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EVALUATION OF THE EFFECT OF MASONRY INFILLS ON RESPONSE OF RC BULDINGS EQUIPPED WITH FPS DEVICES: A CASE STUDY

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Abstract

The assessment of existing structures and infrastructures located in areas with a relevant seismic hazard is one of the challenges for engineers over the last years. With reference to buildings built during 60's and 70's, most of them was realized as reinforced concrete (RC) framed structures disregarding seismic design issues in comparison to the current specifications. In order to enhance the seismic response to seismic actions of existing RC structures, over the years, the isolation system adopting friction pendulum (FP) devices turned out to be one of the most diffused. The aim of this work is to investigate the influence of the retrofitting intervention performed using single-concave friction pendulum devices (FPS) on the seismic response on an existing framed reinforced concrete building located in high seismicity region. The existing building presents both in plane and in elevation irregularities also related to the masonry infills. The effect of the masonry infills on the performance of the building is evaluated in probabilistic terms and useful recommendations for assessment of such kind of structures are reported.

Keywords: Infills, Seismic isolation, Irregular buildings, Assessment, Retrofitting.

1 INTRODUCTION

The quantification of the safety levels of existing structures situated in areas possibly subjected to strong earthquakes has been a crucial issue for engineers over the last decades [1]-[3], also with reference to relevant social consequences. In fact, the major part of reinforced concrete (RC) buildings built in Italy over the ‘60s have been designed almost disregarding the seismic actions focusing mainly to effects of gravity loads. This leads to a large group of structural typologies that presents substantial weakness to the accelerations induced by ground motions [4]. Over the years, with the aim to improve the seismic response of RC buildings, the adoption of isolation systems as the friction pendulum devices (FPS) has been widely investigated [5]-[6]. In the same context, the presence of masonry infills in RC buildings can affect significantly their behavior and extensive investigations are available in literature [7]-[8]. Nevertheless, the interaction between the masonry infills and isolator devices as FPS has been still not deeply investigated with the particular case of both in-plane and in-elevation irregular buildings [9].

According to the mentioned above issues, this investigation relates to the evaluation of the performance of a retrofitting intervention exploited by single-concave FPS devices on an existing irregular RC framed building including the influence of the actual distribution of masonry infills. This evaluation is presented by means of a bi-variate probabilistic approach able to take into account the correlation between the structural responses in longitudinal and transversal directions of the building (i.e., X and Y). In detail, the building is located within the significant seismic area of the central Italy close to the city of L’Aquila. The main materials and geometrical features have been determined according to the “knowledge levels” approach [10]. The RC structure of the building has been modelled by means fiber-based cross section approach using the software SAP2000 [11]. The non-linear dynamic analyses have been exploited selecting 21 natural ground motions according to [5]-[6] each one characterized by the three accelerometric components. In particular, the structure supposed to have fixed-base (FB) and isolated-base (BI) has been analyzed considering the case of bare and masonry infilled frames configurations. Then, the results of the NL dynamic numerical simulations have been useful to estimate, in probabilistic terms, the capability of the isolator devices to improve the seismic performance of the RC-framed building having both in-plane and in-elevation irregularities. The engineering demand parameters (EDPs) herein adopted are the non-dimensional interstory drift (i.e., interstory drift index—IDI) and the in-plane displacement of the slider of the FPS. The EDPs have been modeled through bi-variate lognormal probabilistic model [6]. Finally, the influence on the overall performance on actual distribution of masonry infills and their interaction with the retrofitting intervention by FPS devices is discussed.

2 MODELLING OF SINGLE-CONCAVE FPS DEVICES AND MASONRY INFILLS

This section relates to the basic theoretical aspects associated to the modeling of the single-concave FP devices and full and partial masonry infilled RC frames.

2.1 Behavior of single concave friction pendulum (FP) devices

The adoption of FP devices permits to disconnect the superstructure from the foundation level and allow to absorb the major part of the seismic demand in terms of displacements. Furthermore, these devices are able to provide high energy dissipation through friction mechanism that takes place on the sliding surfaces [6]. The single concave FP devices are realized by means of a slider that move on a surface having a concave shape (characterized by constant curvature radius R) and appropriate friction coefficient μ_d [12]-[14]. The adoption of single-concave FP leads to the main advantage of having the first natural period of the base-isolated structure T_{is} dependent only on the curvature radius R and gravity acceleration constant g and can be evaluated according to [6]. The basic response in terms of the dynamic equilibrium of the FPS device is represented in Figure 1a by means of the free-body diagram of the slider and the related equilibrium equation with respect to translation along the tangent direction to the sliding surface.

