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Seismic evaluation of old masonry buildings. Part I: Method description and application to a case-study

Rafaela Cardoso*, Mário Lopes, Rita Bento

DECivil, Instituto Superior Técnico, Av Rovisco Pais, 1, 1049-001 Lisbon, Portugal

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Abstract

This paper describes a method developed to evaluate the seismic performance of old masonry buildings, which allows identifying the expected structural collapse mechanism of the structure. The collapse mechanism is identified by the accumulation of several damaged structural elements in specific points of the structure. The methodology allows simulating the non-linear behaviour of masonry buildings by making use of an iterative procedure, where the structure is changed at each step according to the cracking, yielding or collapse of structural elements at the previous steps. The method was applied to an old masonry building from the city of Lisbon that includes a three-dimensional timber structure enclosed in masonry walls aimed at providing seismic resistance. Discussion is made regarding the advantages of the iterative procedure for the identification of the expected structural collapse mechanism of old masonry buildings. The method limitations will also be discussed.

Keywords: Seismic vulnerability; Old masonry buildings; Iterative method; Wood structure

1. Introduction

Old masonry buildings are an important percentage of the building stock of many cities. These buildings are still being used and their main functions at present days are mostly housing and services (offices of companies and banks). The importance of the preservation of the cultural heritage and the functions that old masonry structures still maintain in our days justify the concern about their structural safety, including under earthquake actions. Recent earthquakes showed a deficient performance of masonry buildings under seismic actions. Poor seismic performance of these buildings may be expected due to the following reasons: (i) age and consequent degradation of structural materials leading to a decrease of local and global stiffness and strength; (ii) the high number and variety of structural changes that these structures suffered during service time, without considering the effect on the seismic performance; (iii) most of these buildings were not built according to seismic codes because they did not exist at the time of their construction.

The design of strengthening and/or large modifications of old masonry buildings to perform new functions leads to the definition of a numerical model of the building. Besides the correct characterization of strength and stiffness parameters of structural materials (masonry and wood), there are also difficulties in modelling its non-linear behaviour. In fact, the behaviour of a masonry structure is non-linear mainly due to crack opening and to the rupture of connections between structural elements. Considering the existence of timber structural elements, the rupture of connections between timber elements and masonry walls is also a source of non-linear behaviour.

The proposed method was developed aiming at (i) accounting explicitly for most of the main sources of non-linear behaviour of masonry structures and (ii) being a tool applicable in current design practice of seismic assessment and strengthening of old masonry buildings. For these reasons, non-linear analysis is performed by an iterative method where each step is a linear three-dimensional analysis of the structure. The structure analysed at each

* Corresponding author. Tel.: +351 218418265; fax: +351 218497650.
E-mail address: rafaela@civil.ist.utl.pt (R. Cardoso).

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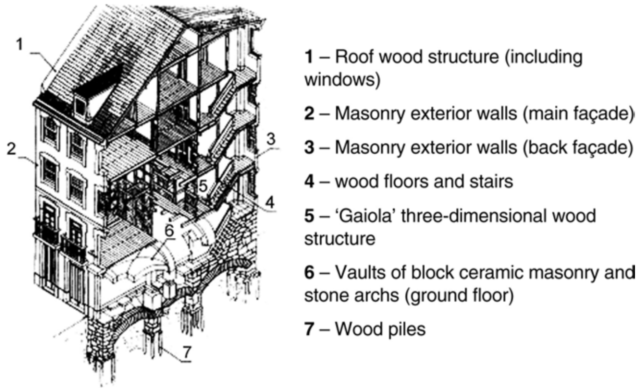


Fig. 1. Structural details of a 'Pombalino' building (*adapted from* [2]).

iteration results from the structure analysed at the previous step with the removal of collapsed connections or stiffness changes due to cracking or yielding occurred at that step. Calculations will be performed with a commercial program (SAP2000® [1]) and linear three-dimensional dynamic analyses will be performed considering the seismic action defined by response spectra.

Discussion will be made about the results of the application of the methodology to the analysis of an existing building from Lisbon Downtown. The building analysed is a 'Pombalino' building, which is an old masonry building with timber structural elements enclosed in masonry. The criterion used to define the collapse of the connections and the collapse of the structure will also be discussed in this paper.

2. Description of the building

'Pombalino' buildings, as shown in Figs. 1 and 2, are old masonry buildings that can be identified by the presence of a three-dimensional timber structure named 'gaiola pombalina' enclosed in interior masonry walls above the first floor. The other interior walls (partition walls) are wooden panels without structural functions. Façades are made of masonry columns and beams without the 'gaiola' structure. The doors and windows define the geometry of the façade masonry structural elements. Roofs are made with timber truss and ceramic tiles and may include window openings. Floors are wood slabs and should be considered as flexible diaphragms. Ground floor interior walls are masonry walls supporting a system of vaults made of blocks of ceramic masonry and stone arches. Foundations include short and small diameter woodpiles connected by a wood grid (Fig. 1).

The wood structure of 'gaiola' is like a birdcage made of vertical and horizontal elements braced with diagonals named St Andrew's Crosses, as shown in Fig. 3. The conception of the 'gaiola' three-dimensional wood structure aimed at providing resistance to horizontal forces. The connections between timber elements of the floors and 'gaiola' and masonry walls are supposed to be done with iron elements. Nevertheless, experience shows that sometimes

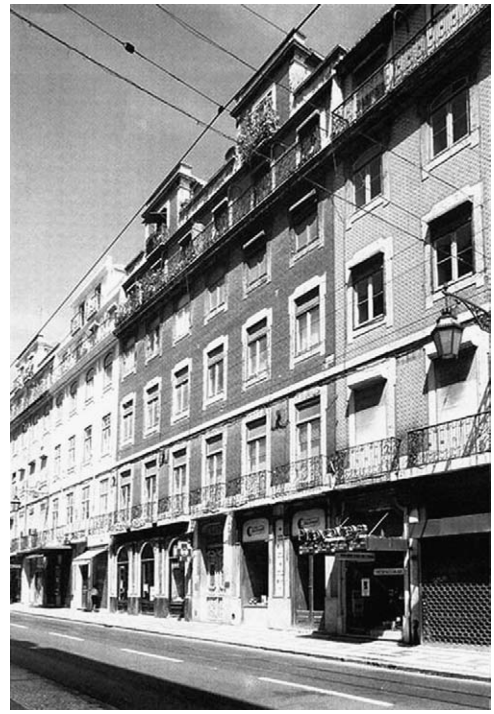


Fig. 2. 'Pombalino' building from Lisbon Downtown [3] (Prata Street, 210 to 220).

