Experimental and numerical analysis of the seismic performance of concrete block masonry buildings

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ABSTRACT: The increase in housing demand and the construction cost of the necessary houses for its satisfaction have motivated the study of economical and efficient solutions aiming at developing new construction methods**.** A new solution for the construction of residential houses is presented. Dynamic seismic tests by using a shaking table were performed on two masonry buildings**.** The first experimental model incorporates steel reinforcement according to the Eurocodes, while the second was tested as an unreinforced solution. A numerical model for the unreinforced solution was prepared by using macro-modelling approach. Five non-linear phased dynamic analyses with time integration representing the same seismic amplitude tests implemented during the experimental campaign were made**.** In terms of experimental results, the quantitative parameters for both models and the crack patterns are presented. Comparisons between the experimental results and those from the numerical simulations are also presented.

Keywords: concrete block masonry; shaking-table tests; seismic performance; numerical simulation; non-linear analysis

1 INTRODUCTION

The implementation of masonry materials for the construction of structures dates since the early civilization of human societies. It is possible to observe how many of the masonry structures constructed centuries ago have remained until nowadays. The economical production of its materials, the easy of its construction process and the resistance capacity given by this construction system are the main reasons for its successful application.

As a material for the construction of buildings, it presents good performance in non-seismic prone areas. Its high resistance to compressive loads and the comfort properties given by the material itself become masonry in an attractive solution for residential buildings. However, some disadvantages like the low tensile strength and its poor ductility behavior arise as drawbacks for its implementation in seismic areas [1, 2]. At the department of civil engineering in the University of Minho, researchers have been working in solutions to those disadvantages. It is believed that with an engineering solution masonry can be effectively considered as an alternative low cost structural system for the construction of low to medium residential buildings.

The present paper presents the results and analyses from an experimental campaign and numerical simulation of a new construction system solution for residential buildings. The researchers´

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objective is to provide a safe/economical structural solution for the construction of low to middle masonry buildings. Detailed investigation about its materials i.e. blocks, mortar and steel reinforcement, as well as lateral in-plane experimental tests of walls, has been previously done [3]. For the validation of the proposed system as a whole, dynamic seismic tests by using shaking table were planned; here, artificial accelerograms based on the elastic response spectrum given by the Eurocode 8 were imposed as inputs. For the tests, two symmetric buildings were constructed. Both of them were similar in materials and geometrical configuration. The only difference consisted in the implementation of steel reinforcement. One building was unreinforced and the other was provided with the minimum steel reinforcement ratio requested for the Eurocodes. From this experimental program, important information regarding the seismic performance of the constructive system was obtained. Complementary a numerical model, calibrated based on the experimental results, was also carried out. From the experimental and numerical studies was obtained not only important information regarding seismic performance, but also significant future design parameters.

2 CONSTRUCTIVE SYSTEM

It is intended that the new constructive masonry system be implemented for the construction of residential houses up to two-story in low to high seismic prone areas. Therefore, the system is composed by concrete block units together with modified mortar and pre-fabricated truss type reinforcement, following always the standards given by the Eurocodes [4, 5]. The concrete block units show a new concept design in terms of geometry, they were designed with three hollow cells and frogged ends. This geometry aims to provide two main purposes: a versatile and efficient construction. The system will be versatile in the sense that for low seismic areas no steel reinforcement is needed, when needed vertical and horizontal truss reinforcement could be added to the system by using the same masonry unit. The modified mortar contributes also to the efficiency of the system, it can be used for both the laying of the block units and the filling of the vertical hollow cells, were the steel reinforcement is placed. With this configuration, the constructive system can be built following different bond patterns i.e. the traditional masonry bond pattern and an alternative masonry bond pattern that include continuous vertical joints formed by the frogged ends of the blocks.

2.1. Materials

For a structural solution, the quality of the construction materials plays an important role in the performance of the whole structure during earthquakes. From previous investigations, important information has been obtained. According to Eurocode 8 [4], units used to build masonry structures in seismic areas should have: Normalized compressive strength, normal to bed joints, not lower than 5MPa and normalized compressive strength, parallel to bed joints, not lower than 2MPa. Results in proposed masonry units show a compressive strength of 10MPa and 7MPa respectively. For the mortar, a minimum compressive strength of 10MPa is recommended in case of reinforced masonry structures in seismic areas. The modified mortar used registered values over 11MPa. From direct tensile tests, the pre-fabricated truss reinforcements showed an average of 580MPa of yield stress.

It is important to note that in spite of mortar has been often neglected in terms of structural analysis of masonry structures, it is well known nowadays that it influences the final behavior of them, in affecting parameters like the compressive strength and deformability. From the 2010 Darfield earthquake, six mortar samples were extracted from damaged masonry buildings and tested in compression. Results revealed values lower than 2MPa [6]. After the earthquake, it was very common to find considerable quantity of walls that failed due to a poor mortar quality, while the masonry units remained in good conditions.

The construction bond pattern was also studied. From the two bond pattern options it was found that when steel reinforcement is used the continuous vertical joints make the construction process easier, and is preferably as good performance was found in masonry shear walls´ tests [7]. For unreinforced masonry solutions, traditional bond pattern with dry head joints is better as the construction is much faster in this way.

