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Seismic evaluation of old masonry buildings. Part II: Analysis of strengthening solutions for a case study

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Abstract

This paper describes the application of the iterative method described in Part I (Cardoso R, Lopes M, Bento, R, Seismic Evaluation of Old Masonry Buildings. Part I: Method Description and Application to a Case Study, Engineering Structures, 2005) to the seismic strengthening design of irregular block masonry structures. The method was applied to an old masonry building from the city of Lisbon, which includes a three-dimensional wood structure braced with diagonal elements, aiming at providing seismic resistance to the building. Three different strengthening solutions were defined, based on the collapse mechanism obtained. This paper presents the analysis performed for each solution and the discussion regarding its qualitatively and quantitatively effects in the seismic structural behaviour: the identification of the expected collapse mechanism after strengthening and the seismic intensity for which it occurs. This is necessary to define the more efficient and economic strengthening strategies.

Keywords: Seismic vulnerability; Old masonry buildings; Strengthening; Wood structure

1. Introduction

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 and Portuguese buildings are not expected to be an exception.

A ‘Pombalino’ building, which is a typical old masonry building from Lisbon Downtown built after the 1755 Lisbon Earthquake, was analysed (Part I) and its expected collapse mechanism due to seismic actions was identified. The methodology proposed allowed simulating, by an approximate manner, the non-linear behaviour of irregular block masonry structures with structural timber elements. 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 low value of factor γ_{sis}^{max} obtained ($\gamma_{sis}^{max} = 0.25$) justifies the concern about the seismic performance of these structures and the need to improve their seismic resistance. Note that the values of γ_{sis}^{max} mentioned in this paper are not multiplied by the q-factor (or force reduction factor), as it would be necessary to evaluate the seismic intensity at collapse. As discussed in Part I, it was considered that a value of $q = 1.5$ could be reasonable for these type of building if the collapse would take place after significant horizontal displacements of the interior ‘gaiola’ walls. The energy dissipation capacity of these walls was already been observed experimentally [1], as mentioned in Part I.

The rupture of relevant structural elements (masonry cracking and the rupture of connections) was introduced in the model in each step of calculation, until the collapse. This procedure allowed identifying the weakest links of the structure, mainly the connections between interior ‘gaiola’

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walls and its perpendicular masonry walls, providing useful information to the seismic strengthening design. The rupture of the relevant structural elements mentioned allowed identifying the fall out-of-plane of the front façade as the expected collapse mechanism.

The iterative method previously described was also used to analyse the effects of three different strengthening solutions, usually adopted in seismic strengthening. The expected collapse mechanism was identified and the corresponding factor $\gamma_{\text{sis}}^{\text{max}}$, was used to quantify the efficiency of the solution by comparing it with the one obtained for the original building, before strengthening. The solutions studied are described further in this paper.

The effects on global behaviour of each strengthening solution will be discussed, aiming at providing information to the definition of the more adequate strategies to improve the seismic resistance of old masonry buildings.

2. Description of the building and numerical model

'Pombalino' buildings 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 wood structure of 'gaiola' is like a birdcage made of vertical and horizontal elements braced with diagonals named St Andrew's Crosses. The other interior walls are partition walls made of wooden panels and should not be considered as having structural functions.

The first floor is composed of a system of vaults made of regular masonry blocks and stone arches and the foundations include short and small diameter woodpiles connected by a wood grid. Floors above the first are wood slabs and should be considered as flexible diaphragms, and the roof is made with timber truss and ceramic tiles and may include window openings. A more detailed description of 'Pombalino' buildings can be found in [2].

A commercial program was used (SAP2000® [3]) and the numerical model of the structure is described in Part I.

The masonry of the exterior walls is made of irregular blocks of calcareous stone and lime mortar with very poor strength capacity. The Young's modulus, E, adopted for the structural materials were 600 MPa for masonry, 8000 MPa for timber and 3000 MPa for stone. The Poisson coefficient of all materials was assigned the value 0.2.

According to the Portuguese Code [4], a uniform service load (1.2 kN/m^2) acting at all the floors was considered. As previously described, linear dynamic modal analysis was performed by response spectrum. The seismic action was based on the response acceleration spectrum also defined in the mentioned code, acting along the two horizontal directions.

3. Strengthening solutions adopted

According to the study of the building presented in Part I, the expected collapse mechanism of the building is the fall out-of-plane of the front façade at the top floor, eventually bringing down other parts of the building. Therefore, the primary concern of the studied strengthening solutions, which are presented in Fig. 1, is to increase the building resistance to this mechanism.

