

PROBABILISTIC RISK ASSESSMENT OF RC BUILDING STRUCTURES WITH MASONRY INFILLS

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1. INTRODUCTION

The study of the seismic vulnerability of existent buildings in urban areas with moderated/high seismic risk is of extreme importance. Earthquake engineering experts, public authorities and general public alike agree on the idea that the assessment of the seismic safety and performance of the built environment is a matter of high priority.

The proposed paper aims to carry out a structural fragility study where a representative RC building structure is modelled considering different characteristics (without masonry infills, with masonry infills and considering different mechanical characteristics of the infills). As the definition of fragility curves for the different analysis cases is the main objective of the study, the general outcome of the study will enable to assess the global importance of the referred material and mechanical uncertainties in the final fragility values and, therefore, in the potential mitigation measures that will be based on such results.

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 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 (Figure 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 reinforced concrete (RC) transversal plane frames that have the same geometric characteristics for all beams and columns. 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, the necessary conditions are met for the development of a soft-storey mechanism at the ground storey level in case of an earthquake event.

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 longitudinal direction. Moreover, since there are no beams in that direction, equivalent beams with linear elastic behaviour were considered in the numerical model that simulate the existent RC slab. In terms of the masonry infill panels, these were considered in the numerical model according to the details found in the existing building.



3. DESCRIPTION OF THE CONSIDERED BEHAVIOUR MODELS

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 previously presented building under earthquake loading was performed using the computer program PORANL (Varum, 1995), that contemplates the nonlinear bending 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 element 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 (Figure 2). The nonlinear monotonic behaviour curve of a given cross-section is characterized by a trilinear moment-curvature envelope while the cyclic behaviour is represented using the hysteretic rules of the Costa-Costa model (Costa and Costa, 1987; CEB, 1996), 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 (Figure 3).

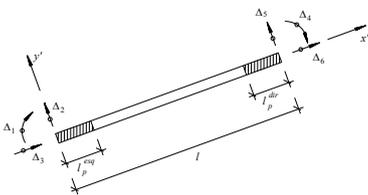


Figure 2: Frame macro-element (Varum, 1995)

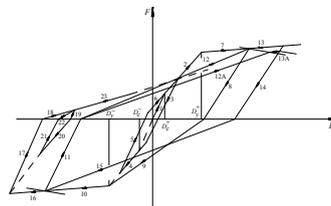


Figure 3: Hysteretic model for RC elements

To represent each infill masonry panel, an improved macro-model is used (Figure 4) and that is based on an equivalent bi-diagonal strut model. The proposed macro-model was implemented in the nonlinear structural analysis program PORANL (Rodrigues, 2005). The considered macro-model is able to represent the nonlinear behaviour of an infill masonry panel, thus enabling the integration of its influence in the global structural behaviour of the building under static or dynamic loading (Figure 5).

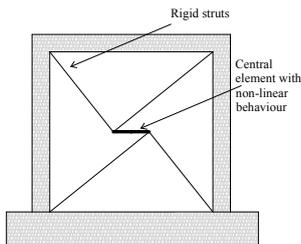


Figure 4: Infill masonry panel macro-model

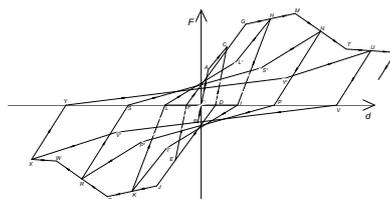


Figure 5: Hysteretic model for infill masonry panels

4. INFILL MODELLING CONSIDERATIONS

The proposed case study is made of several numerical analyses consisting of 5 different structural modelling assumptions. One of the models considered for the analyses was considered without any masonry infills while the remaining four were considered with masonry infill panels. With respect to these four models with infill panels, the first one was defined with the original monotonic curve for the infill masonry panel, obtained and calibrated based on the empirical procedure proposed by (Zarnic and Gostic, 1998) (Figure 6) while the remaining three models were defined by considering that the first branch of the envelope curve represented the existence of a gap between the infill masonry panel and the surrounding RC frame. Three levels of gap were

considered with values of 5, 10 and 15 mm (see Figure 6), which are assumed to represent part of the existing uncertainty about the masonry infill properties.

