Steel based retrofitting solutions for masonry structures

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Abstract

The purpose of this work was to assess the efficiency of retrofitting masonry structures using steel strengthening solutions. A stone masonry benchmark structure was used as application example of the different rehabilitation techniques. The aim was to use advanced, yet commercially available, analysis methods to determine the weaknesses of the initial structure and to test the benefits of different rehabilitation and strengthening solutions. Non-linear modelling techniques have been explored for the evaluation of the structural performance. ABAQUS was utilized as the main modelling tool.

The earthquake performance of the benchmark structure was evaluated using pushover analyses. An initial assessment highlighted the weaknesses of the building: inadequate connections between walls and low out-of-plane bending strength of the walls. It was determined that tying the walls has to be the priority intervention. Further improvements could be obtained if rigid diaphragm was provided at the slabs. As a last resort, the walls could also be strengthened to increase their lateral load bearing capacity.

Keywords: stone masonry, retrofit, steel based solutions

1. Introduction

The purpose of this work was to study the efficiency of retrofitting masonry structures using steel strengthening solutions, within a project dedicated to seismic retrofit and upgrade of existing constructions [1]. More specifically, a "masonry benchmark structure" from Italy has been used as application example of the different rehabilitation techniques. The aim was to exploit advanced analysis methods to determine the weaknesses of the initial structure and to test the benefits of different rehabilitation and strengthening solutions.

The geometry of the structure was based on the description of Braconi *et al.* [2] and is briefly reproduced in Fig. 1. The masonry building has been built according to geometrical considerations, as it was typical at the beginning of the 20th century. The main walls are stone masonry, while thin hollow brick masonry infills were used for compartmentation. These walls have been neglected in the assessment. Two kinds of floors are present in the building. The lower floor slabs are timber structures supported by brick vaults, while the top slab is a simple timber floor. The roof has a timber support structure.

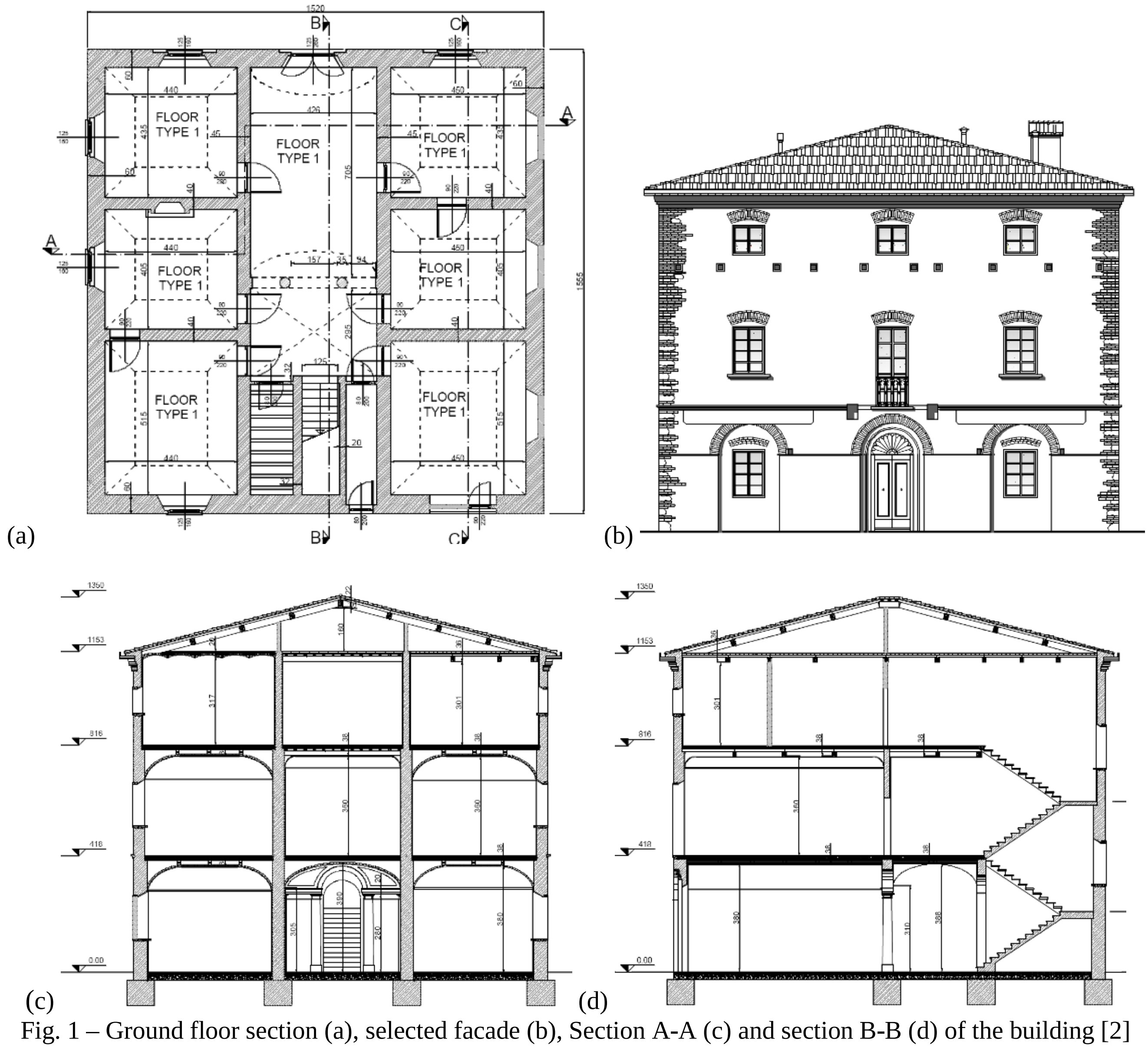


Fig. 1 – Ground floor section (a), selected facade (b), Section A-A (c) and section B-B (d) of the building [2] (dimensions are in *cm*)

2. Description of the models

Both linear and non-linear modelling techniques have been explored for the evaluation of the buildings performance. An elastic linear model was developed in the SAP 2000 commercial software, as combination of shell and beam elements, and used for preliminary assessment of several intervention techniques. This model was also used to benchmark the more complex, non-linear models presented in this paper.

