Evaluation of seismic performance of old existing RC residential buildings accounting for shear effects

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Abstract

Reinforced Concrete (RC) buildings constitute a significant part of the building stock of European countries, the majority built before the introduction of modern seismic codes. The occurrence of past earthquakes highlighted the poor behaviour of these structures when subjected to horizontal loads, which emphasises the relevance of analysing the seismic performance of this type of buildings. This study focused on the characterization of old RC buildings, with special attention given to the structural characteristics that may be associated with the collapse in the case of a seismic event. A survey was conducted in the Alvalade area of Lisbon and the data was compiled into a database that allowed grouping the buildings into different classes. From the analysis of this database, a RC structure built between 1960 and 1980, representative of one of the identified classes, was selected and analysed in terms of seismic performance and economic losses.

The building structure was modelled in OpenSees and the seismic performance assessment was carried out through nonlinear static analysis, followed by the safety verifications recommended by Eurocode 8, which mainly consisted of the evaluation of the demand/capacity ratio (DCR) for all the vertical structural elements (columns and shear walls). The results showed that the seismic performance was controlled by the shear behaviour for early stages of deformation.

The economic losses were estimated through the application of the FEMA P58 methodology. The building´s structural response was evaluated via incremental dynamic analyses and the damage assessment was performed using component based-fragility functions for four different damage states: DS1 (light damage), DS2 (moderate damage), DS3 (severe damage) and DS4 (collapse). It was also assumed that for this building typology, only damage to vertical elements (columns and RC walls) was relevant for the global seismic performance. Based on the results of the initial seismic assessment, which indicated collapse due to deficient of shear strength capacity of the vertical structural elements, the probability of collapse was estimated through two different approaches: (i) assuming the largest probability of any vertical structural element to reach its shear strength capacity, as recommended by EC8-3, and (ii) assuming the largest probability of any vertical structural element to reach the lateral deformation associated to DS4. The results indicate an underestimation of the collapse probability and, consequently, the total losses evaluation, when the shear failure is incorporated in the loss methodology through component-based fragility curves based on drift deformations instead of shear strength verifications, as proposed by EC8-3.

Keywords: Existing RC Buildings; Seismic Assessment; Nonlinear analysis; Building Characterization; Loss assessment

1. Introduction

Past earthquake events, such as the 1999 Kocaeli Earthquake, the 1999 Chi-chi Earthquake, the 2011 East Japan Earthquake, the 2011 Christchurch Earthquake and the 2017 Puebla Earthquake exposed the seismic vulnerability of reinforced concrete (RC) buildings built prior to the introduction of modern codes.
Several studies have been conducted to characterize and evaluate the performance of RC buildings located in the most seismic prone areas. This study focused of the characterization and seismic performance of old RC residential buildings located in in Lisbon.
A survey was conducted in the Alvalade area and the characterization of this type of buildings was made based on the buildings date of construction and the structural configuration. An assessment of the structural characteristics that may be associated with the collapse in the case of a seismic event was also made through the analysis of the blueprints available at the "Arquivo Municipal do Lisboa" in ArcGIS [1]. A list of indicators, common to all building typologies, was collected, such as (i) low confinement and tie reinforcement in vertical structural elements; (ii) insufficient longitudinal reinforcement in all structural elements; (iii) smooth longitudinal reinforcement rebars; and (iv) variation in height of the vertical structural elements section and (v) absence of masonry infills at the ground level.
A selected building, representative of one of the classes identified, was evaluated in terms of seismic performance and economic losses. Similar studies have been conducted [2] based on approaches considering the performance of different building classes, such as the Global Earthquake Model (GEM) [3] and HAZUS [4]. The seismic performance assessment of this building, a RC structure built between 1960 and 1980, which means designed without seismic provisions, was carried out through nonlinear static analysis and the evaluation of demand to capacity ratio (DCR) recommended by Part 3 of Eurocode 8 (EC8-3) [5]. Herein, the shear and flexural failures are assessed in terms of demand to capacity strength and deformation (rotation) ratios, respectively.
The economic losses were estimated following the procedure adopted by Caruso et al. [6], which is based on the PEER-PBEE framework [7]. This approach entails the use of nonlinear dynamic analysis to assess the building's structural response and the application of component based-fragility functions to assess damage in structural and non-structural elements. The component based-fragility functions selected for the RC columns and walls are the ones proposed by [8] and [6], respectively. Both refer to four limit states that consider light damage (DS1), moderate damage (DS2), severe damage (DS3) and collapse (DS4). Shear failure is incorporated in the methodology through the drift deformation associated to DS3 while flexural failure is defined based on a drift deformation associated to DS4, which corresponds to loss of vertical carrying capacity and, consequently, collapse.

