PROBABILISTIC RISK ASSESSMENT OF A MODERN ARCHITECTURE BUILDING: ANALYSIS OF THE ORIGINAL BUILDING AND RETROFITTING PROPOSALS

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ABSTRACT :

The seismic vulnerability of an existing building representative of modern architecture style is presented herein. To improve its seismic response, a retrofitting solution consisting of an x-bracing system associated to a shear-link dissipater was analysed. The effectiveness of the retrofit was measured by carrying out a seismic fragility analysis, considering a numerical model accounting for the nonlinear behaviour under earthquakes of both RC elements and infill masonry walls. It is seen that currently available numerical tools, in combination with probabilistic assessment procedures, make the structural safety assessment of buildings an increasingly feasible methodology based upon which it is possible to propose effective seismic risk mitigation measures.

KEYWORDS: RC structure, retrofit, fragility analysis, infill masonry wall modelling.

1. INTRODUCTION AND OBJECTIVE OF THE STUDY

The study of the seismic vulnerability and safety of existent buildings in urban areas with moderated/high seismic risk according to the recently proposed international codes and recommendations is of extreme importance. Moreover, in the case of buildings exhibiting an irregular shape of some kind, the referred study gains particular importance since it may reveal an inadequate structural behaviour under earthquake loading that may require strengthening measures in order to mitigate such negative effects. In this subject matter, attention is drawn to the high number of buildings constructed in Lisbon, Portugal, during the 1950s, with a particular modern architecture style that led to construction of buildings with characteristics that are inadequate under seismic loading.

In the present paper, the seismic vulnerability of an existing building representative of the referred modern architecture style is studied. Based on the behaviour of the original building, several configurations of a low intrusive retrofitting solution with dissipative devices were tested to improve its seismic behaviour and to maintain its architectural characteristics. The effectiveness of the different retrofit configurations was measured by performing a probabilistic seismic fragility analysis of the different solutions. The proposed paper presents the study of one of the retrofit configurations, emphasizing some of the considered modelling assumptions and the benefits resulting from the probabilistic analysis carried out to assess the retrofit effectiveness.

It can thus be seen that currently available refined numerical tools, in combination with probabilistic assessment procedures, make the structural safety assessment of existing buildings an increasingly feasible methodology based upon which it is possible to propose effective seismic risk mitigation measures.

2. DESCRIPTION OF THE STUDIED STRUCTURE

The building under study is located in the western part of Lisbon and exhibits some of the Le Corbusier's 1920s architectural ideas. The bulk of the structure is lifted off the ground and is supported by *pilotis* – reinforced concrete (RC) columns – which allowed for the definition of an open floor plan.

The building block plan is rectangular with 11.10m width and 47.40m length (Fig. 1). The building has the height of 8 residential storeys plus the pilotis height at the ground floor. The building structure is defined by twelve RC transversal plane frames that have the same geometric characteristics for all beams and columns and that are spaced by 3.80m (Fig. 2). However, from the mechanical point of view, three different frame-typologies were identified, according to the reinforcement detailing. The most peculiar structural characteristic of these buildings, with direct influence in the global structural behaviour, is the open floor plan, i.e. without infill masonry walls. Moreover, the ground storey columns are 5.5m height while all the remaining upper storeys have an inter-storey height of 3.0m. Considering these two structural aspects, it is found that the necessary conditions exist for the development of a soft-storey mechanism at the ground storey level in case of an earthquake event.

Figure 1. General views of the building block under analysis.

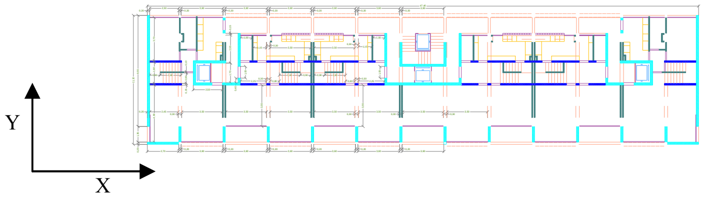

Figure 2. Plan view of the structural system.