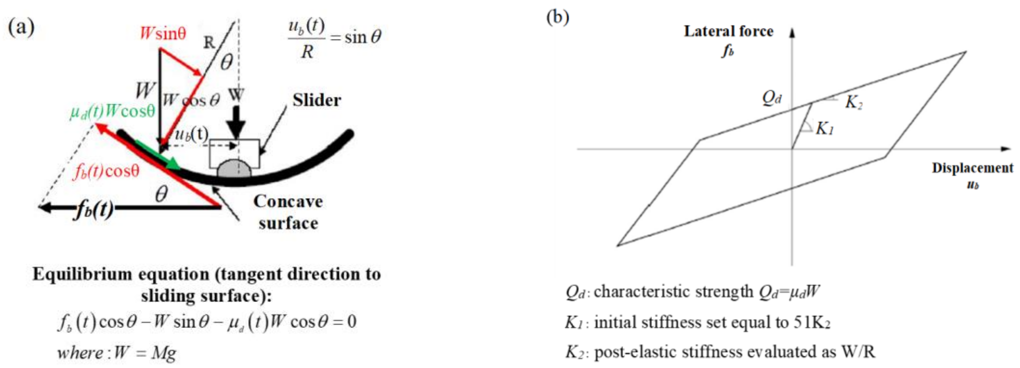

Figure 1: Free-body diagram of the single concave FP device under horizontal force. (a) Hysteretic response of the FP device in non-linear regime (b).

The restoring force $f_b(t)$ of the FP bearing device can be determined as follows:

$$f_b(t) = \frac{Mg}{R} u_b(t) + \mu_d(t)\,\mathrm{Mg}\,\mathrm{sgn}(\dot{u}_b(t)) \tag{2}$$

where M is the mass of the building pertaining to the specific FP device, $u_b(t)$ is the projection on the horizontal plane of the slider displacement relative to the ground, t represents the time, $\mu_d(t)$ represents the coefficient of friction in a dynamic regime, and $sgn(u_b(t))$ determines the sign of the velocity of the slider $u_b(t)$. The behavior under cyclic loading of single-concave FP devices may be idealized according to the non-linear hysteretic model proposed by [12] and represented in Figure 1b. Finally, the evolution of the dynamic friction coefficient $\mu_d(t)$ can be reproduced by means of the model of [13]-[14].

2.2 Modeling the behavior of masonry infills

In this study, the macro-modeling approach [8] has been adopted with the aim to simulate the response of fully and partially masonry infilled frames under seismic action.

With reference to the fully infilled panels, they have been simulated by means of a single-strut model able to take into account the non-linearities of masonry external partitions. In detail, the semi-empirical approach proposed by [8] has been adopted. Concerning the case of partially infilled panels, limited literature researches have been detached. For instance, they have been reproduced by means of a single-strut model with elastic behavior in order to take into account unfavorable effects related to shear demand on columns. With the aim to characterize the equivalent macro-model, a detailed micro-model of the partially infilled panel and of the surrounding RC frame has been defined using SAP2000. The width of the single strut has been calibrated until the same displacements at the top of the panel in the presence of horizontal force were achieve concerning the macro-models with respect to the micro-models. This particular calibration has been defined for all the several geometrical configurations of the partial infills related to the case study. The properties of masonry associated to both fully and partially infilled panels have been defined according to the results of material tests as explained next. Concluding, the masonry panels with large openings have been not included within the NL numerical models because of their limited contribution to global stiffness of the building.