ABAQUS/Explicit has been the tool for the non-linear modeling, utilizing continuum shell elements and the in-build concrete damage-plasticity (DP) model for the material properties. The DP material definition is both flexible and stable, but it has the limitation of not taking into account the orthotropic behavior of the masonry. VUMAT subroutine based material models, developed by Lourenço [3, 4] and implemented in ABAQUS by Haider [5] were also tested, but it was judged that keeping the models within the limits of commercially available software overrides the benefits of using more sophisticated modeling approaches. Hence, the results presented here are based on the simpler DP modeling technique.

The structure has been studied in detail by Varelis *et al.* [6] with load combinations being defined there. In the seismic load combination the total mass was 1539t, most of which is the weight of the walls. For simplicity, the floor based approximation was used to deduce pushover forces, with masses resulting m_G=628t, m_1=541t and m_{2+}=370t. A linear increase of the lateral displacement was assumed with height [7]. The resulting horizontal load distribution is 23.2% to the 1st slab level, 39.1% to 2nd slab level and 37.7% to the roof level. It has to be noted that assigning seismic masses to floors, as shown in Fig. 2, is not entirely realistic in this structure where masses are more uniformly distributed with height.

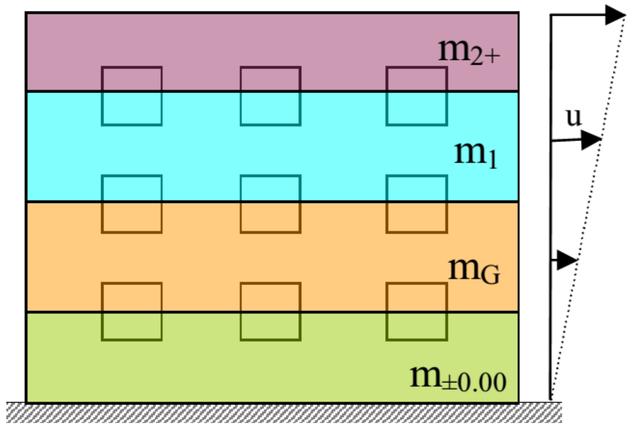


Fig. 2 – Concentration of masses at floor levels.

The seismic load design target was for peak-ground acceleration (PGA) of 0.24×g, with a Type 1 elastic spectrum on Soil B according to EN 1998-1 [7]. The value of the damping ratio was taken 5% for the design, and the structure was classified as importance Class II. In EN 1998-1, the design value of the seismic action (A_{Ed}) can be derived by taking into account: a correction due to the importance class of the building (γ_1=1); the possibility to reduce base shear with the factor λ=0.85 for buildings with more than two floors; and the use of a behavior factor “q” larger than q=1. A_{Ed} corresponding to a dominantly elastic behavior (q=1) in the low period range (T<0.5s) results in a spectral acceleration (PSA) of 5.75m/s^2. Using an equivalency of the structure with a single-degree of freedom system, the 1539t seismic mass therefore generates F_b= 8855kN base shear. This can be used as a first estimate for the expected demand.

2.1 Material models

As mentioned, the in-build ABAQUS/DP material model and a VUMAT based model have been used in the study. For both material models the uniaxial compression strength was 1.5 N/mm^2 [2], and the uniaxial tension strength was assumed 0.125 N/mm^2. In the post-failure range, material models are characterized by degradation dependent on strain in compression, and softening stress-strain response depending on the crack opening in tension (Fig. 3). Because the model is used in pushover analysis only, no damage parameters have been incorporated in the material definition.