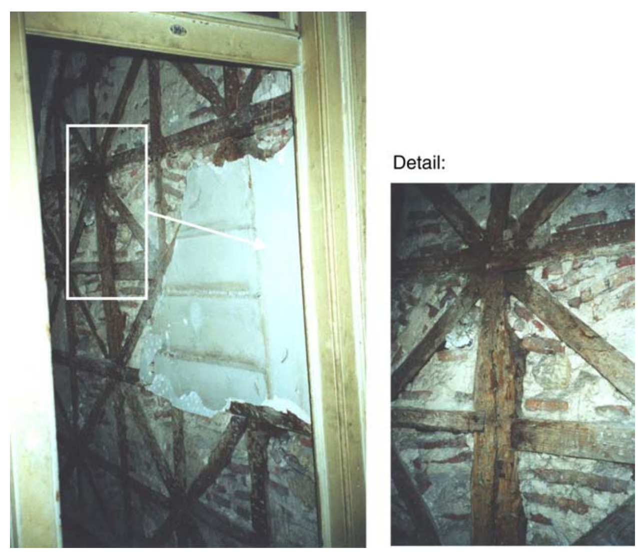


Fig. 3. Timber elements of 'gaiola' enclosed in masonry interior walls.

these elements do not exist. Most of the time, timber elements of 'gaiola' are notched together or connected by nails or iron ties, according to historical information about construction techniques [4]. Therefore, there are some doubts about the possible intention of allowing the overturning of the façades after detaching from the interior 'gaiola' walls, in order to avoid complete collapse of the building. This conception may be efficient for one- or two-floor buildings, as is shown in Fig. 4, which represents a building whose exterior walls have fallen out-of-plane

Fig. 4. Fall out-of-plane of the exterior walls without complete collapse (1998 Azores earthquake).

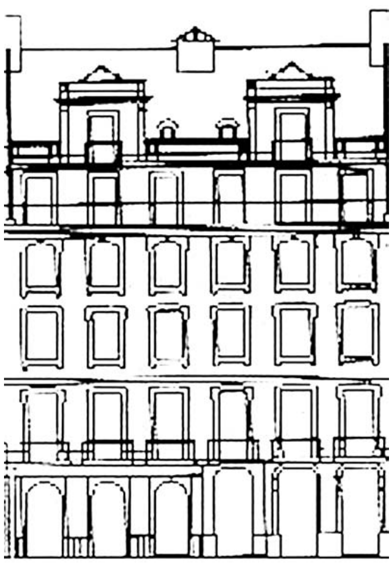


Fig. 5. Main façade of ‘Pombalino’ building from Lisbon Downtown (Prata Street, 210 to 220) [3].

in the 1998 Azores earthquake. However, there are some uncertainties regarding its efficiency in buildings with more than two floors because the out-of-plane fall of the façades may bring down other parts of the building.

Figs. 5 and 6 show, respectively, the main façade and the plan of the five-floor ‘Pombalino’ building that was analysed. The building is located at Lisbon Downtown (Prata Street, 210). According to original conception of Lisbon Downtown, rebuilt after the 1755 earthquake, Pombalino buildings should have similar characteristics, such as number of floors, spans, materials, structural conception, in order that, in each block, all buildings would perform in a similar manner. Therefore, the chosen building was analysed as a single building, assuming that the error by disregarding interaction between adjacent buildings is small, as is also discussed by Ramos and Lourenço [5].

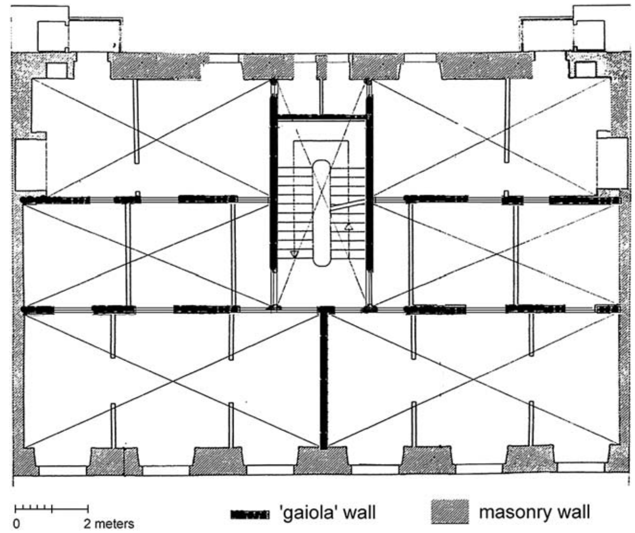


Fig. 6. Plan of a floor (above ground floor) of a ‘Pombalino’ building from Lisbon Downtown (*adapted from* [3]).

The masonry of the exterior walls is made of irregular blocks of calcareous stone and lime mortar with very poor strength capacity. Masonry infill of the ‘gaiola’ can be stone (rubble) or clay bricks, similar to the bricks used at ground floor vaults. It is usual to find both type of masonry at interior walls. The wall’s average thickness is 0.8 m for masonry exterior walls, 0.18 m for the interior walls of ‘gaiola’ and 0.12 m for the other interior walls. A more detailed description of ‘Pombalino’ buildings can be found in [6].

3. Numerical model

As mentioned before, a commercial program was used (SAP2000®) in order to perform the iterative procedure adopted, where linear dynamic modal analysis by response spectrum is used at each step.

Fig. 7 shows the numerical model of the building. Masonry exterior walls were simulated by thin bi-dimensional elements (shell elements) considering only bending deformation in and out of plane. For the interior walls of ‘gaiola’, only timber elements were considered. These were simulated by bars that transmit only axial forces—rotations are free at the connections. The exclusion of masonry from the model of the interior walls was decided after a previous study [7] where experimental results of ‘gaiola’ panels were compared with the results of numerical models of these panels, modelled with timber and masonry elements with different meshes and sizes. This study led to the conclusion that the stiffness of the panels obtained with the numerical model and considering timber and masonry elements was twice the one obtained in the experimental study. However, the numerical and experimental stiffness would be similar if, in the numerical model, masonry elements were removed and the connections of diagonal timber elements under tension were not considered.

