



EXPERIMENTAL INVESTIGATION ON THE SEISMIC BEHAVIOUR OF NEW CONCRETE BLOCK MASONRY BUILDINGS

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The present work deals with the experimental validation of a new structural solution for concrete block masonry buildings. Dynamic tests of two identical two-story concrete block masonry models were performed on a shaking table in reduced scale 1:2, with focus on the global behavior. Both models were tested in the two orthogonal horizontal components with uncorrelated artificial accelerograms compatible with the elastic response spectrum defined by Eurocode 8. The first model was tested in reinforced conditions following the same code, while the second building was tested as an unreinforced solution. The identification of the dynamic properties using modal analysis (based on input-output techniques) as well as the seismic evaluation of both buildings is presented.

In the experimental tests, various input motions with incremental amplitude were implemented. The damage identification through stiffness degradation is studied. Furthermore, the experimental analysis encompasses parameters as the cracking patterns with consequence collapse mechanisms. In-plane and out of plane behavior in terms of displacements and lateral drifts are discussed. Findings related to the global dynamic behavior and comparisons between the results of the two buildings are also presented.

Keywords: *New masonry system, Shaking table test, Dynamic identification, Seismic performance*

INTRODUCTION

Masonry constructions have had an important role in the history. It is possible to observe how many of the structures made with this material have prevailing until nowadays. At the moment the same construction system is still used and even it is a kind of construction material that has become popular mainly due to its fire resistance, thermal capacity and durability. Its construction advantages together with its proved capabilities (e.g. its high resistance to compression) and with the increased research on it, become masonry in a competitive alternative to the construction of low to medium residential buildings. For it masonry has to be planned, designed and built as an earthquake resistance structure.

The low tensile strength and ductility appear as the most important drawbacks well known for the research community to its implementation in buildings (Magenes 2006, Tomažević 1999).



As part of the effort to overcome it several investigations have been carried out for the civil department of the University of Minho, enabling the development of a new construction system that involves an innovative hollow cell concrete block design whose geometry among other facilitates the inclusion of vertical steel. Studies and improvements regardless the mortar and steel properties to be used on the system have already been made. In brief the structural masonry proposed is based on three cell concrete blocks and a modified general purpose mortar to be used for filled both horizontal and vertical steel reinforcement. Good agreements between force-displacements, energy dissipation and lateral strength results have been found by Haach et al. (2010) in in-plane static cyclic tests developed in masonry walls by using this system.

As step forward the present paper contains results of the first series of shaking table tests conducted in buildings models by using the proposed system subjected to a series of ground motions excitations. At present, the use of shaking table testing approach to assess the seismic behavior of civil engineering structures (wood, concrete and masonry) has revealed to be suitable regardless accuracy in comparison with the real behavior exhibit for similar structures during seismic events. The intent from project conception was to develop a concrete block masonry system that provided improved seismic performance, with respect to traditional masonry construction with focus in residential construction following requirements of no collapse and damage limitation. To accomplish this, it is mandatory to clearly understand what the global behavior (in buildings more than in individual elements) and the out of plane responses are when subjected to dynamic loads. It is expected that the proposed solution together with the inclusion of both horizontal and vertical steel reinforcement helps to provide these enhancements; furthermore it is believe that the unreinforced implementation of the system could be safety used in non-seismic prone areas.

EXPERIMENTAL PROGRAM

The residential prototype building is a typical individual two story house with regular geometry exiting in typical belonging to modern residential aggregates, with an interstory height of 3.0m, 2 opposite facades with a percentage of openings of approximately 14% and two gable walls without openings, corresponding to the walls bounding the contiguous houses. Slabs are built in reinforced concrete aiming at working as a rigid diaphragm. From it two models of structural masonry buildings have been considered. In consistence with Eurocode 8 (2004) and Eurocode 6 (2005), a reinforced (RM) solution is studied, in which minimum requirements regarding reinforcement ratios are following, the second one correspond with an unreinforced (UM) solution. The comparison between reinforced and unreinforced masonry should clarify the influence of the reinforcement on the seismic performance of structural masonry. For the UM building the traditional masonry bond pattern is used, whereas for the RM vertical continuous joints are considered in which vertical reinforcement will be placed.

