Paper 85

Proceedings of the Eighth International Conference
on Computational Structures Technology,
B.H.V. Topping, G. Montero and R. Montenegro,
(Editors), Civil-Comp Press, Stirlingshire, Scotland.

Seismic Evaluation of Old Masonry Buildings: Performance and Strengthening

H. Varum†, R. Vicente†, H. Rodrigues† and J.A.R. Mendes da Silva‡
† Department of Civil Engineering
University of Aveiro, Portugal
‡ Department of Civil Engineering
University of Coimbra, Portugal

Abstract

The concern and need to assess seismic vulnerability, particularly of the traditional masonry buildings under seismic actions is a key issue, that should be a priority in the mitigation of the seismic risk, definition of strengthening requirements needs and minimization of possible damages due to seismic actions, in the identification of critical buildings and safeguarding of built heritage.
This paper provides information on the constructive and structural details of the traditional buildings in Coimbra, Portugal, and interprets the potential structural damage and discusses its seismic behaviour, identifying structural fragilities and consequently their vulnerability. It also presents the main results obtained in the numerical studies, and verifies the global stability and dynamic response to seismic actions. Three different strengthening techniques to improve the global behaviour of these buildings were modelled and analysed. Efficiency of the strengthening strategies is also discussed in terms of deformation demands and cost-benefit analysis.

Keywords: masonry buildings, seismic vulnerability, strengthening techniques.

1 Introduction

Old load-bearing masonry buildings exist all around the world, with special significance in urban historical city centres. These buildings, besides their patrimonial, cultural and architectural heritage value, frequently present a high level of degradation, urging for the need of conservation and strengthening actions.

Recently, the consciousness of the public opinion begun to evidence to this need through the creation of safeguarding and preservation policies for the architectural valued buildings and urban aggregates. The inoperativeness of the responsible agents and the lack of strategies and policies in the last half of the XX century in this domain drove the built urban stock to a situation of deep degradation in a great number of historical centres [1]. Worsening this context, it is witnessed the adoption of intrusive and inadequate rehabilitation and conservation practices, using new materials and construction techniques (concrete) on structural and non-structural elements, moving away the knowledge of traditional practices, the capability and connection of the solutions with the existent, leading to the discharacterization of the urban and patrimonial image.

A great percentage of the built urban stock of the historical city centre of Coimbra is constituted by buildings dated of XVIII to the mid XX century (after the 1755 Lisbon earthquake), most of them built without any earthquake resistant design (no specific construction rules). Even the later constructions do not follow the seismic resisting system "gaiola pombalina", developed after the Lisbon earthquake, neither appropriate construction rules nor techniques.

In prone areas of seismic action (Central and Southern Portugal), the need to take preventive measures of structural strengthening to minimise the damages, or avoid losses of incalculable value is surely a priority. Such measures require a previous evaluation of the expected seismic response through modelling representative buildings of this type of construction.

In this paper is presented a study of an aggregate of four typically masonry buildings representative of the constructive typology of the old masonry buildings in Coimbra, Portugal. In this study were performed numerical analysis of four buildings and was tested the efficiency of three typical strengthening techniques. Finally, it is discussed the seismic behaviour, identified structural fragilities and the efficiency of the strengthening techniques studied concerning cost analysis.

2 Aggregate description

The aggregate of buildings studied is included in the irregular urban mesh of the old city centre of Coimbra (see Figure 1). In this area of the city, a renewal and rehabilitation process is taking its first steps as a collaborative framework between the local authorities and the University of Coimbra [1].

The buildings studied belong to the oldest area of the historical city centre, featuring architectural aspects (one direction staircase, stone framing and window glazing characteristics) which evidence that these buildings belongs to the period between the XVIII and XIX century (see Figures 2 and 3).

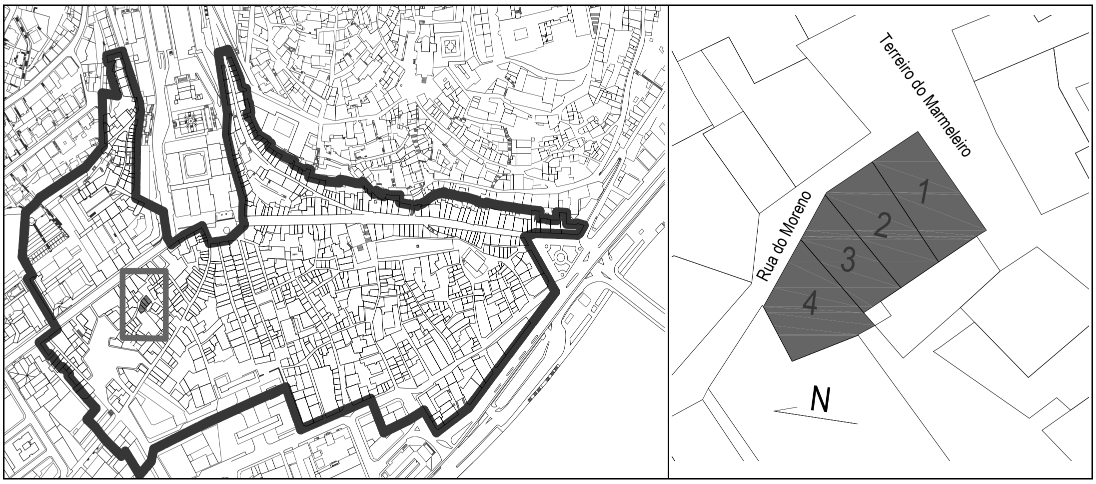


Figure 1: Perimeter of the old city centre and building's aggregate studied

Figure 2: Building drawings and layout

Figure 3: Building façades of the four buildings

An important aspect is the evolution of the urban layout, because of the chronological construction process in which: adjacent buildings share load-bearing masonry walls and others use existing masonry and partition walls for floor and roof support and connections. The buildings do not constitute independent units given that they share the mid-walls with adjacent buildings. This way, the buildings do not have an independent structural behaviour, but they interact amongst themselves, particularly for horizontal actions and so the structural performance should be studied at the level of the aggregate and not for each isolated building.

This reality is important not only for the vertical load-bearing capacity but also for seismic actions, and hence seismic vulnerability. Most of the buildings lack of good connections between walls and particularly at wall angles. Cracking and collapse of the front and back façades during earthquakes is the most frequent failure mechanism, caused by their fragility and particularly to the deficient connection to the perpendicular load-bearing walls

Based on the analysis of the geotechnical reports, the four buildings are founded on silty clay and sand soil layers with some gravel and filling material. Each of those buildings has approximately a rectangular plan, with exception of building 4 located in the N-W corner of the group, which possesses a trapezoidal configuration in plan.

Regarding the geometry in height, buildings 1 and 2 (in the S-E quadrant) are constituted by ground floor, two elevated floors and an attic. Buildings 3 and 4 are composed by ground floor, three elevated floors and an attic. Typically, these buildings have no basement, since the major area of this part of the historical centre of the town is quite close to the river.

Architectural typology and construction solutions are variable in function of the dimension and nobleness of buildings. In respect to housing buildings, very simple structural schemes are found: load-bearing external stone masonry walls and wooden floor slabs and roofs (see Figure 4).

In the majority of buildings that were inspected, and in particular these four buildings, it was observed the systematic use of wood, in the floors, roofs and interior partition walls. Mainly, it was registered the abundant use of dolomitic limestone in external load-bearing walls and the wall thickness varies, normally, in height from a mean value of 50cm (at ground level) to 26cm at roof level. The use of river sand for bed joints and external mortar renderings is also very common. In most cases roofs are covered with clay tiling. Window sashes are predominantly in wood with simple glazing windows. Interior partition walls are thin and sometimes suffer warping, revealing some kind of structural deformation, often as consequence of creep and aging phenomena.