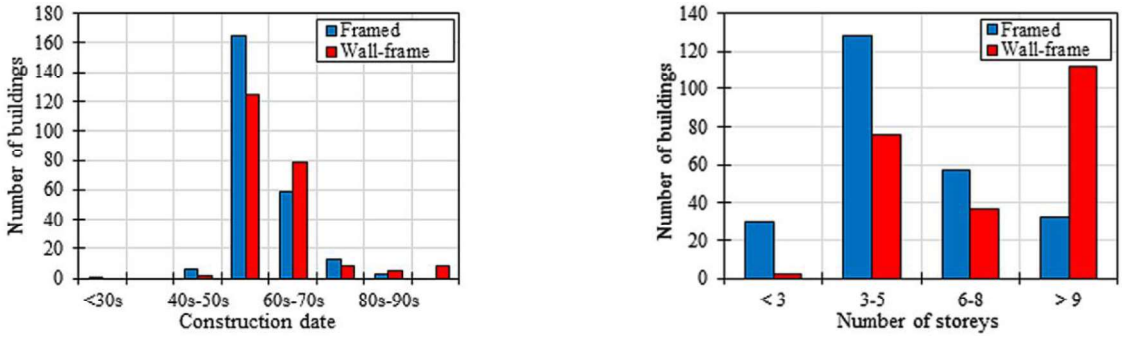
2. Characterization of the RC buildings located in Alvalade, Lisbon

In Portugal, the first design codes that proposed some provisions regarding seismic action were the RSCCS [9] and the RSEP [10] introduced in 1958 and 1961 respectively. The RC structures built following these codes were mainly designed to resist gravity loads and exhibit several seismic vulnerabilities regarding their geometry (e.g. vertical and horizontal irregularities) and structural design (e.g. presence of indirect supports beam-beam, insufficient longitudinal and transverse reinforcements, smooth rebars for pre-1970 buildings, insufficient length of embedment for the anchorage into the foundations, poor construction materials) [11]. It was only in 1983 that was introduced the RSA design code [12] that established seismic performance requirements. Hence, it is useful to classify the existing RC concrete buildings accordingly to their year of construction since it can be used to address the level of ductility of a building by associating the structural design codes that were in effect at the time of construction.

A survey of the buildings located in the Alvalade area of Lisbon was conducted, followed by its characterization, based on the buildings date of construction and their RC structural configuration. This assessment included 2249 buildings: 28% are RC structures, 71% are mixed masonry-concrete structures ("Placa" building), and the other 1% represents "Gaioleiro" buildings, garages and some unknown buildings

(due to the lack of data). The assessment has been carried out by analysing and gathering the data from the blueprints available at the "Arquivo Municipal do Lisboa" in ArcGIS [1].

Regarding the year of construction and as proposed in [2], the buildings constructed before 1958 were classified as pre-code (PC), buildings constructed between 1958 and 1983 as mid-code (MC) and buildings constructed after 1983 as post-code (C). From the total number of RC buildings, it was only possible to identify the structural configuration in 88% of them, where 46% are framed, and 42% are wall-frame. Fig. 1(a) shows the distribution of the RC structures identified in Alvalade according to their structural configuration, and Fig. 1(b) shows the distribution of the RC buildings according to their structural configuration and the number of storeys.



(a) (b)

Fig. 1 – Distribution of RC buildings regarding the structural configuration and the (a) construction date and (b) number of storeys

Most of the RC buildings were built before 1980 (97% of all), while 70% of these were constructed between 1950 and 1970 (when RC structures started to be built). Regarding Fig. 1(b), the majority of the framed structures (51%) are medium height buildings of three to five storeys, while in the case of wall-frame structures, most of them (49%) are tall buildings with more than nine storeys. Regarding other structural characteristics and as common in old existing RC buildings, a high percentage of them (57% for the framed and 67% for the wall-framed) presents irregularities in elevation. Regarding the wall-frame buildings, 62% of them are also characterised by the absence of infill walls on the ground storey, which usually leads to a soft story mechanism.

3. Case study

3.1 Building description

The RC building selected is a five-storey wall-frame RC building built in the 1950s. The building has a total height of 15.4m with a typical floor heigh of 3.0 m, except for the ground floor, which height is 3.4m. It has three and five frames in X and Y directions, respectively. A shear wall is positioned at the middle of the building, oriented in the X direction, on the first three storeys (Fig. 2).

This analysis of the building's blueprints allowed the following observations: (i) RC columns and walls with low confinement and tie reinforcement; (ii) insufficient longitudinal reinforcement as buildings were designed to resist only gravitational loads; (iii) smooth longitudinal reinforcement rebars; and (iv) the section of the columns decreases in height, simultaneously at the floor level, which reduces the stiffness of the system suddenly at different levels in height. Moreover, the building only has masonry infills in the upper storeys, leading to the generation of soft-storey mechanisms at the ground floor caused by vertical discontinuity of masonry infill. The variation of the reinforcement ratios and the geometrical properties of the structural elements is listed in Table 1.