Figure 1a shows RM building ready for testing. Tests run were carried out as a series of incremental seismic inputs on the 3D shaking table at the national laboratory of civil engineering (LNEC) in Lisbon-Portugal which have a platform plan dimensions of 4.6m x 5.6m and a payload of 400 KN (Bairrao and Vaz 2000). Due to dimensions limitations caused by the size of the shaking table, tests were developed in a scale 1:2. For it Cauchy similitude law was implemented (Carvalho 1998), see Table 1 in which *P* and *M* designate prototype and

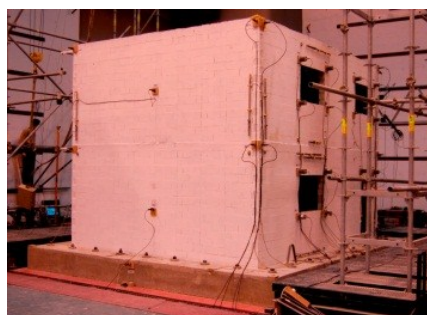
model respectively. Following this scale factors, the experimental model buildings have 4.20m x 3.40m in plan and 3.0m in height (outside dimensions), with a wall thickness of 0.1m. The interstory height is 1.4m with windows openings of 0.8m x 0.5m and door openings of 0.5m x 1.1m. The composition of the concrete for the blocks was designed so that the compressive strength of the units was the same as the minimum of 10MPa required for the prototype. The mortar was handmade and fine sand was used also to comply with the reduced scale of the bed joints. A cement mortar 1:3 with a water/cement ratio of 0.9 was used so that appropriate consistence was achieved enabling to use the mortar in bed joints and in the vertical reinforced hollow cells.

The tests models were placed in a reinforced concrete foundation that was clamped to the shaking table, so that fixed connection was provident with zero deformation. The input was imposed to the table in the two horizontal directions. Two uncorrelated accelerograms with a total duration of 25.6seg, one for each direction was generated from the inelastic response spectrum given for the Eurocode 8 for Lisbon region, ground type A and 5% damping, following similitude law the resulting spectrum was also scaled as shown in **Figure 1b**.

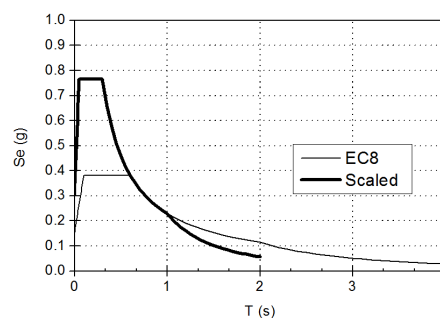
Table 1: Cauchy similitude law

Property	Cauchy scale
Length [L]	$L_p/L_M = \lambda$
Young's Modulus [E]	$E_p/E_M = 1$
Specific mass [ρ]	$\rho_p/\rho_M = 1$
Displacement [d]	$d_p/d_M = \lambda$
Acceleration [a]	$a_p/a_M = \lambda^{-1}$
Time [t]	$t_p/t_M = \lambda$
Frequency [f]	$f_p/f_M = \lambda^{-1}$

With regard to the instrumentation, a total of 28 accelerometers were placed in identical position for each building corresponding with slabs and openings corners. From the acceleration time history registered for them and by means of mathematical double integration displacements were obtained.



(a)



(b)

Figure 1: (a) masonry building ready for testing and (b) standard and scaled elastic response spectrum.

As mentioned the tests input were imposed to the table by incremental amplitudes. Table 2 summarizes the PGA measured at the base of the models for each input run. In the UM a second test of 250% was performed however before substantial building collapse occurred and due to equipment protection most of the instrumentation was removed. Hence the input motion at the base of the model was not measured.



Table 2: Input series and corresponding PGA

Test	Reinforced model		Unreinforced model	
	PGA gable wall (m/s ²)	PGA wall with openings (m/s ²)	PGA gable wall (m/s ²)	PGA wall with openings (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 (1.00g)	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)	-	-

TEST RESULTS

Figure 2 depicts the crack pattern and final damage for both buildings. It should be stressed that these were the final damage for RM after test run of 400% (PGA=1.61g) and 250% (PGA=1.33g) for UM. It should be noted that the onset damage on RM model appeared after input of 100% (PGA=0.39g) and 80% of the final damage was occurred after test of 250% (PGA=1.26g). It was observed 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.