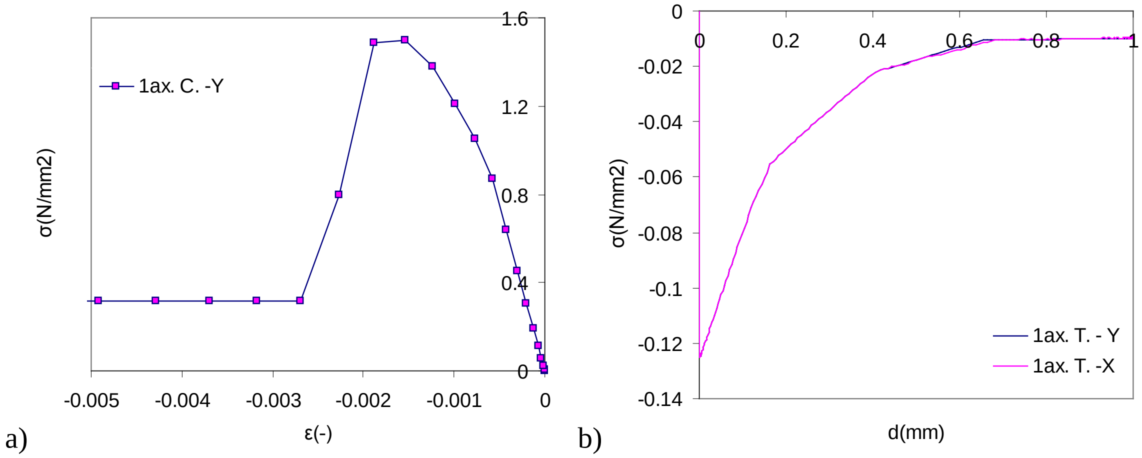


Fig. 3 – (a) Uniaxial compressive and (b) tensile behaviour of the DP model

The material models have been tested under combined in-plane axial and shear loading in order to evaluate their behavior. The sequential loading scheme used in this testing is presented in Fig. 4, and it consists of (1) a step to load the element with axial stress and (2) then subject it to shear while maintaining the axial pre-load. The plots in Fig. 4.c and Fig. 4.d were drawn using calculated values in points of the σ+σ plane. The red diamond symbols correspond to points on the edge of the interaction surface in the σ+σ plane (shear stress is τ=0); while the cyan circles correspond to σ+σ+τ interaction points. Based on the results, the iso-surfaces in terms of τ were drawn (Fig. 4.c d). The uniaxial compression strength of 1.5 N/mm^2 and tension strength of 0.125 N/mm^2 can be identified, together with the maximum shear capacity corresponding to the range of biaxial compression (τ<0.8 N/mm^2).

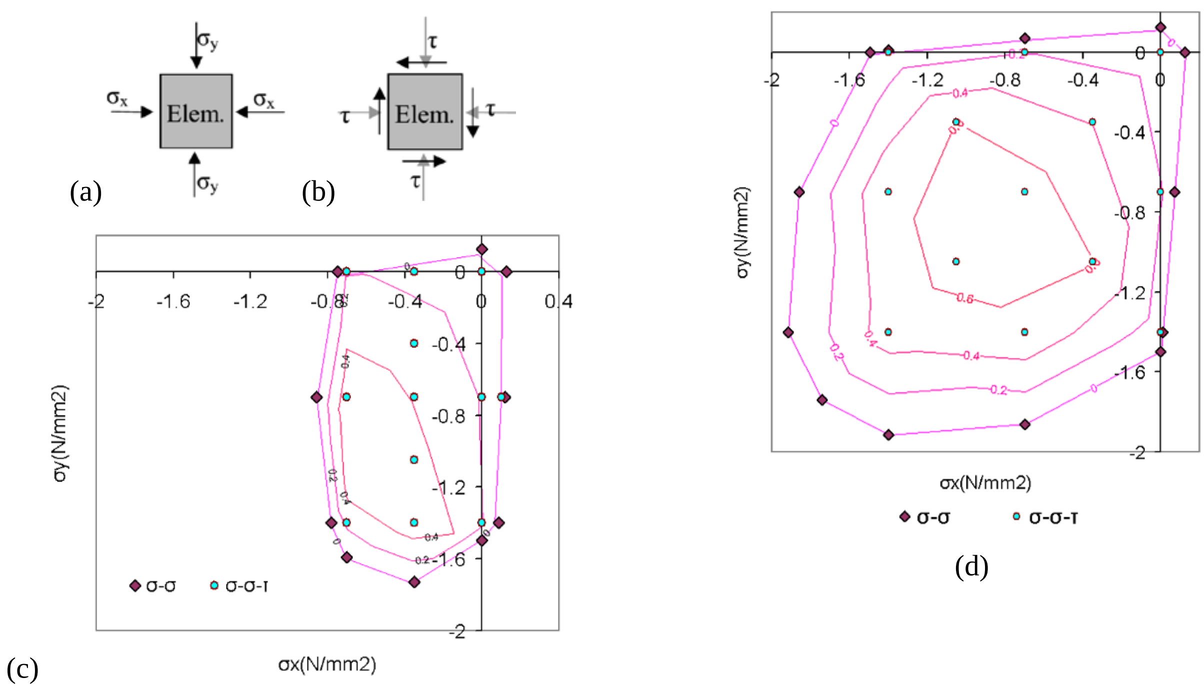
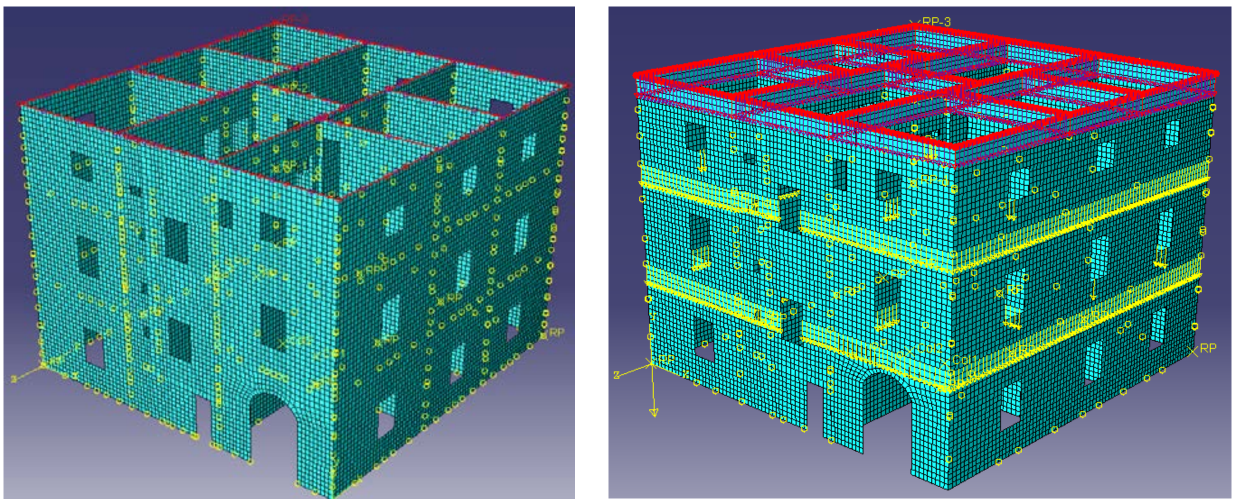


Fig. 4 – Testing the material models. (a) Axial pre-loading followed by (b) shearing of the element with normal stress maintained and (c) mapped σ+σ+τ interaction for the VUMAT and (d) the DP model.