3 EXPERIMENTAL CAMPAING

The residential prototype building is a typical individual two story house with regular geometry, commonly found in modern residential aggregates, with an interstory height of 3.0m, 2 opposite facades with a percentage of openings of approximately 14% and two opposite walls without openings, corresponding to the walls bounding the contiguous houses. The slabs are made with reinforced concrete aiming at working as a rigid diaphragm. From this prototype, two models of structural masonry have been considered. In consistence with Eurocode 8 [4] and Eurocode 6 [5], a reinforced (RM) solution is studied. Here, minimum requirements regarding reinforcement ratios are followed. The second building corresponds with an unreinforced (UM) solution. The comparisons between reinforced and unreinforced masonry buildings should clarify the influence of the reinforcement on the seismic performance of structural masonry. For the UM building, traditional masonry bond pattern is used, whereas for the RM model vertical continuous joints are considered in which vertical reinforcement will be placed.

Due to shaking table´s geometry and payload limitations, the experimental models were scaled. Cauchy scale law was adopted for their construction and input design. Figure 1 shows the final geometry of the scaled masonry buildings. Such a type of scaled models to be tested on a shake table can be used when the stresses induced by the gravity loads are small and negligible with respect to the stresses induced by the seismic forces, e.g. when the lateral load´s resisting system consists primarily of shear walls.

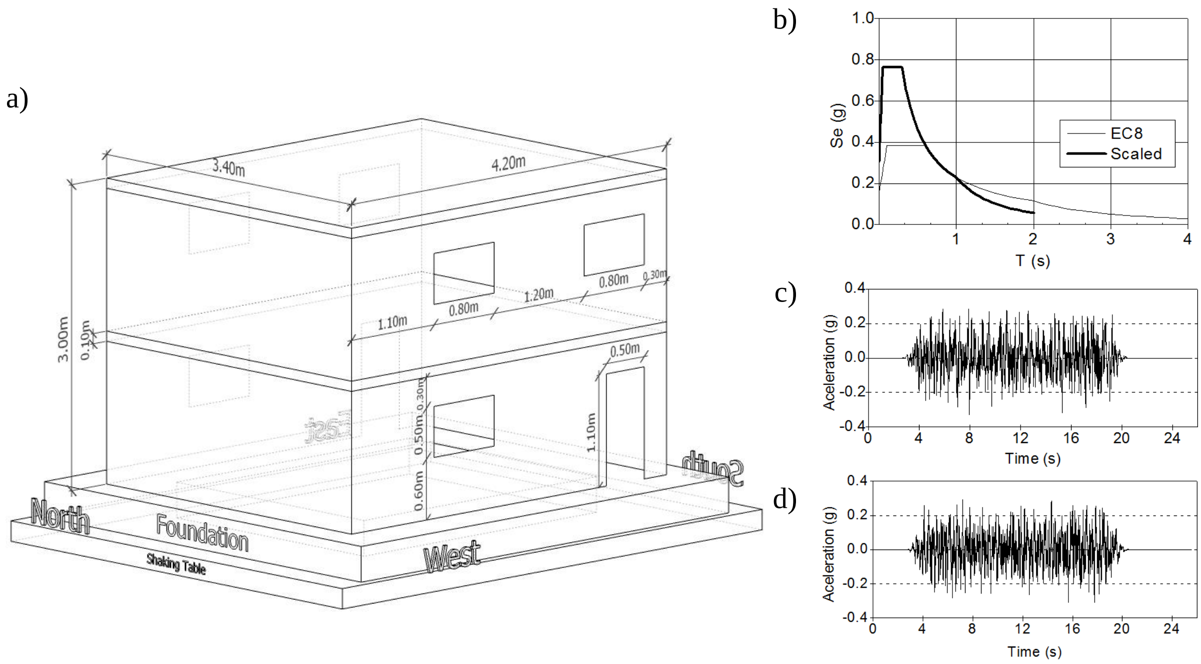


Figure 1. (a) Geometry of RM and UM experimental models; (b) standard and scaled elastic response spectrum; (c) artificial accelerogram North-South direction and (d) artificial accelerogram East-West direction

The tests were developed by imposing simultaneous base excitations, acting along the two orthogonal horizontal directions. Two uncorrelated accelerograms based on type one elastic response spectrum for Lisbon zone, with ground type A and 5% damping, were implemented, one for each direction respectively. Following the scale law, the final response spectrum is shown in Figure 1b, and corresponding accelerograms in Figure 1c and 1d. Each building was subjected to a series of dynamic input tests with incremental ground motions, obtained after scaling the accelerograms. The resulting input accelerograms have durations about 17s and a spectral acceleration in the plateau region of about 0.75g.

For all the tests the measurements in the models were made by a total of 28 accelerometers, which were placed at the base, the slabs and openings´ corners in both directions. From the accelerations time histories registered and by means of mathematical double integration, displacements were also obtained. Table 1 summarizes the inputs and corresponding PGAs registered for each building.