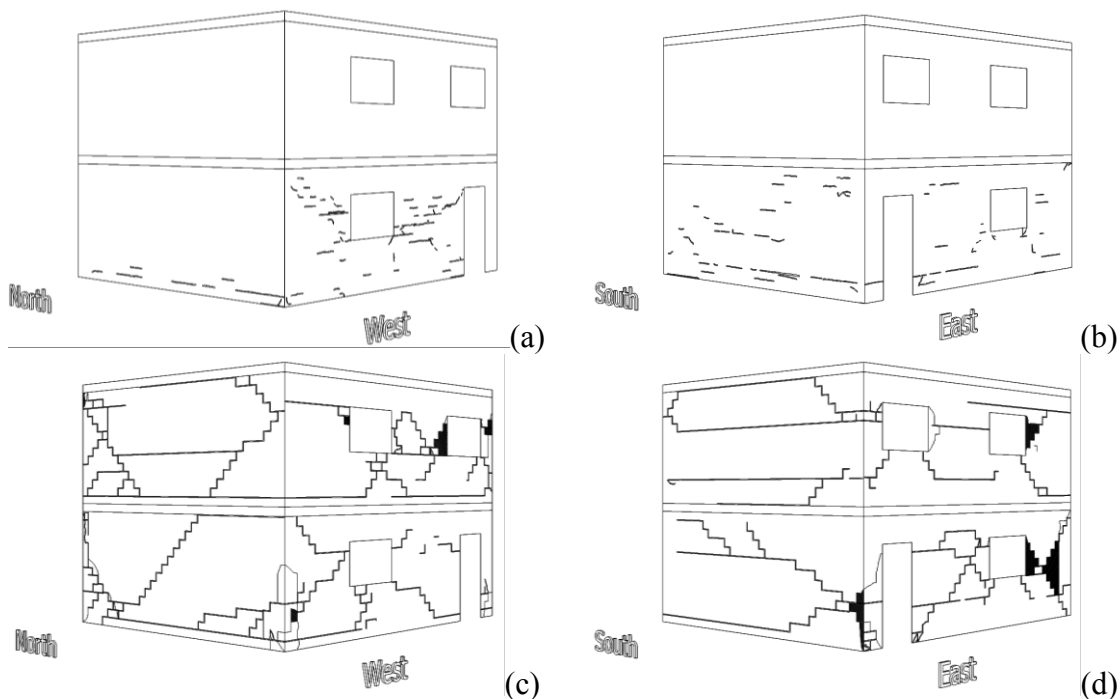


Figure 2: Final damage patterns in: (a, b) reinforced and (c, d) unreinforced building.

On the UM the seismic inputs causes more severe damage at an earlier stage than RM. The maximum input motion attained by UM building represents 62.5% of the maximum input attained by the RM model, however well-defined and localized continuous cracks and



disconnection of blocks was observed. It is noted how for this building the damage occurs in the entire model, visible diagonal stair step crack are presented and a remarkable horizontal cracks were observed at the first course of units in both stories. Furthermore a sliding mechanism was clearly observed during testing mainly through the horizontal cracks presented in the gable walls on **Figure 2 c** and **d**.

The solution of the eigenvalue problem, yielding eigenvalues (natural frequencies) and eigenvectors (mode shapes), gives an intuitive overview of the problem and a considerable insight into the dynamics of the structure. The dynamic properties of both models were studied. Input-output techniques were performed by means of low amplitude white noise excitation in between the main tests run. For both buildings the first natural frequency occurs in the transversal direction (in-plane with the gable walls). RM registered a value of 11.90Hz and UM a value of 11.27Hz. A second frequency was also obtained, here RM registered a value of 20.02Hz and UM a value of 16.12Hz in the longitudinal direction (in-plane with the walls with openings). **Figure 3** presents the mode shapes obtained for both models. It is noted that even the frequency values are different the directions and shapes of the modes are similar. As expected this means that both building present similar global behavior, but RM possesses always higher frequencies values. In spite of the geometry and materials of the masonry building models are the same, this behavior is explained for three important factors, the inclusion of steel reinforcement, the filled of the vertical joints with mortar and the distinct masonry bond used, facts that increase the stiffness of the RM structure in both directions.