2.2 Vertical load introduction

As the intention was to keep the assessment generic, the tributary areas of the floors were not worked out in detail. Floor loads can be treated approximately also because most of the weight is concentrated in the walls and their weight distribution is taken into account correctly by gravity loads. In this scenario there were two options to introduce floor loads in the model, as exemplified in Fig. 5. In the option given in Fig. 5.a, a master node located at the center of gravity of the floor is used to apply the entire floor load as a concentrated force. The nodes on the wall are slaved to the master node, reproducing the presence of a rigid floor. Hence, the force distribution in the walls depends primarily on the stiffness of the different wall segments. In the second option (Fig. 5.b) floor loads are distributed along wall segments uniformly, and the stiffness of the floor is ignored. In this second case, uneven vertical deformation of the walls is permitted, hence resulting in more realistic stress distribution in the walls.

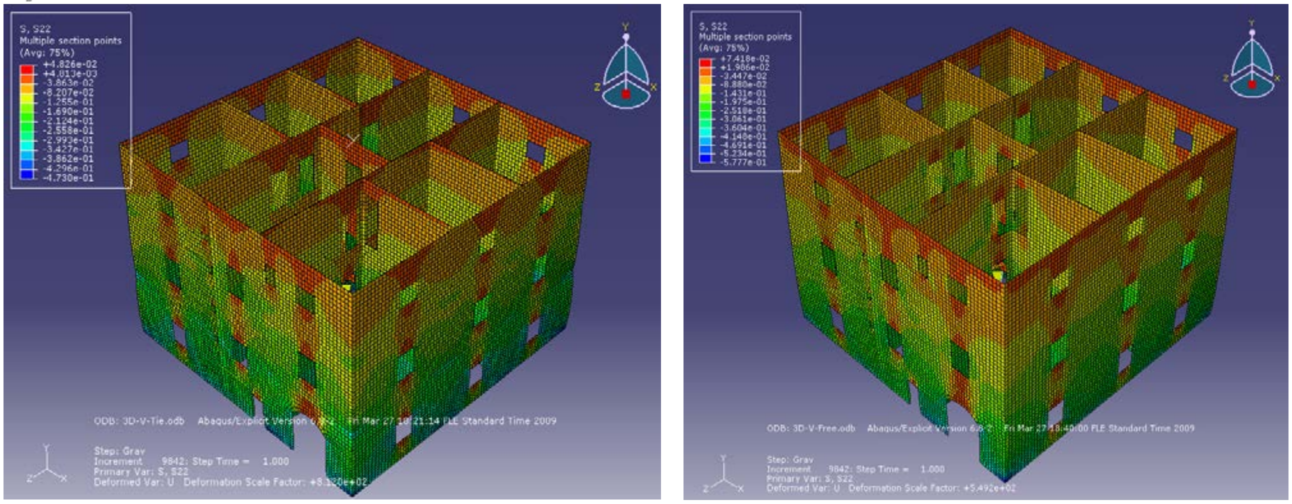


(a) (b)

Fig. 5 – (a) Rigid constrained upper floor, with load applied to the reference point and (b) model without floor constraints loaded with vertical load [8].

Vertical loads were applied using an ABAQUS/Explicit load step with smooth force introduction. The step time for introducing the loads was controlled to correspond to a quasi-static loading scenario. The kinetic energy was monitored to make sure that it does not exceeded 2% of the total energy at any time during the load introduction.

Under vertical loads the walls will develop stresses which will affect the performance to earthquake loads. The stress state under vertical loads is presented in Fig. 6.a for the rigid floor model and in Fig. 6.b for the model without floor connections. As it can be seen, the largest compression stress (σ_y) is -0.47N/mm^2 and -0.57N/mm^2 respectively. This is about 30% of the uniaxial compressive strength of the masonry. The stress patterns are fairly similar in the two cases. The load versus vertical displacement curves were plotted for the two models. As expected, the model using the rigid floor assumption was slightly stiffer but without a very significant difference. This is probably because 74% of the vertical load is the mass of the walls, so the distribution of the remaining 26% load is not crucially influencing the loading in the vertical direction.



(a) (b)

Fig. 6 – Normal stress component σ_{yy} from vertical (Y direction) loads for the (a) rigid floor, (b) and no floor connection assumptions [8]. Units are N/mm^2.

3. Response to horizontal loads / Expected performance

With the original floor construction, the building can be considered to have very limited floor stiffness and strength in the horizontal direction (i.e. no diaphragm action). If the model without floor is analyzed with increasing horizontal forces, the deformed shape presented in Fig. 7 is obtained. With increasing loading the failure of the structure is based on a local mechanism in both loading directions. One main failure mode is due to separation of the heavy external walls from the transversal ones. This happens at small values of the base shear (F_b=800kN). Since an order of magnitude larger base shear is expected, the primary goal of the rehabilitation has to be to tie together the walls in order to avoid these localized failure modes.

