 2: Dimensions of the isolation bearings and superstructure yielding force.

4 PROBABILISTIC ANALYSIS RESULTS

Results of the parametric probabilistic analysis carried out on the set of case studies previously presented are summarized in this section in the form of demand hazard curves of the two main parameters: the maximum bearing's shear strain (γ_{is}) and the maximum superstructure's relative displacement among the x and y directions (u_s). In each figure, two horizontal lines are added: a grey dotted line identifying the design hazard MAF ν_d = 0.0021 1/year and a green dashed line representing the target reliability level $\nu_{target} = 2 \cdot 10^{-4}$ 1/year, consistent with Codes. Two further horizontal lines (the orange and the magenta ones) represent the MAF of the CLS of the Italian NTC2018[5] code and the MCE of the American ASCE-7 [9].

The demand hazard curves of the rubber shear deformation γ_{is} of the bearings are reported in Figure 4 comparing results obtained by adopting different design shear strains. In general, the slope of shear deformation curves follows the hazard trend, suggesting a controlled increase of the bearing response. Due to the rubber hardening behavior at large strains (limiting the growth of the shear deformation with increasing seismic actions) the slope of the hazard curve increases as the design shear strain increases. The horizontal axis of the charts in Figure 4 are extended up to the threshold related to the *HDR* bearing capacity, assumed equal to 350%, on the basis of the available technical literature [32]. Such value also corresponds to the maximum displacement that could be imposed during a displacement capacity tests according to EN15129. In fact, in the hypothesis of a design shear deformation equal to 250% (the maximum allowed value) by applying the reliability factor and the amplification factor a maximum shear strain equal to 250·1.2·1.15=345% is obtained. In theory, every manufacturer working in the European context should be able to guarantee a deformation of about 350%.

It can be noted that only the cases with γ_d =1 completely fulfil the reliability target with a collapse MAF lower than the reliability target, the case with γ_d =1.5 is borderline (with a MAF very close to the target value) whereas the cases with γ_d =2 is not in compliance with it. Thus, it can be concluded that by assuming a design MAF ν_d = 0.0021 1/year (i.e., a design return period T_R=475 of the ULS) the design shear strain cannot be larger than 1.5, consequently EN15129 indications do not always guarantee the required reliability level and isolation system designed according to them might be unsafe. On the other hand, the national Italian code prescribe to design the isolation system considering a design MAF ν_d = 0.001 1/year (i.e., a design return period T_R=978 of the CLS) with a maximum shear strain equal to

200%. As can be seen form the orange horizontal lines in Figures 4, case with γ_d =1.5 at the ULS corresponds to a shear strain just above 2 at the CLS. This means that by performing a similar design procedure at the CLS with γ_d =2.0 the reliability level is attained, thus indications of the national code always guarantee the target reliability level. It is worth to note that safe cases (with γ_d =1.0 or γ_d =1.5) also correspond to a design shear strain of 200% and 300% respectively, by assuming a MAF $\nu_d = 4{\cdot}10^{-4}$ 1/year, i.e., a design return period T_R=2475 typical of the MCE considered in the American seismic codes [9]. A final remark is about the overstrength of the superstructure, which proved to have a negligible influence on the isolation system reliability.