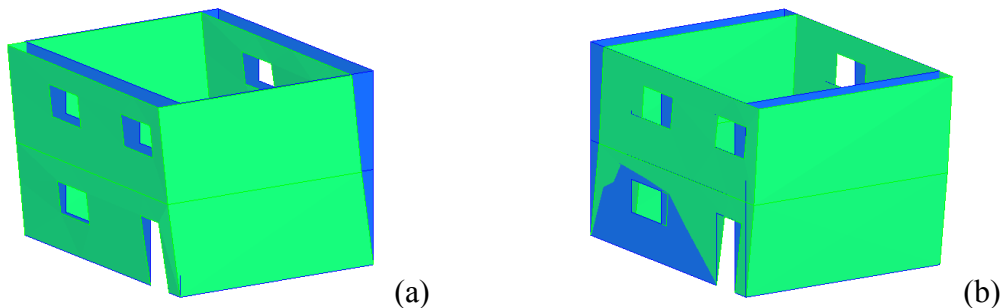


Figure 3: General mode shapes for the masonry building models: (a) first mode-transversal direction and (b) second mode-longitudinal direction.

Because there is a logical relation between the damage presented in the structures and their stiffness degradations, the change of the frequencies through tests run was evaluated. The values obtained after each incremental input are summarized in Table 3. It can be observed a relationship between the stiffness degradation and the progress of the tests in RM in which for the two orthogonal directions the frequencies values decreases.

Table 3. Evolution of the fundamental frequency (Hz)

	Test run	Initial	50%	75%	100%	150%	200%	250%	300%	400% 1	400% 2
RM	long	20.02	19.17	18.19	17.21	15.69	15.32	14.40	14.40	13.18	12.57
	trans	11.90	11.66	11.66	11.35	11.11	11.11	11.05	10.99	10.99	10.74
UM	long	16.12	15.05	-	13.98	15.40	14.69	13.63	-	-	-
	trans	11.27	11.14	-	11.02	11.73	12.09	11.85	-	-	-



Not similar situation was presented in UM in which frequencies decreases only until test of 100% (PGA=0.51g), after that a not very well defined trend is obtained. The reason for this effect is considered to be the sliding at the bed joint cracks presented on the entire model after tests of 100% which causes modifications to the vibration properties.

The in-plane behavior of the buildings models for all the input series is given in **Figure 4**. The values shown correspond with the maximum obtained for each test. Similar behavior was found between gable walls and walls with openings. The results show a considerable difference between models with a maximum displacement during the test run of 400%_2 over 5mm for both directions in RM. The low displacement in this model explain the minor damage observed, whereas UM presents an interesting difference between directions, here a maximum value over 35mm is found in the gable walls and a value lower than 30mm is registered in the walls with openings. This implies that the sliding mechanism presented in this building was more remarkable in the in-plane direction corresponding with the gable walls. The damage in RM is concentrated in the first floor, which is in accordance with the rapid increase on the displacements at this level. The evolution of displacements in the second floor is much slower, which reflects the minor damage developed on it. On the contrary for UM the displacements profile is practically linear in height, meaning that the damage develops in both floors of the building, like actually occurred. The significant increase on the displacements of the walls occurs for the earthquake simulation corresponding to 250% of the reference seismic input loading (PGA=1.33g) and its repetition. The remarkable increase on the displacements is associated to the increase on the crack opening of the diagonal shear cracks. It was during these two tests that the collapse of block units happened. Finally it is interesting to note that the repetition of the earthquake simulation for the seismic input run of 250% results in the important progress of damage.