3 CHARACTERIZATION OF THE RC EXISTING BUILDING

The RC-framed building considered for the investigation is located in central Italy in a region subjected to significant seismicity (PGA, peak–ground–acceleration, superior to 0.25 g and associated with an exceedance probability of 10^{-1} in 50 years referring to life safety limit state [10]). The RC building has been realized around the 1960s according to an Italian design code that was not conceived for seismic design according to current specifications in terms of required conception and detailing. The structure, built during the 1960s, is constituted by a cast in situ RC structure that consists of orthogonal frames dislocated along X (i.e., longitudinal) and Y (i.e., transverse) directions. The foundations are stiff, inverted, continuous RC beams along the direction of the RC frames. The in-plane sizes of the building are 59.8 m in the X direction and 12.2 m in the Y direction. Figure 2 illustrates, schematically, the column disposition with the related frames identified as X1, X2 in the X direction and Y1, Y2, Y3 along the Y direction.

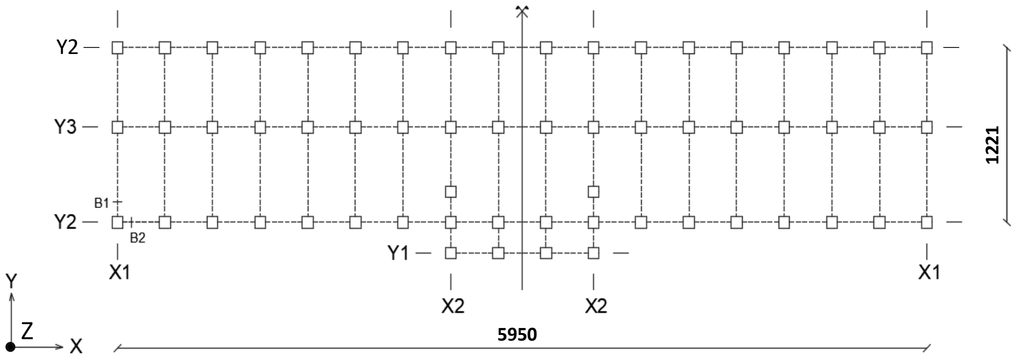

Figure 2: Plan view of the RC building; characterization of the RC frames according to X and Y orthogonal directions.

Concerning the distribution of the building in elevation, the maximum height is equal to 15.9 m measured from the foundation plan. The RC structure presents 4 stories plus the roof level. It is worthy to mention that around the structure for a height of 3.5 m measured from the foundation level is present a soil embankment that is able to restrain the displacements in the X and Y directions of the story 1. Data from inspections denotes that the concrete cover (in the average) is equal to 3 cm for all structural component. The floors are realized using a lightened “latero-concrete” technical solution with RC joists and a top slab. The RC joists are 16 cm high with a base of 10 cm with 0.5 m as center-to-center distance in the transverse direction. They carry the loads along the longitudinal direction (X) in line to the short beam pattern. The top slab is 4 cm thick and connects the joists realizing the floor and it can be considered enough with the aim to realize a rigid floor capable to distribute the actions to the longitudinal and transverse RC frames [10]. The external masonry infills (considered within the numerical modelling) have a total thickness equal to 34 cm and are realized with the following stratigraphy: external facing in exposed brick (8 cm); air cavity (13 cm); internal facing in perforated bricks (12 cm); and internal cement plaster (1 cm). The disposition of the partially in-filled and totally infilled masonry panels within the external RC frames of the structure is reported in Figure 3. More details about the reinforcement arrangement and geometry of the RC frames can be acknowledged in [5]. The adoption of FPS devices for the here described retrofitting intervention has been performed by means of the introduction of a disjunction between the foundation level and superstructure in correspondence of the columns of story 1. Moreover, with the aim to enhance the robustness of the system, connection RC beams between the columns have been introduced in correspondence of the isolation level [6].

Figure 3: Actual disposition of the external infills in the RC frames.

The determination of the materials' mechanical properties has been performed by means of destructive and non-destructive experimental tests with the aim to achieve the knowledge level 3 in line to [10]. The mean values of material properties have been determined according to appropriate statistical analysis of test results. In detail, the mean value of cylinder concrete compressive strength f_{cm} has been estimated as 25.2 MPa. As for the steel reinforcement bars (FeB38k), the mean value of the tensile yielding strength f_{ym} turned out to be equal to 374 MPa. Moreover, tests on masonries were carried out to determine the mechanical parameters; elastic modulus $E_{m1,2}$ (in horizontal and vertical directions, respectively) equal to 4325 and 4804 MPa, respectively; the shear strength f_{vm} equal to 0.75 MPa. The vertical compressive strength of the masonry f_{m2} has been derived according to specifications of [18] assuming that the horizontal compressive strength f_{m1} is equal to the 50% of f_{m2}. All the material properties not experimentally determined have been assumed in line to [18].