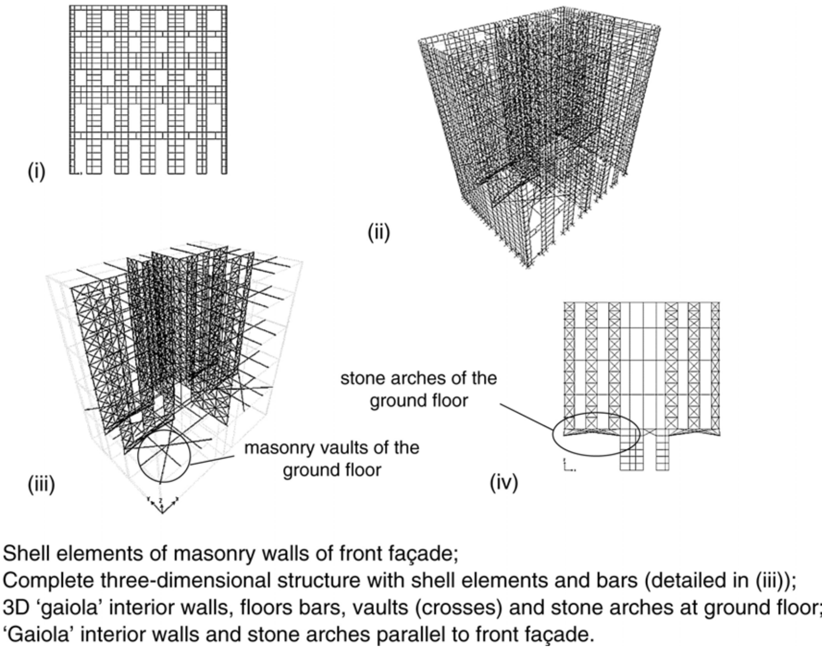


Fig. 7. Numerical model of the building [7].

In fact, historical data regarding construction practices and the observed gaps between the different timber elements of 'gaiola' and the absence of iron elements at the connections indicate that the connections of the diagonal bars could not transfer tensile forces. This study also contributed to the decision on mesh dimensions for masonry elements. It was concluded that it was not worth refining the mesh for masonry elements more, bearing in mind that the level of accuracy does not need to go beyond the accuracy in the evaluation of the material properties as input. Moreover, the main objective of studying crack/crush in masonry elements was the evaluation of the extension and location of masonry damage, since it is fundamental to identify the type of collapse mechanism. The mesh size adopted seems to be appropriate for this purpose since the results obtained are similar to the collapse observed in buildings of the same type, also presented in this paper (Fig. 4).

The floors were modelled as truss bars with free rotations at the connections to the walls, simulating flexible diaphragms and restraining out-of-plane relative displacements of parallel walls. The connections between timber elements of the 'gaiola' and perpendicular masonry walls were simulated considering short bars that only resist axial forces, intending to simulate the strength of the connection. No iron elements were considered in the evaluation of the strength of the connections due to the uncertainties about their real existence in the buildings.

As can be seen in Fig. 7, the roof structure was not included in the model. Its self-weight was assigned to the nodes at the top of the building. Two rigid diagonal bars (Fig. 7(iii)), with free rotations in their connections to masonry walls, simulated the masonry vaults of the ground floor. A triangular truss of rigid bars (Fig. 7(iv)) modelled the stone arches of ground floor. The connections between these bars allow relative rotations transmitting compression forces simulating the arch effect. These arches were connected to the interior 'gaiola' walls of the first floor parallel to the main façade and were supported by ground floor masonry walls. The foundations were simulated by built-in connections. Since the pavements cannot be considered rigid in their own plan, the mass was of each floor was distributed by the nodes of that floor.

Table 1 presents the Young's modulus, E, of the structural materials adopted in the numerical model. The Poisson coefficient of all materials was assigned the value 0.2.

According to the Portuguese Code [9], a uniform service load (1.2 $\mathrm{kN/m^2}$) acting at all the floors was considered. The seismic action was based on the response acceleration spectrum also defined in the mentioned code, acting along the two horizontal directions.

4. Methodology of analysis

4.1. Introduction

The aim of this analysis is to evaluate the building's potential seismic performance, establishing a relation between the intensity of the seismic action and different damage states up to collapse.

In this study, the seismic action was modelled by means of acceleration response spectrum, scaled by a parameter γ_{sis} that, in this manner, defines the intensity of the seismic action. Therefore, in this work, the intensity of the seismic action designates the scale factor of this action (the level of seismic action) and not the usual Mercalli seismic intensity. The reference acceleration response spectrum used in the

Table 1
Properties of structural materials considered in the numerical model

	Materials and structural elements	Young's modulus E (MPa)
Masonry	Façades and walls between buildings	600[a]
	Shell elements between perpendicular masonry walls	150[a]
Timber	Bars of 'gaiola'	8000[a]
	Bars of the connections between timber elements and masonry walls	
	Bars of the floors	
Stone	Arches and vaults (ground floor)	3000[b]

[a] Values adopted are discussed in [8].
[b] Calcareous stone: value adopted is discussed in [7].

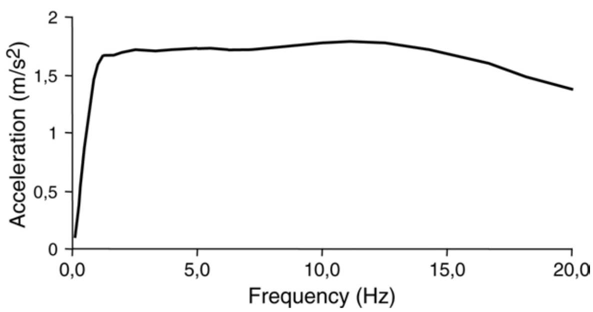


Fig. 8. Response spectrum of accelerations according to Portuguese Code [9] (Seismic Action type 2, Soil type III—soft soil).

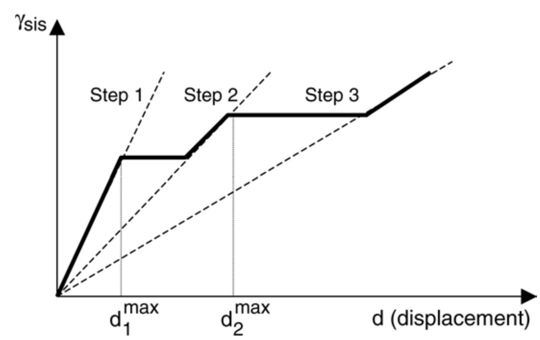


Fig. 9. Relationship between γ_{sis} and displacement d, considered in the iterative method.

analysis performed in this work, corresponding to $\gamma_{\text{sis}} = 1$, is presented in Fig. 8. It is intended to represent a far-distant earthquake, with low frequency contents, which is the one that induces higher accelerations in these type of structures. The average peak ground acceleration of accelerograms associated with that response spectrum is $0.12g$.

Changing the value of the parameter γ_{sis} allows the consideration of different intensities of the seismic action. The purpose of the analyses is to obtain the value of γ_{sis} that defines the intensity of the seismic action corresponding to the collapse of the structure, quantifying, in this manner, its seismic resistance. The value of γ_{sis} obtained at the end of each analysis is called $\gamma_{\text{sis}}^{\text{max}}$.