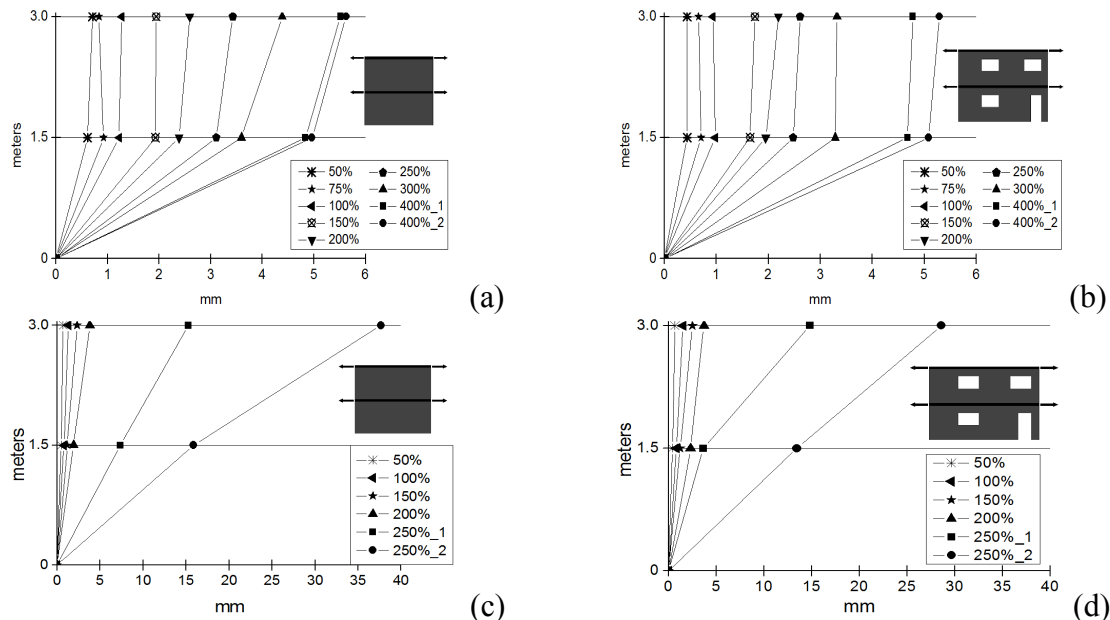


Figure 4: In-plane displacements: (a) RM gable wall; (b) RM wall with openings; (c) UM gable wall and (d) UM wall with openings.

In **Figure 5**, maximum interstory drifts for each input test run are shown and compared between the two masonry models. It can be seen that in agreement with previously results the

drifts corresponding to the UM are much greater than those from the RM under the same level of input ground motion. Commonly in the RM model (Figure 5 a and b), the maximum lateral drifts are higher in the first floor, particularly in case of longitudinal walls, which is associated to level of damage developed on them. In general the lateral drifts recorded in this model for a PGA of 1.61g (test run 400%) are associated to minor and controlled damage. In the UM model, the maximum drifts are recorded at the second floor and attains the maximum value of 2.5%.