The connections between 'gaiola' walls and masonry exterior walls (identified in Fig. 2) play an important role

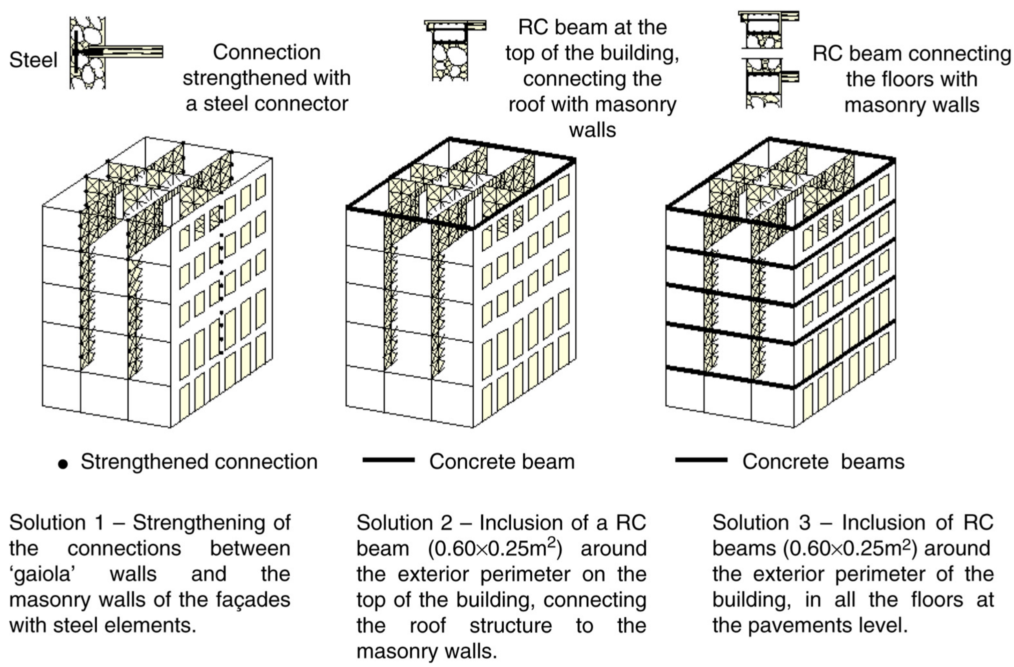

Fig. 1. Strengthening solutions adopted.

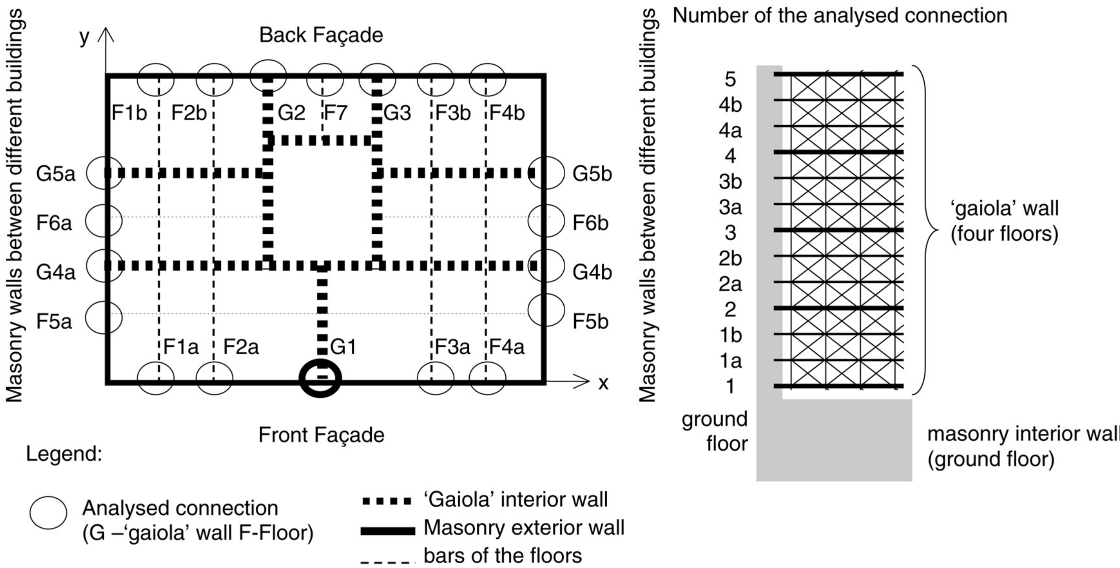

Fig. 2. Analysed connections between timber elements and masonry walls.

in the building's seismic behaviour because the fall out-of-plane of the façade can only occur after the rupture of its connections to the perpendicular 'gaiola' wall(s). Therefore, strengthening these connections with steel elements was one of the strengthening solutions adopted (Solution 1). The other solutions considered reinforced concrete beams (0.60×0.25 m^2) placed around the exterior perimeter of the building, firstly only at the top floor (Solution 2), and secondly in all the floors, at the pavement level (Solution 3). Bar elements connected to the nodes of the shell elements in the corresponding places of the structure modelled the reinforced concrete beams (RC beams).

4. Methodology

The aim of the analysis of each strengthening solution is to evaluate its efficiency, which can be quantified by the increment of the building seismic resistance, measured by the value of $\gamma_{\text{sis}}^{\max}$ for each solution. Each solution is studied by comparing the results obtained in the study of the structure before and after strengthening (dynamic behaviour and expected collapse mechanism).