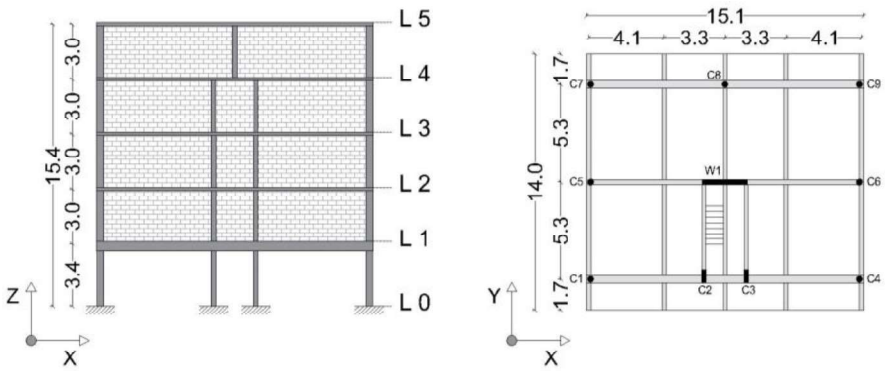


Fig. 2 – Elevation and plan of the selected building

Table 1 – Geometrical properties and minimum/maximum reinforcement ratios of the structural elements (T and B refer to top and bottom beam reinforcements, respectively).

Element	Columns	Beams	Walls
Dimensions (cm)	23×25–25×87	13×31–25×68	250×25
Cross Section (cm^2)	575–2175	403–1700	6250
Longitudinal rebar (cm^2)	0.63–7.29	T :2.26–29.76 B: 3.93–33.58	15.78
Transversal rebar (cm^2)	0.18–0.71	1.48	3.15
Spacing of stirrups (cm)	5-19	20–50	15

3.2 Numerical modelling

The building was modelled and analysed with the OpenSees software [14]. A distributed plasticity modelling approach was adopted to simulate the inelastic response of the structure subjected to seismic action.
The structural elements were modelled using a force-based formulation considering one element per beam, column and wall. The cross section of each element is represented by a fibre section and subdivided into quadrilateral fibres with one centimetre of thickness. At the fibre level, two material models were used: the Popovics model, designated as Concrete04 material in OpenSees, to simulate the concrete, and the uniaxial Giufre-Menegotto-Pinto model, defined as Steel02 material in OpenSees, to model the reinforcement steel. The mean values of the material properties were obtained from the review of the blueprints of this type of buildings, available at the "Arquivo Municipal do Lisboa", and comply with the concrete requirements established in the corresponding codes applicable in this period (1950–1980). It should be mentioned that the concrete tensile strength has been simulated as 10% of the compressive strength. The behaviour of the smooth rebars was modelled by modifying the steel constitutive law, reducing the Young's Modulus and maximum strength of the reinforcing steel to simulate the increase in member flexibility due to strain penetration effects [15].

The torsional stiffness was incorporated in the 3D model by computing the elements torsional stiffness and assigning it through the 'section aggregator' program command, while the rigid in-plane stiffness of the floor slab was modelled considering a rigid diaphragm multi-point constraint, through the program command 'rigid diaphragm'. Second-order effects were accounted for in the model by adopting a P-Delta geometric transformation for the columns and wall. The infill panels were also modelled by assuming the two-diagonal truss approach [16]. The mass, which was obtained through the conversion of dead and live loads, was lumped at the centre of mass at each floor level. The dead loads include the self-weight of the RC elements, masonry infills and coatings, i.e., 8 kN/m^2, while the live loads have been defined according to Part 1 of

Eurocode 8 (EC8-1) [17]. The fundamental periods of vibration of the building are 0.31s and 0.41s in X and Y directions, respectively.
Two types of analyses were performed: nonlinear static (pushover) and incremental nonlinear dynamic analyses (IDA). The nonlinear static analysis (pushover analysis) was performed assuming a (i) modal load pattern, proportional to the fundamental mode of vibration, and (ii) a uniform load pattern, proportional to the masses of the floors. Only the results corresponding to the modal pattern were considered since they correspond to a capacity curve with lower resistance and, consequently, more restrictive in terms of performance. The pushover analyses were performed in X and Y directions, with positive and negative signs and, despite the slight asymmetry in the Y direction of the building, no significant differences were observed between the positive and the negative orientations. Hence, only the analyses in the positive direction have been adopted in the assessment stage.