Table 1. Experimental inputs and corresponding PGAs

Reinforced model			Unreinforced model	
Test	**PGA NS (m/s^2)**	**PGA EW (m/s^2)**	**PGA NS (m/s^2)**	**PGA EW (m/s^2)**
50%	2.06 (0.21g)	1.74 (0.18g)	2.62 (0.27g)	1.99 (0.20g)
75%	2.90 (0.30g)	2.82 (0.29g)	-	-
100%	3.84 (0.39g)	3.71 (0.38g)	5.01 (0.51g)	4.26 (0.43g)
150%	6.24 (0.64g)	5.53 (0.56g)	7.88 (0.80g)	6.64 (0.68g)
200%	9.80 (1g)	7.13 (0.73g)	10.90 (1.11g)	8.51 (0.87g)
250%	12.32 (1.26g)	8.90 (0.91g)	13.04 (1.33g)	10.42 (1.06g)
300%	13.03 (1.33g)	10.14 (1.03g)	-	-
400%_1	15.83 (1.61g)	12.71 (1.30g)	-	-
400%_2	15.49 (1.58g)	13.36 (1.36g)	-	-

As observed, RM attained a maximum PGA input of 1.6g. In this model, the test was stopped because it was attained the maximum displacement capacity of the shaking table´s actuators. Maximum PGA for UM registered a value of 1.33g. Here, a second test of 250% was developed, but due to the imminent model´s collapse some equipment were removed and no data at the base was registered. The test was finally stopped as result of no collapse.

4 TESTS RESULTS

After each test, the accelerations were processed and analysed. The signal process developed consisted mainly in the removal of the quasi-static components and noise by means of a Fourier filter between 0.4Hz and 35Hz. Furthermore, signal offset correction and signal crop were implemented. Finally, damage was accurately mapped from where failure and collapse mechanisms were identified.

4.1. Damage

Figure 2 depicts the crack patterns and final damages for both buildings. It should be stressed that these were the final damages for the RM building after test run of 400% (PGA=1.61g) and for UM building after test run of 250% (PGA=1.33g). The onset damage on RM model appeared after input of 100% (PGA=0.39g) and 80% of the final damage occurred after test of 250% (PGA=1.26g). It was noted how the damage in this model was represented only by cracking, most of it affecting only the bed and head mortar joints and located in the first story. Here the density of cracks is clearly higher in case of the walls with openings.

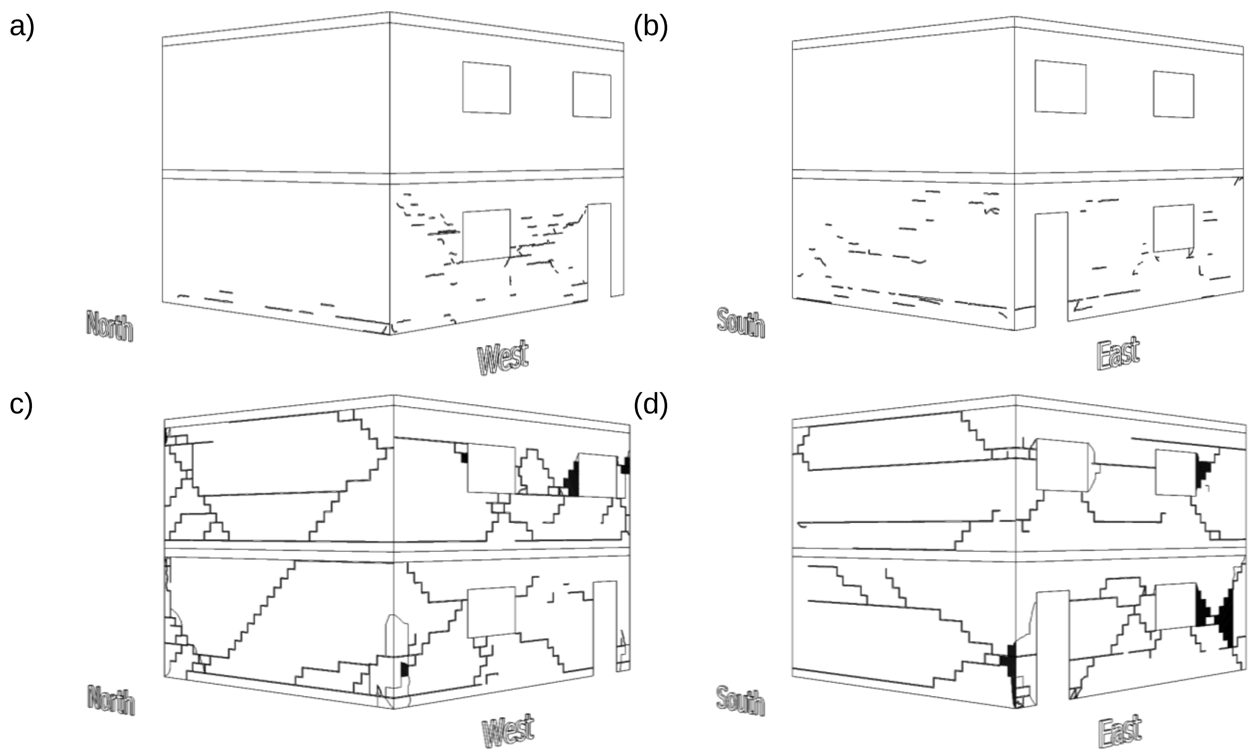


Figure 2. Final damages for: (a, b) RM after nine test runs; (c, d) UM after five test runs

On the UM model, the seismic inputs cause more severe damages. Furthermore, they started during an early stage than RM. The maximum input motion attained by the UM building represents 62.5% of the maximum input attained by the RM model, however in the first one well-defined and localized continuous cracks and disconnections of blocks were observed. It was noted how for this building the damages occurred in the entire model. Visible diagonal stair step cracks are presented in all facades and remarkable horizontal cracks were observed at the first course of units in both stories. In addition, a sliding mechanism was clearly observed during testing, mainly through the horizontal cracks presented in the walls without openings, see Figure 2 c and d.