4 NUMERICAL MODELING AND NL DYNAMIC ANALYSES

The RC building under investigation has been modeled according to the actual boundary conditions, geometrical and material characteristics using the program SAP2000 [11].

The main structural elements, characterized by RC columns and RC beams, have been modeled by means fiber cross-section approach and concentrated plasticity philosophy (SAP2000 [11]) by means the definition of plastic hinges (inclusive of the interaction between axial force and bi-axial bending). The non-linear response of the hinges takes place within a predetermined length L_p that has been modeled by means of the approach of [16].

The concrete constitutive law according to [17]-[18] has been used to simulate non-linear responses in compressions core (i.e., confined) and the cover concrete. The tensile behavior has been modeled by means of LTS (i.e., linear tension softening). The degradation of the mechanical properties due to cyclic loading has been accounted for with the model of [19]. The constitutive relationship for bar reinforcement has been assumed as elastic with perfect plastic response limiting the ultimate strain at a value of 7.5%. The modulus of elasticity has been assumed equal to 200,000 MPa.

The partially infilled panels have been modelled using "elastic frame elements" in SAP2000 [11] with active response only in compression. The fully infilled frames have been modeled adopting the link element "Multilinear plastic" [11] active in compression only.

The FP devices have been modeled by means link element with non-linear behavior denoted as "Friction Pendulum" in line to the SAP2000 [11]. The parameters affecting the response of the isolator have been determined as a function of the axial load on the specific FP device [5],[13]. The related value has been determined in the function of the axial load applied on each device with reference to the empirical approach of [30]. As reported in Figure 4, four different numerical models have been realized, differentiating between the bare fixed-base/base-isolated structure and the infilled fixed-base/base-isolated.

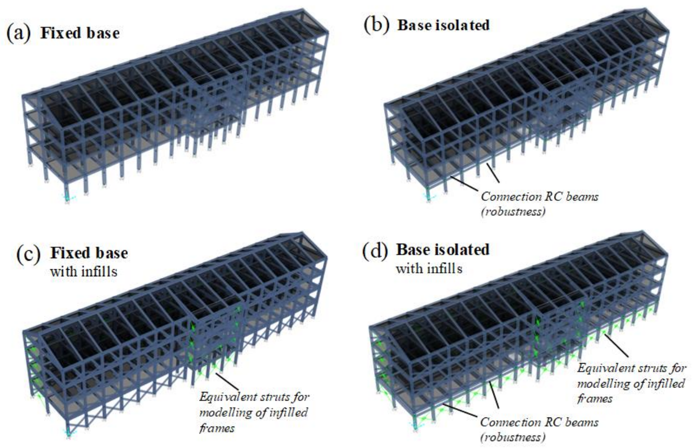

Figure 4: Different configurations for the NL models.

The seismic demand has been determined by adopting as an intensity measure (IM) the value of the pseudo-acceleration S_a in the elastic regime. The site-specific response spectrum associated with 50 years as a reference period for limit state of life safety has been adopted in according to [10]. The values of the damping coefficient ξ have been distinguished between fixed-base and base-isolated structures, adopting 5% for the former and 2% for the latter one [6].