Masonry walls combined with the wooden floor slabs constitutes the dominant structural scheme resulting in a very simple box-type structure. The masonry fabric is constituted by stones of small to medium dimension, linked with lime and clay mortar. Some of the thinner masonry (near openings and staircase structures) incorporate crossed timber elements. These stone masonry walls expect to have globally a good behaviour in compression, usually induced by gravity forces, and a poor performance for out-of-plane flexural, in-plane shear or tensile actions, depending on the geometric characteristics of the masonry, to their connection, and to the materials characteristics (stone size, masonry arrangement and stone laying, type of transversal connection between wall faces, type of natural stone and type of mortar).

The floors are considered as flexible diaphragms with small beams and joists with sections of $0.10x0.20m^2$, disposed perpendicular to the mid-walls (parallel to the façades). The wood frequently used is Portuguese pitch-pine wood and, in some cases, oak and chestnut.

The roofs are typically sloped in two directions, the timber roofing structure is constituted by timber elements of $0.10x0.16m^2$ for rafters and beams and 0.12x0.20m2 for the roof ridge beam. These roofs exert an outward thrust on the supporting walls and other are framed as to impose just a vertical resultant reaction to the supporting walls.

Types of load-bearing masonry walls	Wooden floors	Timber roofing system	Interior partition walls (lath work with mortar)

Figure 4: Typical construction details of old housing in Coimbra

3 Numerical simulation

To understand the behaviour of the old constructions, an aggregate of four buildings was modelled with a finite element tool. The results of these models will aid in the identification of fragile areas of the buildings and in the vulnerability evaluation of the aggregate. With this numerical analysis it is intended: i) to estimate the natural frequencies and vibration mode shapes, for the original structure and for different strengthening solutions; and, ii) to understand the global seismic response of the structure through global parameters in terms of top-displacements and drifts.

3.1 Finite element model and material properties

Numerical analyses were preformed in a finite element program, Robot Millenium v17.5 [2]. The buildings geometry was defined from available drawings in digital format (CAD) and was confirmed with technical visits. The elements used in the definition of the global three-dimensional model were four-node shell elements for the masonry panels, and two-node frame elements for timber beams, joists and rafters, as shown in Figure 5.

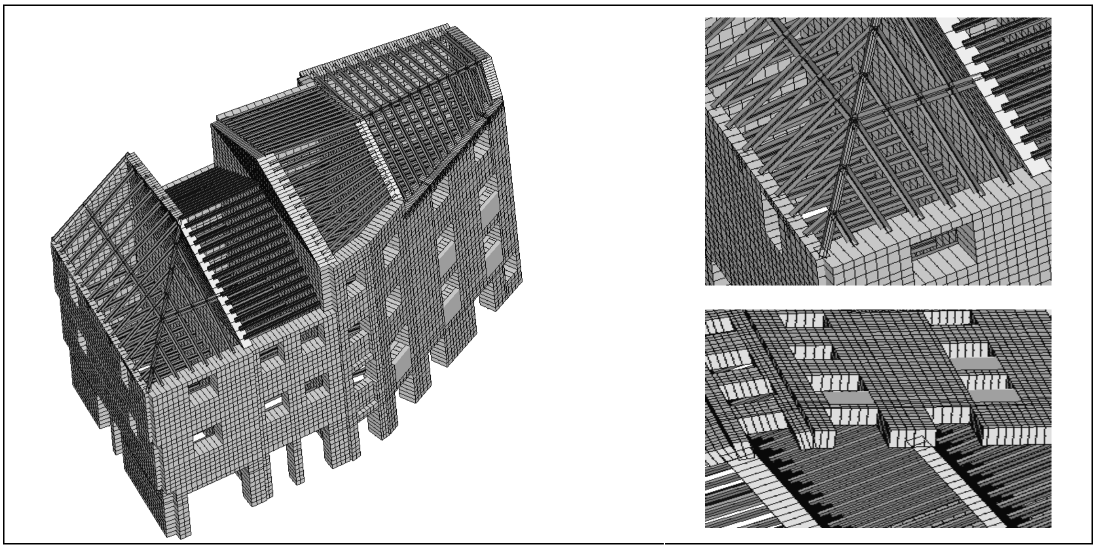


Figure 5: Global three-dimensional model

The linear elastic models can supply important results for a first global evaluation of traditional structures, particularly in what concerns the identification of critical regions and also helps in the analysis of potential causes of observed structural damages.

A finite element model should be capable of well representing the global behaviour of the construction and in detail particular regions with distinctive behaviour (elements connection and compatibility, linkage quality, material characteristics). Therefore, some basic assumptions were considered and must be put forward:

- Two types of masonry materials were used, namely one for common masonry walls and other for the thinner stone panels (under window panes);
- Consideration of linear elastic behaviour for all structural elements;
- Rigid support conditions in all points at the base of the walls, restraining the displacements and out-of-plane rotation;
- Assumed behaviour factor equal to 1, corresponding to the typical characteristics of these materials (poor ductility and energy dissipation capacity);
- The roof structure system of the building number 2 (see Figure 2) was rehabilitated in the last decade and is constituted by precast concrete beams.

Regarding the structural elements, representative values collected from bibliography were used for timber and stone masonry mechanical properties [3, 4]. In Table 1 are shown the material properties considered in the analysis.

Material properties	Masonry	Stone panels	Timber elements	Concrete beams
Modulus of elasticity, E (MPa)	E = 320	E = 3000	E = 6000	E=35000
Volumetric weight, γ (kN/m^3)	γ = 19.6	γ = 20.0	γ = 6.0	γ = 25.0
Poisson ratio, ν	ν = 0.2	ν = 0.2	ν = 0.2	ν = 0.2
Compression strength, σ_c (MPa)	σ_c = 1.0	σ_c = 3.0	σ_c =11	σ_c = 17.0
Tensile strength, σ_t (MPa)	σ_t = 0.05 (theoretically zero)	σ_t = 0.05 (theoretically zero)	σ_t = 18.0	σ_t = 2.5
Shear strength, τ_u (MPa)	τ_u = 0.04 (depends on normal stress)	τ_u = 0.05 (Mohr-Coulomb)	τ_u = 2.0	--

Table 1: Properties of the structural materials considered in the numerical model

3.2 Static loads and spectral analysis considerations

In order to evaluate the seismic performance of the buildings, a spectral analysis was performed modelling the seismic action by means of a acceleration response spectrum, acting along the two independent horizontal directions. The acceleration spectrum used, presented in figure 6, is based on the Portuguese Standard [5] (seismic action type II - far-distance earthquake, soil type II – coherent soil, 2% damping and seismic zone C).