The iterative method presented in Part I will be applied to each solution studied. As a brief description of the iterative method used, damage in the structural elements or connections between elements are identified in each step and the structural system changed accordingly. The factor γ_{sis} is a scale factor of the response spectrum used to perform a linear elastic dynamic analysis in each iteration. This factor was used to define the seismic action associated to each damage state, according to Eq. (1). This equation defines the design action effects (internal forces) in structural elements, F_{Sd}, where F_{Perm} are the effects of vertical permanent loads and F_E are the effects of the code prescribed seismic action.

$$F_{\text{Sd}} = F_{\text{Perm}} \pm \gamma_{\text{sis}} F_E. \quad (1)$$

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 $F_{\text{Sd}} \geq F_{\text{Rd}}$.

The structural elements where damage was analysed are the masonry elements from the exterior walls (resistance values: 1.3 MPa—compression, 0.1 MPa—tension and 0.1 MPa—shear), the connections of timber elements (without tensile strength) and the connections between timber elements and masonry walls (strength value of 5 kN). The values adopted are justified in Part I.

5. Collapse mechanisms considered

The main collapse mechanisms analysed were the out-of-plane fall of the front façade, as described in Part I, and the global base shear mechanism. Damage patterns (masonry cracking and yielding or rupture of connections) allowed identifying both mechanisms.

To identify the out-of-plane fall of the façade, damages at the masonry walls due to bending (tension and compression) were analysed. Damage due to shear in the vertical direction was also analysed at the elements close to the connections between perpendicular masonry walls. The yield or collapse of the connections between the interior walls of 'gaiola' and the exterior masonry walls of the façades, considered together with masonry damages, allowed the identification of this mechanism. At the limit, the rupture of these connections corresponds to the overturning of the façades.

The global base shear mechanism was identified by the formation of a continuous slip surface in the vertical masonry elements of the ground floor, parallel to the façades. The elements analysed were the masonry walls of the façades because their resistance to shear, due to the doors and window openings, is smaller than the resistance of masonry walls without openings, such as the walls between adjoining buildings.

It was considered that the formation of the horizontal slip surface would be associated with reaching the shear capacity in all the columns of a façade. Even though this hypothesis may not be considered conservative, it is known that the error is not too large for two reasons: (i) the columns have similar dimensions and therefore it is likely that shear effects are close to the shear capacities in most columns when the effects match the capacities in a given column, and (ii) since the columns are made of irregular block masonry, it is acceptable to assume that the elements possess reasonable post-ultimate strength mobilized by friction between blocks, allowing some redistribution of forces. This is equivalent to admitting ductile behaviour of the columns. Therefore, the seismic capacity associated to the collapse of the façade due to shear in its own plane can be evaluated by comparing the overall shear strength of all columns with the overall shear force in the plane of the façades.

In all the cases studied, the possibility of failure of the columns due to the combined effects of bending and axial forces was also studied but it was concluded that it is not critical as it led to similar results.

As described in Part I, the lower value of γ_{sis} will allow us to identify the collapse mechanism. Although the collapse mechanism of the building was the fall out-of-plane of the front façade, described in detail in Part I, the global base shear mechanism was also analysed for the original building. Assuming that other collapse mechanisms would not be developed before the global base shear mechanism, a value of $\gamma_{\text{sis}} = 0.70$ was obtained.

6. Results

6.1. Solution 1

Solution 1 is the strengthening, with steel elements, of the connections between 'gaiola' walls and perpendicular masonry walls. The rupture of these connections was not considered since it was assumed they would be designed with enough strength to not fail before the collapse of the building occurred in other modes.

Solution 1 does not change element dimensions; therefore it does not modify the global stiffness of the structure. Moreover, since the rupture of connections will no longer occur, one of the main reasons for the reduction of the global stiffness is eliminated during an earthquake. In fact, as discussed in Part I, the rupture of the connections between 'gaiola' walls and the façades strongly influences the evolution of the dynamic behaviour of the structure since it gives rise to local vibration modes with reduced mass participation.

The effects of the seismic action evaluated from the calculation of the first iteration of the original building

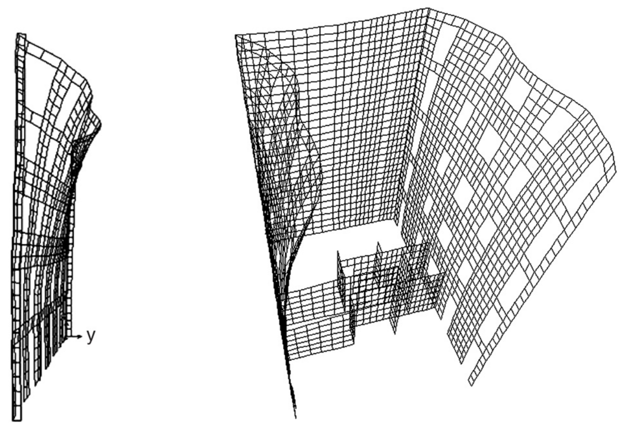

Fig. 3. Deformed shape of the front façade due to seismic actions.

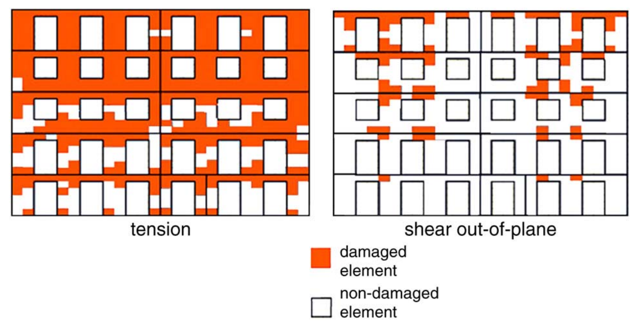

Fig. 4. Damage patterns in masonry elements at the collapse obtained for the building strengthened with Solution 1 ($\gamma_{\text{sis}} = 0.60$).