As previous analyses of both directions of the building showed the longitudinal direction (X) to be the most vulnerable [1], only this direction is analysed herein. Given the symmetry and the replication of the identified RC frames, a simplified planar model was defined which is able to represent the structural behaviour of the building in the X direction. Moreover, since there are no beams in that direction and in order to simulate the existent 0.20m thick RC slab with an influence width of 1.25m, equivalent beams with linear elastic behaviour were considered in the numerical model. In terms of the masonry infill panels, these were considered in the model according to the details found in the existing building.
Taking into account the referred symmetry, the numerical model considered to study the building in its longitudinal direction represents only a quarter of the total building (Fig. 3). This model has six columns, where inelastic behaviour is considered, connected at the storey levels by the horizontal equivalent elastic beams simulating the RC slabs. Since there are no full-bay infill panels in the longitudinal direction, an external simplified global infill masonry was considered (Fig. 3), which is slaved to the RC structure at the storey levels.

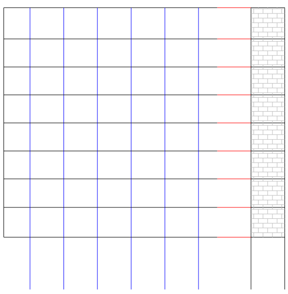

Figure 3. Structural system for the numerical model of the longitudinal direction (X)

3. BEHAVIOUR MODELS, SEISMIC DEMAND AND GENERAL MODELLING DATA

The current state of development of nonlinear mechanical behaviour laws and hysteresis models offers a considerable advantage for the analysis of structures subjected to earthquake ground motions, as they make way for a more rigorous representation of the seismic structural response. The numerical simulation of the structural behaviour of the studied building under earthquake loading was performed using the program PORANL [1], that contemplates the nonlinear flexural behaviour of RC elements (beams and columns) and the influence of the infill masonry panels in the global response of the buildings by numerically modelling the global nonlinear behaviour of the panels. Each RC structural member is modelled by a macro-element defined by the association of three bar finite elements, two with nonlinear behaviour located at the ends of the element (plastic hinges) and a central element with linear behaviour (Fig. 4 a)). The nonlinear monotonic behaviour curve of a given cross-section is characterized by a trilinear moment-curvature envelope obtained using a fibre analysis of the cross section (see Fig. 4 b)) accounting for the geometric characteristics of the cross-sections, its reinforcement, the characteristics of the material properties and the existing axial force. The nonlinear cyclic behaviour of the cross sections is represented using the hysteretic rules of the Costa-Costa model [2], thus enabling the representation of the response evolution of the global RC section to seismic actions and contemplating mechanical behaviour effects such as stiffness and strength degradation, pinching and slipping effects (Fig. 4 c)). In addition, the numerical formulation also enables the consideration of P-delta effects. With respect to this aspect of the numerical modelling, a sensitivity study was carried out to assess the influence of P-delta effects for the structure under study and to determine whether such influence is dominant for the purpose of the presented study. Results of this assessment are presented in Section 5.

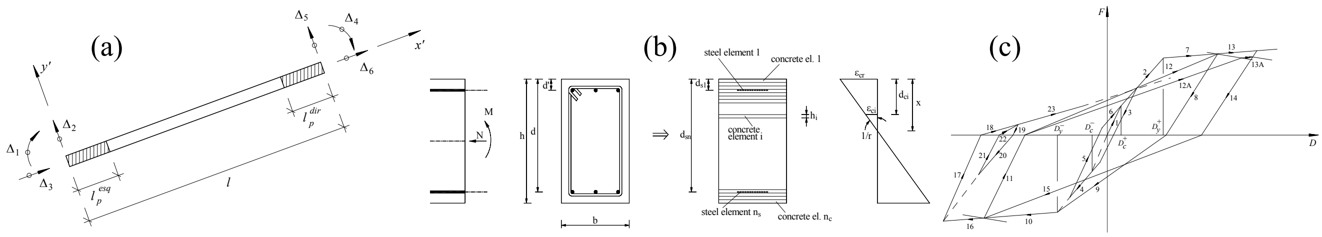

Figure 4. (a) Member macro-element; (b) fibre model of cross sections and (c) hysteretic model of RC members.

