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On the Seismic Strengthening of Old Masonry Buildings

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Summary

A numerical model of an old masonry building from downtown Lisbon was developed for the seismic assessment of its structure. The expected collapse mechanism of the building under earthquakes was thus obtained, as a reference case. The same model was then used for the study of three different strengthening solutions, usually adopted in the strengthening design of old masonry buildings of the same type. These solutions were defined according to the collapse mechanism obtained for the original building. The changes produced in terms of the dynamic behaviour of the structure are then presented, along with the collapse mechanism obtained for each strengthened building. The paper closes with a discussion about the efficiency of each strengthening solution, quantified by the improvement in terms of the seismic resistance of the building.

Introduction

Old masonry buildings are an important percentage of the building stock of most cities. Recent Earthquakes (Italy, 1997, Azores Islands, 1998, Iran, 2004) showed a bad seismic performance of these structures, justifying the concern with their structural safety under seismic actions. Seismic assessment of these structures is fundamental to define anti-seismic provisions regarding the safety of the persons that live and work daily in these buildings. The definition of the expected collapse mechanism of old masonry buildings under Earthquakes usually permits the characterization of the global seismic resistance of the building, therefore its seismic vulnerability. Moreover, the knowledge of the collapse mechanism also allows identifying the weakest links of the structure providing useful information for the design of efficient seismic strengthening solutions.

This paper presents the study of three different strengthening solutions defined according to the expected collapse mechanism of an old masonry building from downtown Lisbon, aiming at providing useful insight to the mitigation of seismic risk of these constructions.

Description of the model

The building analyzed is a ‘Pombalino’ Building from downtown Lisbon with 4 floors above the ground floor, as shown in Figure 1. It is an old masonry structure built after the 1755’s Lisbon Earthquake and includes a three-dimensional timber structure called ‘gaiola pombalina’. This structure is enclosed in the interior of masonry walls above the first floor. The masonry of the exterior walls (masonry walls without the

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'gaiola' structure) is made of irregular blocks of calcareous stone and lime mortar with very poor strength capacity. The ground floor interior walls are masonry walls supporting a system of vaults made of blocks of ceramic masonry and stone arches. Floors are timber slabs and should be considered as flexible diaphragms. A more detailed description of 'Pombalino' Buildings can be found in Cardoso *et al.*[1].

The numerical model developed is presented in Figure 2. A commercial program (SAP2000®[2]) was chosen to perform the structural analysis. *Shell* elements modelled the exterior masonry walls. The floors were modelled as truss bars orientated perpendicular to the façades and simulating flexible diaphragms, restraining the relative out-of-plane displacements of parallel masonry walls. Two crossed rigid diagonal bars connected to masonry walls simulated the masonry vaults of the ground floor. A triangular truss of rigid bars modelled the stone arches of ground floor, which were connected to the interior 'gaiola' walls of the first floor parallels to the main façade and were supported by the interior masonry walls of the ground floor. The foundations were simulated by built-in connections. A more detailed description of the model can be found in Cardoso[3].

Figure 1 – Analyzed building (Prata Street, 210 to 220, *in* Santos[4])

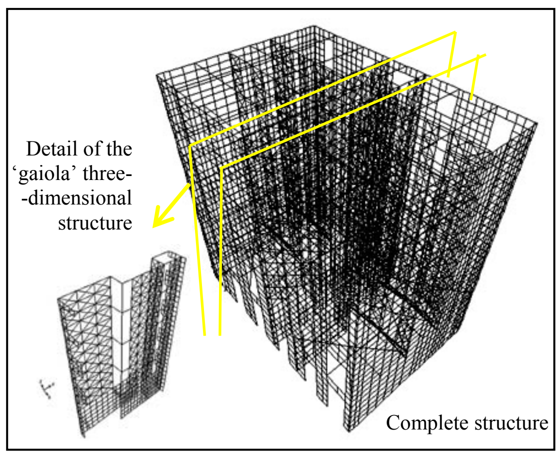

Figure 2 – Numerical model of the building (Cardoso[3])

Table 1 presents the Young's modulus, E, of the structural materials adopted in the numerical model. For the Poisson coefficient of all materials a value of 0.2 was assigned. The self-weight of the roof structure was included in the nodes of the *shell* elements at the top of the building. According to the Portuguese Code RSA[5], a uniform service load (1.2kN/m^2) acting at all the floors was considered. The seismic action was defined by the acceleration response spectrum presented in the mentioned code, acting along the three orthogonal directions. Since the floors cannot be considered rigid in their own plan the mass was distributed by all the nodes of the model.