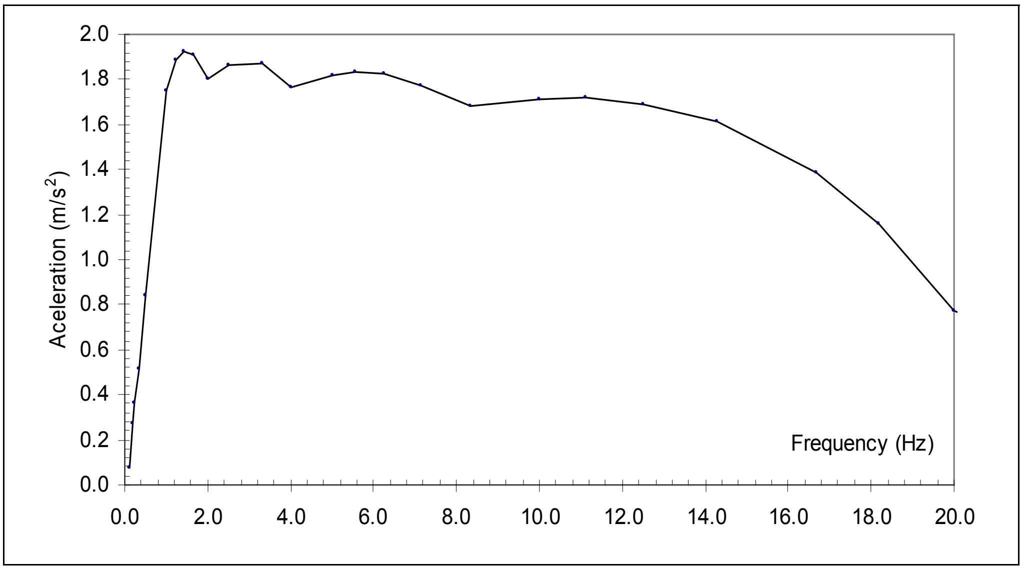


Figure 6: Response spectrum (Portuguese standard, RSA)

According to the Portuguese Code, the modal analysis is performed for the serviceability limit state combination ($1.00 \cdot G_k + 1.00 \cdot \psi_2 \cdot Q_k$). The permanent loads ($G_k$) contemplate the self-weight of the structural and non-structural elements, as presented in Table 1 (masonry walls, timber roof and floor members, coverings and interior partition walls). The live load (Q_k) considered for the floors is 2.0kN/m^2 and for roofing structures 1.0kN/m^2.

3.3 Strengthening solutions studied

Rehabilitation and structural interventions to improve the seismic behaviour of traditional masonry buildings should respect the original building materials and construction techniques [6]. The numerical model developed was also oriented in the sense of evaluating suitable strengthening solutions. Three strengthening solutions were modelled intending to reduce the building's seismic vulnerability namely: floor stiffening, tie-rods and masonry consolidation.

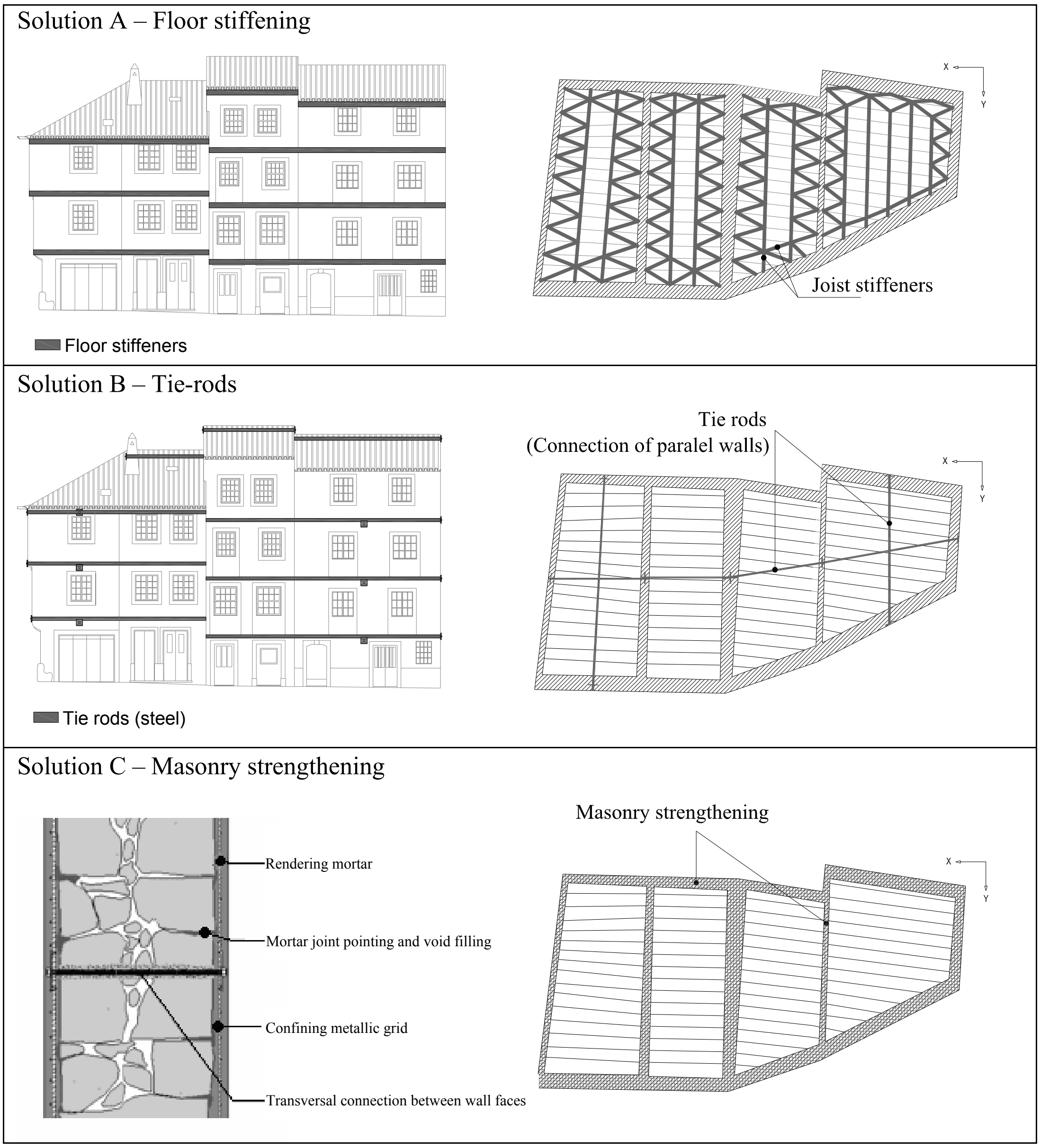


Figure 7: Rehabilitation schemes

A possible action to improve the structural seismic behaviour could be through the floor stiffening. The in-plane stiffening of the floor diaphragms was modelled by introducing diagonal and orthogonal timber bars with similar characteristics to the original wooden slab framework, as shown in figure 7 (Solution A). The introduction of tie-rods at floor level and roof-ridge level to retain and prevent the out-of-plane mechanisms was another studied solution. Steel tie-rods are the less intrusive rehabilitation technique proposed (see Figure 7 – solution B). The tie-rods were modelled with the truss elements only with tensile strength, with the geometrical and mechanical characteristics indicated in Table 2. Taking into account that the typical stone masonry of these buildings has reduced shear and flexural strength, a third solution studied is the wall strengthening, based on the improvement of bond conditions using transversal wall connectors, mortar joint pointing, void filling and confining stainless steel mesh embedded in a plaster mortar layer (see Figure 7 - solution C). This measure was modelled by increasing the average elasticity modulus of the masonry in 75%, value adopted from the analysis of experimental studies performed by Costa [7]. Even though the connection quality between walls is not evaluated in this study, it is underlined the crucial importance of an efficient connection between main structural elements (walls-floors, roofs-walls, walls-perpendicular walls) in the global response.