Since it was also intended to develop a methodology that can be used in current design practice, it would be useful that the analyses could be done with commercial programs that perform essentially linear dynamic analyses. This restriction led to the choice of an iterative procedure — a step by step procedure — in order to simulate, in an approximate manner, the non-linear behaviour of the structure. In fact, by introducing a number of changes in structural configuration, it is possible to simulate the main sources of non-linear behaviour: (i) cracking of the masonry elements and (ii) failure of the connections between the timber elements and masonry walls. It is also possible to simulate the respective sequence of ruptures, which is relevant in the structure global behaviour as the rupture of some structural elements can lead to the rupture of others. Masonry mechanical behaviour in compression is also non-linear but it is usual to represent masonry compressive stiffness by mean of a constant secant Young's modulus up to a given stress level. This approach is considered quite reasonable, given the large variability of material properties and, moreover, can be easily implemented by means of linear analyses.

The intensity of the seismic action whose effects lead to cracking or rupture of structural elements corresponds to a given value of factor γ_{sis}. If γ_{sis} obtained for each iteration is expressed as a function of a given control variable, for instance a displacement, d, the relationship may be as shown in Fig. 9. It can be observed that the building's global stiffness (the slope of each step line) is reduced during the iterative process due to the evolution of damage states.

Since in each step the seismic action is fully considered, the effect of the duration of the seismic action is neglected. If the seismic action was defined by an accelerogram, this would be equivalent to starting each step from the beginning of the accelerogram and not from the part of the accelerogram where the last step finished. Obviously this is a conservative procedure, especially if the accelerogram has few peaks with amplitude similar to the maximum amplitude. This problem can only be accounted for in time history analyses, which is obviously more complex. It is probably much less important in the case of long duration accelerograms with several peaks with amplitude similar to the maximum, which is the base of large distance earthquakes.

Another disadvantage of the method is that it does not account directly for hysteretic energy dissipation. Thus, in this type of analysis, this phenomenon can only be accounted for in an approximate manner, for instance by means of changing the viscous damping coefficient, and/or by means of a behaviour coefficient (q-factor in EC8 [10]). Cyclic tests on real 'gaiola' panels [11] have shown that these elements possess reasonable energy dissipation capacity. This may be either due to friction between elements of the panel, displacements associated with opening and closing of gaps between the timber elements of the 'gaiola', or to metallic elements at the connections. However, this is not well characterized yet; therefore, the energy dissipation capacity was directly accounted for by means of a constant relative viscous damping coefficient of 10%. This may be a conservative procedure, especially if large deformations of the 'gaiola' take place before rupture. It is thought that a q-factor of 1.5 can be considered in this situation.

The proposed method cannot be used in regular block masonry buildings because, for these structures, the non-linear behaviour of the joints, as well as the geometrically non-linearity, should be included in the analyses.

The method, besides making use of accessible computational tools, has the advantage of identifying the weakest links of the structure. Therefore, it provides useful information about the potential efficiency of possible strengthening interventions, as discussed in a companion paper (Bento, R, Cardoso, R, and Lopes, M, 2005—Seismic evaluation of old masonry buildings. Part II: Analysis of strengthening solutions for a case study, Engineering Structures, 2005).

4.2. *Description of the iterative method*

The sequence of the iterative procedure is as follows.

1. A linear model of the structure is defined, intending to represent the existing structure.
2. A linear dynamic analysis is performed, in which the seismic action is defined according to the prescribed acceleration response spectrum (γ_{sis} does not scale this response spectrum).
3. One value of γ_{sis} is set, assuming a given intensity of seismic action acting in the structure:
   - The design action effects (internal forces) in structural elements, F_{Sd}, are defined by Eq. (1), where F_{Perm} are the effects of vertical permanent loads and F_E are the effects of the code prescribed seismic action. Therefore, the value of the permanent load effects is constant and only the value of earthquake effects is scaled by means of parameter γ_{sis}.

     $$F_{\text{Sd}} = F_{\text{Perm}} \pm \gamma_{\text{sis}} F_E. \tag{1}$$

   - For each structural element or connection, the design action effects, F_{Sd}, will be compared with respective resistances, F_{Rd}, identifying their rupture, cracking or yielding if $F_{\text{Sd}} \geq F_{\text{Rd}}$.
   - A damage level is defined, according to the cracking, yielding or collapse of a set of elements or connections in the structure, as presented further in this section.
   - It is admitted that, if the rupture of the set of elements that define the current damage level can change structural behaviour, as explained soon after in this section, a damage state is identified. In this case, the associated value of γ_{sis} is named $\gamma_{\text{sis}}(n)$. If not, γ_{sis} is incremented and step 3 of the iterative process is repeated.
4. The damage state for $\gamma_{\text{sis}}(n)$ is modelled by removing collapsed elements or connections or reducing the stiffness of the elements or connections which suffered significant yielding or cracking. The stiffness can be reduced gradually or all at once, depending on how sudden is the effect been modelled. The number of elements to be removed at each time depends on the precision required in the analysis.
5. If the accumulated damage is considered to correspond to structural collapse, the process stops. If not, the previous sequence is repeated, including the changes in the structural model. The value of γ_{sis} at the collapse will be the value of $\gamma_{\text{sis}}(n)$ of the last step and will be named $\gamma_{\text{sis}}^{\max}$.

The value of γ_{sis} allows controlling the iterative procedure because (i) in each step, it defines damage levels in the structure until a damage state is reached and (ii) between steps, it defines the intermediate damage states reached until the collapse of the structure.

The structural elements where damage was analysed are the masonry elements from the exterior walls and the connections of timber elements. The connections considered relevant in the analysis are (i) the connections of the diagonals to the other elements of 'gaiola' and (ii) the connections between timber elements ('gaiola' and floors) and masonry walls. These connections were chosen due to their bracing function of the structure against horizontal actions. The connections analysed are identified in Fig. 10.

In this study, a complete drop in stiffness was considered to model the rupture of connections within the 'gaiola' walls and between these and the masonry walls. For masonry elements an average state in each element was considered to identify damage. In some masonry elements, a partial stiffness reduction (50%) was considered. However, a designer may wish to adopt a different criterion for stress evaluation aiming at higher precision.

