Simplified evaluation of seismic vulnerability of early 20th century masonry buildings in Lisbon

A. Simões & R. Bento
CERIS, ICIST, Instituto Superior Técnico, Universidade de Lisboa, Lisbon, Portugal

S. Lagomarsino
University of Genoa, Genoa, Italy

P. B. Lourenço
ISISE, University of Minho, Guimarães, Portugal

ABSTRACT: This paper evaluates the seismic behaviour of a group of masonry buildings through the application of a simplified method. The method is based on the definition of a vulnerability index supported on few characteristics of the buildings and provides an estimation of the expected damage on the buildings as a function of the intensity of the seismic action. The application of the method to a large group of buildings enables to conduct loss estimation studies to support seismic risk management. In this work, the method was applied to an aggregate of old masonry buildings in Lisbon. An overview of the procedure and its inherent uncertainties is presented. In the future, this method will be calibrated based on the results from detailed analytical models of the buildings. Then, the study would give an important contribution to generalize and apply the procedure to other aggregates of masonry buildings in Lisbon and to conduct a city scale seismic assessment.

1 INTRODUCTION

The assessment of existing buildings aims to verify the behaviour of the structure and to identify possible fragilities. This work addresses the seismic evaluation of an aggregate of unreinforced masonry buildings built in Lisbon, Portugal, in the beginning of the twentieth century. These buildings, vulgarly called “gaiolero”, correspond to the typology with highest structural weaknesses of the housing stock of the city. This fact is in part consequence of a period of real estate speculation, which occurred in the transition of the centuries and affected the quality of construction of the buildings, and also due to the lack of maintenance over the years, the change of use from housing to offices and the implementation of structural modifications to the original buildings. For instance, there are many buildings with internal walls replaced by steel or reinforced concrete beams to create larger spaces or buildings with the removal of masonry piers on the ground floor façade wall to open shop windows. The problem is that, in some situations, these modifications may have compromised even more the structural behaviour of these buildings.

The aggregate defined as case study was characterized in terms of architectonical and structural features based on the information available in the Municipal Archive of Lisbon and after in-situ inspection to the majority of the buildings. Starting from this survey, a simplified assessment method was applied to determine a vulnerability index, which provides a first estimation of the expected seismic performance of the buildings.

The method is based on the procedure from the Gruppo Nazionale per la Difesa dai Terremoti – GNDT II level approach (GNDT, 1994) proposed to evaluate the seismic behaviour of masonry buildings. Vicente (2008) adapted the method for the masonry buildings in Portugal and has applied it to different city centers: Coimbra (Vicente, 2008; Vicente et al., 2011), Seixal (Ferreira et al., 2013) and Faro (Maio et al., 2015).

Based on the vulnerability index attributed to each building, it is possible to generate vulnerability and fragility curves, which are function of different seismic intensity levels, and to determine the expect damage on the buildings. This paper provides an overview of the procedure and its application to an aggregate of old masonry buildings.

2 AGGREGATE OF BUILDINGS

The buildings under study are mid- to high-rise unreinforced masonry structures with flexible timber floors. The aggregate is limited by “Avenida da República”, “Avenida Visconde Valmor”, “Avenida 5 de Outubro” and “Avenida Elias Garcia” and is composed by 29 units (Fig. 1): 19 unreinforced masonry buildings, 9 reinforced concrete (RC) buildings and 1 unoccupied
unit (which used to be a masonry building, but at the moment, only the façade wall remains supported by a containment structure).

The characterization of the buildings was supported on the information gathered in the Municipal Archive of Lisbon which includes original drawings, modification drawings and reports from official inspections. In three cases it was not possible to obtain any drawings of the buildings. In other cases, the information was sometimes incomplete (e.g. absence of original and modification drawings or specification of the materials). In-situ inspections to the buildings were carried out to confirm the data collected and to verify the state of conservation of the buildings. It was not possible to visit seven buildings in a total of nineteen (three of these are unoccupied).