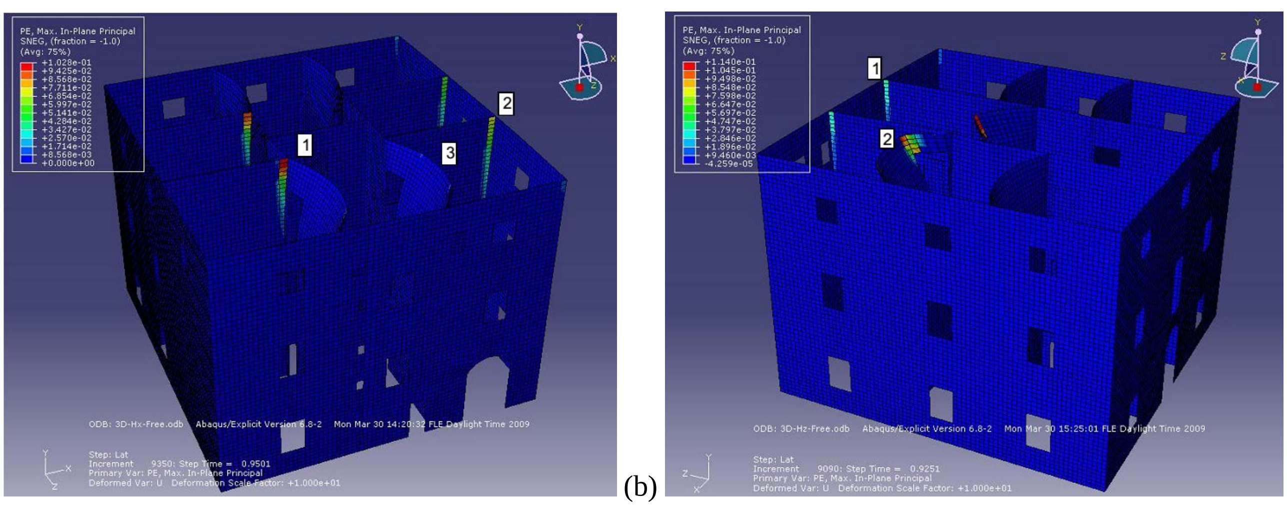


Fig. 7 – Plastic-strain pattern at failure for (a) X direction and (b) Z direction pushover for the model without floors. High values of plastic strains indicate locations of tensile cracks in the material.

As far as the building configuration is concerned, the following structural characteristics and potential deficiencies have been identified based on the analysis of the linear and non-linear model outputs:

- The structure is almost symmetrical and has similar behavior in the two main directions. Torsion does not affect the performance.

- The largest part of the seismic mass is given by the walls. Both the weight of the floors and the mass resulting from live loads are secondary.
- The main weakness of the structure is the lack of diaphragm effect at the level of the floors and roof. The walls are not tied together and local failure governs the behavior. An effective tying between the walls has to be a priority of the rehabilitation.

4. Tested rehabilitation measures

Based on the observations concerning the behavior of the structure, the following rehabilitation techniques have been tested:

- Tying of the walls at the intersections primarily at roof level (Horizontal elements).
- Establishing diaphragm at the roof level (Horizontal elements).
- Providing diaphragm at the roof level (Horizontal elements), coupled with strengthening the external ground floor walls (Vertical elements).
- Providing diaphragm at each floor level (Horizontal elements), coupled with strengthening the external ground floor walls (Vertical elements).

For each of these generic rehabilitation techniques in the horizontal plane, several potential application options were considered, modelled and their economic feasibility assessed. These were:

- Steel tying systems to connect the existing walls;
- Providing a ring beam at the roof level;
- Horizontal trussed girders at the roof level;
- Horizontal bracing systems at the roof or floor level.

Similarly, for strengthening the vertical elements, the following steel based options were considered:

- Steel shear walls;
- Steel bracing elements;
- Steel frames;
- Steel braced frames;
- Steel strips elements integrated in the wall.

Furthermore, the possibility of strengthening of the foundations by steel micro-piles has been assessed.

5. Discussion of the effects of selected rehabilitation techniques

As mentioned, one of the weaknesses of the original structure is that walls are not tied together and they have the tendency to separate (Fig. 7). Therefore, the first rehabilitation solution proposes the tying of the wall segments, but without creating diaphragmatic effect (Fig. 8). This is a simple, cheap and low-intrusive technique which can improve the performance of the structure. By tying the top of the transversal walls using Ø24 mm, f_y=350 N/mm^2 steel bars, the dominant failure mode could be changed to out-of-plane bending of transverse walls. The corresponding base shear was approximately doubled compared to the initial values, reaching F_b=2000kN.