Material properties	Strengthened masonry walls	Steel tie-rods
Modulus of elasticity, E (MPa)	E = 560	E = 210000
Volumetric weight, γ (kN/m^3)	γ = 19.6	γ = 77.0
Poisson ratio, ν	ν = 0.2	ν = 0.2

Table 2: Properties of the materials considered in the strengthening schemes studied

4 Result analysis

4.1 Natural frequencies and mode shapes

To control the structural behaviour changes induced by the structural strengthening actions, it is important to estimate the dynamic characteristics of the constructions under analysis (natural frequencies and vibration modes). Therefore, four different models were studied: i) Original structure; ii) Retrofitting solution A (tie rods); iii) Retrofitting solution B (floor stiffening); and, iv) Retrofitting solution C (masonry strengthening). The results for natural frequencies are summarized in Table 3.

Model	Frequency (Hz)			
	1st Freq	2nd Freq	3rd Freq	4th Freq
Original structure (masonry walls with timber roof and floors)	2.390	3.257	3.841	4.523
Retrofitting solution A (tie-rods)	2.610	3.374	4.050	4.763
Retrofitting solution B (floor stiffening)	3.104	3.546	4.733	6.151
Retrofitting solution C (masonry strengthening)	3.002	4.170	4.930	5.547

Table 3: Natural frequencies

About 70% of the total building´s mass is due to the masonry walls. Therefore, the total mass of the structure does not change significantly with the strengthening strategies studied (tie-rods or floor stiffeners). From the analysis of the natural frequencies estimated, the following can be concluded:

- The increase of the first natural frequency due to installation of the tie rods is about 9.4% in relation to the original structure. However this technique is most effective in the out-of-plane deformation control;
- The use of floor stiffeners increases the first natural frequency of about 30%, inducing a global rigid mechanism response;
- The masonry strengthening technique increases the first natural frequency of around 26%.

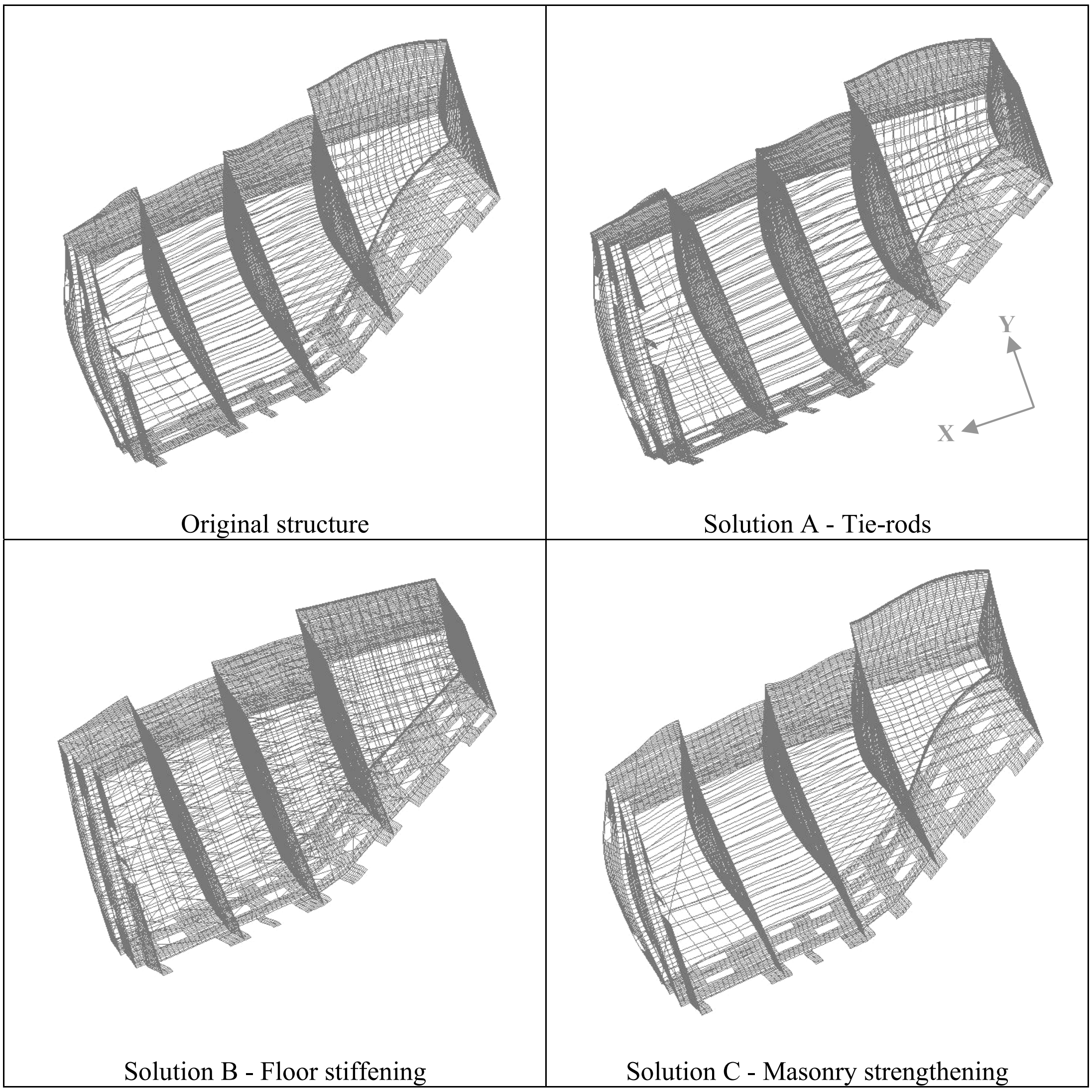
From the analysis of the modal shapes, presented in Figure 8, the following comments can be made:

- The first mode, for all the structural systems analysed, shows essentially the translation along the longitudinal direction (X). For the first modes, the ground floor presents a significant deformation, due to the number and large dimensions of openings in direction X. The first mode shapes evidences the high vulnerability of some masonry walls to the out-of-plane mechanism (façade wall of buildings 1 and 4, and internal mid-walls);
- With the retrofitting solution A (tie-rods), for the first natural mode, the out-of-plane mechanism of the masonry walls is smaller than in the original structure;
- With the retrofitting solution B (floor stiffening), the increased in-plane stiffness of the floors reduces significantly the out-of-plane mechanism of the walls;
- The masonry strengthening solution (C) produces a similar first mode shape than the estimated for the original structure, with an increase in frequency of 26%;

In situ measurements were carried out, using a seismograph, with the purpose of calibration of the numerical model. However, due to the complexity of the structural aggregate and difficulties in the excitation of the structure, the signal measured has not produced credible results, therefore these dynamic results were not used in this study.

4.2 Influence of the diaphragm stiffness in the structure response

These structures have a significant use of timber elements, such as for floor and roof structures, which can alter significantly the structural response. To evaluate the influence of timber elements stiffness of, it was performed a parametric study varying the stiffness of these elements in the original structure model, evaluating the changes in terms of first natural frequency and corresponding modal shapes (see Figure 9).



Original structure | Solution A - Tie-rods | Solution B - Floor stiffening | Solution C - Masonry strengthening

Figure 8: First vibration modes

From the variation of the timber stiffness, we observed that the global mode shapes resulting are similar to the original structure mode shape and do not produce significant changes in the first mode shape. Hence that this characteristic must be verified in order to conclude the following:

- The timber structure stiffness decrease does not change significantly the first frequency, compared to an increase of the same range (e.g: K_f/K_i=0,01 and K_f/K_i=100);
- Taking, for example, a situation representative of a decrease of stiffness - degradation of mechanical characteristics and physical properties

(deformation and poor condition state) this result reveals a reduced effect on the global behaviour. However, the increase of the timber elements stiffness improves the global response of the structure. As a measure of increased stiffness of the timber structure, the floor stiffening solution is equivalent to an increase of about 60 times in relation to the original structure.