The buildings from this aggregate were built between 1908 and 1929. They have rectangular plan shapes which can be divided into four types: Type I – buildings with small size façade walls and one side air shaft (6 buildings); Type II – buildings with medium size façade walls and one air shaft (4 buildings); Type III – buildings with large façade walls and more than one air shafts (6 buildings) and Type IV – buildings on the corner of the aggregate (3 buildings). Buildings type IV have quite different dimensions. Two of them have radial symmetry but the other one has a quite irregular structure.

The foundation system of the buildings is defined by continuous rubble stone masonry walls with larger thickness than the load-bearing walls. In some cases, the building plan refer to the use of hard limestone masonry with air lime mortar with a lime:sand ratio of 1:2 or 2:5 (there is also one case with 5:9). The façade walls are made of rubble stone masonry and air lime mortar (with the same lime:sand ratio used in the foundation) and variable thickness along the height of the building. The front façade walls have typically 0.60 m to 0.90 m thickness on the ground floor level and are slightly thinner on the back façade (with a variation of 0.05 m to 0.10 m).

The side walls and the walls from the air shafts are made of clay brick masonry and air lime mortar. In some cases the use of hollow clay bricks is referred. On the other cases there is no reference to the type of brick used. The thickness of the side walls varies between 0.20 m and 0.50 m and is slightly larger on the air shafts varying between 0.28 m and 0.50 m. In both situations thickness of the walls is constant along the height of the building.

The internal walls are made of clay brick masonry. Again, in some cases the use of hollow clay bricks is stated. In two buildings these brick walls are reinforced by vertical and horizontal timber elements. The existing drawings suggest that the main internal walls have, in general, 0.15 m of thickness. The secondary or partition walls can be made of clay brick masonry or a light timber structure made of vertical and diagonal boards and crossed laths known as “tabique” walls. These walls have approximately 0.10 m of thickness.

Floors are made of timber beams covered by timber boards or by mosaic tiles on the kitchens and bathrooms. Pine wood (Pinus pinaster Ait.) was often used. The geometry of the beams ranges between 0.07 m and 0.08 m for the width and 0.16 m and 0.22 m for the height. The beams are placed perpendicular to the façade walls and are disposed parallel to each other distancing 0.35 m to 0.45 m. The beams are embedded on the façade walls and restrained by smaller beams in the perpendicular direction. The buildings are covered by pitched roofs composed by timber trusses placed perpendicular to the façade walls. In most cases the roofs have dormer or mansard windows which were also used for housing.

The balconies on the back façade walls are made of steel sections connected with jack arches. Commonly T sections with 0.06 m, 0.08 m or 0.10 m (both width and height dimension) are used. The span of the balconies varies between 1.0 m and 2.2 m. This floor structure is embedded on the façade wall and supported on a steel frame made with beams and circular cast iron columns. Steel staircases make the access the interior yard of the aggregates.

These buildings have experienced several structural interventions during their lifetime. These were mostly motivated by changes of use of the buildings or maintenance works that not necessarily improved the structural behaviour of the building. There is the record of sixteen buildings with the elimination of internal walls (Fig. 2). In three buildings larger shop windows
were opened at the ground floor (Fig. 3) and in one building a larger opening makes the access to a garage. These type of interventions introduce important weaknesses and stiffness variations on the buildings. In addition, depending on the orientation of the walls, the actual support to the masonry façade walls might be compromised. In addition, the removal of the masonry piers from the façade wall reduces the shear strength capacity in the base of the building. These actions generate artificial soft-stories which result on the support of a rigid element (upper floors) over weaker and flexible elements.

Still during the construction period, the increasing population in Lisbon and the real estate speculation influenced the alteration of these masonry buildings with the addition of new floors or the conversion of the pitched roofs for housing. A total of six buildings have an additional floor and in one building two new floors were added. The direct consequence is the increment of the weight of the structure at the top of the building resulting on the increasing inertia horizontal forces and displacements during the seismic action. The opening of semi-basements starting from the ventilation boxes on the ground floor is also common. These semi-basements have been also subjected to additional modifications with the elimination of walls. All these alterations have an impact on the foundation system and on the stability of the building.

There is no record about the replacement of the timber floors in these buildings. There is only one case where the floors from the kitchen and bathroom were replaced by jack arches equivalent to the ones used on the balconies from the back façade wall. However, it was verified that in ten buildings these balconies were replaced by reinforced concrete slabs.