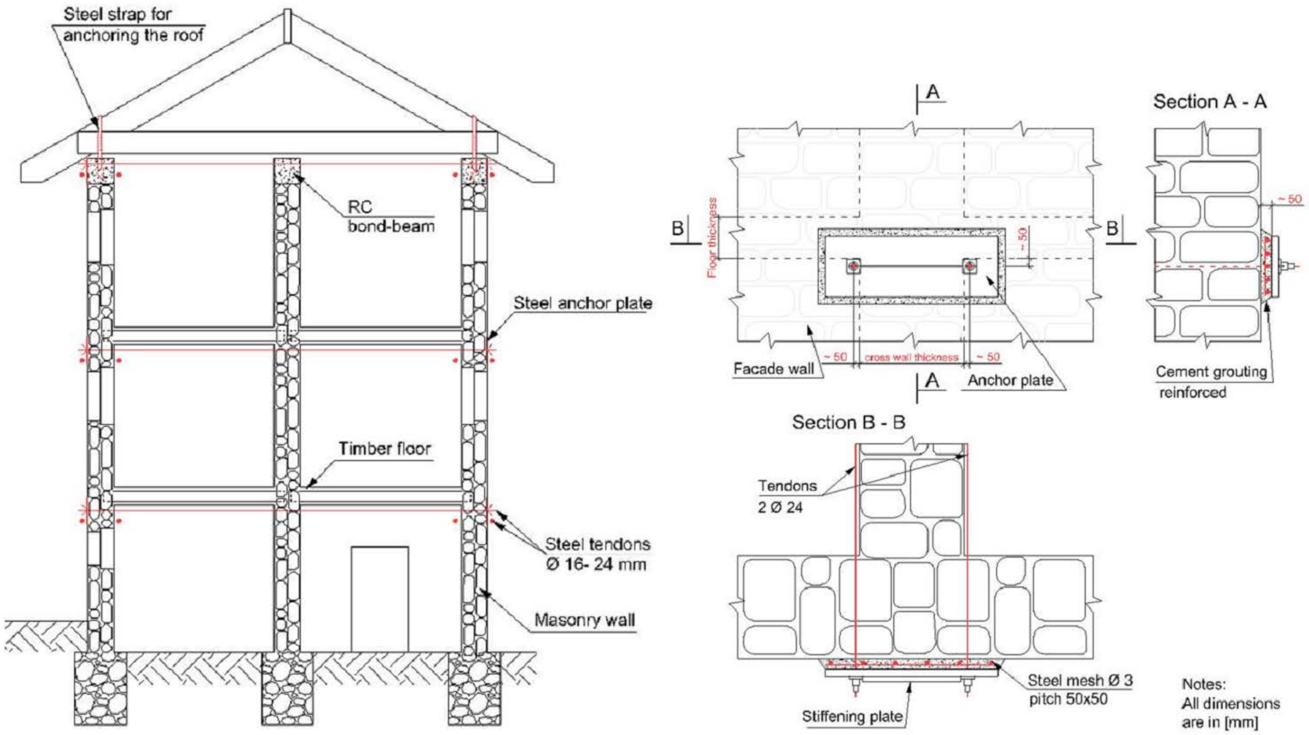


Fig. 8 – Horizontal tying elements and possible anchoring techniques [9]

Another level of intervention involving the horizontal planes would be the creation of horizontal diaphragms at roof and/or floor levels. This intervention is more intrusive and costly, especially if it involves internal floors. The techniques considered involved the use of horizontal bracing ties, horizontal truss systems and ultimately replacing floors with concrete slabs.

One major difficulty in all these cases is handling the anchoring forces at the interface of the newly added steel elements and the old masonry. In Fig. 9.a and Fig. 9.b, possible anchor arrangements are presented for brace-like ties in the horizontal planes. In Fig. 9.c the possible arrangement for a ring beam in the roof plane is shown. The Fig. 9.c arrangement could be used for anchorage to a horizontal truss system in the roof plane. Since the force transmitted by individual anchors is limited, it is important to provide multiple anchor points for the newly introduced structural elements.

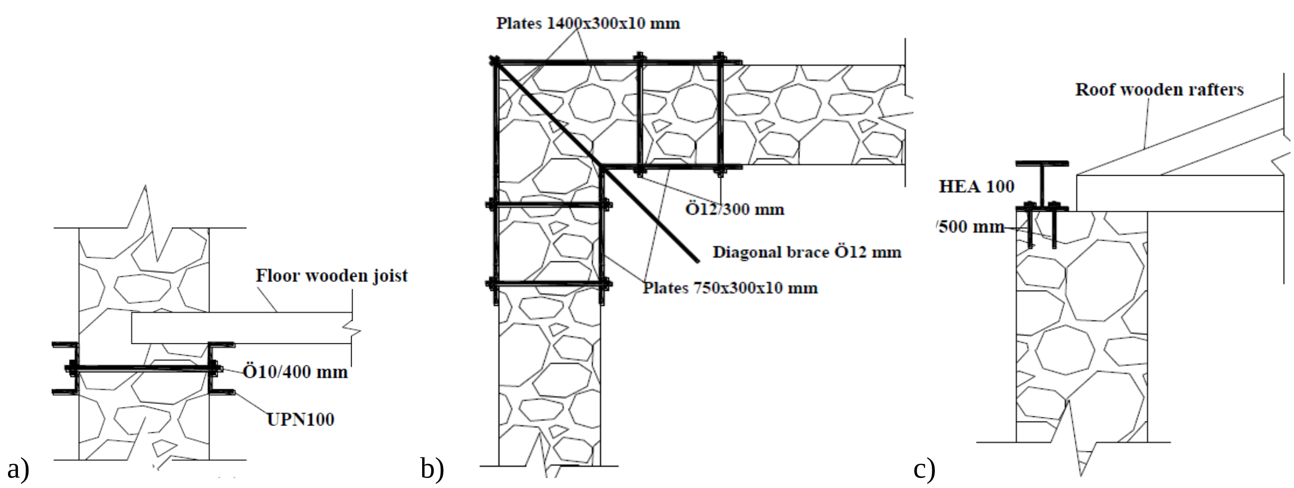


Fig. 9 – Possible anchoring techniques for horizontal stiffening elements

This case has been modeled in ABAQUS by constraint for nodes at the top of the walls. Deformation shape under pushover loading is presented in Fig. 10. As it can be observed, the cracking of the walls is distributed over large areas, which is beneficial. However, localized failures may still be present in the form of separation of vertical wall connections at intermediate floor levels, or out-of-plane failure of wall segments. Supplementary local intervention and strengthening, besides the roof level diaphragm, may be required to eliminate these failure modes.