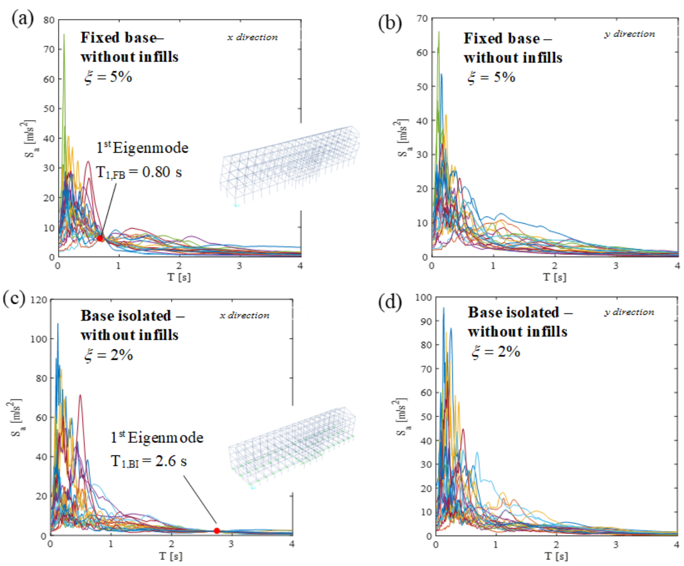

Figure 5: Scaled spectra of pseudo-acceleration for fixed-base (a,b) and base-isolated (c,d) building in X and Y directions. Bare frames.

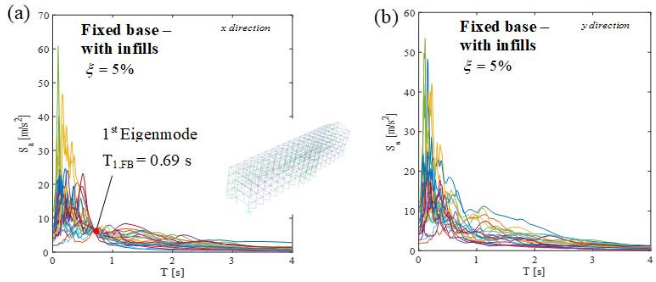

Figure 6: Scaled spectra of pseudo-acceleration for fixed-base (a,b) building related to natural seismic inputs along X and Y directions. Infilled frames.

The set of ground motion inputs to realize the NL dynamic simulations is represented by 21 natural records composed of 3 acceleration components along the in-plane and vertical directions according to the selection of [5]-[6] from the ESM (European Strong Motion Database) [20]. With the aim to ensure spectrum compatibility with [10], the spectra related to the 21 natural records have been properly scaled for what concerns the X-direction component. In detail, the IM evaluated in concomitance with the fundamental period T_1 of the structure related to the elastic spectrum of the specific record has been scaled to the IM of the design spectra of [10]. This operation has been performed for both fixed-base and base-isolated structures, relating to both bare and infilled structural configurations. The so far scaled response spectra related to the selected natural records and associated with the different structural configurations are shown in Figures 5 and 6. It is worthy to highlight that the base-isolated infilled frame shows scaled records that are almost equal to the ones corresponding to the base-isolated bare frame.

The non-linear dynamic analyses have been executed solving the non-linear equations of motion under seismic input by means of the method of direct integration in line with [21]. The outcome of the NL dynamic analyses has been measured in terms of the peak value during the ground motion of the inter-story drift index (IDI) for each story of the building for what concerns the in-plane directions (i.e., X, Y). Moreover, the horizontal displacement of the FP slider has been monitored with reference to base-isolated models. The outcomes of the NL dynamic analyses have been submitted to probabilistic analysis as discussed in next section.

5 BI-VARIATE PROBABILISTIC ANALYSIS OF THE RESULTS

The results from the NL dynamic simulations have been quantified in terms of the horizontal relative displacements of the isolators d_{FPS} and the *IDIs*. In particular, a lognormal probabilistic model have been adopted for both both d_{FPS} and *IDIs* [6]. The mean value μ and standard deviation σ have been estimated by performing a statistical analysis [22]-[30] of the data. The parameters of the probabilistic distribution have been computed adopting the maximum likelihood method according to [31].

In detail, the tri-dimensional response of the fixed-base structure or the base-isolated structure can be performed by evaluating the degree of correlation between the abovementioned parameters *IDIs* and d_{FPS} along the two directions X and Y. Then, the joint log-normal distributions can be computed according to [6]. The probability of failure in the case of jointly distributed variables can be calculated by integrating the generic joint lognormal between the responses in X and Y directions. Figures 7–10 illustrate mainly the level curves of the joint PDFs, together with a certain limit state area, considering the cases with and without masonry infills and for both fixed-based and base-isolated buildings. Concerning the superstructure, only results related to the 4th story are herein presented.