3 VULNERABILITY INDEX METHOD

The seismic assessment of the aggregate was carried out by assigning a vulnerability index ($I_V$) to each building. This is based on the weighted sum of specific parameters that influence the seismic performance. The method was initially defined by the GNDT and was supported on post-earthquake damage observations of masonry buildings in Italy (GNDT, 1994). Vicente (2008) adapted the method for the Portuguese masonry buildings and introduced some modifications. The parameters were reorganized into four groups and additional information was introduced for a more clear classification of the parameters. Three new parameters were also introduced accounting for the interaction between buildings and other overlooked building features.

The current version of the method (Vicente et al., 2011) comprehends a total of fourteen parameters characterized with a weight ($p_i$) ranging from 0.50, for the least important parameters, to 1.50, for the most important in terms of the seismic behaviour (see Table 1). Each parameter is then related to one of four classes of growing vulnerability: $A = 0$, $B = 5$, $C = 20$ and $D = 50$ depending on the intrinsic characteristics of the building. The vulnerability index ($I_V$) results, as referred, from the weight sum of these parameters. For convenience, the index is afterwards normalized to the range 0–100.

In addition to the definition of three new parameters ($P_5$, $P_7$ and $P_{10}$), Vicente et al. (2011) also revised the weight ($p_i$) attribute to the parameters based on experts’ option. The definition of these weights is an important source of uncertainty of the method and should be validated based on more detailed study (e.g. results from numerical analyses) which will be defined in future work.

The attribution of the vulnerability class is supported on the information about the buildings (drawings and in-situ inspections). However, due to the difficulties in accessing to the interior of all buildings,
some hypotheses had to be considered in the equation. Some features of the buildings were supported on expert opinion or derived from the comparison with the similar buildings in order to attribute a vulnerability class to all the parameters and obtain in this way the vulnerability index of all buildings.

Figure 4 plots the variation of the vulnerability index ($I_V$) for this group of buildings. The index ranges between 38.7 and 74.0 being the mean value equal to 54.0. From the 19 buildings analysed, 15 (78.9%) have a vulnerability index higher than 45 and, due to this, should be considered for a more detailed seismic analysis (Vicente et al., 2011). In fact, these first results highlight the high seismic vulnerability of these unreinforced masonry buildings.

Figure 5 presents the distribution of the vulnerability classes (A to D) for each parameter. Parameters P1 – Type of resisting system, P9 – Height regularity and P11 – Horizontal diaphragms are the most critical features as more than 80% of the cases were classified with class D. As to the parameters with higher weight ($p_r$) – P3, P5 and P7 – there is not a prevailing vulnerability class. Nevertheless, it is important to highlight that in case of in parameter P3 – Conventional strength, 5.3% of the buildings were classified with class B, 26.3% with class C and 68.4% with class D showing the limited strength of the buildings in general.

Figure 6 shows the distribution of the vulnerability index on the aggregate where it is possible to conclude that the buildings from the end of the row have a higher vulnerability index. In addition, it is evident that the vulnerability is independent of the building configuration (type I – IV).

4 SEISMIC ASSESSMENT

Starting from the vulnerability index obtained for each building it is now possible to generate vulnerability and fragility curves, which are function of different seismic intensity levels, and to determine the expect damage on the aggregate of buildings. A combination between the vulnerability curves proposed by Petrini (1993) within the GNDT II level approach (1994) and the macroseismic method defined by Giovinazzi & Lagomarsino (2004) is considered to this end.

Petrini (1993) proposed vulnerability curves based on damage surveys after the earthquakes of Friuli (1976) and Abruzzo (1984) in Italy. These vulnerability curves correlate the damage ($d$ – ratio between the cost of repairing and the cost of rebuilding the building), the seismic intensity expressed in terms of the Peak Ground Acceleration (PGA) and the vulnerability index ($I_V$) normalized to the range 0–100.

The macroseismic method (Giovinazzi & Lagomarsino, 2004) was developed to evaluate the seismic vulnerability of buildings located in European regions and towns. The method is based on the European Macroseismic Scale – EMS-98 (Grünthal, 1998) and was created to resolve the uncertainties related with the EMS-98 Damage Probability Matrixes (DPM).