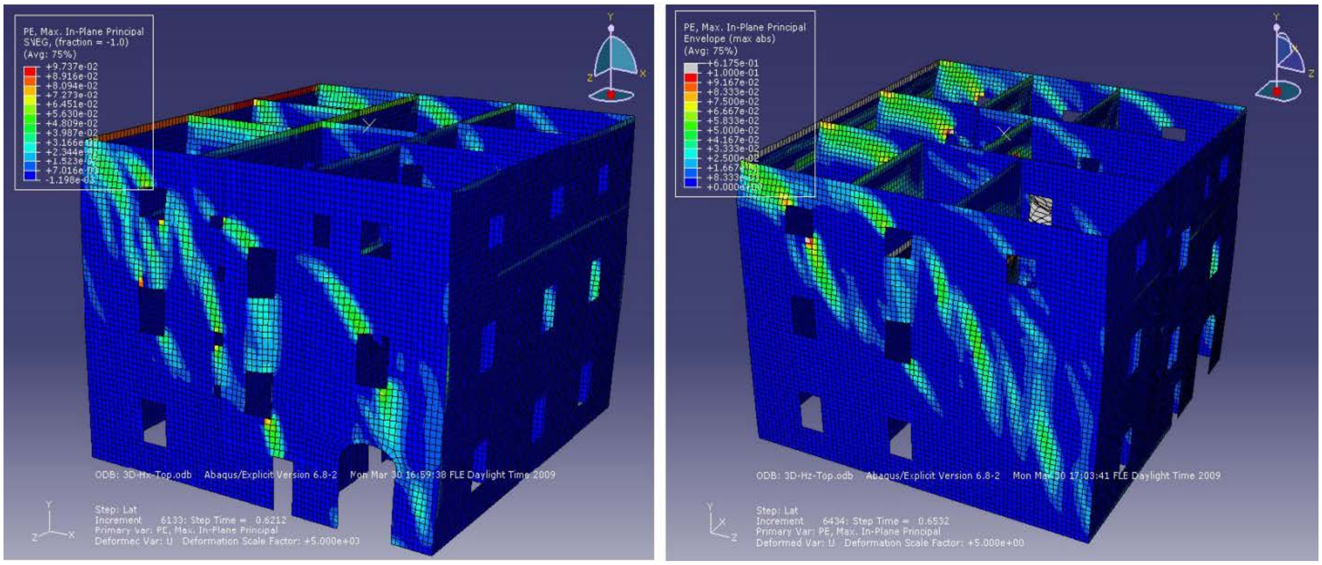
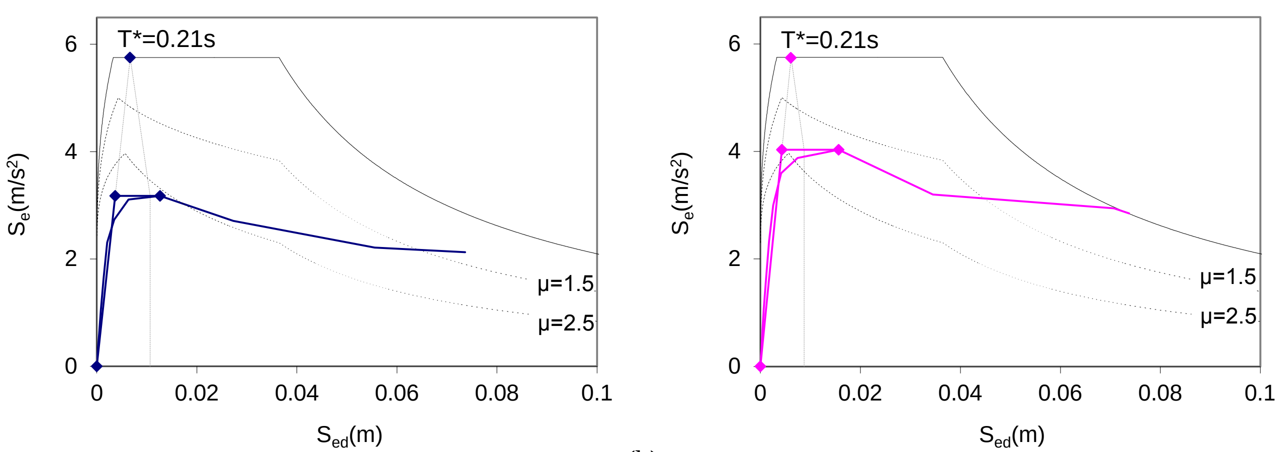


Fig. 10 – Views of the deformed shape and distribution of plastic strain for the X and Z direction forces.

The overall performance of the structure is very advantageous. As one can observe from the curves in Fig. 11, the rehabilitated structure has sufficient strength and ductility to withstand the design earthquake load in both X and Z direction. The plots presented in Fig. 11 are given in capacity spectrum format, calculated based on the N2 method of Eurocode 8 [7], which can be used for displacement-based design of structures [10]. The structure is very stiff, with an estimated equivalent SDOF period of 0.21s (~5Hz), and it is weaker in the X direction (Fig. 11). The expected performance in the X direction indicates ductility demand slightly above 2.5, while in the Z direction in the range of 2. This performance is in line with behavior factors 1.5-2.5, given in Eurocode 8 for unreinforced masonry structures designed for seismic regions (Table 9.1, EN 1998-1); but it would exceed the expected performance for simple unreinforced masonry structures.



(a) (b)

Fig. 11 – PSA-SD spectral plot over-lapped with the pushover curve transformed to SDOF format with the N2 method for (a) the X and (b) Z directions.

It was also noted that the roof level diaphragm provides less strength but substantially more ductility compared to the technique involving stiffening each floor slab, due to the evenly distributed damage with height. When slabs are provided at each floor level, disadvantageous storey failures were noticed. In that case wall cracking is also more concentrated to single floors.