(Part I) are similar to those obtained with Solution 1 because they correspond to the situation in which all connections are still active. As obtained before strengthening, the masonry damages concentrated mainly on the top of the building. Fig. 3 shows the deformed shape of the front façade, where the bending out of plane of this masonry wall is presented. As it can be observed in Fig. 3, the connections G1 (between the 'gaiola' interior wall and the front façade, identified in Fig. 2), provide support to the façade, as well as the masonry walls perpendicular to the main façade (walls between adjoining buildings).

Since it was admitted that the rupture of connections between 'gaiola' walls and the façades cannot occur, the identification of the out-of-plane fall mechanism must be identified essentially by masonry damage. Since all the main sources of non-linear behaviour affecting the development of this mechanism are eliminated by the strengthening solution (Solution 1), a linear analysis provides a reasonable estimate of the seismic intensity at collapse.

According to the damage patterns obtained for the front façade, it was considered that the one reached for $\gamma_{\text{sis}} = 0.60$ (not multiplied by the q-factor of 1.5) corresponds to 'unacceptable' damage. As shown in Fig. 4, for this seismic intensity, masonry damages (tension and shear in the out-of plane direction) reach all the levels. In Fig. 5, the vertical line in the middle of the façade corresponds to connection

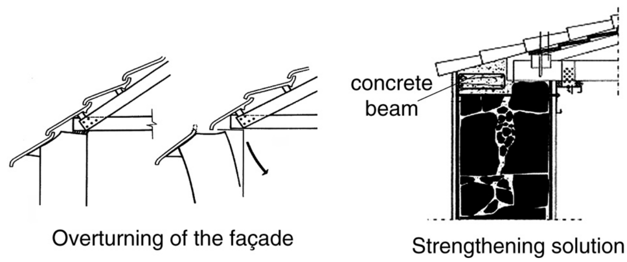

Fig. 5. Usual strengthening solution adopted to prevent fall out-of-plane of façades ([5] and [6]).

G1 (Fig. 2). The other vertical line at the ground floor is the connection between the façade and an existing interior masonry wall.

According to the results presented above, the seismic resistance of the building increases from $\gamma_{sis}^{max} = 0.25$ (Part I), before strengthening, to $\gamma_{sis}^{max} = 0.60$. Due to strengthening, the out-of-plane displacement of the front façade is reduced from 3.7 cm (third step of calculation, Part I) to 2.88 cm. It corresponds to a value 22% inferior to the one obtained before strengthening.

The global base shear mechanism was also analysed and the procedure previously presented was adopted. A value of $\gamma_{sis} = 0.70$ was obtained, equal to the one obtained for the original building.

6.2. Solution 2

Solution 2 consisted of the inclusion of a reinforced concrete beam (RC beam) around the exterior perimeter of the building with a height of $h = 0.25$ m and a width of $b = 0.60$ m, on the top, as shown in Fig. 5.

Table 1
Rupture of connections G1 after strengthening the building—Solution 2

Number	$\gamma_{sis}(1)$	$\gamma_{sis}(2)$	$\gamma_{sis}(3)$
5		*Strengthened connection*	
4b	**0.40**		
4a	0.60	**0.45**	
4	0.54	**0.35**	
3b	0.50	**0.26**	
3a	**0.42**		
3	**0.42**		
2b	0.49	**0.30**	
2a	0.54	0.75	**0.10**
2	0.61	**0.37**	
1b	1.00	1.58	1.59
1a	1.43	0.63	4.72
1	0.98	0.66	0.67

The analysis of the strengthened building led to the identification of the collapse mechanism in three iterations. The connections removed along the iterative procedure are presented in Table 1. This table shows the maximum values of γ_{sis}, for which rupture of different connections G1 (defined in Fig. 2) occurs, and the connections removed in each iteration. These values are the scale factors corresponding to the intensity 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}.

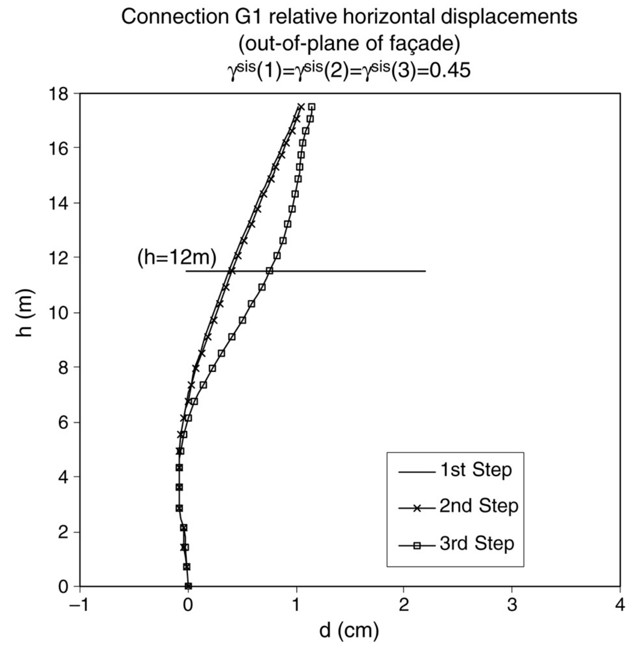

Fig. 6. Evolution of the horizontal displacements out-of-plane of the façade relative to the corner of the building in the iterative process ($\gamma_{sis}(1) = \gamma_{sis}(2) = \gamma_{sis}(3) = 0.45$).