To represent each infill masonry panel, an improved macro-model is used (Fig. 5 a)) that is based on an equivalent bi-diagonal strut model. The proposed macro-model was implemented in the nonlinear structural analysis program PORANL [1]. The considered macro-model is able to represent the nonlinear behaviour of an infill masonry panel under cyclic loading, thus enabling the integration of its influence in the global structural behaviour of the building under static or dynamic loading. The monotonic behaviour curve of each panel depends on the panel dimensions, the existence of openings (their dimensions and position), the material properties (namely of the bricks, mortar and plaster), on the quality of the handwork and on the interface conditions between the panel and the surrounding RC elements. For the present study, the referred monotonic curve was obtained and calibrated based on the empirical procedure proposed in [3]. The nonlinear behaviour of the infill masonry panels subjected to cyclic loads is controlled through a hysteretic procedure, with rules that are illustrated in Fig. 5 b) and that is able to represent mechanical effects such as stiffness and strength degradation, pinching, and that is also able to account for the existence of internal cycles.

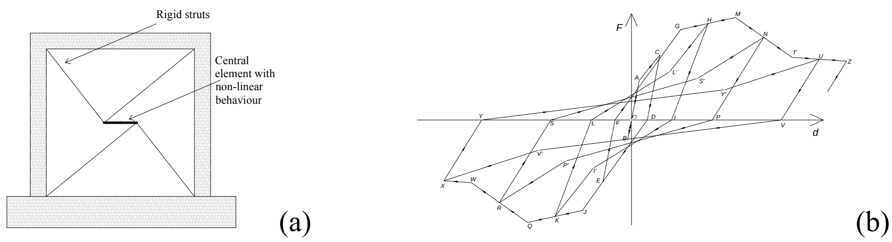

Figure 5. (a) Infill masonry panel macro-model and (b) hysteretic model for the infill masonry panels.

The vertical loading considered in the analyses consists in uniform loads of 8.0kN/m^2 on all storeys distributed on the beams. These loads represent the self-weight of the beams, of the slabs and of the infill walls, the finishes and the quasi-permanent value of the live load. Damping was assumed to be of Rayleigh type with a critical damping equal to 1% for the first and second mode periods. The mass of the structure was assumed to be concentrated at the storey levels and distributed on the beams. Each storey has a mass of about 4Mtons, which represents the

self-weight of the structural and non-structural elements, of the infill walls and finishes, and of the quasi-permanent value of the live loads.
Before running the analyses, the numerical model of the structure was calibrated by comparing experimentally measured and analytically estimated natural frequencies. Experimental frequency measurements were carried out with a seismograph measuring ambient vibrations. For the longitudinal direction, which is of interest herein, the computed fist mode numerical frequency was 1.08Hz while the experimentally measured one was 1.17Hz, which shows there is a good agreement between the dynamic properties of the numerical model and those of the actual building. Furthermore, it can also be seen from Fig. 6, where the first mode shape for the longitudinal direction is presented, that, as expected, earthquake actions will induce a soft-storey mechanism on the structural response.
In terms of seismic demand, the proposed study considers, as earthquake input, a set of synthetic ground motions defined for the city of Lisbon according to a non-stationary stochastic finite fault seismological simulation model based on random vibration theory [4]. These ground motions reflect both the close and distant earthquake scenarios for the city of Lisbon and were defined for several return periods: 73, 225, 475; 975 and 2000 years. For each earthquake scenario and each return period, a set of 10 ground motions was considered.

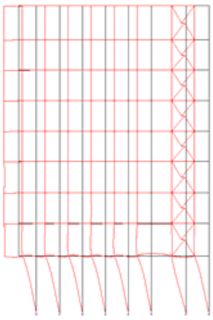

Figure 6. First mode shape for the longitudinal direction

3.1 Definition of the retrofit solution and corresponding modelling

In order to improve the seismic response of the building under study, a retrofitting solution aiming to reduce the effect of the soft-storey mechanism by reducing the deformation demand at the ground floor level, was analysed. More specifically, the retrofit proposal consists of an x-bracing system associated to a shear-link dissipation device (see Fig. 7 a)). The retrofitting solution is based on a solution studied in [8] and defined by 4 shear links made out of HEB140 steel sections with 60 cm length, each one located on one side of the stairway shafts, Figs. 7 b) and c). This solution is able to increase both the lateral stiffness and the damping properties of the building, thus leading to a reduction of the lateral deformation demand. The adoption of x-braces as a retrofitting solution is due to their efficiency in reducing the deformation demand but also due to the fact that, since they are only applied at the ground floor and in very specific locations, it is considered that they do not affect significantly the architecture of the building.
To be able to include the effect of the retrofit solution on the structural behaviour of the building, an additional numerical model was developed and implemented into the considered analysis program. This new model is able to simulate the nonlinear behaviour of the bracing devices and was calibrated with experimental results resulting from a full-scale cyclic test performed on a frame retrofitted with the same dissipative device [8]. The hysteretic behaviour and rules of this model are presented in Fig. 8 a) while Fig. 8 b) presents a sample of the calibration results obtained for the referred model.