For the incremental nonlinear dynamic analysis a group of 30 ground motions (Fig. 3) were selected, with both record components applied simultaneously, first in the XY direction and after rotated 90º [18], resulting in a total of 60 analysis. The selection of the ground motion records was conducted with the SelEQ tool [19]. The records were first pre-selected from the PEER database based on geophysical parameters, considering both horizontal components, and after scaled (both components of the record with the same scale factor) in order that the average of the group has a good match with the Portuguese Response spectrum (Type 1; Soil type B, PGA = 0.15g) in the period range of interest (0.2T1 and 2T1 in accordance to EC8-1, where T1 is the fundamental period of the structure). The viscous damping was modelled using the Rayleigh damping formulation considering a damping matrix proportional mass and tangent stiffness with 5% damping ratios for the first and fifth vibration modes.

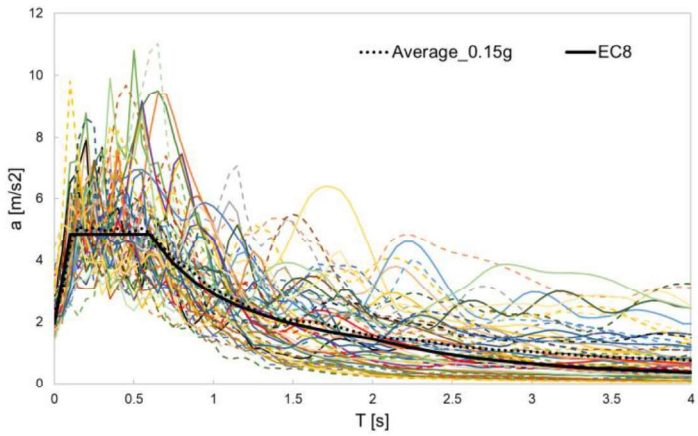


Fig. 3- Average and targeted elastic response spectrum for ground motion group of 30 records

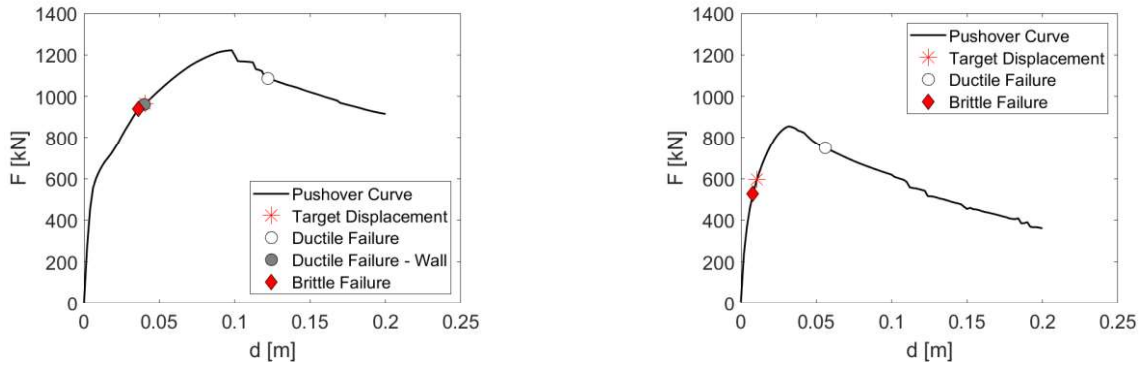
4. Seismic assessment according to EC8-3

The seismic safety of the buildings was evaluated in terms of the demand/capacity ratio (DCR) for each vertical structural element (columns and walls) following the procedure recommended in EC8-3 [5], which consists of comparing the chord rotation and shear demand with the capacity values of ultimate chord rotation and shear strength. The target displacement was determined with the N2 method, also as prescribed in EC8-3.

As established in EC8-3, the return periods to consider in the assessment of the existing buildings are: 73, 308 and 975 years for the damage limitation (DL), the significant damage (SD) and the near-collapse (NC) limit states, respectively. According to each of the limit states, the reference ground acceleration (agR) is then multiplied by a coefficient. Only the response spectrum for the Type 1 seismic action (far-field earthquake) was considered, therefore, the agR is 0.15g (Lisbon, Zone 1.3). The Portuguese Annex of EC8-3

[20] recommends that, for existing residential buildings, the assessment should be performed considering only the significant damage (SD) limit state.