The results led to $\gamma_{sis}(1) = \gamma_{sis}(2) = \gamma_{sis}(3) = 0.45$, equal to γ_{sis}^{max}. As observed before strengthening and described in Part I, the collapse of the structure is sequential at the same seismic intensity because γ_{sis} is equal for all steps.

The collapse mechanism obtained after strengthening the building with the top beam is still the out-of-plane fall of the front façade, triggered by the collapse of the connections with the 'gaiola' wall. However, the sequence of collapse starts by the connections at third floor level and not at the top (Table 1). Fig. 6 shows the relative displacement pattern along the height for each step of the iterative procedure, scaled by the corresponding values of γ_{sis} ($\gamma_{sis}(1) = \gamma_{sis}(2) = \gamma_{sis}(3) = 0.45$). These displacements are the difference between the horizontal displacements observed in the connection G1 (Fig. 2) and the displacements in the same direction observed in the left corner of the front façade of the building, as described in Part I.

Table 2 shows the increases of the deformation of the front façade at the top and at 12 m high in the out-of-plane direction, to illustrate that the non-linear behaviour of the building induces larger deformations at $h = 12$ m. This difference is compatible with the evolution of damage in the masonry walls and the pattern of rupture of connections G1 (Fig. 2) presented in Table 2. The observed collapse mechanism may indicate that the introduction of a metallic tie or a

Table 2
Horizontal relative out-of-plane displacements at connection G1 ($\gamma_{sis}(1) = \gamma_{sis}(2) = \gamma_{sis}(3) = 0.45$)

	1st Step	2nd Step	3rd Step
Relative[a] displacement TOP (cm)	1.03	1.05	1.15
Increase in displacement (cm)	–	0.02	0.10
Total increase		0.12 cm (12%)	
Relative[a] displacement $h = 12$ m (cm)	0.44	0.47	0.82
Increase in displacement (cm)—$h = 12$ m	–	0.03	0.35
Total increase—$h = 12$ m		0.38 cm (87%)	

[a] Relative to the corner of the building.

concrete beam in all the floors of the building, connecting the façade to the floors and to the 'gaiola' walls at these levels (strengthening Solutions 1 and 3, respectively), can be adequate to increase further the seismic capacity of the building.

According to the results obtained, the expected collapse mechanism of the building before and after strengthening will be the fall out-of-plane of the front façade. Nevertheless, the strengthening solution increases the global seismic resistance of the building because γ_{sis}^{max} was increased from 0.25, before strengthening, to 0.45, after strengthening.

The global shear mechanism was also analysed and a value of $\gamma_{sis} = 0.63$ was obtained, which will be discussed further.

6.3. Solution 3

Solution 3, which was an extension of Solution 2, considers the inclusion of RC beams (0.60×0.25 m^2) around the exterior perimeter of the building, at the pavement level of all the floors. It was assumed that all the connections between timber elements (pavements and 'gaiola' interior walls) and the façades were strengthened at this level; therefore their rupture was not considered in the analysis.

Table 3
Rupture of connections G1 after strengthening the building—Solution 3

Number	$\gamma_{sis}(1)$
5	*Strengthened connection*
4b	**14 260**
4a	1.78
4	*Strengthened connection*
3b	0.82
3a	**8361**
3	*Strengthened connection*
2b	0.70
2a	0.90
2	*Strengthened connection*
1b	0.71
1a	0.83
1	*Strengthened connection*

Table 3 presents the values of γ_{sis} for which rupture of the connections G1 occurs (values obtained as those presented in Table 1). The values marked bold in Table 3 correspond to the connections for which rupture is not likely to occur.

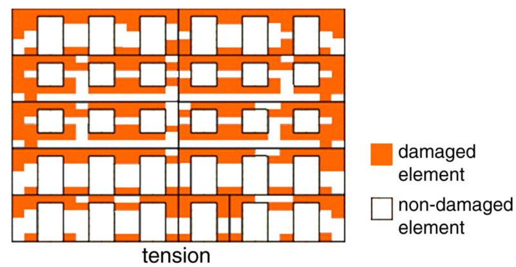

Fig. 7. Damage levels in masonry elements from the main façade at the collapse ($\gamma_{sis} = 0.50$), obtained for the building strengthened with Solution 3.

The values presented in Table 3, and the damages obtained in the masonry walls of the front façade for $\gamma_{sis} = 0.50$ (Fig. 7), indicate that the collapse due to the out-of plane fall of the front façade can only occur due to bending of the façade between the RC beams, since there is no rupture of connections. The damage pattern shown in Fig. 7, obtained for $\gamma_{sis}^{max} = 0.50$, corresponds to an 'unacceptable' situation, which was considered to identify the collapse mechanism. However, the masonry damages due to this mechanism cannot be clearly distinguished from those that allow identifying the bending parallel to the plane of the façades.