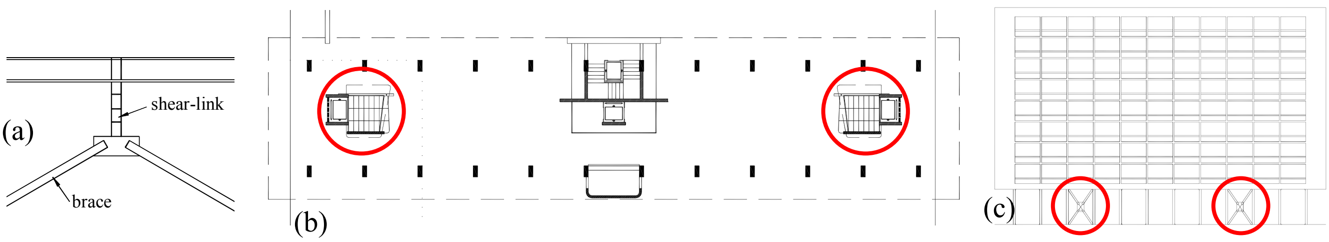

Figure 7. (a) Shear-link dissipater; (b) and (c) location of the shear link in the longitudinal direction.

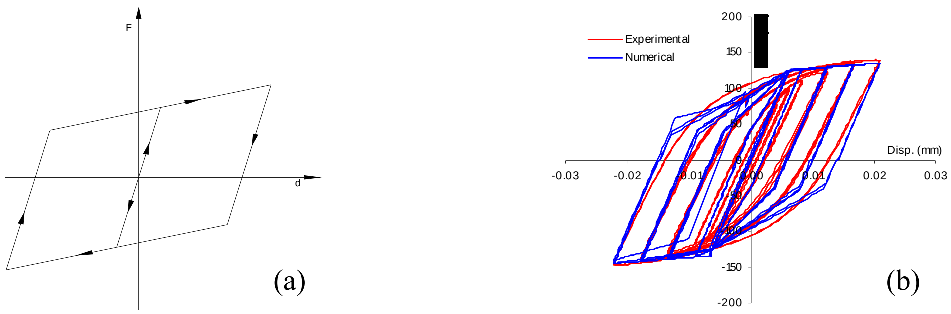

Figure 8. (a) Implemented hysteretic behaviour of the shear-link and (b) sample of the calibration results of the numerical model for cyclic behaviour.

4. GENERAL DESCRIPTION OF THE PROBABILISTIC ANALYSIS

In the context of the proposed study, probabilistic analyses were carried out to define global behaviour fragility curves associated to different limit states and different building modelling strategies. The referred fragility curves were assumed to be lognormal functions, a widely considered assumption (see e.g. [5][6]), that describe the probability that the demand/damage measure θ reaches or exceeds a given limit state. The conditional probability of being, or exceeding, a damage state y given a peak ground motion intensity measure (IM) is defined by:

$$P\left(\theta_{\max} \geq y|_{IM=x}\right) = \Phi\left[\frac{1}{\beta_y}\ln\left(\frac{x}{IM_y}\right)\right] \tag{1}$$

where IM_y is the median value of the selected IM for which the building reaches the threshold of damage state y, β_y is the standard deviation of the natural logarithm of the selected IM for the damage state y, and Φ is the standard normal cumulative distribution function. For the purpose of the considered analysis, the selected demand measure θ is the interstorey drift. Therefore, the measure θ_{max} represents the maximum interstorey drift over the height of the building. In terms of the selected limit states, these were defined according to the values presented in Table 1 which correspond to the limit values proposed in [7] and were considered as deterministic limits for simplicity.

Table 1 – Considered inter-storey drift limits according to the VISION 2000 proposal [7].