Table 2 presents tensile strength values considered for the mentioned connections. It also presents masonry strength values obtained from a bibliographic search [8]. The masonry strength values are average values that cannot be considered the same for all masonry buildings, mainly due to the large variability of the properties of structural materials and to the variety of structural solutions found in old masonry buildings. The tensile capacity of the connections between timber elements and masonry walls is probably below real average values of the strength of the

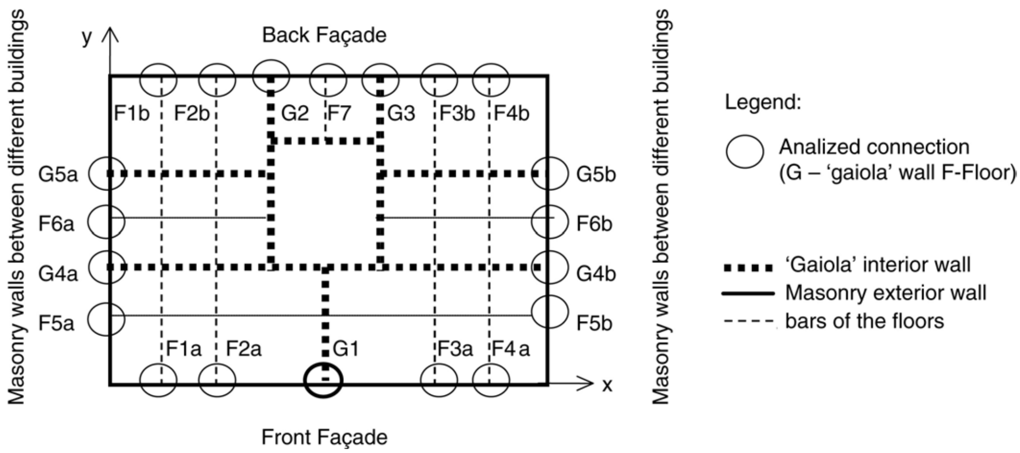


Fig. 10. Analysed connections between timber elements and masonry walls.

Table 2
Strength values adopted for damage calculation in relevant structural elements

	Structural element	Strength values—F_{Rd}
Connections[a]	Braced timber bars in 'gaiola' (only tension)	0 kN
	Timber bars – masonry walls (only tension)	5 kN
Masonry	Compression	1.3 MPa
	Tension	0.1 MPa
	Shear[b]	0.1 MPa

[a] Values justified in [7].
[b] This was considered an average shear stress in a cross-section related to diagonal cracking.

connections to inner 'gaiola' walls. This is because there is a large uncertainty about the details of these connections. Considering a poor detail, a strength value of 5 kN was adopted for the connections.

The connections achieve rupture only in tension, corresponding to timber elements being pulled out from the masonry walls. Even though the connections may continue to transfer compressive forces this is enough to allow the detachment of the exterior walls from the inner 'gaiola' walls. Therefore, it is considered that the connections are no longer effective and they are removed from the model.

4.3. *Collapse mechanism identification*

The definition of a collapse mechanism is associated to a damage state considered "unacceptable". Therefore it cannot be defined with complete precision. For instance, the out-of-plane fall of the façades is considered collapse, even if the inner structure of the building remains standing, since the building will be left in an irrecoverable state. However, it does not mean that it leads to human causalities among the occupants. The identification of this collapse mechanism is therefore associated to a given level of damage in the masonry walls and large out-of-plane displacements that are only possible after the rupture of part of the connections to the inner 'gaiola' walls.

Besides the level of damage in the structural elements or connections, the evolution of the values of γ_{sis} in the iterative process may also lead to the identification of the collapse mechanism. Due to the brittle nature of the behaviour of many elements and connections, it is possible that the seismic intensity associated to the new significant damage in a given iteration is equal to or inferior to the seismic intensity registered in the previous iteration. This identifies a situation in which damage increases in the structure without an increase of the seismic intensity, which may lead to sequential collapse in a manner similar to a 'domino effect'.

5. Method application

5.1. *First iteration*

The evolution of damage levels in the first step of the iterative procedure can be observed in Fig. 11, which presents the masonry elements from the main façade for which cracking due to tension occurred (the darkest elements), for different values of γ_{sis}. The vertical line in the middle of the façade above the first floor represents the connection G1 identified in Fig. 10. In Fig. 11, the vertical lines at the ground floor represent the connections of the existing interior masonry walls to the façade.

Fig. 11 shows that masonry damaged elements are mainly located at the top of the building. The obtained damage patterns identify the expected collapse mechanism, corresponding to the fall out-of-plane of the front façade at the top floor. This mechanism is one of the typical collapse mechanisms of masonry buildings which was observed in buildings that collapsed during recent earthquakes in Europe, as shown in Fig. 12 [12].

The table in Fig. 13 presents the maximum values of γ_{sis} for which rupture of different connections G1 occurs. These values are the scale factors corresponding to the intensity

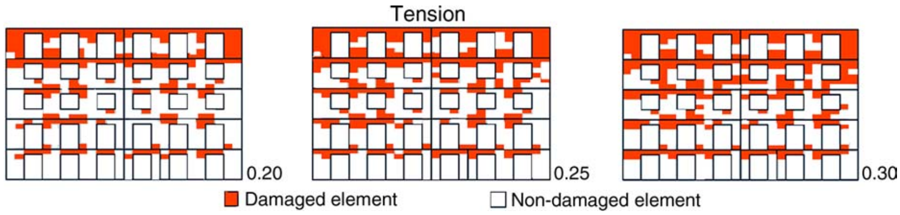


Fig. 11. Damage levels in masonry elements from main façade for increasing values of γ_{sis}.

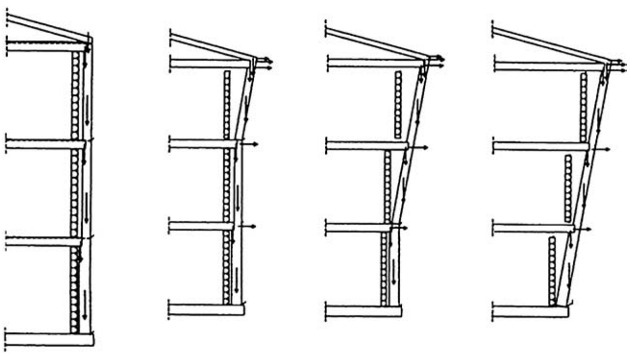
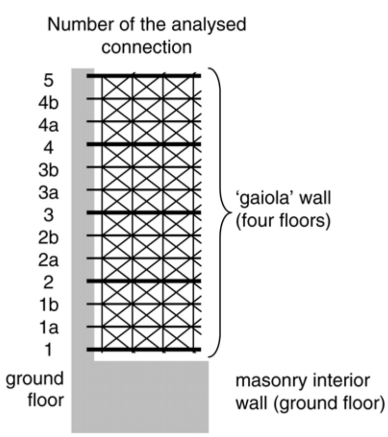


Fig. 12. Bending out-of-plane of masonry buildings as a typical collapse mechanism due to seismic actions [12].