Since the masonry damage pattern is evident in all the floors for values of seismic intensity inferior to those corresponding to the rupture of the connections, the collapse does not depend on the rupture of the connections. In this condition, one iteration is enough to identify the failure mechanism, as for Solution 1.

The global base shear mechanism of the structure corresponding to Solution 3 was also analysed and the value of 0.55 was obtained. Due to similarity of the values of γ_{sis} obtained for both collapse mechanisms ($\gamma_{sis} = 0.50$ to the bending of the façade and $\gamma_{sis} = 0.55$ for the base shear), it is expected that the collapse mechanism combines both mechanisms and takes place at a seismic intensity value $\gamma_{sis}^{max} = 0.50$.

Solution 3 proves to be efficient because the global seismic resistance of the building increases from 0.25, before strengthening, to 0.50, after strengthening. This solution

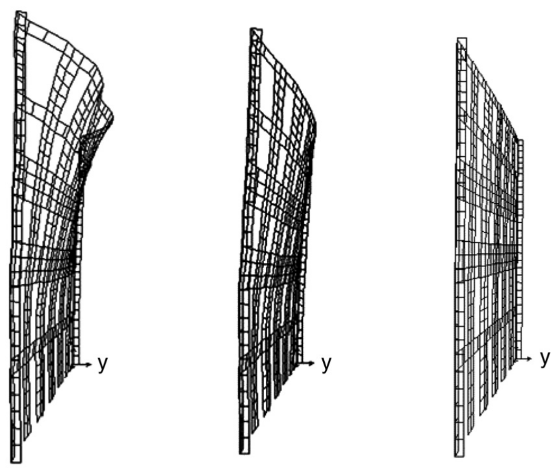

Fig. 8. Deformed shape of the front façade of the building before and after been strengthened with Solutions 1, 2 and 3 (same scale factor).

increases seismic capacity more than Solution 2 (from 0.25, before strengthening, to 0.45, after strengthening), which confirms the relevant role of the connections between 'gaiola' walls and the façades on the structural behaviour. In fact, the rupture of these connections decreases the maximum seismic intensity the structure can withstand, as the analysis of Solution 2 has shown. A more detailed discussion on the influence of Solution 3 in the global behaviour will be presented further in Section 7.

6.4. *Comparison of the strengthening solutions adopted*

The deformed shapes of the front façade of the buildings before and after strengthening are presented in Fig. 8. It can be observed that the deformed shape of the front façade after strengthening with RC beams is smoother than before strengthening. This result shows the increment of the out-of-plane stiffness of the front façade due to the presence of these concrete elements. However, after strengthening, the out-of-plane displacements are still relevant, justifying the collapse mechanism obtained for all the analysed solutions.

Fig. 9 shows the out-of-plane displacements along the height of the front façade at the collapse, corresponding to the deformed shapes presented in Fig. 8. The displacements in Fig. 9 were obtained in the connections C (connection G1—Fig. 2) and W (the left corner of the building—connection between perpendicular masonry walls), also identified in Fig. 9. As previously mentioned, the out-of-plane deformations of the front façade can be measured by the difference between the displacements obtained in connections C and W.

According to the displacement patterns shown in Fig. 9, the strengthening solutions where RC beams are used (Solutions 2 and 3) reduce the out-of-plane displacements of the front façade. This result indicates the efficiency of these solutions in preventing the expected collapse mechanism (fall out-of-plane of the façade) of the original building.

The building global stiffness is changed also by the strengthening solutions, which can be confirmed by analysing the frequencies and configurations of the first three vibration modes of the buildings, shown in Fig. 10.

By comparing the values of the frequencies of the structures strengthened with Solutions 2 and 3 with the corresponding values for the original building, presented in Fig. 10, it is evident that there is an increase of the global stiffness. This increment is higher for Solution 3 in which a higher number of RC beams were considered. In the case of Solution 1 there is no increase in stiffness as compared with the original unstrengthened building.

As expected, the increase of stiffness when RC beams are adopted is more relevant for the direction perpendicular to the front façade, the out-of-plane direction. According to Fig. 10, the first modes with relevant out-of-plane displacements of the front façade are the first mode, before strengthening/Solution 1, and the second mode, after strengthening (Solutions 2 and 3). The increment of the frequency is due to the increment of the global stiffness of the building in this direction.

Table 4 shows the values of γ_{sis} corresponding to the fall out-of-plane of the front façade and to the global base shear mechanism. As shown in Table 4, the expected collapse is the fall out-of-plane of the front façade for all the strengthening solutions studied (marked bold). Solution 1 is the most efficient of all because it corresponds to the highest increment of the seismic resistance (140%). As mentioned while presenting Solution 3, the seismic resistance of the structure for the global base shear mechanism is similar to the resistance obtained for the fall out-of-plane of the front façade, being expected a mechanism that combines both mechanisms due to the similarity of the corresponding values of γ_{sis}.