Limit State	Fully Operational	Operational	Life Safety	Near Collapse
Inter-storey Drift Limit	0.2%	0.5%	1.5%	2.5%

5. INFLUENCE OF THE P-DELTA EFFECTS BASED ON NONLINEAR DYNAMIC ANALYSIS RESULTS

As previously referred, a preliminary set of nonlinear dynamic analyses was carried out to assess the influence of P-delta effects for the structure under study, considering the type of demand that is required for the probabilistic performance assessment. Four groups of analyses were carried out for the close earthquake scenario ground motions, all the return periods, and considering the possible combinations of structure (with and without retrofit) and analysis (with and without P-delta effects). Observation of the results led to conclude that, for all the analysis cases, the maximum interstorey drift θ_{max} is always at the bottom storey. For the analysis cases where the retrofitted structure is considered, this is the reflection of having selected a “light” strengthening configuration. For a more “heavy” retrofit solution, it is foreseen that θ_{max} can occur over any storey. Moreover, for the considered cases, it is seen that the more important effects in terms of deformation increase due to P-delta effects are located in storeys above the bottom one. That is, at the bottom storey where the relevant demand parameter is measured, P-delta effects produce a reduced increase in lateral deformation that is mostly felt for some of the larger earthquake intensities. Fig. 9 presents a sample of the obtained results in terms of interstorey drift profiles for the non-retrofitted structure, for analyses situations with and without P-delta effects, for the “near” earthquake scenario and for all return periods. The plotted interstorey drift for each storey represents the maximum interstorey drift over the 10 ground motions of each return period. It is noted that interstorey drifts are defined in percentage values and plotted in log scale. As can be seen from these results, P-delta effects have a considerable influence over the lateral deformation above the bottom storey, while for this storey their influence is much less important. Hence, in order to reduce computation time, P-delta effects were not considered in the remaining analysis carried out for the probabilistic assessment.

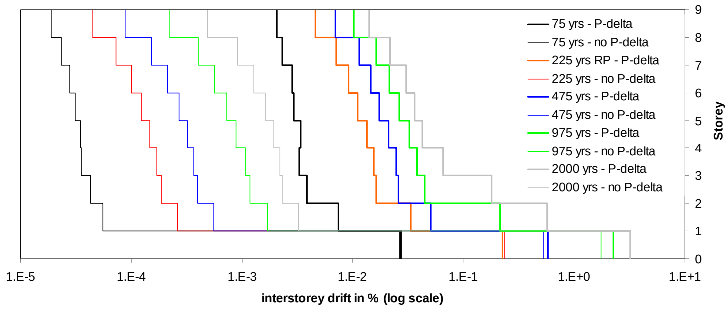

Figure 9. Interstorey drift profiles for the non-retrofitted structure, with and without P-delta effects.

6. RESULTS OF FRAGILITY ANALYSIS

A sample of the obtained fragility analysis results is presented in the following. The results comprise fragility curves for the different limit states previously defined, for the two considered earthquake scenarios and for the original and retrofitted structures. Although the selected ground motion IM was the peak ground acceleration (PGA), it is noted that since the ground motions were generated by a stochastic process based on magnitude and distance criteria, ground motions associated to a given return period might have different PGAs. In order to have a common IM for the ground motions of a given return period, the average PGA of the records was selected. Figs. 10 a) and b) present the fragility curves of the original structure for the several limit states and the two earthquake scenarios, while Fig.10 c) presents an overlap of the two earthquake scenarios. As can be seen, the "distant" scenario is dominant over the full range of limit states. Figs. 11 a) and b) present the fragility curves of the original and retrofitted structures for the several limit states and the two earthquake scenarios. As can be seen, there is a considerable reduction of the fragility values (i.e. of the risk) for a given limit state and IM value, as the several fragility curves of the retrofitted structure are shifted to the right. In order to have a more thorough understanding of the effectiveness of the retrofit solution and of the risk reduction it provides, Fig. 12 a) presents a simple analysis of the decrease in risk from the original structure to the retrofitted one for a couple of fixed levels of PGA while Fig. 12 b) presents an analysis of the increase in PGA from the original structure to the retrofitted one for a couple of fixed levels of fragility. When performing these two types of analysis for all the limit states, for all the PGA levels and for all the fragility levels, Figs. 13 a) and b) are then obtained. As expected, from the analysis of Fig. 13 a), the fragility reductions for the limit states of Fully Operational and Operational take place for low PGA values, while for the limit states of Life Safety and Near Collapse they also occur for larger PGAs. From the analysis of Fig. 13 b), it can be seen that the dissipative devices shift the fragility curves to the right such that, for a given fragility value, the shift has the same proportion for all limit states.