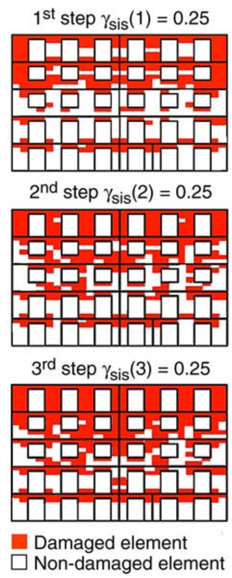
Rupture of the connections G1 (1st step)

Number	γ_{sis}
5	**0.06**
4b	**0.14**
4a	1.15
4	0.47
3b	**0.22**
3a	**0.21**
3	0.32
2b	0.38
2a	0.52
2	0.72
1b	1.18
1a	1.18
1	0.94

Fig. 13. Values of γ_{sis} corresponding to the rupture of the connections G1 ('gaiola' wall to façade masonry wall).

of the seismic action whose effects in the structure, F_{Sd}, defined by Eq. (1), are equal to the resistance of each connection, F_{Rd}.

Considering the damage levels of the front façade presented in Fig. 11, defined to values of γ_{sis} between 0.20 and 0.30, and, simultaneously, the rupture of the connections presented in the table in Fig. 13 (bold values), the damage state associated to $\gamma_{sis}(1) = 0.25$ was adopted to define the first step. According to the values presented in the table of Fig. 13, the rupture of the connections number 5 (at the roof level) occurred at values of γ_{sis} inferior to 0.25. An intermediate step introducing their rupture in the structure was not justified mainly due to the reduced damage level of masonry elements of the front façade at those levels.



Rupture of connections G1 along the iterative process

Number	$\gamma_{sis}(1)$	$\gamma_{sis}(2)$	$\gamma_{sis}(3)$
5	**0.06**		
4b	**0.14**		
4a	1.15	**0.02**	
4	0.47	**0.06**	
3b	**0.22**		
3a	**0.21**		
3	0.32	**0.05**	
2b	0.38	**0.16**	
2a	0.52	0.36	**0.04**
2	0.72	1.96	0.62
1b	1.18	1.54	**0.17**
1a	1.18	2.26	0.31
1	0.94	0.98	1.38

Fig. 14. Results (main façade)—all steps of iterative method.

5.2. *Results of all the iterations*

In the following steps of analysis higher levels of damage associated to the out-of-plane displacements of the front façade were obtained without increasing the value of γ_{sis}. The collapse mechanism was reached after three steps of calculation. As all γ_{sis} values were equal to 0.25, the collapse of the structure is sequential, as described in Section 4.3. This type of mechanism was observed in small buildings during the 1998 Azores earthquake, as shown in Fig. 4.

The relevant masonry damage states analysed for the three steps are presented in Fig. 14. The table in this figure shows the connections G1 suppressed during the iterative procedure (bold values). According to these values, the value of γ_{sis} associated with the rupture of some connections in the second and third iterations decreases, which is a consequence of the increase in the tensile forces in the connections due to the removal of the adjacent connections from the model that suffered rupture in the first and second iterations, respectively.

Table 3
Horizontal relative displacements out-of-plane at the top of connection G1 ($\gamma_{sis}(1) = \gamma_{sis}(2) = \gamma_{sis}(3) = 0.25$)

	1st Step	2nd Step	3rd Step	Building without 'gaiola' $\gamma_{sis}^{max} = 0.25$
Relative[a] displacement (cm)	1.20	2.30	3.70	3.90
Increment of displacement (cm) between successive steps	–	1.10	1.40	–
Increment of displacements (cm)		2.50 (217%)		0.2 (5%)[b]

[a] Relative to the left corner of the front façade.
[b] Relative to the last step of the iterative procedure for the building with 'gaiola'.

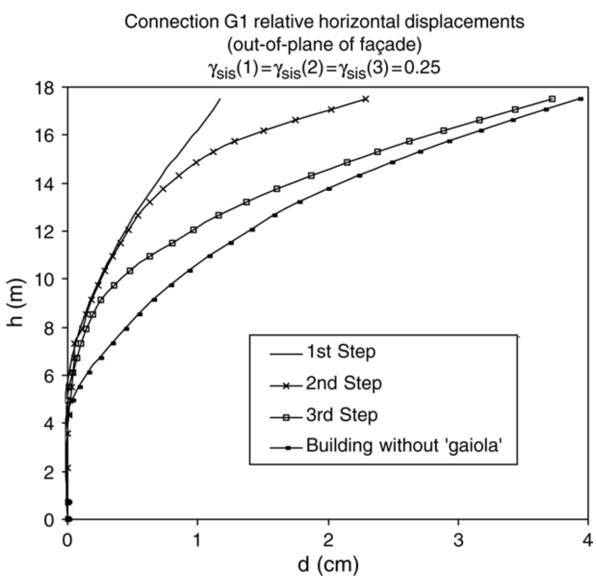


Fig. 15. Horizontal displacements out-of-plane of the façade, relative to the corner of the building.

The collapse mechanism can be identified by the out-of-plane horizontal displacements associated with the deformation of the main façade. Those displacements, presented in Fig. 15, are the difference between the horizontal displacements observed in the connection G1 (Fig. 10) and the displacements in the same direction observed in the left corner of the front façade of the building. This procedure was adopted to better understand the real deformed shape of the front façade in the out-of-plane direction. The maximum values were obtained at the top of the building and are presented in Table 3 for each step of the iterative procedure, scaled by the corresponding values of γ_{sis} ($\gamma_{sis}(1) = \gamma_{sis}(2) = \gamma_{sis}(3) = 0.25$). In this table, the displacements observed at the end of the iterative procedure (step 3) at the top of the building were 2.50 cm higher than the value obtained in the first iteration. The observed displacement corresponds to an increase of 217% of the displacement observed in the linear analyses (step 1), which indicates that the iterative procedure leads to a more realistic evaluation of displacements.

Table 3 and Fig. 15 also include the displacements observed in the numeric model of the same building without 'gaiola'. It can be observed that displacements measured at the end of the iterative procedure were 0.2 cm (5%) smaller than the displacements measured in the same building without the 'gaiola' (Table 3). This is due to the remaining active connections after the collapse of the building (third step) that still contributes to its stiffness.