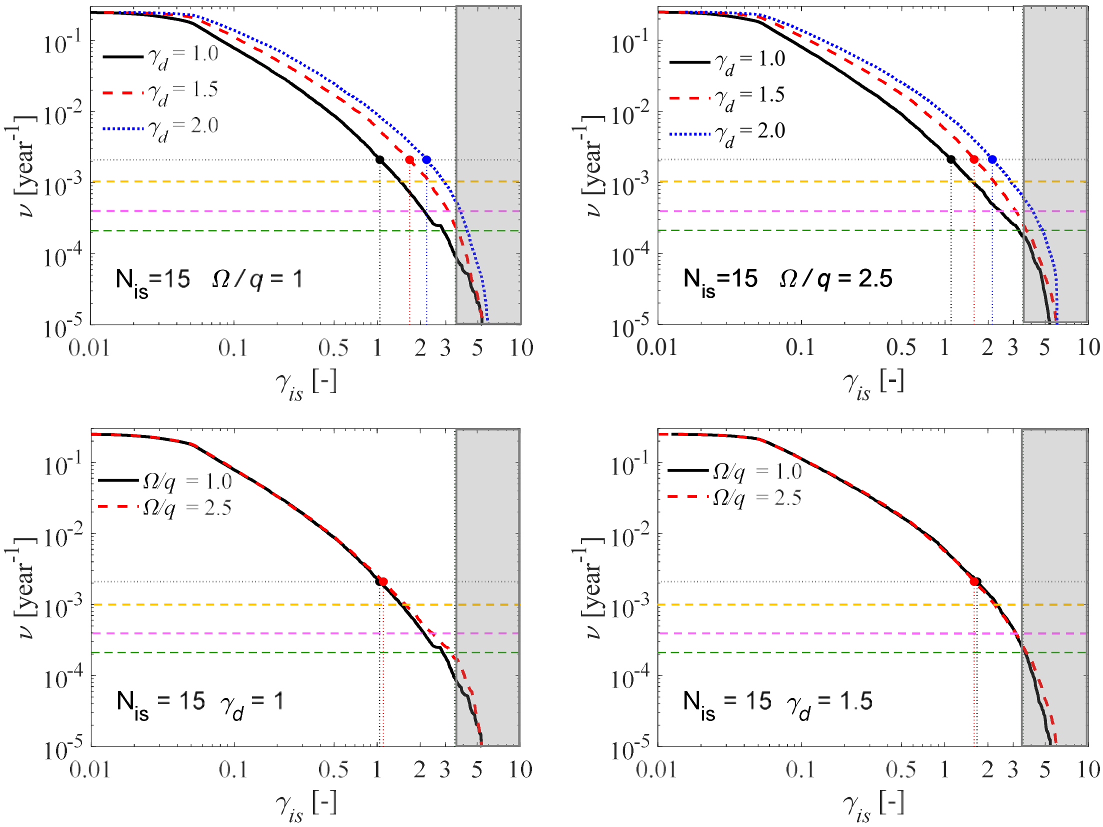

Figure 4: Hazard curves of the maximum bearings shear strain

Regarding the superstructure behaviour, Figure 5 shows the hazard curves of the maximum relative displacement of the superstructure. In all the figures the initial branches of the curves, representing the elastic range of the superstructure response, are overlapped. In the case Ω/q=1 the superstructure attains the yielding limit at ν_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 [33]. The same behaviour is recognized in the case Ω/q=2.5, but it is postponed (lower MAF) due to the larger yielding strength available on the superstructure. In both the cases, the after-yielding part of the curves, unlike the elastic branches, have a higher sensitivity to the design shear deformation. By assuming an average inter-storey drift of 2% as the threshold value beyond which the superstructure stability could be strongly compromised, an equivalent relative displacement of 0.16m is obtained and reported in the graphs. From the last two pictures of Figure 4 (where cases with Ω/q=1.0 and Ω/q=2.5 are compared for the two design shear strains leading to reliable isolation systems) it can be easily noted that only the case Ω/q =2.5 fulfils the reliability target, i.e., the collapse condition is attained with a MAF lower than $\nu_{target} = 2{\cdot}10^{-4}$ 1/year. In Figure 6. Thus, it is possible to conclude that if the superstructure is designed at the ULS (T_R=475 years according to [3] and [5]), the collapse MAF of the cases with Ω/q =1 is larger than the target reliability level, both for γ_d=1 and even more for γ_d=1.5. Differently, for both these cases the assumption Ω/q = 2.5 (i.e., q=1 and Ω=2.5) can guarantee the reliability of the superstructure.