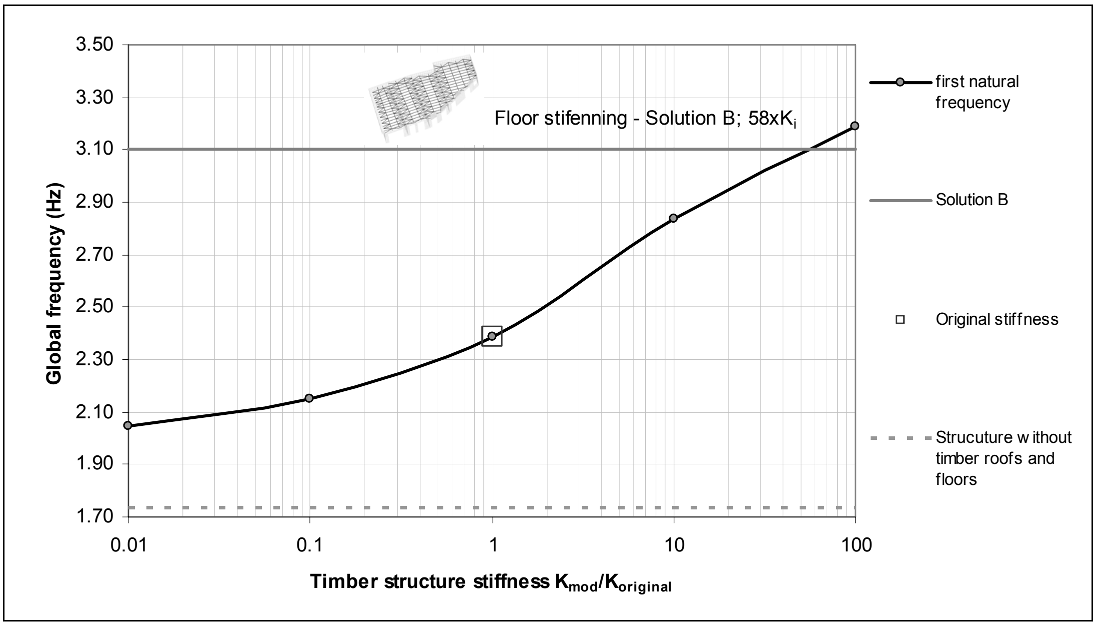


Figure 9: Modification of structural response due to timber structure stiffening

4.3 Displacement profiles and collapse mechanisms

From the observation of damaged masonry buildings, in recent earthquakes, it is evident the concentration of damage at regions with highest demand, such as corner angles and façades with large openings. In this section are presented the displacement profile, numerically evaluated, at crucial points of the structure: corner angles, front façade and internal mid-walls. For both directions (X and Y) the most relevant results obtained with the spectral analysis for the design earthquake level for Coimbra region are presented, since sixteen points were controlled.

Analysing control point P2 it was evaluated the efficiency of the simulated strengthening solutions in the reduction of the out-of-plane masonry façade wall mechanism. From the displacement profile at this point (see Figure 10), it is clear that masonry wall strengthening better reduces globally the displacements (reduction of about 25%), with exception of the upper level. However, the floor stiffening solution reduces the top-displacement more efficiently (reduction of about 36%).

The tie-rod strengthening solution have a insignificant effect on the deformation control of the corner angle (as shown in Figure 11, for point 3), but does help in mobilizing the global response of the structure as a whole. Hence, the principal function of the tie-rod, referring to its reduced axial stiffness, is to control the out-of-plane deformation of the façades, and not to stiffening the structure, as produced by the masonry strengthening. The floor diaphragms stiffening have a negative effect in the in-plane displacements. The masonry strengthening is clearly the retrofitting technique that most reduces the top-displacement (of about 37%).

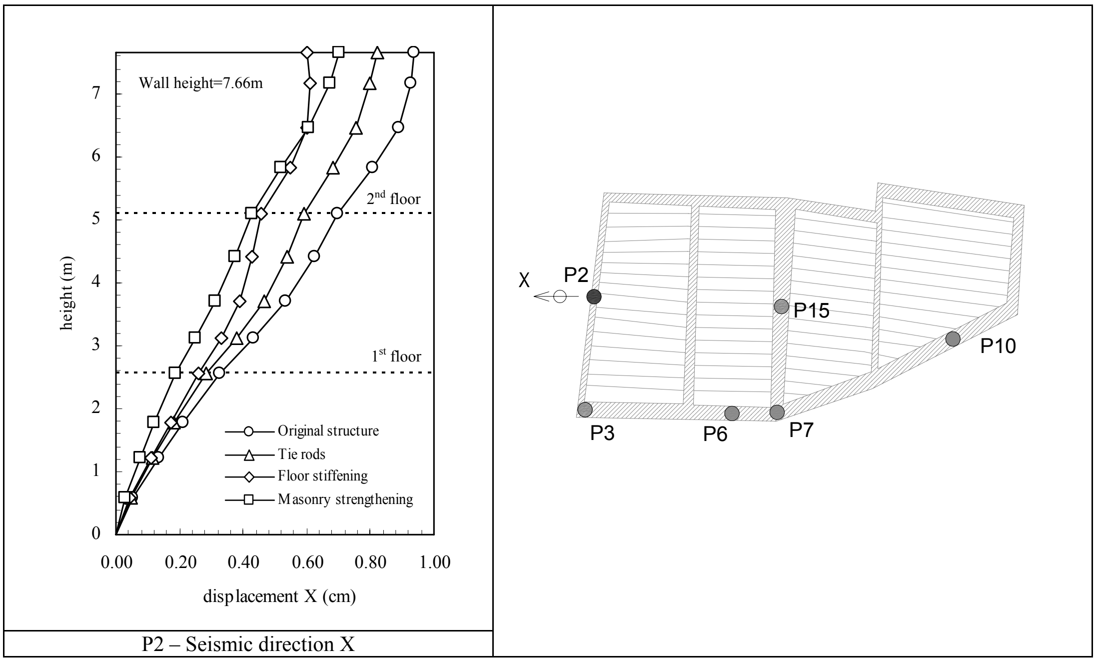
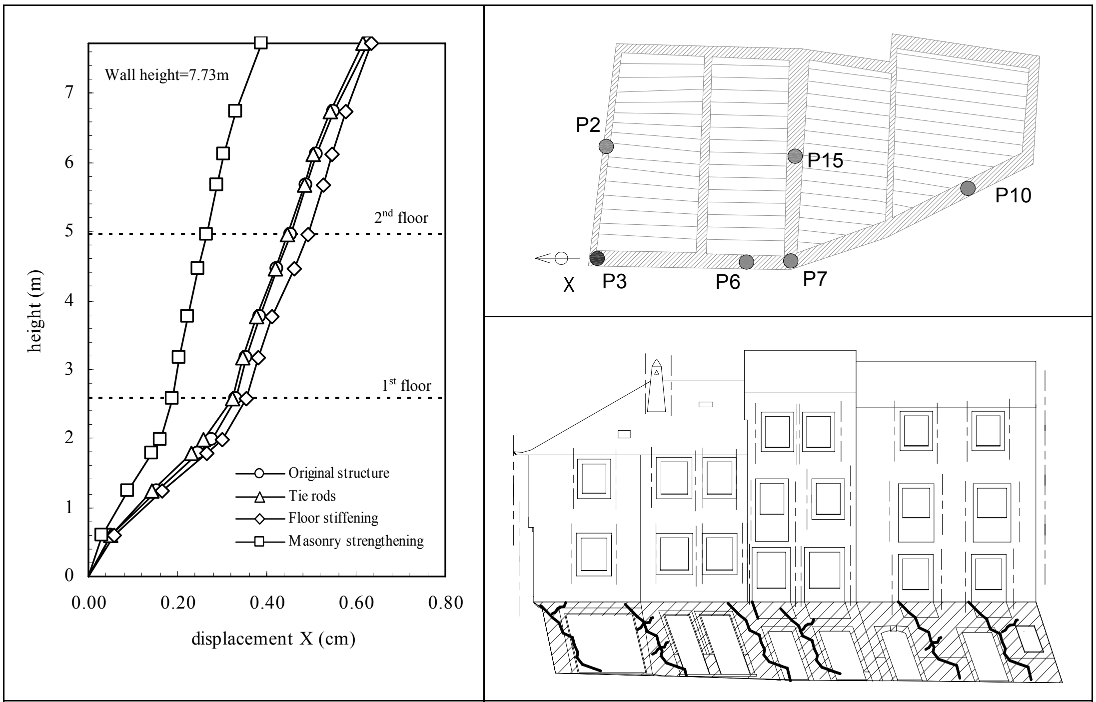