A previous study [7] allowed understanding that the presence of 'gaiola' in the building braces masonry walls. The analysis showed that independent (local) vibration modes of each masonry wall and relative displacements between parallel walls were restrained at the first iteration. The frequency of the first mode of vibration was $f = 0.942$ Hz, higher than $f = 0.398$ Hz obtained for the same building without 'gaiola'. Fig. 16 shows the configuration of the first three modes for both buildings.

Fig. 16 and the mentioned results show that the 'gaiola' are effective in bracing masonry walls and give an important contribution to the global stiffness. The dynamic behaviour of the structure analysed in the last step is closer to the dynamic behaviour of the building without 'gaiola' due to the high number of removed connections. This can be observed in Table 4, which presents the values of the frequencies for the first vibration modes of the building with 'gaiola' analysed in the iterative procedure.

The reduction of the frequencies between iterations also confirms the reduction of the global stiffness of the building during the iterative procedure, explained by the rupture of the connections between wood structural elements and masonry (floor elements, 'gaiola', etc) and wood elements between themselves (elements of 'gaiola').

The reported results, regarding out-of-plane displacements of the façade and the frequencies, indicate that the building collapse is due to the overturning of the front façade.

5.3. *Evolution of the building global behaviour along the different steps*

Table 5 presents the evolution of the global stiffness of the building, K, for each step ($\gamma_{sis}(1) = \gamma_{sis}(2) = \gamma_{sis}(3) = 0.25$). The global stiffness of the structure, K, was obtained with Eq. (2), considering the effects of the seismic action obtained in the analyses, the global base shear reactions, F, and the average of the displacements of all the nodes of the top of the building, d, for both horizontal directions.

$$K = F/d. \tag{2}$$

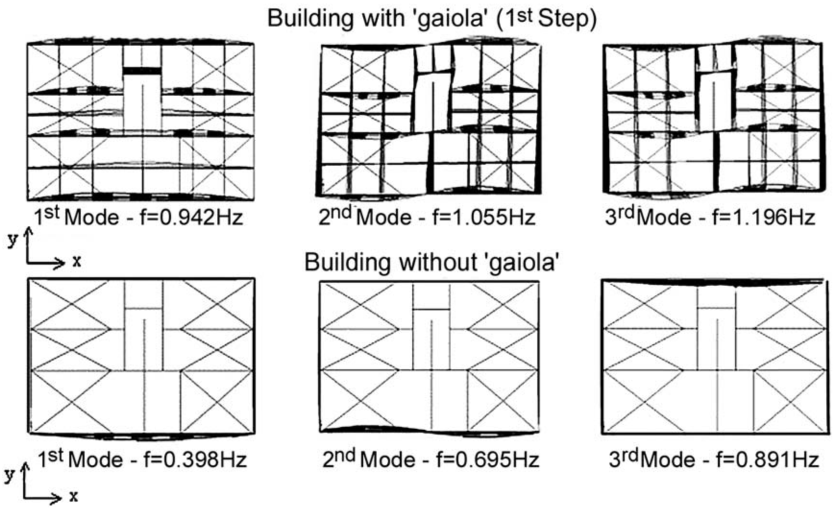


Fig. 16. Dynamic shapes of the building with and without 'gaiola'.

Table 4
Dynamic analysis—frequency values obtained during the iterative process

Mode	Mode of vibration	1st Step f (Hz)	2nd Step f (Hz)	3rd Step f (Hz)
1	Translation parallel to y-axis	0.942	0.671	0.438
2	Translation parallel to x-axis with torsion	1.055	0.806	0.622
3	Translation parallel to x-axis with torsion	1.196	1.096	0.700

Fig. 17 relates the base shear reactions in each step, with the displacements of the top of the building, according to the values presented in Table 5.

As shown in Table 5 and in Fig. 17, the global stiffness is reduced along the iterative procedure for both horizontal directions. The global base shear reactions in the out-of-plane direction of the front façade are also reduced along the iterative procedure.

The decrease in the overall stiffness of the building from iteration to iteration is due to the reduction in the number of effective connections between the 'gaiola' walls and the masonry walls, as well as the effects of masonry cracking.

Another consequence of the reduction of the effective number of connections is the fact that the masses of the façades and of the inner part of the building do not vibrate together. This changed the mode shapes and is responsible for the reduction of the mass mobilized in the first modes, as shown in Table 5.

6. Discussion

By applying the iterative procedure it is possible to obtain more realistic out-of-plane horizontal displacements of masonry walls and corresponding damages, so the collapse mechanism of the structure will be more realistically characterized. Because the structure is changed at each step, the analysis allows modelling non-linear behaviour. Moreover, the iterative procedure allows for detecting sequential collapse.

The main advantages of defining the value $\gamma_{\text{sis}}^{\max}$ are that it provides, with a single variable, information about the seismic vulnerability of the buildings and also gives information about the relevance of different collapse mechanisms.

The definition of the damage state in each step, according to the number and location of the structural elements that have yielded, collapsed or reached a given stress level, may not be obvious and straightforward. Therefore the interpretation of the results, like those presented in Fig. 14, is not always simple. Thus, the accuracy in the evaluation of γ_{sis} at each step depends on the accuracy of the definition of damage states.

Even if the value of $\gamma_{\text{sis}}^{\max}$ cannot be evaluated with the precision that there would be for a reinforced concrete or steel building, the value evaluated can be considered a parameter that allows for quantifying the building's potential performance under seismic actions, and therefore its seismic vulnerability.

In the structural design of new structures, the design action effects including the effects of the seismic action are defined by an equation similar to Eq. (1), in which γ_{sis} is replaced by the safety factor, γ_s. Since the safety factor γ_s scales the seismic action, it can be directly compared with the factor $\gamma_{\text{sis}}^{\max}$. The low values obtained for this factor show a low strength of these structures for seismic actions, justifying the concern about the potential seismic

Table 5
Evolution of the global stiffness of the structures analysed in each iteration, for all the analysed modes (values obtained for $\gamma_{\text{sis}}(1) = \gamma_{\text{sis}}(2) = \gamma_{\text{sis}}(3) = 0.25$)

	x direction parallel to the front façade			*y* direction perpendicular to the front façade		
	Step 1	Step 2	Step 3	Step 1	Step 2	Step 3
F (kN)	619	626	611	527	408	370
d (cm)	0.912	0.976	0.972	0.941	1.159	1.432
K (kN/m)	67 870	64 154	62 855	55 930	35 156	25 848
% of the mass mobilized[a] in the first mode in each direction	27.9 (3rd mode)	44.1 (3rd mode)	39.2 (5th mode)	35.1 (1st mode)	22.2 (1st mode)	16.2 (1st mode)

[a] Percentage in the mass mobilized in all the modes considered in the analysis (over 100 modes, which include local modes of vibration of the timber elements of 'gaiola').