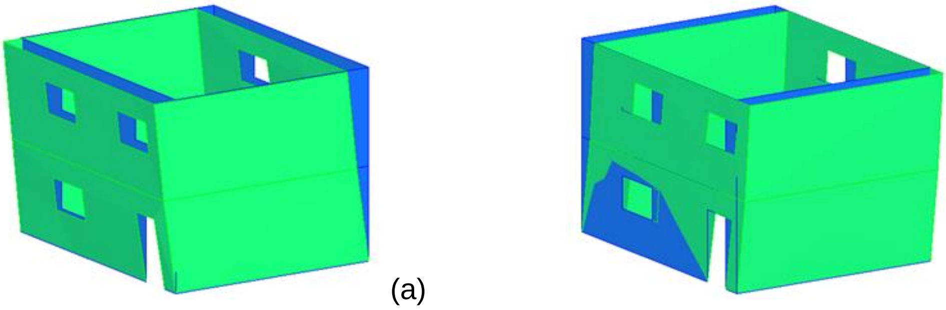
4.2. Frequencies and mode shapes

The solution of the eigenvalue problem, i.e., yielding eigenvalues (natural frequencies) and eigenvectors (mode shapes), gives an intuitive overview and a considerable insight into the dynamics of the structure. The identification of modal parameters of the two structures were obtained after they were placed on the shaking table, these identifications were based on the frequency response functions (FRFs), phases and coherences, estimated through traditional methods of signal analysis.

Results from these identifications have been processed by using the software LNEC-SPA and ArteMis [8, 9]. A first identification without any filter has been done aiming to identify the higher quantity of modes, however for both buildings not any clear mode was detected beyond the frequency capacity of the shaking table. Then, a low pass Fourier filter of 40Hz was applied to all signals. A DC offset process was also implemented. Intended to avoid aliasing, no decimation was applied to any signal, so that a sampling frequency of 250Hz corresponds to a sampling rate of 0.004s. Finally, boundary noise reduction by means of cropping of the signals' extremes was performed. As second step for the correct identification of the natural frequencies, it was implemented a series of frames with standard overlap of 2/3 and a reduction of leakage by using a Hanning window.

For the masonry buildings the first two natural frequencies were clearly identified by well define peaks, summarizing the frequency response of all the accelerometers. For both models, the first natural frequency occurs in the transversal direction. RM registered a value of 11.90Hz and UM a value of 11.11Hz. The second frequency obtained shows a mode shape in the longitudinal direction (in-plane with the walls with openings) in which for RM was registered a value of 20.02Hz and for UM a value of 16.12Hz.

Figure 3 presents the mode shapes obtained for both models. The behavior obtained is considered as global. However for both models it was noted that even the frequency values are different the directions and shapes are similar. Then RM possesses higher frequencies values for the two orthogonal directions. In spite of the geometry and materials of the masonry building models are the same, this behavior could be explained for three important factors, namely the inclusion of steel reinforcement, the filled of the vertical joints with mortar and the alternative masonry bond used, facts that notoriously increased the stiffness of the RM structure in both directions.



(a) (b)

Figure 3. Global mode shapes for the masonry building models: (a) first mode - transversal direction and (b) second mode - longitudinal direction.

4.3. Out of plane displacements

The in plane and out of plane values, namely the quantitative parameters of both experimental buildings were taken from the accelerometers placed on the models. From those accelerations, and after a detailed signal processing, double integration was made in order to obtain the displacements. As widely known, earthquakes induce horizontal forces to the structures, these forces affect buildings in all directions causing not only in plane but also out of plane effects. The combination of them could result in a very dangerous situation for civil structures and may lead in an imminent collapse of any building. In particular, the poor out of plane behavior of masonry is considered one of the main causes for the early collapse of masonry buildings during earthquakes. Figure 4 presents the maximum out of plane displacements registered during each seismic input. It is important to note that damage is more likely to occur if during design was not expected those force combinations or, in case of occurrence, was not implemented during construction a resistant solution to avoid catastrophic consequences.

As observed, the structural behavior of RM building to out-of-plane forces reveals a stiff structure. In this building it is observed that the maximum out-of-plane displacements occur at mid height of the non-openings´ wall at the first floor. Particularly in this wall, the displacements at mid height are always higher than the displacements exhibited by the slabs. In all cases, only minor differences in the displacements of the slabs were recorded, which confirms its monolithic behavior. In a general overview it is possible to conclude that for this model important cracks were developed in all facades when out of plane displacements are about 4mm. Displacements of the second level do not cause damage due to the slab displacements at the first level, which are similar to the ones of the second level.