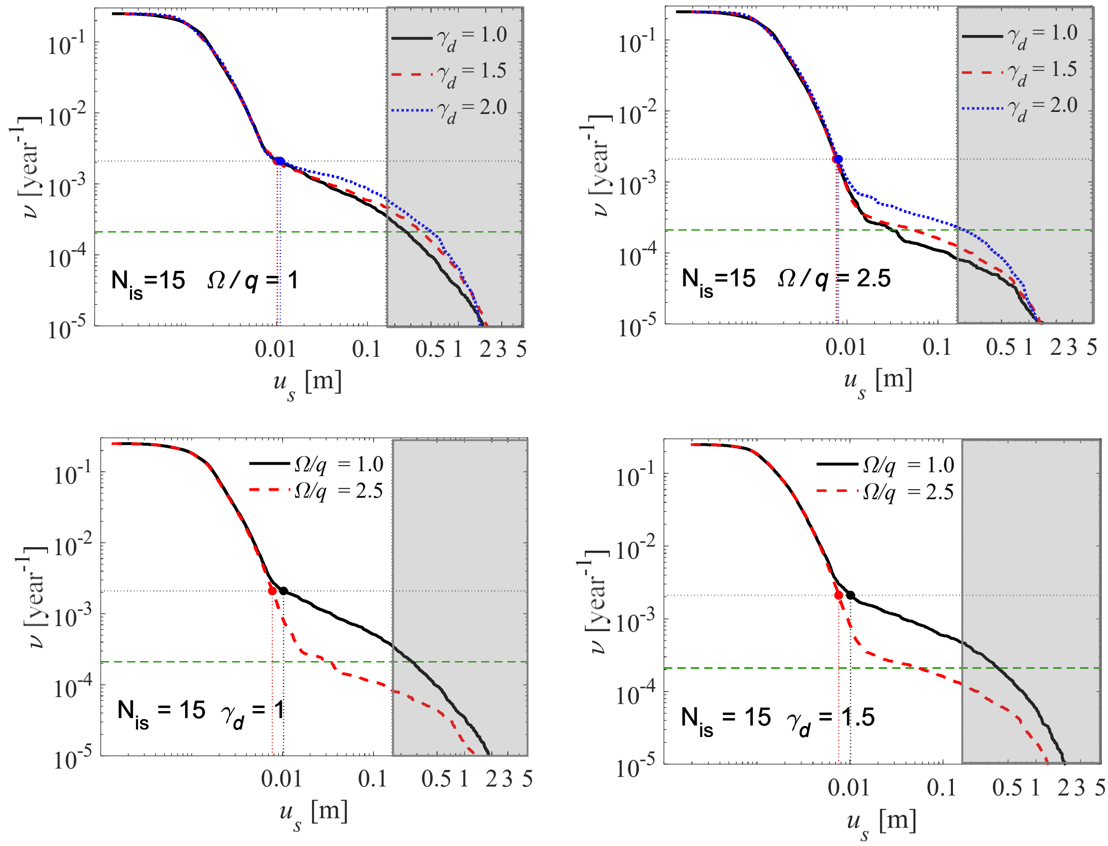

Figure 5: Hazard curves of the maximum superstructure's relative displacement

5 CONCLUSIONS

In this study seismic reliability analysis are performed on structural systems isolated with HDRBs. The influence of the main design parameters of both the isolation system and the superstructure is investigated by using a robust probabilistic framework furnishing the demand hazard curves of the most important system demand parameters. Based on the outcomes, the following main conclusions can be drawn:

- Regarding HDRBs, if a shear deformation capacity of 350% is assumed, prescriptions of EN15129 do not always ensure the target reliability level for high seismicity areas (the case with γ_d = 2 show a MAF of collapse larger than the reliability target); conversely, prescriptions of NTC 2018 always lead to a safe design of the isolation systems.
- Regarding the superstructure, if designed at the ULS (T_R=475 years according to Eurocode 8 and NTC 2018) with Ω/q =1 the MAF of collapse is larger than the target reliabil-

ity level; conversely, the assumption Ω/q = 2.5 (i.e., q=1 and Ω=2.5) can guarantee the reliability of the superstructure.

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

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