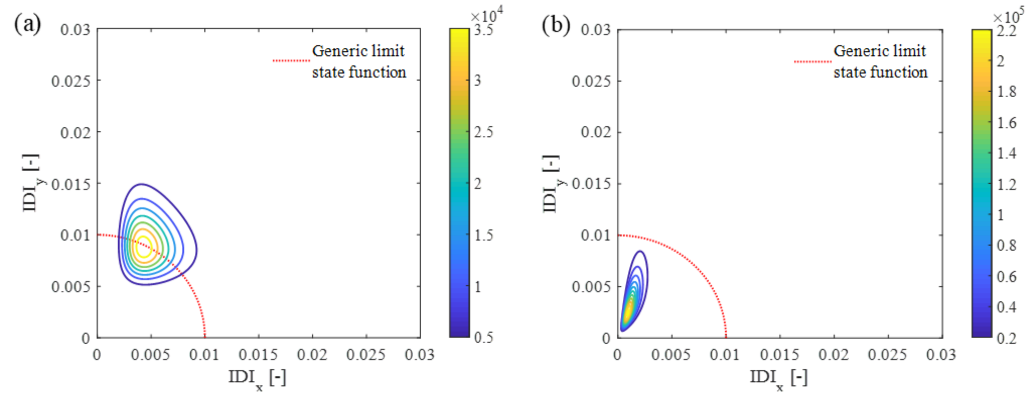

Figure 7: Level curves of the joint PDF for the 4th story with a generic limit state: fixed-base model (a); base-isolated model (b); (without masonry infills).

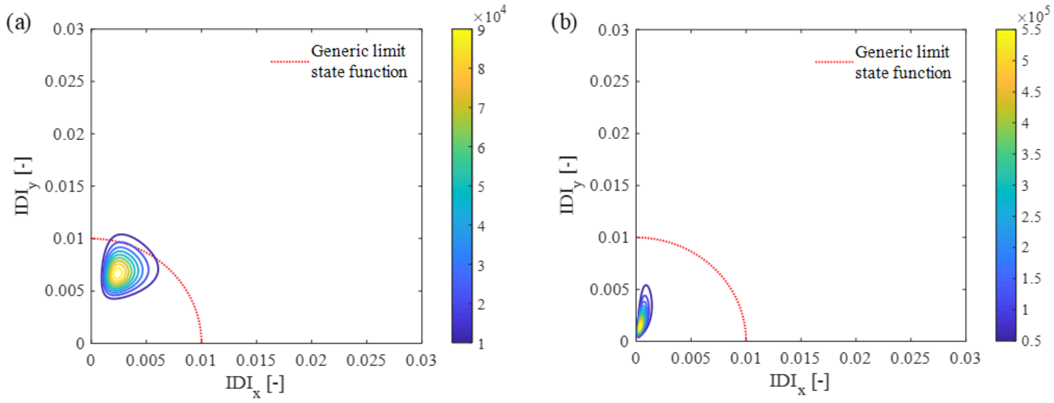

Figure 8: Level curves of the joint PDF for the 4th story with a generic limit state: fixed-base model (a); base-isolated model (b); (with masonry infills).

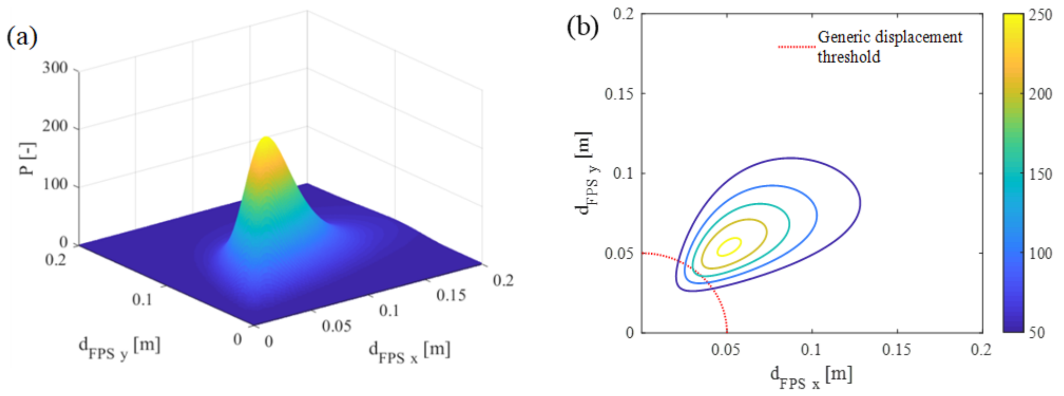

Figure 9: Three-dimensional view of the joint PDF for the isolation story (a) and level curves (b) with a generic displacement threshold (without masonry infills).