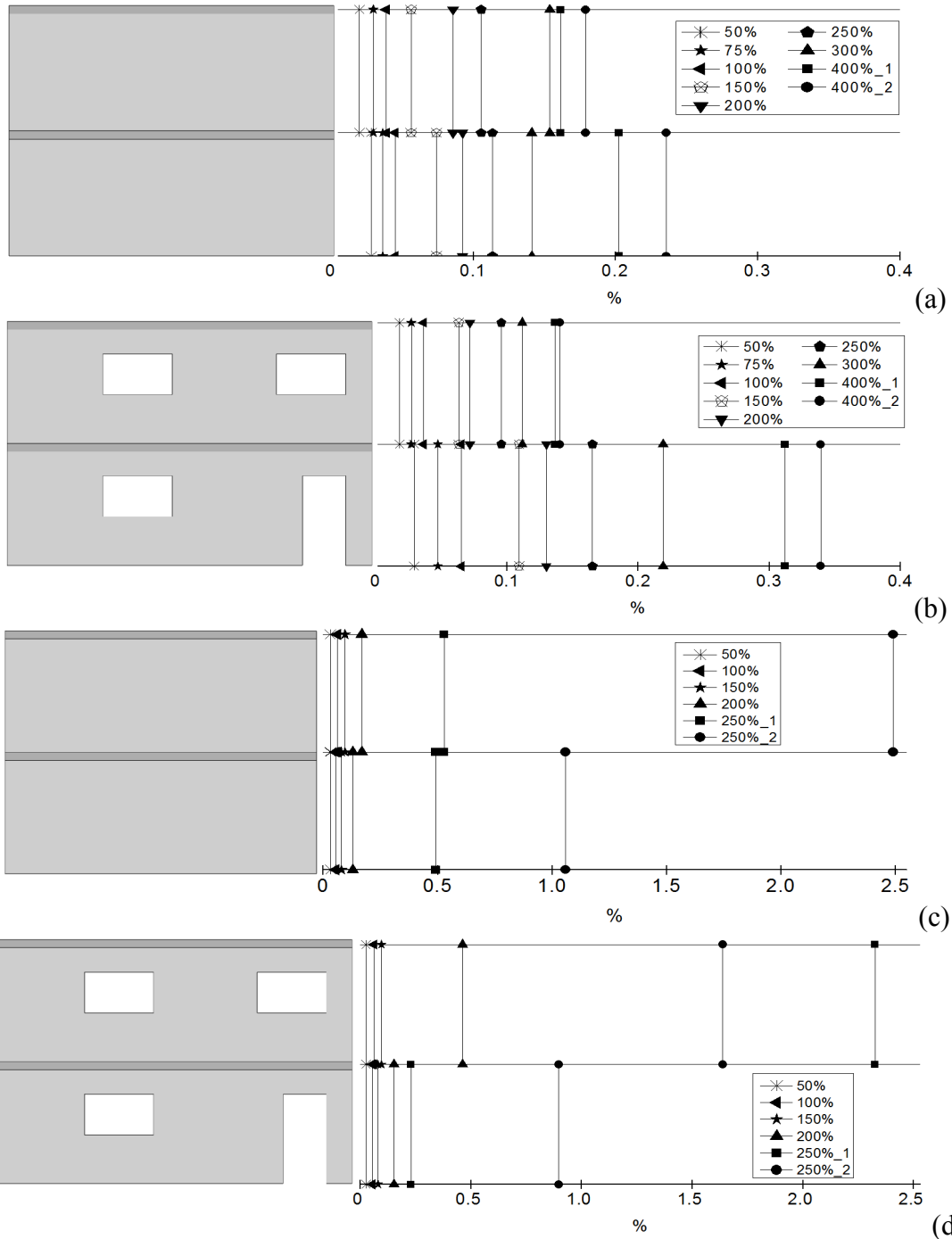


Figure 5: Interstory drifts: (a) RM gable wall; (b) RM wall with openings; (c) UM gable wall and (d) UM wall with openings.



As was seen in the in-plane responses, the lateral drifts are very controlled up to the seismic input loading corresponding to a PGA of 1.11g (test run of 200%) and amplifies clearly for the PGA of 1.33g (tests run of 250%). It is also observed that the lateral drifts for seismic input corresponding to a PGA of 0.51g (test run of 100%) are of 0.085% in the first floor and of 0.089% in the second floor. These results are associated to the development of reduced damage in the UM model for moderate levels of seismic input, which shows the suitability of the unreinforced masonry solution to moderate seismic loading. However the crack failure caused by subsequence tests increased considerably the lateral drift corresponding with a significant enlargement of the damage in this model associated with the increase of the openings at diagonal cracks and correlate sliding mechanism in the horizontal ones. However, it should be noticed that no collapse of the structure was observed, even after the repetition of the last input seismic loading.

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. Damage is more likely to occur if during design was not expected that this force combinations happened or, in case of occurrence was not implemented during construction a resistant solution to avoid catastrophic consequences. As final analysis for the understanding of the behavior of the new system in buildings the out of plane displacements were study. **Figure 6** presents the maximum displacements obtained.