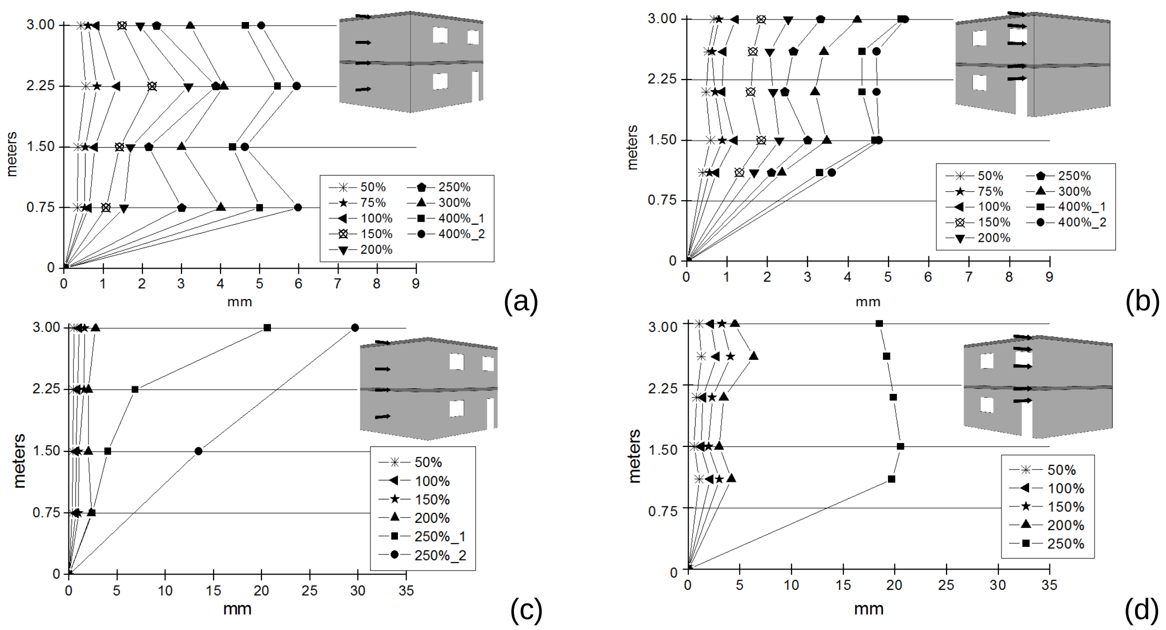


Figure 4. Out of plane displacements: (a) RM no openings wall; (b) RM wall with openings; (c) UM no openings wall and (d) UM wall with openings.

The out of plane behavior in the UM model presented significant differences in trends and values. In all walls, there was a general trend in the displacements up to the test run of 200% (PGA=1.11g). After it, there were remarkable increments of the out-of-plane displacements. Tests of 250% (PGA=1.33g) shows how in the non-openings´ wall the maximum out-of-plane displacements for each story are at the floor levels, achieving a maximum value at the second floor of approximately 30mm, and in the wall with openings the highest displacement is presented in the first slab with a value of approximately 20mm. During the experimental tests, this building presented a clear sliding mechanism at both story levels. Then, it could explain the differences presented between longitudinal and transversal directions. In this model, serious damages started since seismic input of 100% for all facades, those cracks were mainly associated with shear failure, reason why the out of plane displacements do not register important displacements at low tests run.

5 NUMERICAL SIMULATION

Numerical models provide to the research engineers, after validation with experimental results, with an extraordinary tool for the study of different possible scenarios. Its implementation allows save time and costs for the design of new structures or even for the analysis of existing constructions. For a numerical model is mandatory to be reliable and provide confidence. A suitable analysis of a masonry building should include the numerical model of its structure with the constitutive laws accurately describing the mechanical behavior of its materials. Then, after a calibration process, based on experimental results, analysis and conclusions could help in the interpretation and understanding of the structure behavior. Thus, design or strengthening solutions can be formulated. Nonetheless, to be properly implemented and effectively used, most of the current available tools require a large amount of resources: in terms of money, time, computational effort and knowledge [10]

For the study presented in this paper a macro modeling approach was implemented. In this kind of approach, the structure is divided into blocks with a considerable size. Each block represents a large portion of undamaged, or little damaged masonry, and the joints formed by these blocks represent a potential crack [11]. Even this method does not provide a detailed mesh, it made the discretization of the model simpler, resulting in a more straightforward geometrical construction and with less number of degrees of freedom, decreasing considerably the memory requirements and the required computational time.

Due to the importance involved in this project a sequence of non-linear dynamic analysis with time integration were developed. Furthermore, aiming at representing the damage evolution as accurate as possible, phase analysis was implemented. It compromises several calculation phases, one for each seismic input. In each phase a separately analysis is performed, in which the results from previous phases are automatically used as initial values of the next phase. The implementation of this analysis procedure in the numerical model simulates as real as possible the fact that the experimental damage observed after input of 200% was generated not only by the 200% input itself but also by the cumulative damage of previous inputs.

5.1. Description of the model

As observed in Figure 2, the most damaged building was the unreinforced solution. Then, intended to study the most critical case, the unreinforced masonry building was selected for the elaboration of the numerical model. The finite element software DIANA® [12] was used for the simulation. As first step, the geometry configuration and physical configuration follows the geometry dimensions and characteristics of the experimental model. It is composed by the ring beam foundation, the masonry walls and the slabs. Eight node curved shell elements for the walls and slabs and three node curved beam elements for the beam foundation were used.