Figure 10: Displacement profile at point 2

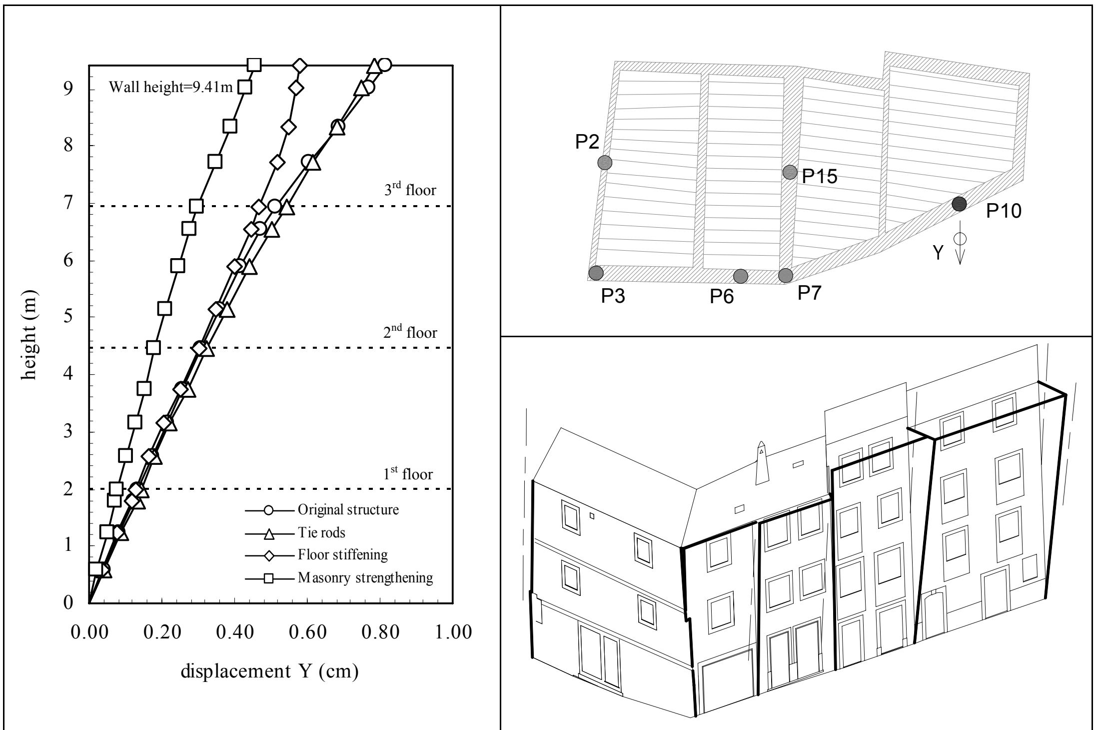
For the original structure, as for all the strengthening techniques analysed, the displacement profile in direction X, as shown in Figure 11, evidences a soft-storey mechanism at the ground level (deformation at the first storey height represents about 50% of the top displacement), which indicates the aggregate vulnerability for seismic actions in this direction.

From the analysis of displacement results at point 10 (Figure 12), the following can be concluded: i) the tie-rod is not efficient in the reduction of the flexural deformation of the façade wall, and slightly reduces the top-displacement (of about 4%); ii) the floor stiffening retrofitting technique reduces the out-of-plane deformation of the façade walls of about 29%, however, this efficiency is only verified if the all the retrofitting is adopted at all floor levels, including the attic level. It has been observed in previous work, that when this action is not taken out for higher floor levels, even though deformation control is interesting for lower levels (usually the attic and roof is not stiffened), the horizontal displacement is amplified, and originates higher displacements and deformation at the superior levels in relation to the original structure. Once again, the masonry strengthening is the most efficient method reducing about 44% of the top-displacement. For all displacement profiles for out-of-plane façade movement it reveals an overturning mechanism as shown in Figure 12.



P3 – Seismic direction X	Soft storey mechanism

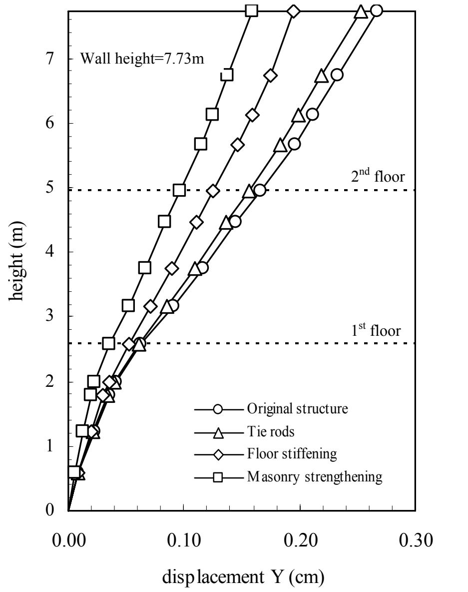
Figure 11: Displacement profile at point 3



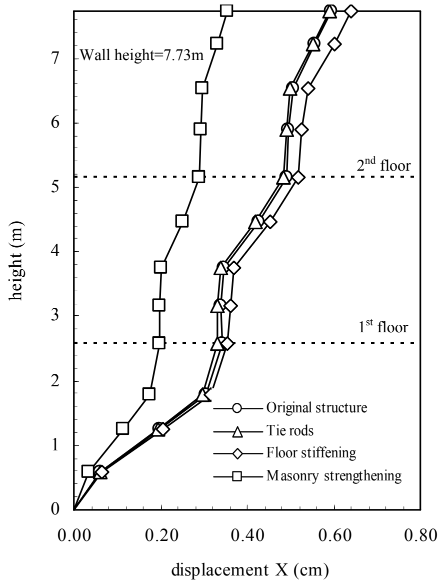
P10 – Seismic direction Y	Global overturning of façade wall

Figure 12: Displacement profile at point 10

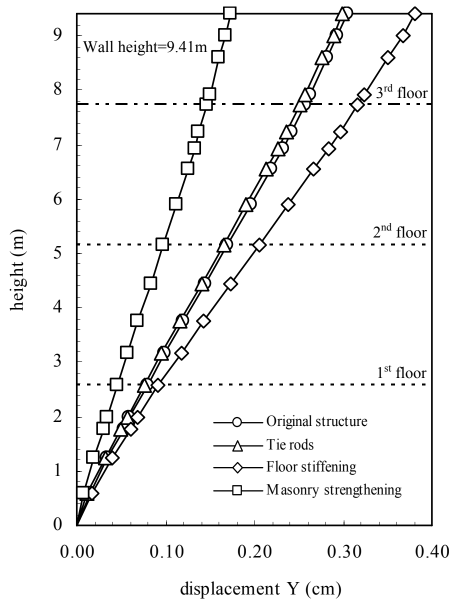
From the previous displacement profiles it is fairly conclusive on the seismic performance and behaviour mechanisms originated. Moreover by observing Figure 13, the point 6 displacement profile shows soft-storeys mechanism at higher storey, due to the high percentage of opening at all floor levels. In the Y direction (in-plane) for control point 3 (façade wall) and 7 (mid-wall) as shown in Figure 13, once again demonstrates the flexural building behaviour in this direction because of the extensive wall development and low opening percentage.