The values of γ_{sis} obtained for the global base shear mechanism (Table 4) decrease with the increase of the global stiffness. The increment of stiffness leads to higher values of the seismic action effects (the inertia forces increase). As the resistance of the structure to the global base shear mechanism does not change, the seismic intensity associated to collapse in this mechanism decreases with the adoption of Solutions 2 and 3.

The seismic intensity corresponding to the global base shear mechanism for Solutions 2 and 3 is $\gamma_{sis} = 0.63$ and $\gamma_{sis} = 0.55$, respectively. These values are inferior to $\gamma_{sis} = 0.70$ and can be explained by the increase in the global base shear forces in the buildings strengthened with RC beams, as presented in Table 5.

According to the values presented in Table 5, the global base shear forces due to the stiffening effect induced by the RC beams are incremented in both directions. This derives from the increase in the inertia forces associated to the increase in stiffness due to the beams. The increment of the global base shear force is more relevant in the out-of-plane

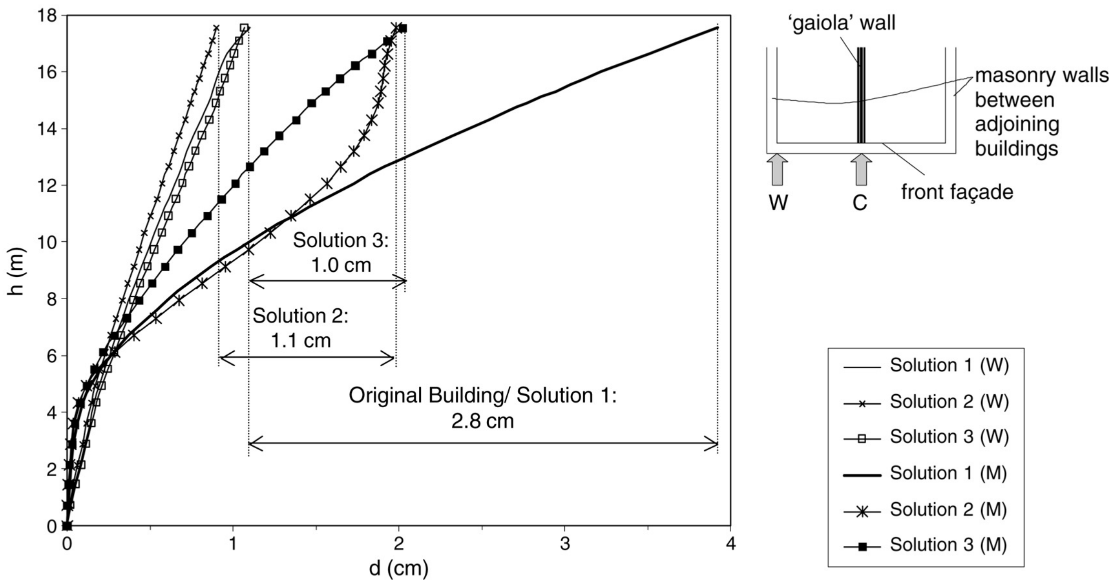

Fig. 9. Displacements of the front façade after strengthening due to seismic actions, at the collapse.

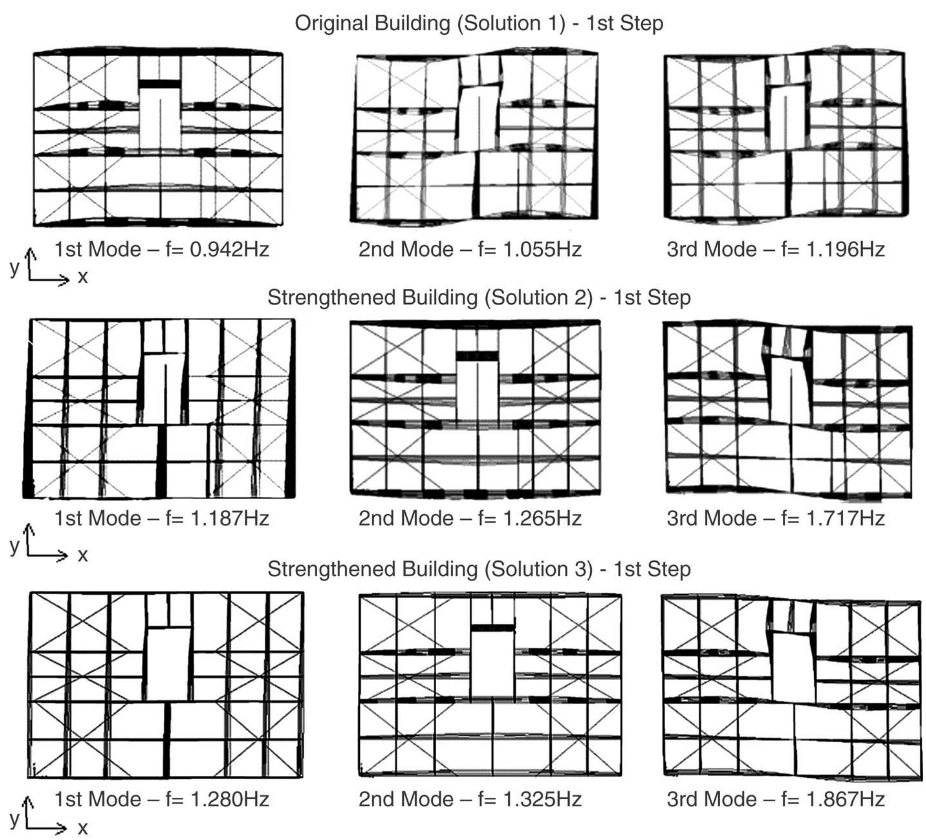

Fig. 10. Mode shapes of the first step of the analyses for the building before and after being strengthened.

direction of the front façade (19% for Solution 2 and 21% for Solution 3) than in the direction parallel to its plane (9% for Solution 2 and 8% for Solution 3).