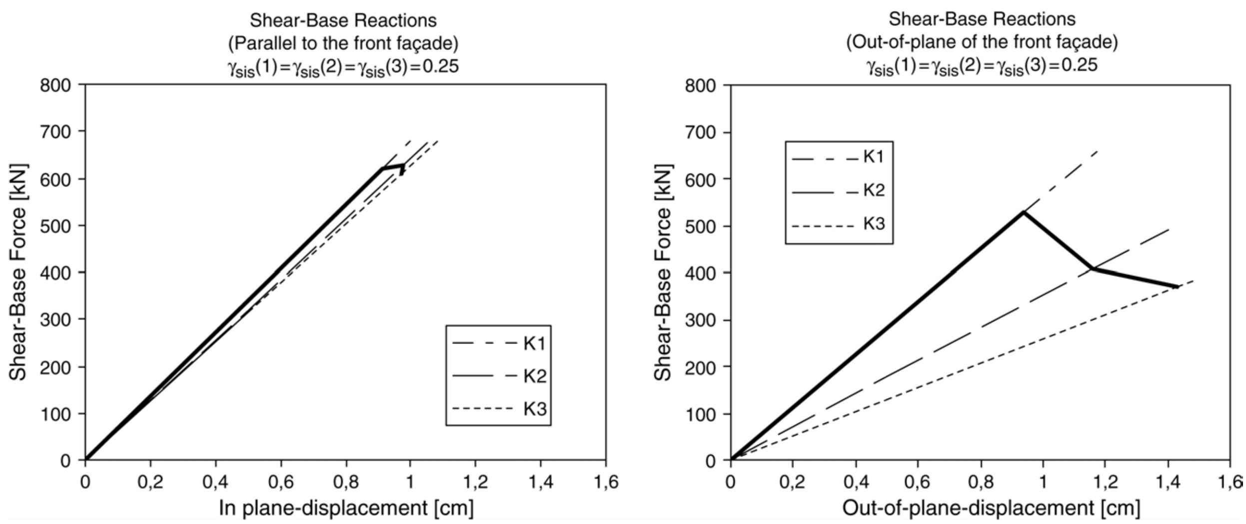


Fig. 17. Evolution of the global stiffness of the analysed structures (values obtained for $\gamma_{\text{sis}}(1) = \gamma_{\text{sis}}(2) = \gamma_{\text{sis}}(3) = 0.25$).

performance of these buildings. However, the final value $\gamma_{\text{sis}}^{\max}$ also depends of the hypothesis adopted to define the model: (i) assuming that the seismic action starts in all the steps by adopting the same response spectrum in all the iterations and (ii) accounting for the hysteretic energy dissipation capacity of 'gaiola' walls to dissipate energy and the one due to the rupture of structural elements during the iterative calculation by adopting a constant viscous damping coefficient equal to 10%. These hypotheses are conservative and lead to an underestimation of the value of $\gamma_{\text{sis}}^{\max}$. The consideration of the q-factor (force reduction factor) equal to 1.5 would increase the values of γ_{sis} by 50% and would be closer to the expected seismic behaviour of the buildings.

The value of $\gamma_{\text{sis}}^{\max}$ obtained also depends on the strength parameters considered, which must be calibrated case by case. The values of the strength parameters adopted, presented in Table 2, are acceptable or below average values. But it should not be forgotten that these values have a large variability due to the variability of the material characteristics and construction techniques in this type of building, as the values found in the literature [8] show. Degradation due to age and structural changes performed during the structure lifetime also justify the large range of some material property values. For all the material properties values presented and adopted in this work, the most relevant that should be reasonably assessed are the tensile capacity of the connections involving timber elements and the masonry mechanical properties. In particular, masonry shear strength is fundamental to quantify the resistance to the global shear mechanism. These values can be calibrated in real cases by non-destructive techniques or 'in situ' tests, which can be prescribed by the structural designer [7].

Only three steps of analysis were necessary to reach the collapse mechanism and to understand the structural behaviour of the original building. Although a higher number of iterations could be necessary if a higher level of accuracy in the definition of the damage states was required, the analyses performed showed that the most relevant information could be obtained with a reduced number of iterations. However, the amount of work associated with the identification of damaged elements all over the structure is significant. Therefore, the automatic implementation of this step-by-step procedure in an interactive manner is probably necessary for design applications. Another advantage of the automatic implementation of the iterative procedure is that it would allow performing different analyses for the same building covering a given range of material properties, in order to deal with uncertainties in the characterization of these properties.

The proposed method allows, within certain limits, the simulation of fragile and ductile behaviour. An example of the first is the behaviour of the connections between the 'gaiola' walls and the façades, and an example of the second is the type of behaviour admitted for the façades in their own plan.

7. Summary and conclusions

An iterative method is proposed for the seismic assessment of old masonry buildings. In each iteration, damage in the structural elements or connections between elements due to collapse (brittle behaviour) or yielding (ductile behaviour) are identified and the structural system changed accordingly. Each iteration comprises a linear elastic dynamic analysis by response spectrum, scaled by a factor γ_{sis} to define the seismic action associated to each damage state. The value of this factor at the collapse of the structure, γ_{sis}^{max}, quantifies its potential seismic performance, and is directly comparable to a safety factor. The fact that each iteration only comprises a linear analysis allows the use of the method in current design practice.

Besides the seismic intensity at collapse, the method allows the identification of the weakest links and connections in the structure and the identification of its expected collapse mechanism, which are relevant information to the design of seismic strengthening solutions.

The proposed method is conservative, as it does not account for the energy dissipation capacity, which is likely to be underestimated by means of using an equivalent linear damping coefficient, and overestimates the effects of the seismic action, as it does not account for its duration. It cannot be applied to regular block masonry, as it cannot simulate the behaviour of the interfaces and the geometrical non-linearity.

The proposed method was applied to an old masonry 'Pombalino' building typical of Lisbon Downtown, built after the Great Lisbon Earthquake of 1755 with the concern of providing seismic resistance.

The results, which can be confirmed by comparison with masonry buildings' collapse mechanisms observed during earthquakes, indicated a sequential collapse at a relatively low seismic intensity, initiated by failure of the connections of the façade to the inner horizontal resisting braced timber structure on the upper floors. This induced the collapse of most of the other connections at the lower floors, followed by the out-of-plane fall of the front façade, all at the same intensity of the seismic action. This type of behaviour is associated to a slight decrease of the inertia forces as the damage increases. This would correspond, in a force–displacement diagram, to a descending branch after the ultimate load was reached and is due to the brittle nature of the behaviour of the connections.

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