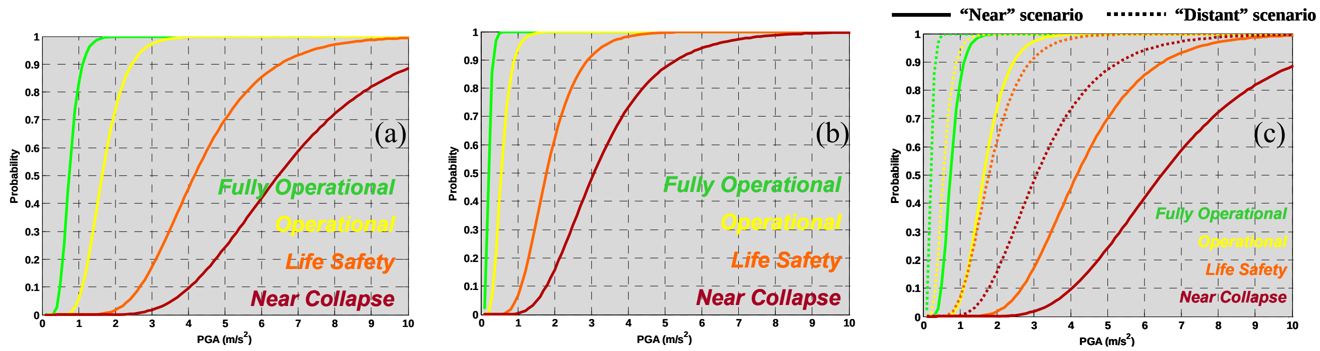

Figure 10. Fragility curves of the original structure for the several limit states and the (a) "near" earthquake scenario; (b) the "distant" earthquake scenario and (c) both scenarios.

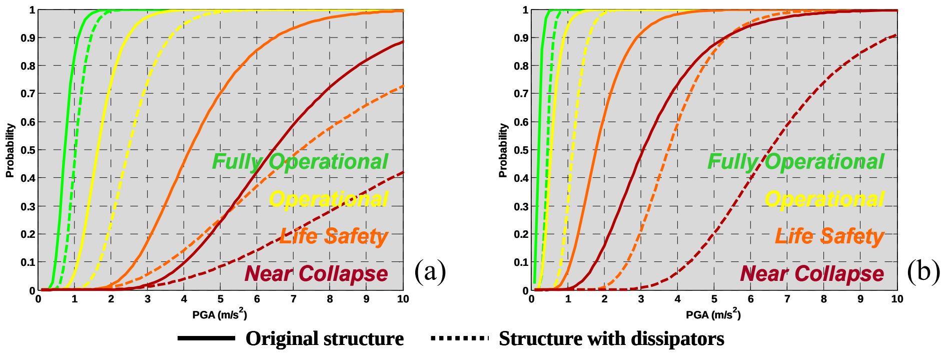

Figure 11. Fragility curves of the original and retrofitted structures for the several limit states and the (a) "near" earthquake scenario and (b) "distant" earthquake scenario.

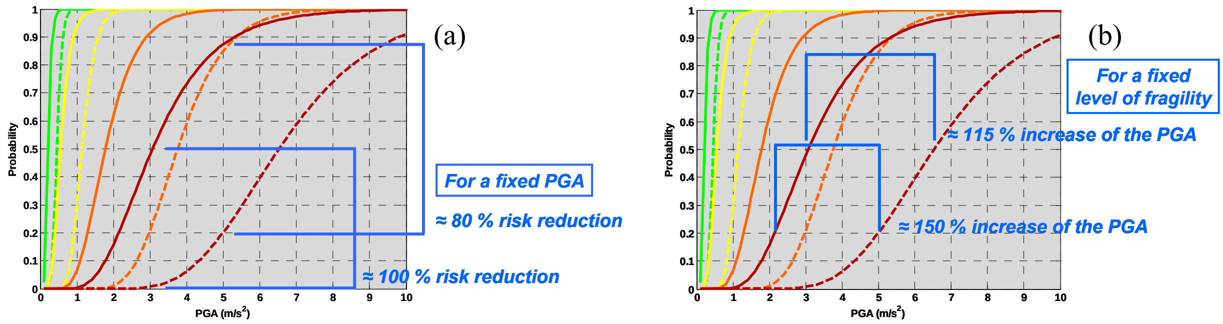

Figure 12. (a) Decrease in risk from the original structure to the retrofitted one for fixed levels of PGA and (b) increase in PGA from the original structure to the retrofitted one for fixed levels of fragility.