A materials characterization was performed in both masonry samples and individual components i.e. blocks, mortar and concrete, in order to identify the materials properties values to be used in the numerical model. It include, among others, direct and diagonal masonry compression in samples made in purpose at the laboratory and samples taken from the debris of the experimental masonry buildings. In Table 2 are listen the results obtained. The reinforced concrete ring beam foundation and slabs are not intended to be study in the non-linear range. Then, only elastic properties were desirable.

Table 2. Material properties

Element	Material	*Young´s Modulus* E *(GPa)*	Poisson´s Ratio ν	ρ *(kg/m³)*	Damping ξ (%)	Tensile strength f_t (MPa)	Mode I fracture energy G_t *(N/mm)*	Compressive Strength f_c (MPa)	Compressive fracture energy G_c *(N/mm)*
Walls	Masonry	5.3	0.2	1200	3.6	0.16	0.04	5.95	9.52
Foundation /Slabs	Reinforced concrete	30	0.2	2400					

The construction of the experimental model was carried out over a strong floor outside the shacking table. After the construction process, suitable cured period and painted, the building was carefully moved to the shaking table. Then, it was attached to the table through post-tensioned steel rods, fixing the beam foundation. However the platform of the shaking table is perfectly leveling, and despite the rigorously care taken during the construction, mainly during leveling of the concrete beam foundation and transportation of the model, the fixation to the shaking table was not totally perfect. In order to simulate this drawback, structural interface elements were adopted numerically between the beam (foundation) elements and the fixed constraint (shaking table). Finally, the same seismic inputs in the two orthogonal directions introduced to the shaking table were used as input for the numerical simulation, thus following the same incremental path.

5.2. Results and comparison with experimental models

As previously study, the experimental shaking table campaign was preceded by a series of forced vibration tests, to estimate frequencies and vibration modes. From them, the calibration of the numerical model was performed. Then, in the numerical simulation using the linear properties of the model and the interface elements of the base as variables, both the frequencies and mode shapes were updated. The results of the numerical eigenvalue solution show a satisfactory fitting in accordance with the experimental ones. Frequencies comparisons are listed in Table 3 and the two mode shapes could be observed in Figure 5.

Table 3. Frequencies comparisons

Direction	Experimental f (Hz)	Numerical f (Hz)
Trans	11.110	11.107
Long	16.121	16.207

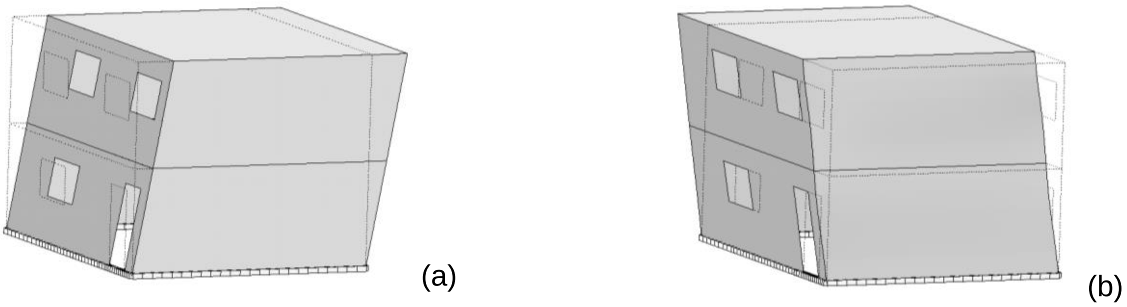


Figure 5. Mode shapes for the numerical simulation: (a) first mode - transversal direction and (b) second mode - longitudinal direction.

The comparison of experimental results with their respective numerical analysis results is a valuable information to assess the realism of a given structural analysis model. From Table 3 it is conclude how the numerical simulation represents accurately the critical natural frequencies of the masonry building in study. It should be noted that due to the geometrical construction in a CAD environment the numerical model possesses a different mass distribution at the corners of the model and at the foundation-building connection. Nevertheless, it is assumed that the mass and stiffness matrix clearly represent the linear behaviour of the unreinforced masonry building. As a proof of that, the comparison of numerical eigenvectors (mode shapes) in Figure 5 with experimental ones in Figure 3 reveals a similar global behaviour of the model. The two mode shapes correspond with the two main directions, with a first mode in the transversal direction (direction with shortest walls) and a second mode in the longitudinal direction (largest walls with openings).Then, it could be assumed that the two experimental modal properties and therefore the linear dynamic behavior of the unreinforced masonry building are accurately represented by the numerical model.

After the preparation and calibration of the numerical model a non-linear analysis with time integration was carried out by phases. Every input motion developed during the experimental campaign was represented by an input phase in the numerical simulation. Total strain rotating crack material model, also known as an orthogonal crack model, which simulate smeared cracking was implemented. This material model corresponds with distributed rotating cracks based on total strains, i.e. the stresses are describe as a function of the strains in the direction which are given by the crack directions. Furthermore cracks can continuously rotate according to the principal directions of the strain vector.

As a summary of the results obtained, the acceleration and displacement achieved by the numerical model during the test of 200% are compared against the experimental results. Figure 6 and Figure 7 show this comparison. These results correspond with the values obtained at the second floor for the two orthogonal directions.