In the case of this benchmark building, the roof-diaphragm rehabilitation technique is sufficient to fulfill the earthquake requirements. If it would be needed, the performance can further be improved by strengthening selected wall segments, especially in the X direction [11]. A general hierarchy of the available refurbishing techniques is presented in Fig. 12. The sequence of possible upgrades ranges from the low-intrusive wall tying to highly intrusive roof diaphragms.

However, it has to be mentioned that the steel based rehabilitation techniques for vertical elements, investigated in this study, mostly proved to be ineffective. This was partly due to the high stiffness of the masonry walls, which would not permit redistribution of loads to other vertical systems, and partly due to the limited number of techniques applicable to stone masonry walls.

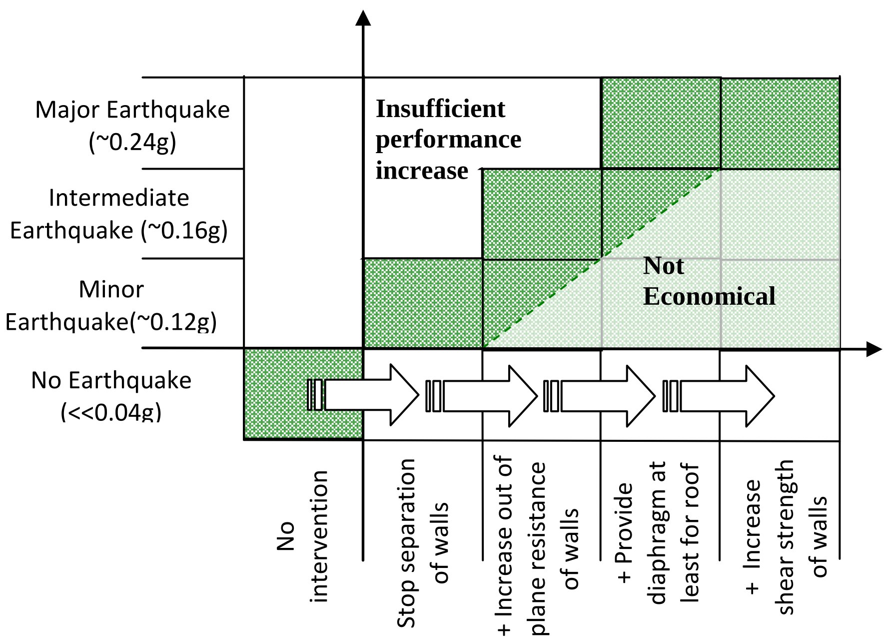


Fig. 12 – Expected performance thresholds as successive rehabilitation techniques are applied.

6. Discussion/Conclusions

In this study non-linear modeling has been explored for the evaluation of the performance of a masonry benchmark structure. One goal was to demonstrate the usefulness of applying relatively simple commercially available non-linear models in order to understand the behavior of masonry structures. Using simple analogy with concrete plasticity, one can create flexible and stable models to understand the main weaknesses of a masonry structure.

For the evaluation of the earthquake performance of the structure under consideration, pushover analysis has been used. An initial analysis highlighted that the main weaknesses of the building are the weak connections between walls and the low out-of-plane bending strength of some walls. It was determined that the tying of the walls has to be the priority intervention. Further improvements were obtained if diaphragm was provided at the roof level. Inserting diaphragms at each floor was not advantageous, because it localized the cracking and failure of the walls. As a last resort, the walls could also be strengthened to increase their shear capacity, a technique which was unnecessary in this case. The rehabilitation techniques analyzed are presented in a hierarchical order in Table 1. The main reasons for the deficient performance are listed in successive rows, indicating whether the intervention technique listed in the columns is providing solution for each deficiency. An intervention can mainly target a specific deficiency, but it can also have beneficial effect to others. For this particular benchmark building, some interventions proved to be sufficient to upgrade the structure to the desired performance level.

Table 1 – Summary of tested rehabilitation techniques and their hierarchy

Nr.	Inadequate / deficient performance of the structure	Cause of the deficient response	Intervention technique							
			Tying wall tops with steel rods		Roof level diaphragm		Floor diaphragms		Wall shear improvement + diaphragms	
			Objective	Solved?	Objective	Solved?	Objective	Solved?	Objective	Solved?
1	Detaching of walls	No connection at vertical lines	Main	Yes	Main	Yes	Main	Yes	Main	Yes
2	Out of plane failure of walls	Low bending resistance	Yes	No	Main	Yes / Partly	Main	Yes	Main	Yes
3	Potential ground-mechanism	Stiffness / strength difference of walls at different levels	-	-	-	Yes	-	No	-	-
4	Low ductility of walls		-	-	-	Yes	-	No	Main	Yes
	Overall assessment of the technique for the benchmark building		Fail		Successful		Fail		Successful	

For instance, tying together the top of the walls partially solved the deficiencies of the structure, but failed to improve the performance to fulfilling the design criteria. On the other hand, the introduction of rigid diaphragms at slab levels changed the behavior. It was found that introducing diaphragm only at roof level is more beneficial than diaphragms at each slab. The roof diaphragm alone forces the structure to behave as a whole, and cracking is evenly distributed on the height of the walls. Hence, global ductility is superior. On the contrary, the presence of diaphragms at floor level increases the chance of localized cracking, therefore reducing the overall ductility.

Thresholds of performance increase can be identified as successive rehabilitation techniques are applied (Fig. 12). Providing sufficient tying between walls and eliminating out of plane failure of walls is sufficient to reach requirements for small/medium values of ground acceleration. The next threshold is reached when roof/floor diaphragms are introduced and provide adequate distribution of seismic forces on the shear walls. In the case for ground acceleration of 0.24×g, this measure proved to be sufficient.

5. Acknowledgements

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