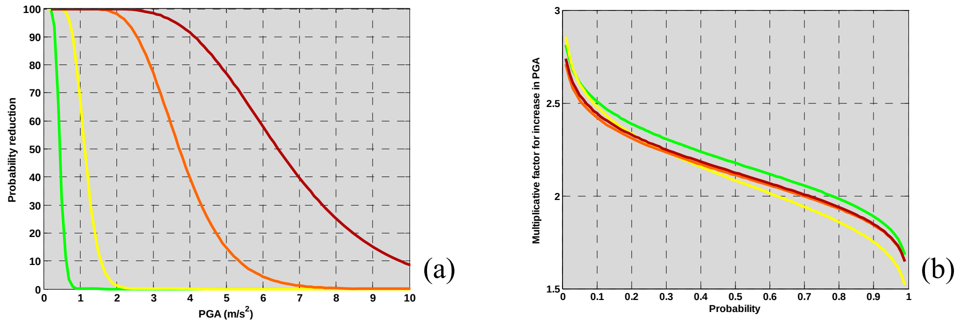

Figure 13. (a) Decrease in risk from the original structure to the retrofitted one for fixed levels of PGA and (b) increase in PGA from the original structure to the retrofitted one for fixed levels of fragility.

7. FINAL OBSERVATIONS

In the present paper, the seismic vulnerability of an existing building representative of modern architecture style is studied. Based on the behaviour of the original building, a low intrusive retrofitting solution with dissipative devices was tested to improve its seismic behaviour and to maintain its architectural characteristics. The effectiveness of the retrofit configuration was measured by performing a probabilistic seismic fragility analysis of the solution. The proposed paper presents the study of the retrofit configuration, emphasizing some of the considered modelling assumptions and the benefits resulting from the probabilistic analysis carried out to assess the retrofit effectiveness. It is seen that currently available refined numerical tools, in combination with probabilistic assessment procedures, make the structural safety assessment of existing buildings an increasingly feasible methodology based upon which it is possible to propose effective seismic risk mitigation measures.
As previous analyses of both directions of the building showed the longitudinal direction to be the most vulnerable, only this direction was analysed. The considered numerical model of the building accounts for the nonlinear behaviour under earthquake loading of both RC elements and infill masonry walls. In terms of structural

modelling, a sensitivity study was also carried out to determine the influence of P-delta effects for the considered structure and for the relevant demand measure for the study. Results indicated that P-delta effects have a considerable influence over the lateral deformation above the bottom storey, while for this storey their influence is much less important. Since the relevant demand measure, maximum interstorey drift over the height of the structure, is always located at the bottom storey, P-delta effects were not considered in the remaining analysis carried out for the probabilistic assessment, in order to reduce computation time.
In order to improve the seismic response of the building under study, a retrofitting solution consisting of an x-bracing system associated to a shear-link dissipation device and aiming to reduce the effect of the soft-storey mechanism by reducing the deformation demand at the ground floor level was analysed. This solution is able to increase both the lateral stiffness and the damping properties of the building, thus leading to a reduction of the lateral deformation demand, namely reducing the influence of the soft-storey mechanism that is developed under earthquake loading.
From the probabilistic analysis that was carried out to determine the effectiveness of the retrofit solution, the "distant" earthquake scenario was seen to be dominant over the full range of the considered limit states. It was also seen that when performing a simple analysis to determine the decrease in risk from the original structure to the retrofitted one for fixed levels of PGA, for all the limit states and for all the PGA levels, the fragility reductions for the limit states of Fully Operational and Operational take place for low PGA values, while for the limit states of Life Safety and Near Collapse they also occur for larger PGAs. Furthermore, when performing an analysis to determine the increase in PGA from the original structure to the retrofitted one for fixed levels of fragility, it was seen that the dissipative devices shift the fragility curves to the right such that, for a given fragility value, the shift has the same proportion for all limit states. Based on these two analyses it is possible to observe that the effectiveness of the retrofit solution provides a uniform protection throughout the several limit states while providing also considerable risk reductions when observing the vulnerability across fixed levels of PGA.

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