Originally these DPM were defined with six vulnerability classes with decreasing vulnerability (from A to F) to which the probability of five Damage States (DSk with $k = 1, \ldots, 5$) was expressed by the linguistic terms “few”, “some” and “many” for a given macroseismic intensity degree ($I_{EMS-98}$ from I to XII).
damage states are defined as follows: DS1 – slight damage, DS2 – moderate damage, DS3 – extensive damage, DS4 – near collapse and DS5 – collapse.

The macroseismic method solved the incomplete damage distribution and the imprecise determination of the damage probability by applying the fuzzy set theory and the binomial distribution (Lagomarsino & Cattari, 2014). The vulnerability is expressed by a vulnerability curve which gives the mean damage \((0 < \mu_D < 5)\) in terms of the macroseismic intensity \((I_{EMS-98})\) as defined in Equation 1:

\[
\mu_D = 2.5 + 3 \tanh \left( \frac{I_{EMS-98} + 6.25V - 12.7}{Q} \right)
\]

where \(V\) is the macroseismic vulnerability ranging between 0 and 1 and \(Q\) is the ductility index. Vicente (2008) states that the ductility index \(Q\) can vary between 1.5 and 3.0 for masonry buildings. In this work it was considered equal to 2.3 as in the original macroseismic method and assuming that the buildings have moderate ductility.

The vulnerability curves from the GNDT II level approach correlate the damage \((d)\) with the seismic intensity (in PGA) for each vulnerability index \((I_F)\). The vulnerability curves from the macroseismic method correlate the mean damage \((\mu_D)\) with the macroseismic intensity \((I_{EMS-98})\). In order to combine the vulnerability curves from the two methods it is necessary to adopt relationships between the seismic intensity (PGA) and the macroseismic intensity \((I_{EMS-98})\) and between the damage \((d)\) and the mean damage \((\mu_D)\).

In the first case, the relationship proposed by Lagomarsino (2008) was adopted as it provides a good approximation in comparison with the other laws proposed in the literature (Equation 2).

\[
I_{EMS-98} = 5 + \frac{\ln(PGA) - \ln(0.03)}{\ln(1.8)}
\]

In the second case, the relationship is based on the occurrence probability of damage in terms of economic losses and consequences as defined in Equation 3:

\[
d = \sum_{k=0}^{5} w_{c,k} \times p_{DSk}
\]

where, \(w_{c,k}\) is a weighted factor (different values are proposed in the literature) and \(p_{DSk}\) is the probability of having a certain Damage State DSk. This probability follows the binomial probability distribution, as proposed in Lagomarsino & Cattari (2014) and is given by Equation 4, which in turns depends on the mean damage \((\mu_D)\):

\[
p_{DSk} = \frac{5!}{k!(5-k)!} \left( \frac{\mu_D}{5} \right)^k \left( 1 - \frac{\mu_D}{5} \right)^{5-k} (k = 0,\ldots,5)
\]

The weighted factors \((w_{c,k})\) proposed in HAZUS (FEMA, 1999) were considered in this study following the suggestion from previous works with masonry buildings in Portugal (e.g. Vicente et al., 2011 and Ferreira et al., 2013). The values proposed in HAZUS (FEMA, 1999) are: \(w_{c,0} = 0; w_{c,1} = 0.02; w_{c,2} = 0.10; w_{c,3} = 0.50\) and \(w_{c,4} = w_{c,5} = 1.00\).

After plotting the mean damage \((\mu_D)\) as a function of the damage \((d)\) the following relationship was determined (Equation 5):

\[
\mu_D = 4.66d^{0.60}
\]

Following the previous steps, the vulnerability curves for the 19 masonry buildings were obtained and are presented in Figure 7. The curves corresponding to the higher macroseismic vulnerability classes: A \((V = 0.88)\) and B \((V = 0.72)\) were also included in Figure 5 putting in evidence the high vulnerability of these buildings.