P3 – Seismic direction Y



P6 – Seismic direction X



P7 – Seismic direction Y

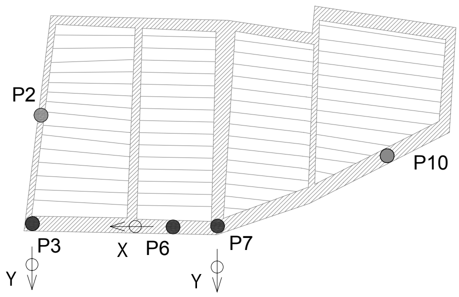


Figure 13: Displacement profile for points P3, P6 and P7

In the next points it will be discussed and compared the efficiency of the strengthening solutions proposed, through drift profiles and cost-benefit analysis.

4.4 Improving structural integrity

To evaluate the efficiency of the retrofitting solutions proposed, it was appraised the reduction of global deformation parameters representative of the aggregate structural response, namely top-displacement at points, P2, P3, P7 and P10 (for out-of-plane and in-plane mechanism), relating the reduction of the deformation with the cost of the strengthening action. Specialized construction contractors in the refurbishment and structural retrofitting of buildings were consulted to budget all tasks for each strengthening solution studied. The obtained mean values are summarized in the Table 4:

Original building aggregate value:	400,000.00€	
Retrofitting technique	Cost	Retrofitting action / Building value
Solution A – Tie rods	8,000.00€	2%
Solution B – Floor stiffening	48,000.00€	12%
Solution C – Masonry strengthening	80,000.00€	20%

Table 4: Original building value and retrofitting costs

In Figure 14 is represented the top-displacement reduction in function of the strengthening cost. The tie rod solution is a very low cost action, representing 2% of global building value, but has a low efficiency for all the studied points in terms of out-of-plane and in-plane deformation reduction.

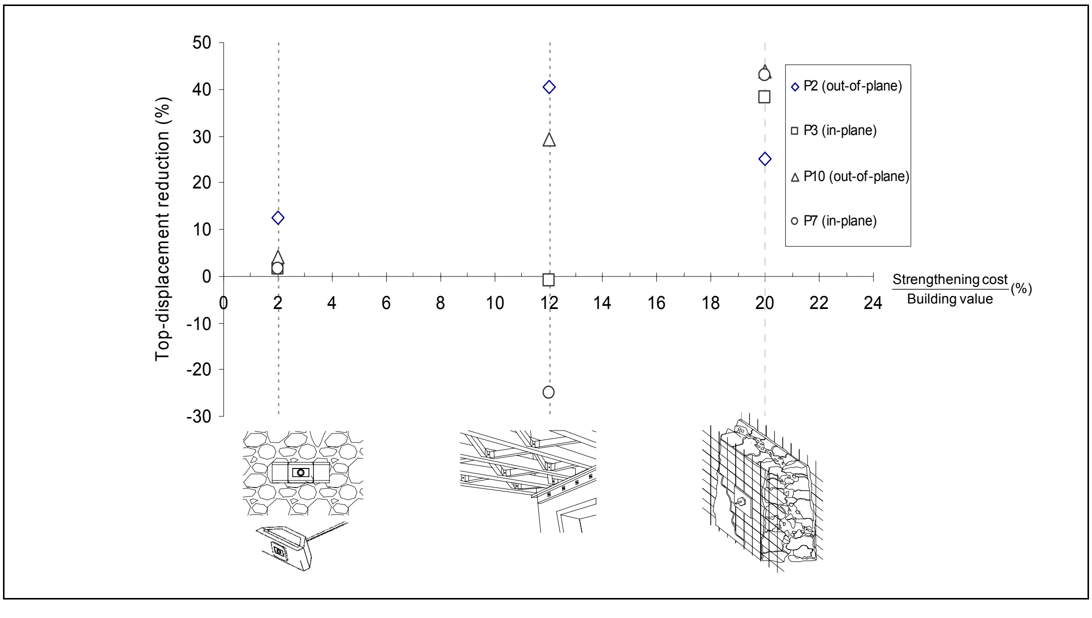
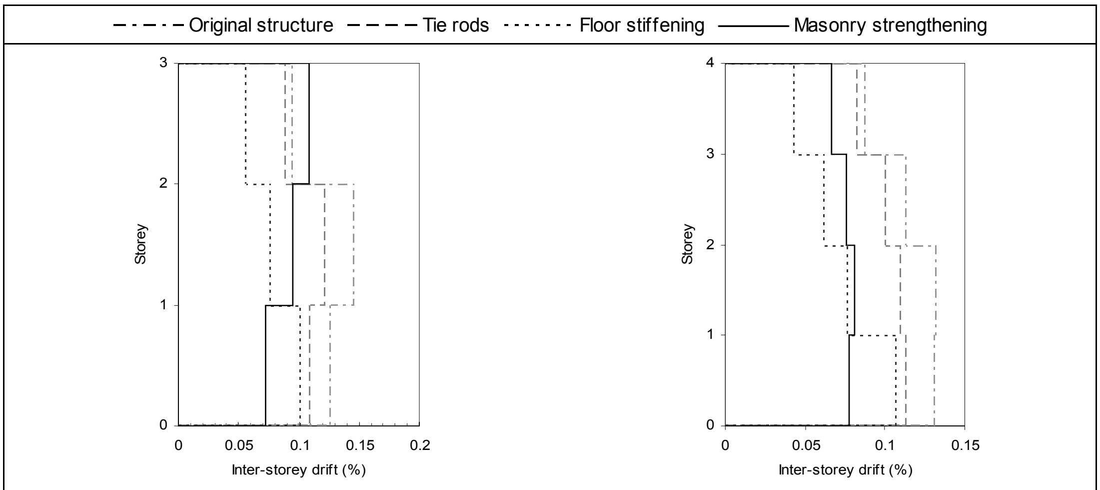


Figure 14: Strengthening efficiency versus strengthening cost

The floor stiffening is very effective in the control of the out-of-plane deformation (point P2 and P10). However, the increase of the floor stiffening induces a negative effect in the in-plane deformation demands at the same points (point P3 and P7). The masonry strengthening is the most globally efficient solution, but the most expensive and costly solution (about 20% of the original building value). Comparing this solution with the floor stiffening technique, it is much more expensive (see Table 4) and the reduction of the out-of-plane deformation is roughly similar. Generally the in-plane top-displacements are very low. However, the most probable collapse is governed by the out-of-plane mechanism of the façade wall deformation concentrated at ground level. If the objective is the strengthening to optimize the ratio cost-benefit, the floor stiffening seems to be the better solution.