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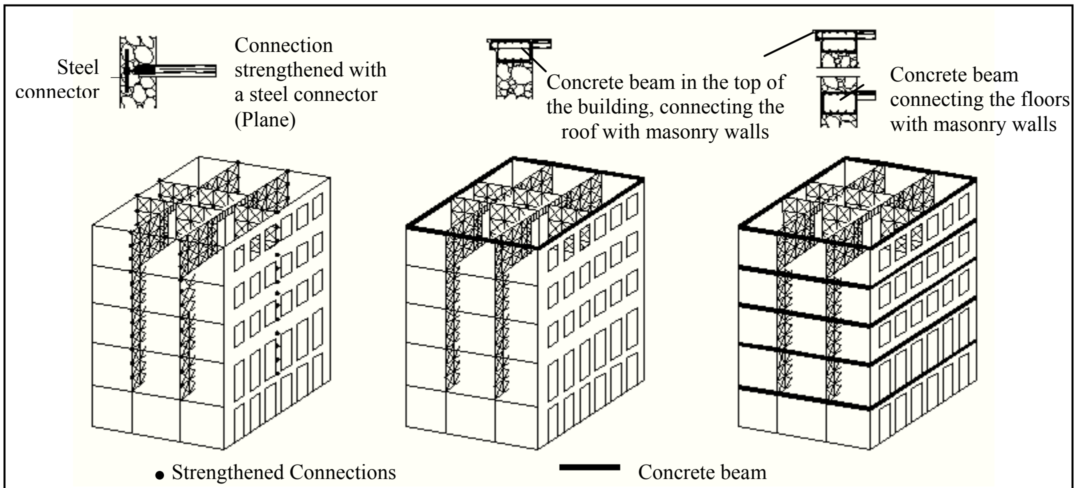
Table 1 – Young modulus adopted in the model for the structural materials (Cardoso[3])

Masonry		*Timber*	*Stone (Calcareous)*
Non-damaged	Damaged (1)	Floors and 'gaiola'	Arches and vaults (2)
600 MPa	150MPa	8000 MPa	3000 MPa

(1) Masonry between perpendicular masonry walls (2) Ground floor

Collapse mechanism of the building and strengthening solutions adopted

According to the results of a previous study (Cardoso et al.[6]), the expected collapse mechanism of the building is the fall of the front façade, starting on the top of the building. With this study it was possible to understand the important role of the connections between 'gaiola' walls and masonry exterior walls in the building's global behaviour and in its collapse. In fact, the overturning of the façade can only occur after the rupture of its connections to the perpendicular 'gaiola' wall. Therefore, the strengthening of these connections is one of the strengthening solutions adopted (Solution 1). Figure 3 shows the strengthening solutions studied, which can be adopted to prevent the collapse due to the overturning of the masonry walls. Beam elements placed around the exterior perimeter of the building, connected to the nodes of the shell elements in the corresponding places of the structure modelled the concrete beams.



Solution 1 – Strengthening of the connections between 'gaiola' walls and the masonry walls of the facades with steel elements.

Solution 2 – Inclusion of a concrete beam ($0.40\times0.25m^2$) around the exterior perimeter on the top of the building, connecting the roof structure to the masonry walls.

Solution 3 – Inclusion of concrete beams ($0.40\times0.25m^2$) around the exterior perimeter of the building, in all the floors, at the pavements level.

Figure 3 – Strengthening solutions adopted

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The rupture of structural elements or connections identifies damages. Damages were obtained by comparing the design actions effects in each relevant element, F_{Sd}, with its respective resistance, F_{Rd} (for F_{Sd} larger than F_{Rd}). Table 2 presents the strength values (F_{Rd}) adopted for the structural elements considered in the analyses. The F_{Sd} values (internal forces) were defined according to equation 1, where F_{Perm} are the effects of vertical permanent loads and F_E are the effects of the seismic action, multiplied by the scale factor γ_{sis} to simulate the intensity of the seismic action.

$$F_{Sd} = F_{Perm} \pm \gamma_{sis} F_E \qquad (1)$$

The collapse mechanism of the building was obtained by observing the evolution of the damage patterns in structural elements, defined for increasing values of γ_{sis}. The value of γ_{sis} corresponding to the collapse mechanism is named γ_{sis}^{max}. This value quantifies the seismic resistance of the structure and allows comparing the efficiency of the different strengthening solutions. For the original building it was obtained γ_{sis}^{max}=0.25 [3].

Table 2 – Strength values adopted for damage calculation (Cardoso[3])

Connections (1)		*Masonry*		
Braced timber bars in 'gaiola'	Timber bars to masonry walls	Compression	Tension	Shear
0 kN	5 kN	1.3 MPa	0.1 Mpa	0.1 MPa

(1) Only rupture due to tension was considered

Results

Figure 4 shows the out-of-plane displacements of the front façade obtained for the three strengthening solutions referred. The displacements in Figure 4 were observed in the connection (M) (between the front façade and its perpendicular 'gaiola' wall) and in the connection (W) – Figure 4. The out-of-plane displacements of the front façade are the difference between the displacements observed in connections M and W. According to the results, the strengthening solutions where concrete beams are used (solution 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.