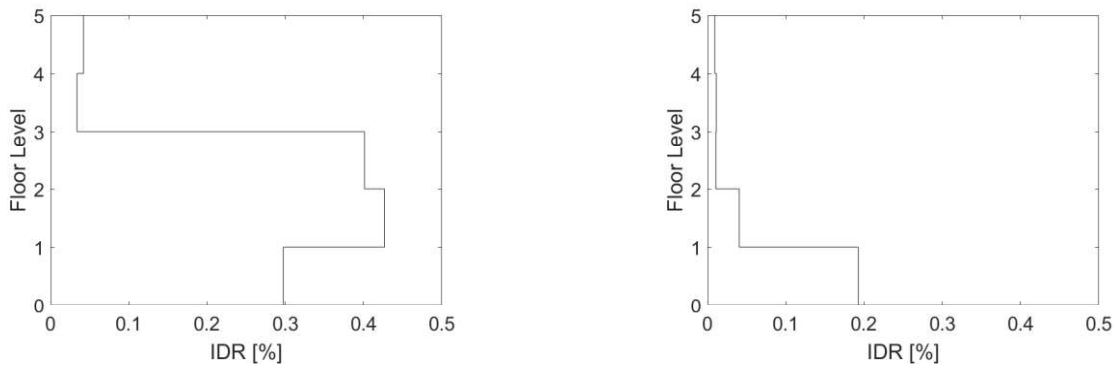
Fig. 4 shows the pushover curves obtained for the X and Y direction with the indication of the attainment of the SD limit state for the brittle and ductile failure and the target displacement. The performance of the structure for the Y direction presents less capacity, which can be explained by the absence of RC walls in this direction. Simultaneously, brittle failure is the conditioning mechanism for both directions corresponding to a displacement of 0.0036 m and 0.0079 m for the X and Y direction, respectively. The occurrence of this type of mechanism at early stages of lateral deformation is common in old RC buildings designed to resist gravity loads, as verified by [21].



(a) (b)

Fig. 4 – Pushover curves for the (a) X and (b) Y direction with the indication of ductile and brittle failure and the target displacement.

Fig. 5 shows the inter-storey drift profile in (a) X and (b) Y directions observed at the occurrence of the brittle failure. At it can be seen, no excessive deformations develop in both directions of the analysis, confirming a failure governed by strength parameters, for a low intensity level of 0.03g ($Sa(T_1)$=0.09g). In the X direction, the wall located at the first three storey levels, that takes most part of the loading, deforms more than the columns, while in Y direction, the first storey appears to be more flexible than the others, revealing the absence of infills at this level.



(a) (b)

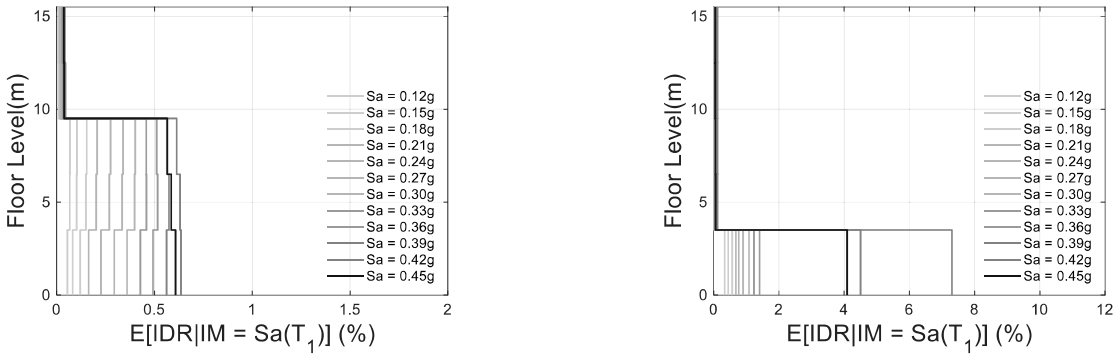
Fig. 5 – Inter-storey drift profiles at failure (a) X direction and (b) Y direction

5. Loss assessment

The methodology selected to perform the loss assessment of the studied RC building is proposed by [6], which is based on the PEER-PBEE framework [7] but specifically developed for the RC wall-frame building typology. The framework consists of applying four analysis (seismic hazard analysis, structural analysis, damage analysis and loss analysis) which will be briefly explained in this section.

This procedure starts with a probabilistic seismic hazard analysis to estimate the site's seismic hazard curve, λSa, i.e., the mean annual frequency of exceeding a specific ground motion intensity. The pseudo-spectral acceleration, $Sa(T_1)$, is suggested as the intensity measure [22]. In this work, T_1 was taken as the average of the first mode building vibration periods in X and Y directions, as proposed by FEMA P-58 for assessing the response of 3D buildings.

For the structural analysis of the building, an incremental dynamic analysis (IDA) was used to evaluate the mean response of the different engineering demand parameters (EDPs). Typically, for old RC wall-frame structures, these parameters are the inter-storey drift ratio (IDR), the peak floor acceleration (PFA) and the residual inter-storey drift ratio (RIDR). Figure 6 shows the mean inter-storey drift ratios (IDR) plotted for different levels of seismic intensity and for the X and Y directions of the case study building.