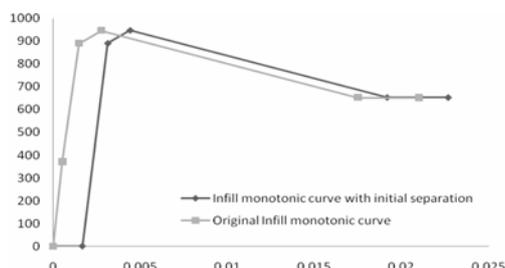


Figure 6: Infill generic monotonic curve

5. EARTHQUAKE INPUT SIGNALS

The proposed study considers 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 (Carvalho *et al*, 2008). The considered ground motions reflect both the close and distant earthquake scenarios for the city of Lisbon and were defined for several return periods.

6. FRAGILITY ANALYSIS

In the context of the proposed study, building fragility curves were assumed to be lognormal functions, a widely considered assumption, that describe the probability that the demand/damage measure θ reaches or exceeds a specific limit state. The conditional probability of being, or exceeding, a particular damage state y given a peak ground motion intensity measure IM is defined by:

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

where IM_y is the median value of the selected IM where 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 (SEAOC, 1995).

Table 1 – Considered inter-storey drift limits according to the VISION 2000 proposal (SEAOC, 1995)

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

7. A SAMPLE PREVIEW OF THE RESULTS

As stated, the main results of the proposed study come in the form of fragility curves that will allow for the quantification of the different selected modelling strategies. Figure 7 and 8 present a sample of the results obtained, in order to illustrate the aforementioned strategies. Figure 7 represents the fragility curves of the

analysed frame considering that no gap exists between the infill masonry and the RC structure, for the four considered limit states (see Table 1). On the other hand, Figure 8 presents, for the limit state of “Fully Operational”, the fragility curves of the analysed frame considering different modelling strategies for the infill masonry panels. As can be seen, for low level earthquake intensities, there is a significant increase in the fragility values due to the consideration of the referred gap between the infills and the RC elements.

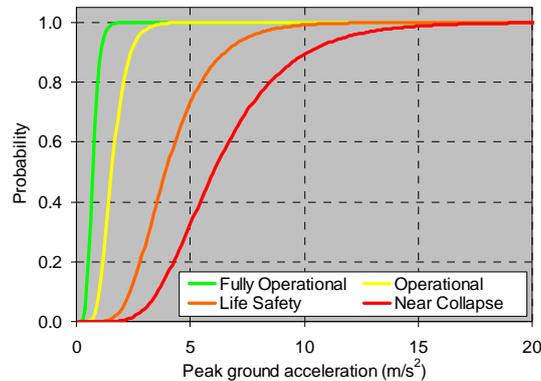


Figure 7: Fragility curves for the four considered limit states.

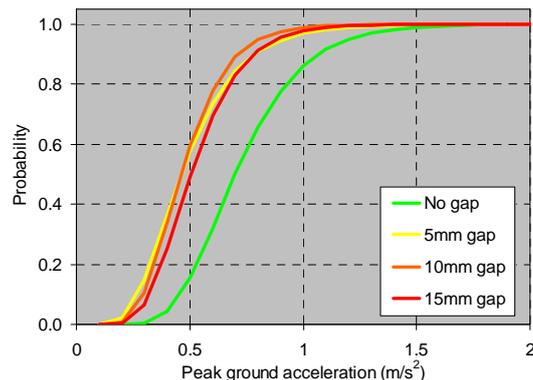


Figure 8: Fragility curves for different modelling strategies of the infill panels.

8. ACKNOWLEDGMENTS

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