4.5 Drift profile

Comparing the drifts with the defined performance limits in FEMA 356 [8], it is revealed that for the low to moderate seismic action adopted in reference to the site location, all the studied points do not verify the first performance level for the in-plane deformation – IO (Immediate Occupancy, drift limit 0.1%), expecting slight damages (small crack openings) entering the non-linear behaviour. In Figure 15 and 16, analysing the inter-storey drift profile for point 2 and 10, it can be seen the good reduction of the in-plane drifts below the IO limit for the floor stiffening and masonry strengthening for point 2 and 3, respectively. Even though the FEMA-356 document [8] does not indicate out-of-plane inter-storey drift limits for unreinforced masonry walls, it however indicates a geometrical control procedure; height-to-thickness ratio and a damage state control, based on floor accelerations and velocities. In spite of this, as shown in Figure 15, for point 10 the drift for the original structure at the last storey level is highly reduced when introduced the floor stiffening measure. On the other hand, the floor stiffening solution does not reduce significantly storey drifts at the lower levels.



Point 2 – In-plane | Point 10 – In-plane

Figure 15: Inter-storey drift profiles

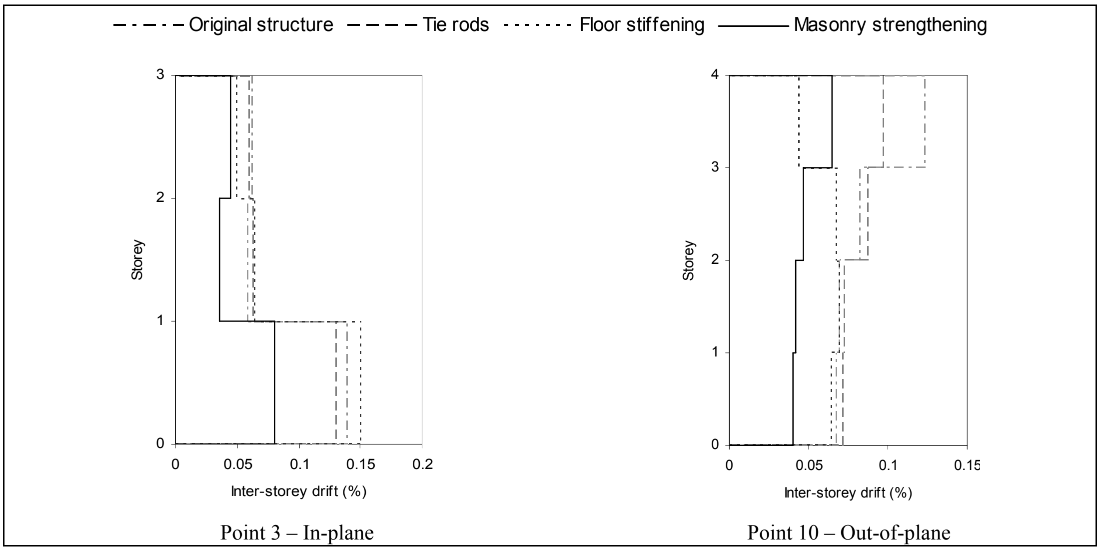


Figure 16: Inter-storey drift profiles

5 Conclusions and final remarks

This study has allowed better understanding the seismic performance of this type of buildings. The numerical analysis performed allows to state:

- Numerous and large openings, particularly at ground storey induces a concentration of deformation and stress at the wall façades, principally for earthquakes acting in the direction of development of this façades. Inter-storey drifts are rather high at ground level, which can originate a soft-storey mechanism under seismic loadings. Openings enlargement or suppress of masonry walls at ground floor, for example, to install commercial spaces or garages is a common and inadequate practice in old buildings in city centres, that should not be overlooked;

- The non-symmetric distribution of openings in the buildings, particularly between the front and posterior façades induces a global torsion mechanism of the group of buildings studied. However, it is recognised that the global behaviour of the overall aggregate, where the four buildings are included, attenuates the torsional effects mentioned;

- Masonry walls are very sensitive to perpendicular seismic actions, inducing out-of-plane mechanisms. Their connections to floor and roof timbered structures and to the orthogonal walls are efficient and economical measures to reduce its vulnerability to the out-of-plane collapse mechanisms. These connections have a crucial influence in the behaviour of the structure, particularly for higher floor levels, as observed in the displacement profiles;
- The numerical model developed admits continuity in terms of displacements and rotations at the wall intersections. This specific feature is a limitation of

the models, which cannot reproduce exactly the real structural behaviour of these fragile regions of old masonry buildings. Tie rods can be used to improve the bonding conditions, threading together the masonry walls at corners, confining the perimeter of all the structure. In the numerical analysis, the connections between the timber joist and the walls were assumed allowing the continuity in terms of displacements, but not of rotations, only force transfer to the masonry walls. This assumption overlooks the poor connection conditions of the horizontal diaphragms that not represent adequately the poor anchoring of the timber floor elements to the walls, which is one of the main reasons for deficient seismic behaviour.

- From the retrofitting techniques studied, masonry strengthening has revealed to be the most efficient technique in reducing the deformations (out-of-plane and in-plane);

- Increasing the diaphragm stiffness can be also an effective retrofitting solution to improve the global behaviour of old masonry buildings. However, when this strengthening technique is not applied at all floor levels, the deformation demands at the upper storeys could be larger than for the original non-strengthened structure;

- Tie-rods can be efficient in restraining the out-of-plane deformations of masonry walls. As was expected, numerical results indicates that tie-rods do not contribute significantly to the in-plane response. Tie-rods are especially effective at roof level, controlling the out-of-plane deformation of the walls;

- The studied strengthening techniques were designed respecting the original conception of the building. Nevertheless, economical cost analysis and intrusion level of these schemes must be considered. Masonry strengthening and floor stiffening are normally costly and intrusive measures, and imply additional costs for the temporary rehousing of residents;

- From the studied retrofitting techniques, the optimum strengthening approach in terms of cost-benefit is the floor stiffening. It is very effective in the control of the out-of-plane deformations. However, a combination of the three studied strengthening actions could probably be a more effective strengthening scheme, for example, floor stiffening at all levels, roof tie-rods and masonry strengthening at ground floor level.

Acknowledgements

The authors would like to thank the building refurbishment and retrofitting contractors, particularly to, Antero Santos & Santos Group, for the valuable help and information provided on the retrofitting costs estimation used in this study.

References

[1] Vicente, R.S.; Varum, H.; Silva, J.A.R., Seismic vulnerability assessment of buildings in the old city centre of Coimbra, International Conference 250th Anniversary of the 1755 Lisbon Earthquake, 1-4 November 2005.

[2] Robot Millennium v17.5®, Three dimensional static and dynamic finite element analysis and design of structures, Robot Office, 2004.

[3] Pagaimo, F.A.L., Mechanical characterization of old masonries. Case study of the historical town of Tentúgal, MSc thesis in Civil Engineering, Faculdade de Ciências e Tecnologia da Universidade de Coimbra, 2004 [in Portuguese].

[4] Cardoso. R., Seismic vulnerability of old masonry structures – application to a “Pombalino” building, MSc thesis in Structural Engineering, Instituto Superior Técnico, October 2002 [in Portuguese].

[5] RSA, Rules for the definition of actions and safety in buildings and bridges. INCM, 1983 [in Portuguese].

[6] Giuffrè, A., Editore Laterza & Figli Spa, Sicurezza e Conservazione dei Centri Storici: Il caso Ortigia, Rome-Bari, 2000.

[7] Costa, A., Determination of mechanical properties of traditional masonry walls in dwellings of Faial Island, Azores, Earthquake Engineering and Structural Dynamics, 31: 1361-1382, 2002.

[8] FEMA-356, Prestandard and commentary for the seismic rehabilitation of buildings, American Society of Civil Engineers and Federal Emergency Management Agency, Washington D.C., 2000.