(a) (b)

Fig. 6 – Mean inter-storey drift ratio (IDR) as a function of the intensity levels for the (a) X direction and (b)Y direction

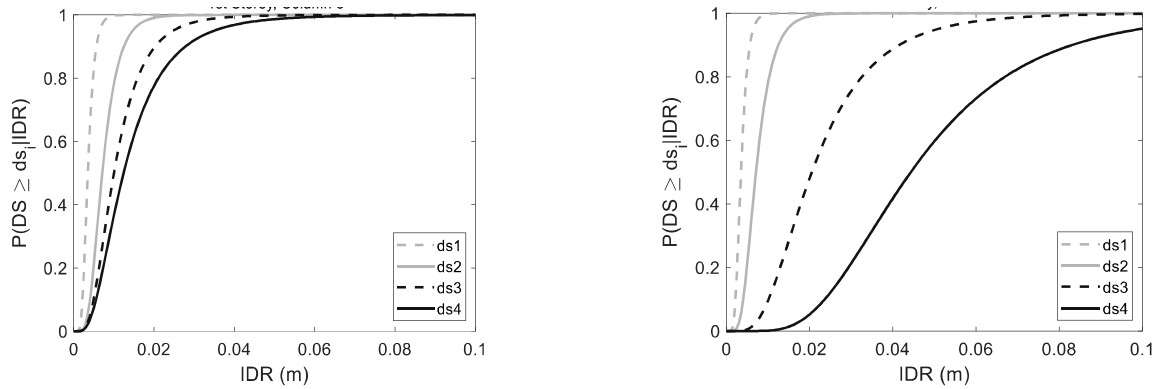
Fragility functions for RC Columns

The fragility functions selected for the RC columns are based on the experimental results obtained by [8], which establish levels of lateral deformations associated with the four damage states: DS1 (light damage) DS2 (moderate damage), DS3 (severe damage) and DS4 (collapse). According to this study, the columns shear failure is expected for a level of deformation that corresponds to DS3, and the collapse is characterized by a level of deformation that corresponds to the loss of vertical carrying capacity (DS4).

The fragility functions calculated for column 8 of the case study building are shown in Fig. 7. As it can be seen, the column at first storey level experiences a higher concentration of damage than at fifth level. For example, the probability of collapse for an interstorey drift of 0.06m is 100% at first storey, while is only 72% at fifth storey level.

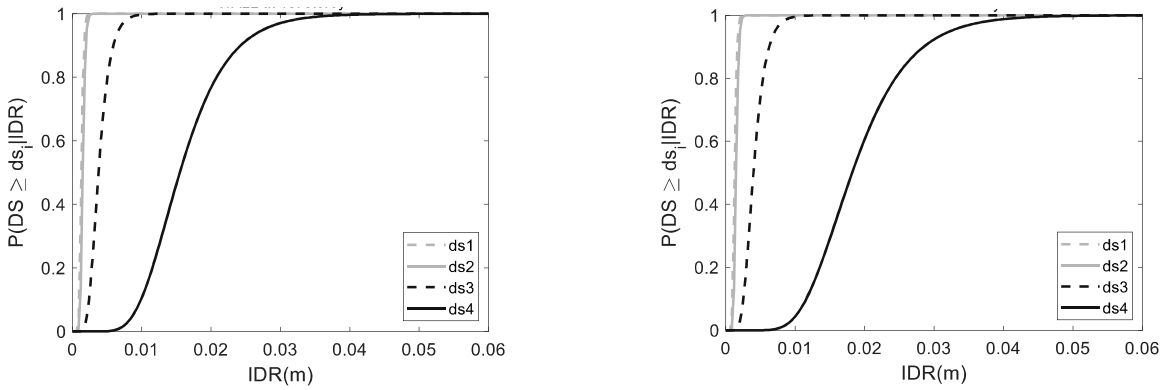
Fragility functions for RC Walls

The fragility functions used for the RC walls are the ones derived by [6] based on a parametric study performed to a slender RC wall representative of constructions without seismic detailing, with a longitudinal reinforcement ratio lower than 1%, a relatively low axial load ratio (N/Agfc) and limited ductility. Fig. 8 shows the fragility functions obtained for the RC wall at first and third storey levels of the case study building, which indicate that the level of damage is similar along the wall height.