This result can also be confirmed by means of the dynamic characteristics of the structure for the three solutions - Table 3. By comparing the values of the frequencies of the strengthened structures 2 and 3 with the correspondent values for the original building it is evident an increase of the global stiffness, mainly for solution 3 in which a higher number of reinforced beams were considered. Nevertheless, the stiffness of the structure is not changed for solution 1. As expected, the increase of stiffness is more relevant for the direction perpendicular to the front façade, the out-of-plane direction. This can be confirmed by the changes observed in the first modal configurations before and after strengthening, marked bold in Table 3.

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In this study, the mechanisms analysed were the overturning of the front façade and the global shear mechanism [3], [6]. The values of γ_{sis} obtained for each mechanism are presented in Table 4. The collapse mechanism is the one corresponding to the lowest value of γ_{sis} (bold values in Table 4).

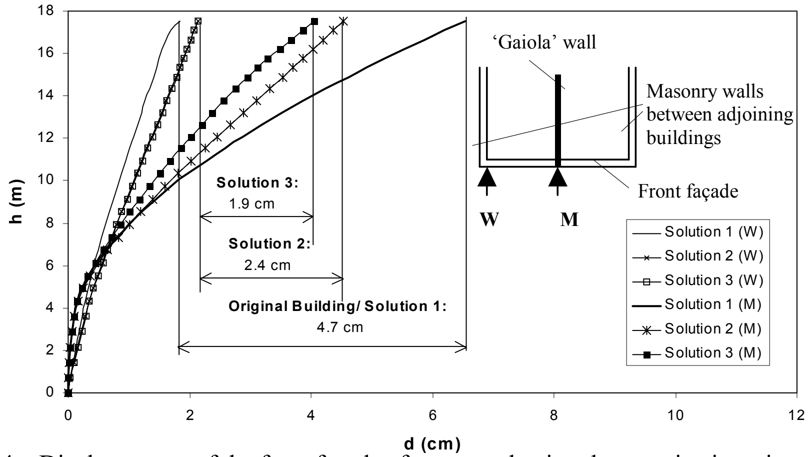

Figure 4 – Displacements of the front façade after strengthening due to seismic actions (γ_{sis}=1.0)

Table 3 – Results of the dynamic analyses of the buildings after strengthening (first modes)

Mode	*Original Building/Solution 1*		*Solution 2*		*Solution 3*	
	F [Hz]	Mode configuration	F [Hz]	Mode configuration	F [Hz]	Mode configuration
1st Mode	0.942	**Translation perpendicular to the front façade**	1.187	Translation parallel to the front façade	1.280	Translation parallel to the front façade
2nd Mode	1.055	Translation parallel to the front façade with torsion	1.265	**Translation perpendicular to the front façade**	1.325	**Translation perpendicular to the front façade**
3rd Mode	1.196	Translation parallel to the front façade with torsion	1.717	Translation perpendicular to the front façade with torsion	1.187	Translation perpendicular to the front façade with torsion

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

Collapse mechanism	*Original*	*Solution 1*	*Solution 2*	*Solution 3*
Overturning of the front façade	**0.25**	**0.60**	**0.45**	**0.50**
Global shear base mechanism	0.70	0.70	0.60	0.55
Increment of the seismic resistance of the original building	-----	140%	80%	100%

Table 4 shows that the expected collapse is still the overturning of the front façade for all the strengthening solutions studied. Solution 1 is the most efficient of all because it corresponds to the highest increment of the seismic resistance (140% for solution 1, 80% and 100% for solutions 2 and 3). For solution 3, the seismic resistance of the structure for

Proceedings of the 2004 International Conference on Computational & Experimental Engineering & Science 26-29 July, 2004, Madeira, Portugal

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global shear mechanism is similar to the resistance obtained for the overturning of the front façade, being expected a mechanism that combines both mechanisms.

The values of γ_{sis} obtained for the global shear mechanism (Table 4) decrease with the increase of the global stiffness. The increment of the stiffness leads to higher values of the seismic action effects (the inertia forces increase). The resistance of the structure to the global shear mechanism decreases and, at the limit, this mechanism may become the collapse mechanism of the structure, instead of the overturning of the front façade.

Conclusions

According to the obtained results, it is possible to infer about the efficiency of the analysed strengthening solutions, as they improve the seismic resistance of the building. Solution 1 proved to be the most effective one with an improvement of 140% in the associated seismic resistance.

The changes on the dynamic behaviour of the structure due to the introduction of new structural elements made of different materials, such as the concrete beams included in the model (solutions 2 and 3), may lead to collapse mechanisms that were not relevant in the original building. Moreover, the seismic intensity associated to collapse mechanisms that were not relevant in the original building may be reduced due to the strengthening intervention adopted; 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 might, not only change the type of collapse mechanism, but also reduce its efficiency.

According to the conclusions of the study, seismic strengthening solutions that do not change significantly the global stiffness and the dynamic characteristics of the structure, would be more efficient.

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