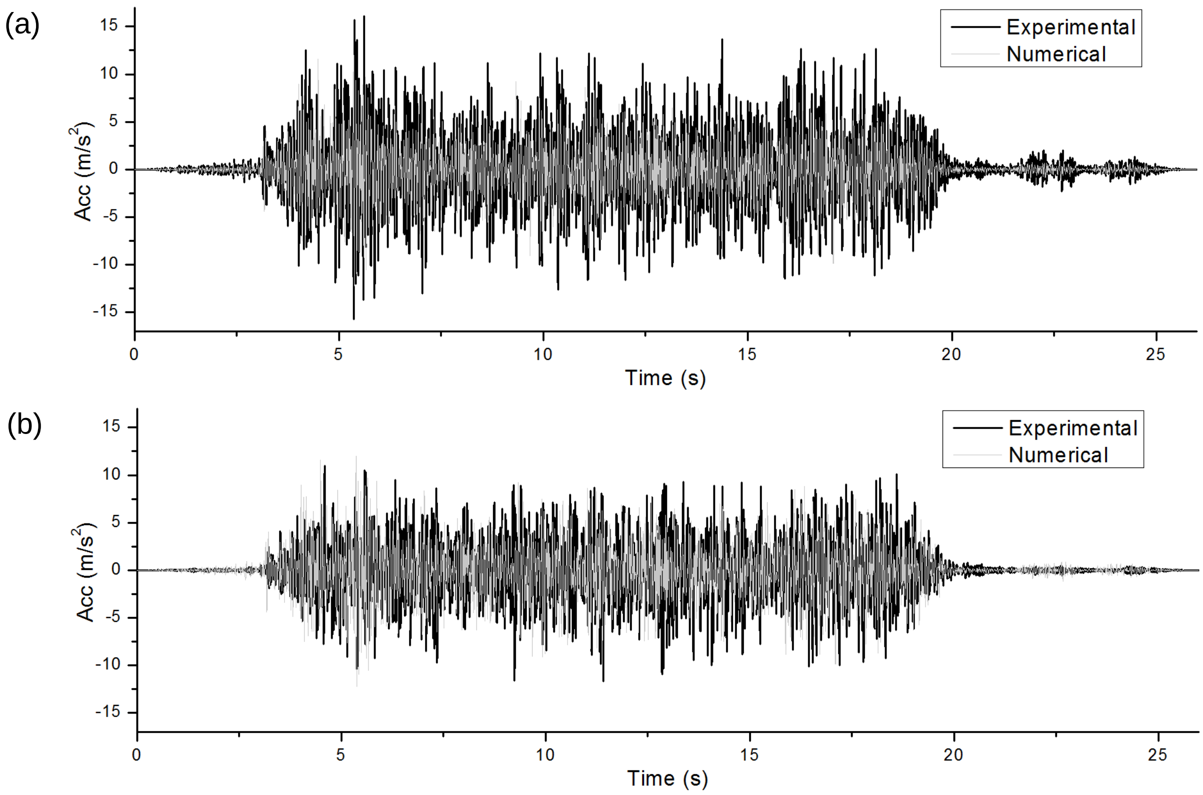


Figure 6. Experimental and numerical accelerations during input of 200% at second floor: (a) longitudinal direction and (b) transversal direction