(a) (b)

Fig. 7 – Fragility functions corresponding to four damage states column 8 at (a) first storey level (b) fifth storey level



(a) (b)

Fig. 8 – Fragility functions corresponding to four damage states for RC wall at (a) first storey level, (b) third storey level

The expected economic losses are obtained through the loss analysis. The total expected loss, as a function of the ground motion intensity, IM, is the sum of three components: (i) losses resulting if the building collapses, E[Loss|C], (ii) losses associated with repairs given that the structure has not collapsed (NC) at a given ground motion intensity, E[Loss |NC ∩ R, IM], and (iii) losses resulting from having to demolish the building due to excessive residual drifts, E[Loss |NC ∩ D, IM].

Equation (1) is used to express the building-specific relationship that relates ground motion intensity to economic monetary loss:

$$E[Loss_T|IM] = E[Loss|C]\cdot P(C|IM) + E[Loss|NC \cap R, IM]\{1-P(D|NC, IM)\}\cdot\{1-P(C|IM)\}+ E[Loss|NC \cap D, IM]\{P(D|NC, IM)\cdot\{1-P(C|IM)\} \quad (1)$$

where P (C|IM) is the probability that the structure will collapse under a ground motion intensity IM = im and P(D| NC, IM) is the probability that the structure will be demolished given that it did not collapse when subjected to an earthquake with intensity level IM = im.

Repair Losses

The expected value of the economic loss associated to the repair of the structure is given by the sum of the losses in structural and non-structural components. In general, the repair losses associated to the structural components represents about 25–30% of the building's total value, while the remaining costs are distributed among the non-structural (drift sensitive and acceleration sensitive) components.

The economic losses of the individual structural components are the losses associated to the repair or replacement actions required when a specific damage state has been observed in that component. The information about the repair actions and associated costs for to each damage state defined for both the RC columns and walls are described in [6].

For old residential buildings, past studies [23] indicate that the contribution to repair losses is mostly characterized by damage to drift sensitive non-structural component, e.g. masonry infills and partition walls, while the contribution of acceleration-sensitive non-structural components is practically negligible. The fragility and loss functions used to compute the repair losses associated to the drift sensitive components are the ones developed by Cardone and Perrone [24]. The fragility and loss functions associated to the acceleration sensitive non-structural components are the ones published by HAZUS [4]. The cost of acceleration sensitive non-structural components is estimated as a percentage of the total construction cost of the building and, as suggested by [25], a value of 20% of the total building cost was assumed.

Probability of Collapse

To estimate the total expected economic loss of the building as a function of the ground motion intensity, E[LossT|IM], using Eq. (1), it is required to estimate the probability of collapse as a function of the ground motion intensity, P(C|IM). The probability of collapse is essential to evaluate the total economic losses but also the collapse safety.

Based on the results obtained from the seismic assessment performed previously, which indicated a shear failure at the early stages of lateral deformation, the probability of collapse P(C|IM) was estimated considering two different approaches: (i) assuming the largest probability of any vertical structural element to reach its shear strength capacity, as recommended by EC8-3, and (ii) assuming the largest probability of any vertical structural element to reach the lateral deformation associated to DS4, which is associated to collapse.

The building structural analysis indicate that the first elements to reach the DS4 drift limit are the RC columns at the ground storey in the Y direction, while the shear strength capacity is attained by several RC columns and wall in the first two storeys, in both X and Y directions. The collapse fragility curves obtained for each approach are shown in Fig. 9. The results indicate a collapse probability of 100% due to shear failure of the vertical structural elements for an intensity level of $Sa(T_1)$ equal to 0.11g, while the same collapse probability due to the attainment of the DS4 drift limit is expected for an intensity level seven times higher, i.e., $Sa(T_1)$ equal to 0.77g. The difference between the collapse probability obtained by these two approaches, using a strength or a deformation criterion, has a huge impact in the collapse losses evaluation and, consequently, in the total losses.

The variation of the total expected losses, defined as a fraction of the replacement value of the building, as a function of the ground motion seismic intensity, computed considering a shear failure or the attainment of the DS4 drift limit are shown in Fıg. 10(a) and Fig. 10(b), respectively. The figure also shows the disaggregation of the total loss into the three components (repair, colapse and demolition components).

As it is evident from Fig. 10 (a), the contribution of repair losses and losses resulting from having to demolish the building due to excessive residual drifts are negligible when the collapse probability is estimated based on the shear strength capacity of the vertical structural elements. The total and collapse losses of the structure reach 100% for intensity levels, $Sa(T_1)$, higher than 0.11g while the repair and demolishing losses are 0%. Conversely, for the same intensity level, when the collapse probability is