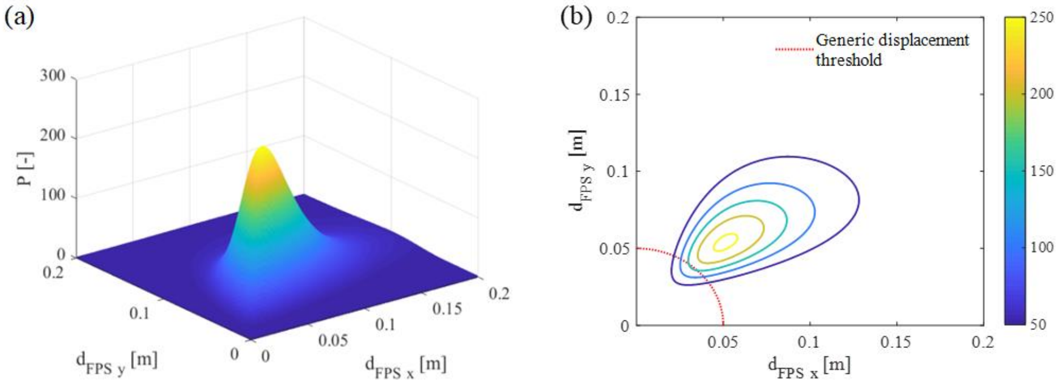

Figure 10: Three-dimensional view of the joint PDF for the isolation story (a) and level curves (b) with a generic displacement threshold (with masonry infills).

Defining specific thresholds in terms of *IDIs* and d_{FPS} it is possible to estimate the bi-variate probabilities of exceedance according to Figures 11 and 12. It turns out clear as the presence of infills leads to lower exceeding probabilities as well as a lower dispersion of the spatial response of the structure along its height, especially for the fixed-base structure, but also for the case of the isolated building.