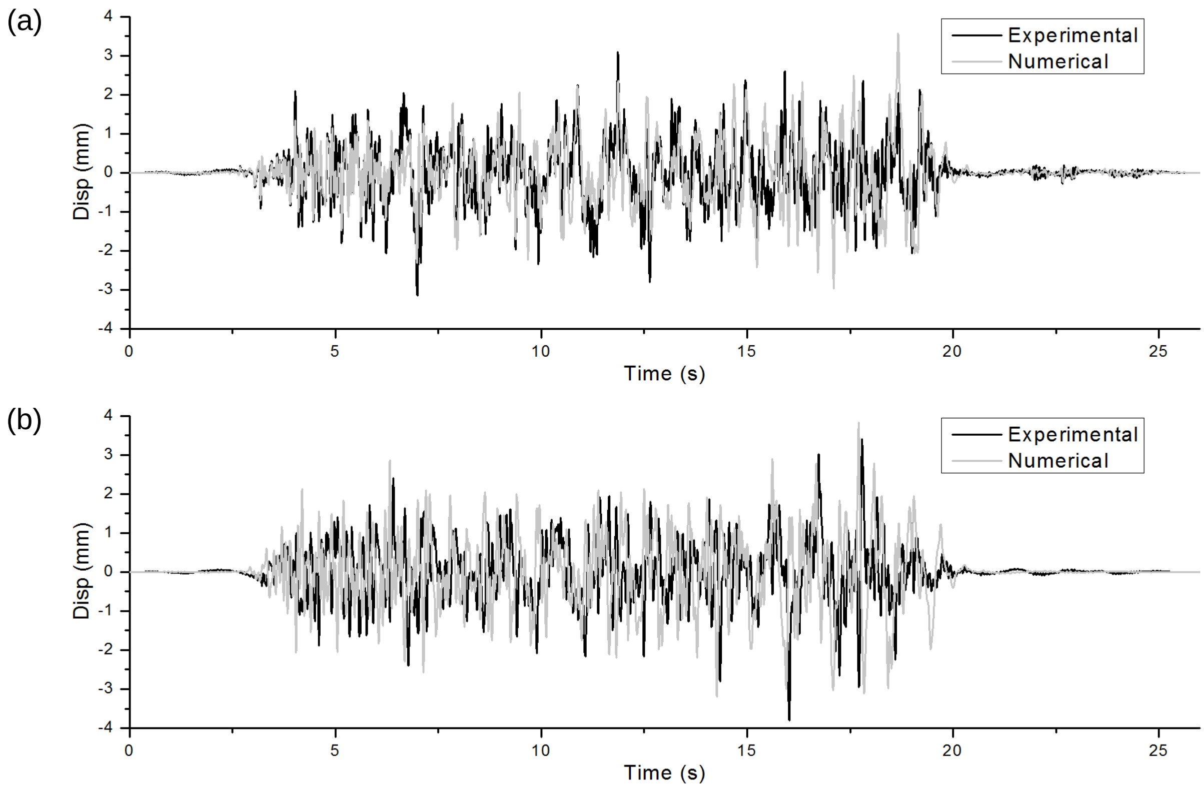


Figure 7. Experimental and numerical displacements during input of 200% at second floor: (a) longitudinal direction and (b) transversal direction

In order to have a general overview of the approximation obtained with the proposed numerical model, Table 4 presents a comparison of accelerations´ and displacements´ peak values obtained during the experimental tests and corresponding values reached by the numerical simulation.

Table 4. Peak values comparison between experimental and numerical results

	Accelerations (m/s^2)				Displacement (mm)			
	Long		Trans		Long		trans	
Test	Exp	Num	Exp	Num	Exp	Num	Exp	Num
50%	6.67	7.67	4.49	6.30	0.66	0.72	1.05	1.43
100%	8.89	10.04	7.37	8.33	1.30	1.07	1.31	1.91
150%	12.47	11.82	9.64	8 .77	2.06	1.46	2.33	2.11
200%	16.08	12.23	11.66	10.88	3.15	3.57	3.81	3.84
250%	20.29	13.41	13.38	12.15	12.96	9.81	18.94	7.24

From Figure 6, Figure 7 and Table 4 is extracted that the accelerations in both orthogonal directions show a good correlation between experimental and numerical results. Longitudinal direction presents larges values than the transversal one, revealing a more weak direction due to the cracks nearby the openings. The numerical results follow the experimental ones in a suitable manner. It could be observed how, not only for both geometrical directions but as well for the positive and negative signals´ range, numerical wave shapes are in agreement with the experimental waves even in the highest peaks.

Regarding the displacements, the results provided by the numerical model show a very good fitting with the experimental values. It is highlighted that until test of 200% maximum values are just about 3.5mm. Wave shapes and maximum values for both directions in the positive and negative range follow in a suitable manner the ones obtained in the experimental model. The experimental model and its numerical simulation present about 3m of high, then the numerical model is representing accurately displacements about 0.12% the high of the model. Maximum differences between results in both directions are just 0,60mm. It should be mention that during the experimental campaign, at the stage of 200%, it was notoriously appreciated the damage generated. The model presented mainly horizontal and stepped diagonal cracks, distributed through all the walls in the two stories.

The numerical model proposed represents with a truthful path the experimental behavior of the unreinforced masonry building as discussed, in both accelerations and displacements, validating the choices made regarding numerical solution, type of analysis, structural elements and material model. Furthermore, the results provide and important evidence about the effectiveness of the connections between the different type of numerical elements, providing confidence and reliability to the proposed numerical simulation.

6 CONCLUSIONS

In order to assess the seismic behavior of a new constructive system based on concrete block masonry, a large experimental campaign based on shaking table tests was carried out. Two symmetrical models were constructed from a prototype structure. Following European codes, one building was designed and built with horizontal and vertical reinforcement and one as a simple and traditional construction. Numerical simulation by finite elements and non-linear time history analysis in phases was performed. Comparisons between experimental and numerical results were analyzed.

The study indicates that in spite of the same seismic inputs were applied to both models the final state of UM building and its nonlinear behavior led to a considerable amplification of the deformations and damages but no collapse. It is also conclude that the combination of vertical and horizontal reinforcement, about the minimum required by Eurocode 8 clearly improves the seismic response of concrete block masonry buildings, leading to adequate structural robustness for very high seismic loading. On the other hand, the unreinforced masonry model has provided very good capacity for moderate to high seismic loading, together with a ductile failure mode with enough capacity for vertical loading even after major damage.

The comparison between experimental and numerical accelerations and displacements at the second floor reveals a good representation by the numerical model of the unreinforced masonry building behavior. The procedure, the structural numerical elements and the constitutive laws adopted fitted with the required accuracy needed for the simulation of dynamic experimental tests. Then, the numerical results reveal a valid numerical model for the posterior design, study and analysis of new masonry structures.

Future analyses and comparisons involving stresses, strains, and damage mechanisms will increase the reliability of the numerical model. Different geometries configurations and a full-scale numerical model with the same properties presented, together with a parametric study involving steel reinforcement and different ground motion zones, will endorse the design method of the new concrete block masonry buildings.

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