determined based on the SD4 drift limit, the total and collapse losses are equal to 7% and 2%, respectively, and a repair loss of 6% is expected. The total losses are mainly dependent of repair losses because only slight to moderate damage is expected for this intensity level. The total loss of 100% is expected for an intensity level of $Sa(T_1)$ equal to 0.44g, which is slightly lower than the intensity level that corresponds to the reference peak ground acceleration in Lisbon ($Sa(T_1)$=0.46g or 0.15g). At this intensity level, the losses due to collapse are equal to 90% of the replacement cost of the structure, while the loss due to repair is practically 0% and the loss due to demolition is around 10%.
In terms of expected annual losses (EAL), the values of 3% and 0.73% are obtained for a probability of collapse estimated based on the structural elements shear strength capacity and the attainment of the SD4 drift limit, respectively. The evaluation of collapse is, therefore, vital, not only for the evaluation of the performance of structure but also to an accurate estimation of the losses.
Moreover, the results reflect an underestimation of the collapse probability and, consequently, the total losses evaluation, when the shear failure is incorporated in the loss methodology using fragility curves based on drift deformations (DS3 drift limit) instead of the shear strength verifications, as proposed by EC8-3.

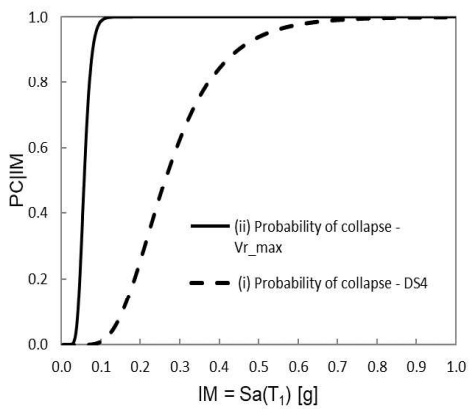
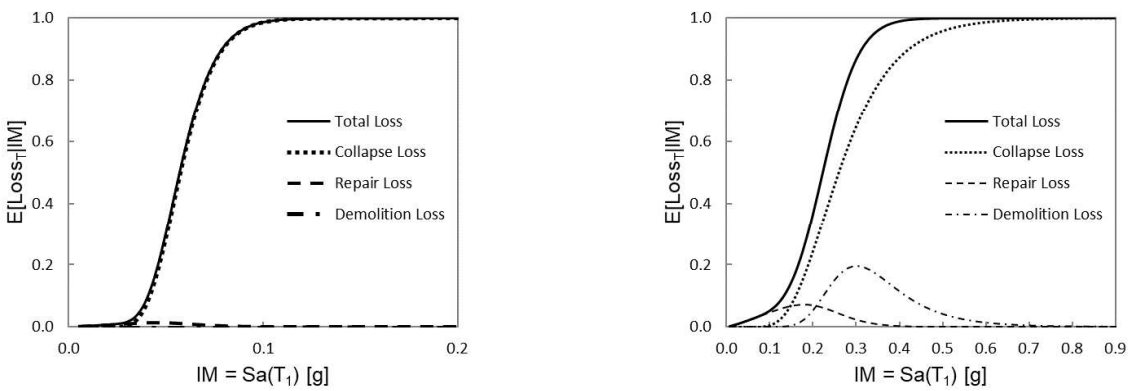


Fig. 9 – Probability of Collapse for the case study building



(a) (b)

Fig. 10 –Variations of the expected loss in the case study building as a function of the ground motion intensity, IM, considering (a) a shear failure of the vertical structural elements, (b) the attainment of DS4 drift limit of the vertical structural elements~

5. Conclusions

This work focused on the characterization and seismic assessment of old existing RC residential buildings. The characterization was based on data compiled in a survey conducted in the Alvalade area of Lisbon. Special attention was given to the structural characteristics that may be associated with the collapse in the case of a seismic event. A building representative of one of the building classes identified in the survey was selected and evaluated in terms of seismic performance and associated economic losses.

The seismic evaluation was made in terms of the demand/capacity ratio (DCR), following the procedure recommended in Part 3 of Eurocode 8 [5]. The results indicated that the seismic behaviour was controlled by the development of shear failure, in both directions, being the Y direction the most restrictive.

The economic losses were estimated using the methodology proposed by [6], developed for the RC wall-frame building typology RC buildings, although the probability of collapse was evaluated through two different approaches: (i) assuming the largest probability of any vertical structural element to reach its shear strength capacity, as recommended by EC8-3, or (ii) assuming the largest probability of any vertical structural element to reach the lateral deformation associated to DS4. The discrepancy observed in the results highlight the importance of an adequate incorporation of the shear failure assessment in the evaluation of both the collapse probability and the economic loss estimation.

6. Acknowledgments

The authors would like to acknowledge the financial support of Fundação para a Ciência e a Tecnologia (FCT, Ministério da Educação e Ciência, Portugal) through the FCT Research Program: MitRisk - Framework for seismic risk reduction resorting to cost-effective retrofitting solutions, POCI-01-01456-Feder-031865.

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