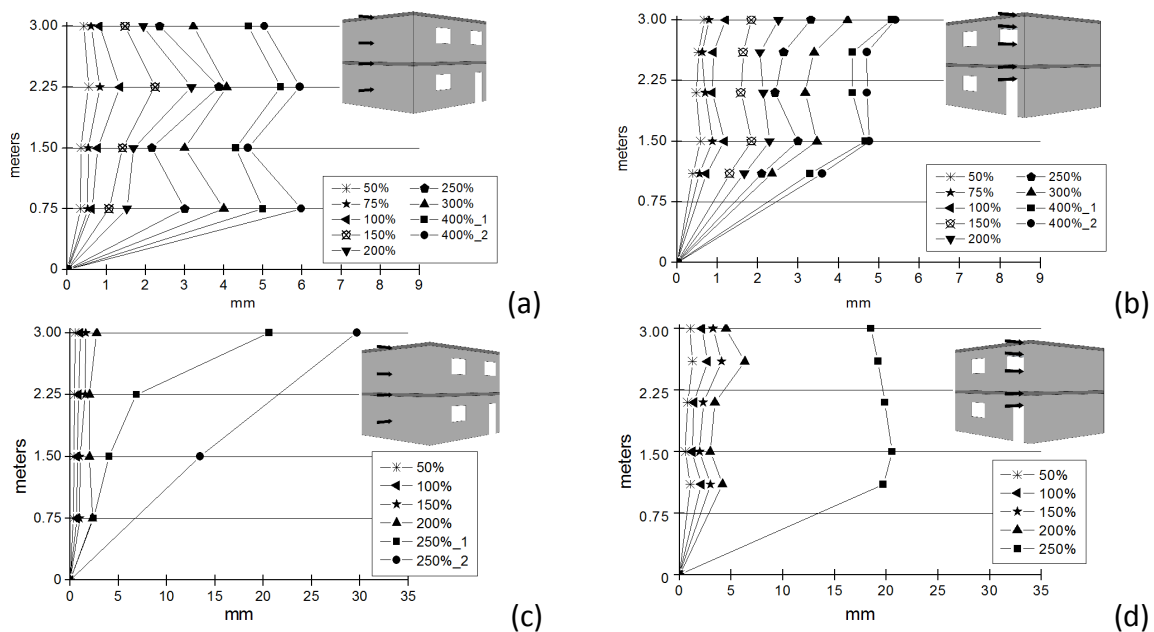


Figure 6: Out of plane displacements: (a) RM gable wall; (b) RM wall with openings; (c) UM gable wall and (d) UM wall with openings.

In the wall with openings was intended to measure the out-of-plane displacements at three different cutaway sections in accordance with the windows corners in the two story levels as well as window and door corners. Unfortunately, in both buildings some of the accelerometers fell during the tests run, preventing the measurement of them. It should be noted that the cutaway studied in this facade correspond with the most near to the corner of the building. As



observed in **Figure 2a**, no damage was registered at the right side of the door opening, then the out of plane displacements may not be representative of the total wall like for instance the out of plane at the middle of the wall in which would be expected higher displacements.

In RM it is observed that the maximum out-of-plane displacements occur at mid height of the gable wall at the first floor. In this wall, the displacements at mid height are always higher than the displacements exhibited by the slabs. In all cases, only minor differences on the displacements of the slabs were recorded, which confirm the monolithic behavior of the RM model. 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 slabs displacements how was observed in the drifts analysis.

The out of plane behavior in the UM model presented significant differences in trends and values. In both the gable wall and in the wall with openings, there were a general trend in the displacements up to the test run of 200% (PGA=1.11g). After this, there is a remarkable increase of the out-of-plane displacements. Tests of 250% (PGA=1.33g) shows how in the gable 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 higher displacement is presented in the first slab with a value of approximately 20mm. This reveals the rotational effects presented, confirming the trend of UM model to slide during the tests. In this model the damage started since 100% for all facades, these cracks are mainly associated with shear failure, reason why the out of plane displacements do not register important displacements at low tests run.

SUMMARY AND CONCLUSIONS

In order to assess the behavior under earthquake loading of masonry buildings built with a new concrete block concept several shaking test inputs were performed in two proposed models. Following European codes, one building was designed and built with horizontal and vertical reinforcement and one as a simple and traditional construction. In spite of the same seismic inputs were applied to both models the damage state of UM building and its nonlinear behavior lead to a considerable amplification of the damage and deformation. The failure mode presented for it was that of shear failure due to the bed joint sliding. This mechanism as seen in the damage pattern presents clear continuous connection among cracks that affecting the whole building in the first and second story. These results confirm the loss of connection through the high of the model. However the connections of the intersecting walls due to the traditional bond pattern implemented revealed to be adequate enough to avoid detachment and out-of plane rotation of the masonry facades. It is conclude that with the proposed system unreinforced masonry solution can sustain a controlled behavior until PGA=0.25g in which no any damage was presented. With a double value, that is with a PGA=0.5g there are presence of cracks in which repair of most of them can be easily be carried out, however a more detailed analysis of it should be made. Therefore, it is essential to consider the effect of steel reinforcement for the practical application of the seismic safety concrete block masonry buildings to be implemented in seismic prone areas.

The study reveals as well that the contribution of the steel reinforcement to the global capacity of the structure enhances the response to seismic excitation. The increase in the stiffness due



to the infill with mortar of the vertical joints as well as the generated better connection between walls and between walls and slabs allow the system to sustain an input excitation with a PGA=1.61g with only smeared localized cracking in the first story. This value represents 21% more of PGA than the one attained by UM building in which considerable damage was observed. The differences in the damage distribution as well as in the responses through the tests run in terms of displacements showed that reinforced masonry buildings are able to sustain relatively high dynamic excitation due to significant level of structural overstrength.

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