The relation between the vulnerability index \((I_F)\) and the macroseismic vulnerability \((V)\) was defined by applying the least square method to the central mean damage value \((\mu_D = 2.5)\) to each building, as suggested in Vicente et al. (2011). Finally, a correlation between these two parameters was obtained (Equation 6).

\[
V = 0.0061 \times I_F + 0.5542
\]

The macroseismic vulnerability \((V)\) ranges between 0.79 and 1.00 being the mean value equal to 0.88. In fact, there is one building with macroseismic vulnerability equal to 1.00, which is the maximum possible value.

In what concerns possible seismic scenarios and the expected consequences to these buildings, Figure 8 plots the mean damage \((\mu_D)\) distribution for the Eurocode 8 (CEN, 2010) seismic action type 1.3 with a PGA = 0.15. On this, the mean damage \((\mu_D)\) ranges between 2.43 and 4.02. According to Vicente et al. (2014), buildings with mean damage \(\mu_D \geq 3\) are expected to suffer severe damage and, therefore, require reassessment with a more detailed methodology. In this case, that corresponds to 15 buildings out of a total of 19. This outcome shows that for this specific
aggregate of buildings the level of damage associated to the seismic code action is considerably high, indicating again the low resistance of these unreinforced masonry buildings.

Fragility curves are another way to represent the expected damage of the buildings. These are defined through a continuous probability function, expressing the conditional cumulative probability of reaching or exceeding a certain Damage State $D_{Sk}$. The fragility curves for different Limit States $L_{Sk}$ are generated based on Equation 7:

$$P_{L_{Sk}} = \sum_{k=1}^{S} P_{D_{Sk}} (k = 1, ..., S)$$  \hspace{1cm} (7)

Figure 9 plots the fragility curves obtained for the average macroseismic vulnerability ($V$) of the 19 buildings, which is equal to 0.88, as a function of the seismic intensity (in PGA).

Figure 10 gives the damage distribution for the seismic action type 1.3. As expected there is a high probability of having damage in these buildings for this action. Taking as reference the average macroseismic vulnerability $V = 0.88$, the probability of having extensive damage ($D_{S3}$) is equal to 34%, the probability of having the near collapse of the buildings ($D_{S4}$) is 28% and the probability of collapse is 9%. These results were obtained for the average macroseismic vulnerability of the aggregate and can only be reduced in case of an effective strengthening intervention on these buildings.

5 CONCLUSIONS

The paper presents a simplified assessment of the seismic vulnerability of an aggregate of masonry buildings. It is based on fourteen parameters that influence the seismic performance of the buildings. The procedure was previously proposed by Vicente (2008) for masonry buildings in Portugal and was applied for the first time to the masonry buildings built in the early twentieth century in Lisbon, which are classified as the most vulnerable typology of buildings from the housing stock of the city.

In fact, the results presented in this paper support these conclusions for this specific aggregate of buildings. The level of damage associated with a possible seismic code action is considerably high. From the 19 buildings studied, 15 buildings are expected to have severe damage (mean damage $u_D \geq 3$). If the results are analysed in terms of fragility curves and considering the case of the average macroseismic vulnerability of the buildings the expected probabilities of damage are: 28% of having heavy damage, 34% of having near collapse and 9% of having collapse of the buildings.

The evaluation was carried out by the definition of a vulnerability class to fourteen different parameters based on the information about the buildings (drawings and in-situ inspections) and some expert judgement. An open subject is the definition of the weight that is attributed to each parameter and the importance of performing a more detailed study to calibrate these values.

Even taking into account the inherent uncertainties and limitations of the method, it is evident that the need of additional measures to reduce the seismic vulnerability of these buildings. In the future, this simplified method will be calibrated based on the results from detailed analytical models of the buildings. Then, this study would give an important contribution to generalize and apply the procedure to other aggregates of masonry buildings in Lisbon and to conduct the seismic assessment at the scale of the city. The final aim is to conduct loss estimation studies to support the seismic risk management and mitigation.
ACKNOWLEDGEMENT

This work was financially supported by Fundação para a Ciência e a Tecnologia (FCT, Ministério da Educação e Ciência, Portugal) through the scholarship PD/BD/106076/2015 within the FCT Doctoral Program Analysis and Mitigation of Risks in Infrastructures – INFRARISK- (more information at http://infrarisk.tecnico.ulisboa.pt).

REFERENCES