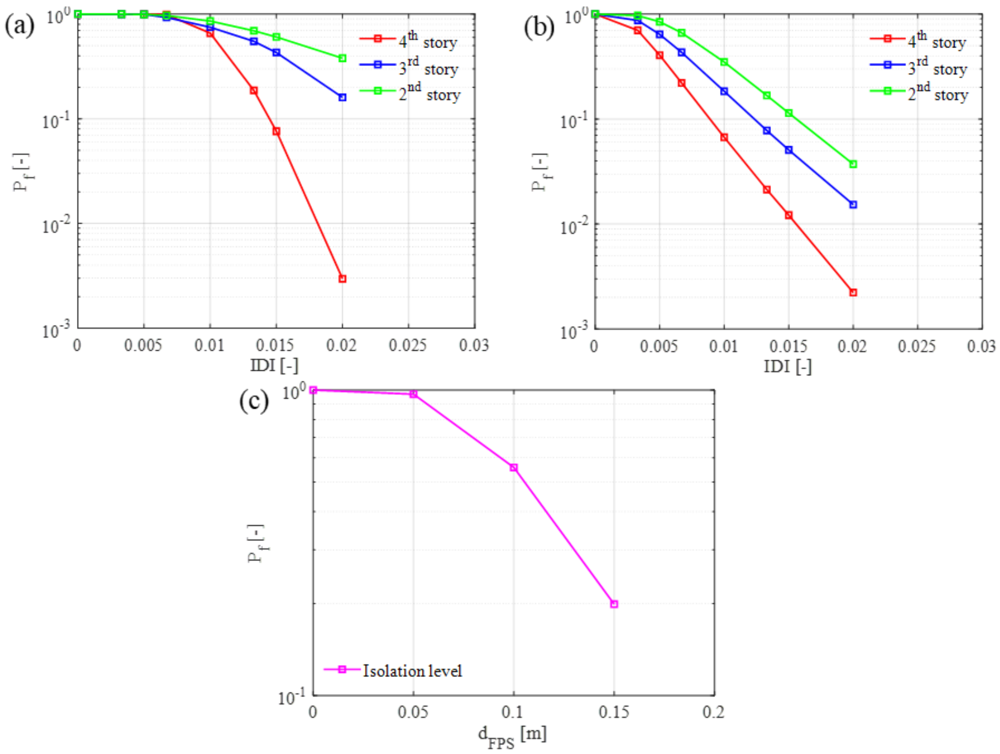

Figure 11: Probabilities of exceedance with bi-variate log-normal assumption in logarithmic scale: fixed-base model (a); base-isolated model (b); isolator devices for base-isolated model (c); (without masonry infills).

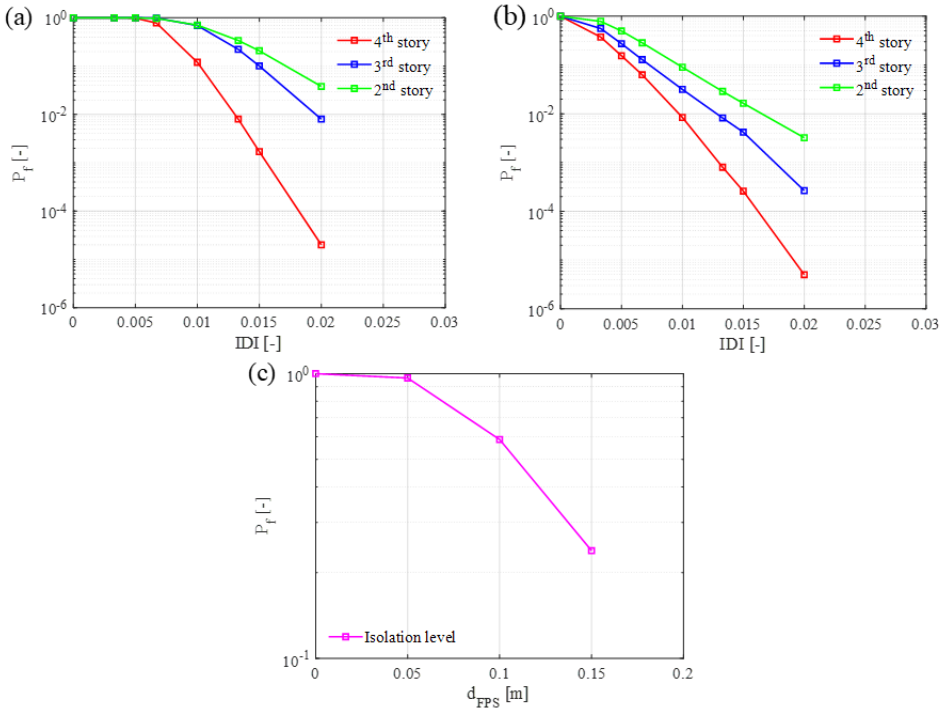

Figure 12: Probabilities of exceedance with bi-variate log-normal assumption in logarithmic scale: fixed-base model (a); base-isolated model (b); isolator devices for base-isolated model (c); (with masonry infills).

6 CONCLUSIONS

The present study is devoted to the evaluation of retrofitting intervention on an irregular RC building using FP devices including the influence of masonry infills panels. In particular, the interaction between the FP isolation system and the irregular distribution of infilled frames has been analyzed. The outcomes of the NL dynamic simulations highlight that the presence of the FPS isolators allows for a relevant reduction in the values of the IDIs. The presence of masonry infills improves the effectiveness of the isolation system by further reducing the displacement demand. The results of the non-linear dynamic analyses have been treated by means of probabilistic approach based on bi-variate lognormal distribution functions. This with the aim to include within the probabilistic analysis the correlation between the response of the building in X and Y directions. The probabilities of exceedance present relevant reduction between the fixed-base building and the base-isolated building. This result is confirmed and magnified by the presence of masonry infills. This is evidence of the hidden safety levels that exist when seismic analysis is carried out without including the structural effects of masonry infills. As conclusive remark, further developments should be carried out in order to estimate the seismic reliability of such a building including also the contribution o the site-dependent seismic hazard.

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