The increment of the force in the direction parallel to the front façade is higher for Solution 2 than for Solution 3, contrary to what happens in the direction perpendicular to the front façade. This may be due to stiffness redistribution in the structure due to the inclusion of the RC beams in all the floors, and to torsion. The global stiffness increment also indicates that the base shear mechanism may become

Table 4
Values of γ_{sis} corresponding to the mechanisms analysed for each strengthening solution

Collapse mechanism	Original building	Strengthening solution		
		Without stiffness increment	With stiffness increment	
		Solution 1	Solution 2	Solution 3
Fall out-of-plane of the front façade	**0.25**	**0.60**	**0.45**	**0.50**
Global base shear mechanism	0.70	0.70	0.63	0.55
Seismic resistance increment	–	140%	80%	100%

Table 5
Seismic base shear forces for the building before and after strengthening corresponding to the formation of the shear-base collapse mechanism

	Building before strengthening/Solution 1	Solution 2 (one RC top beam)	Solution 3 (RC beams in all the floors)
Global base shear force [kN] parallel to the front façade (*x* direction)	1733	1888 (+9%)	1886 (+8%)
Global base shear force [kN] out-of-plane of the front façade (*y* direction)	1474	1756 (+19%)	1785 (+21%)

the collapse mechanism of the structure, instead of the fall out-of-plane of the front façade. The global base shear mechanism probably would be observed if higher stiffness RC beams were adopted.

As shown in Table 5, the value of $\gamma_{\text{sis}}^{\max} = 0.60$ obtained for Solution 1 is higher than $\gamma_{\text{sis}} = 0.45$ obtained for Solution 2. This difference indicates that the front façade resistance to the out-of-plane mechanism increases if a top beam, that provides a continuous support to the façade, is replaced by a connection in a single point in all floors. For Solution 1, the strengthened connections are better distributed along the height of the building and this eliminates the progressive collapse of the façade observed in the study of the building strengthened with Solution 2. This indicates that a better improvement of the seismic resistance can be reached if strengthening interventions are designed based on the knowledge of the building's global behaviour, like Solution 1, and not just to prevent the expected collapse mechanism from occurring, like Solution 2.

7. Summary and conclusions

The previous discussion shows that the strengthening solutions studied increase the building's capacity as regards the resistance to horizontal inertia forces but may have other effects, namely the stiffness increment. This generally leads to an increase of the inertia forces, a negative effect for the building's seismic performance.

Other intervention strategies may aim at producing other types of change in the structural behaviour, such as increasing the energy dissipation capacity, by means of specific devices, or decreasing the inertia forces, for instance by means of base isolation. It is therefore important that the choice of a seismic upgrading strategy consider all the changes in structural behaviour it may induce. Moreover, it is very important to know how the solutions adopted influence the seismic resistance of different collapse mechanisms.

The previous discussion concentrated on comparing some of the most current strengthening strategies. The most usual solutions comprise the strengthening of elements or connections, sometimes without a criterion. In order to assess the efficiency of different solutions, the resistance to the global base shear mechanism and to the out-of-plane fall of the façades was studied. However, it should be mentioned that other mechanisms may exist, and therefore the discussion presented cannot be considered exhaustive, but an illustration of the type of analysis that is considered necessary to decide between different seismic upgrading strategies of old masonry buildings.

By analysing the results obtained, it is possible to infer the relative efficiency of the analysed strengthening solutions. Solution 1 proved to be the most effective one, with an improvement of 140% in the related seismic resistance. Besides, this solution is less intrusive than the introduction of an RC beam, which is a positive factor with regard to the preservation of the cultural (historic) value of the building.

The changes in the dynamic behaviour of the structure due to the introduction of new structural elements, such as the RC beams included in the model (Solutions 2 and 3), may reduce the seismic intensity associated to collapse mechanisms that were not relevant in the original building; this may result, for example, as a consequence of the increment of the global stiffness. If such a mechanism becomes the new collapse mechanism, the strengthening solution may not only change the type of collapse mechanism, but also reduce the efficiency of that solution.

It should also be mentioned that these results strongly depend on the number and distance between the support points of façades (connections to perpendicular walls) and therefore can vary from building to building or even between the façades of the same building.

According to the conclusions of the study, seismic strengthening solutions that do not significantly change the global stiffness and the dynamic characteristics of the structure would be more efficient. The above discussion points to the importance of analysing the efficiency of different upgrading strategies in order to make the best possible use of the funds invested in seismic strengthening.

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