Seismic Response Analysis Of Concrete Block Masonry Buildings: An Experimental Study Using Shaking Table

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SUMMARY
The combination of a series of structural and economic advantages like the high capacity to resist compressive loads, the simple and easy method of construction, comfort properties and structural performance given for the new masonry structures makes the implementation of masonry as a profitable solution for the increased residential houses demand. An innovative solution for the construction of low to medium residential masonry buildings is here analysed. The assessment of the earthquake resistance capacity of this system is evaluated aiming at providing further guidelines for its design and construction. This paper presents a step in this process related to the experimental mechanical validation of the constructive system based on concrete block masonry buildings under seismic loading.

Keywords: New Masonry System, Shaking Table Test, Seismic Performance

1. INTRODUCTION

Unreinforced masonry has been used since early stages of civilization, particularly due to the availability of natural stone materials, irrespectively in seismic hazard and non-seismic hazard regions. The suitable seismic behavior of masonry buildings relies on the shear resisting mechanisms and on the adequate connection between intersecting walls and between walls and floors and ceilings. However, the analysis of the failure mechanisms found in unreinforced masonry buildings has shown its poor lateral force resisting properties. Its large mass together with its low tensile strength are the mainly responsible for the brittle behavior of this type of constructive system (Bruneau, 1994, Magenes, 2006). In fact, as part of the engineering post-earthquake conclusions it has been common that the total or partial collapse occurs due to the poor quality of materials and construction techniques. Besides, the lack of connections between structural members has become a relevant aspect that affects the global behavior and repeatedly unreinforced masonry has proven to perform unsatisfactorily. Several recent examples can be pointed out like the 2007 Peru earthquake that showed the importance of the construction techniques and the poor quality of the construction materials. The absence of lateral connections promoted the premature out-of-plane collapse of the walls leading often to the total collapse of the masonry buildings (San Bartolomé and Quiun, 2008). During the 2010 Chile earthquake the confined masonry buildings presented the in-plane failure as the most common damage observed. Shear cracking was detected in walls built using all types of masonry units. In this type of construction system walls crushed at the wall toe leading the failure of the reinforced concrete tie-columns, mainly shear failure at the tie-columns ends and buckling of longitudinal reinforcement was observed. The few confined masonry buildings that collapsed were due to the soft story mechanism. Similar behavior has been observed in other seismic events on infill masonry buildings, the significant shear strength at the ground level caused by the induced lateral forces in multi-story buildings appears as the main reason (Brzev et al., 2011). Another example of a destructive earthquake occurred on 4 of September, 2010 in Christchurch, New Zealand. The damage found in unreinforced masonry buildings were in accordance with projections for the scale of the earthquake and consistent with observations previously made in large earthquakes (Ingham and

Griffith, 2011). Here, the weak out-of-plane strength was highlighted during the earthquake shakes in which parapets and gable walls were severely affected. These failures often resulted in further structural damage due to masonry sections fell through the roof of neighbouring buildings increasing considerably the self-weight of the landing structure, representing also a very dangerous situation to the people nearby to the falling structure. Inspections to out-of-plane wall failures typically indicated poor or not anchorage of the wall to the supporting diaphragm. In-plane damage has been also observed, in which diagonal pier cracks were the most common failure mode.

As seen, several failure and collapse mechanisms have been already well identified for this type of building construction when subjected to earthquakes, but it is believe that with an adequate construction technique and from the learned experience registered in past earthquakes it is possible to develop a construction system based on masonry materials that satisfices both the safety and comfort conditions for the construction of residential buildings (low to medium rise buildings).

Therefore, the work presented here deals with the evaluation of the seismic behavior of a reinforced concrete block masonry building with a constructive system recently proposed at University of Minho and compared with the performance of an unreinforced concrete block masonry building with an identical configuration. The seismic behavior of the masonry buildings is acquired by experimental shaking table tests carried out at the National Laboratory of Civil Engineering (LNEC) in Lisbon, Portugal. In the subsequent section a detailed description of the experimental procedures and a discussion of some experimental results are provided.

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 (Eurocode, 2004, Eurocode, 2005). The concrete block units show a new concept design in terms of geometry, it was 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. 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 steel reinforcement is placed. With this configuration the constructive system can be built following different bond patterns i.e. the traditional masonry bond and an alternative masonry bond composed of continuous vertical joints formed by the frogged ends of the blocks.

2.1. Materials Properties

As mentioned the quality of the construction materials plays an important role in the performance of the whole structure during earthquakes. The concrete block units, the modified mortar, the truss reinforcement and walls specimens with and without reinforcement have been previously studied (Haach, 2009). From these investigations important information has been obtained. According to Eurocode 8 (Eurocode, 2004), units used to build masonry structures in seismic areas should have a normalized compressive strength normal to bed joints not lower than 5 MPa and a normalized compressive strength parallel to bed joints not lower than 2 MPa. Results in proposed masonry units show a compressive strength of 10 MPa and 7 MPa respectively. For the mortar a minimum compressive strength of 10 MPa is recommended by Eurocode 8 in case of reinforced masonry structures in seismic areas. With a cement mortar 1:3 and water/cement ratio of 0.9, the modified mortar used exhibit values over 11 MPa. From direct tensile test the pre-fabricated truss reinforcement shows an average of 580 MPa 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 that it influences the final behavior of the masonry structures such as compressive and bond strengths, and deformability. From the 2010 Darfield earthquake, 6 Mortar samples were extracted from damaged masonry buildings and tested in compression, resulted in compressive strengths lower than 2 MPa (Ingham and Griffith, 2011). 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 a good condition.

The construction bond pattern was also studied. From the two bond pattern options it was found that when steel reinforcement is needed the bond pattern with continuous vertical joints makes the construction technology easier, and is preferably as good performance was found in masonry shear wall tests (Haach et al., 2010). For unreinforced masonry solutions traditional bond pattern with dry head joints is better as the construction is much faster in this way.

3. EXPERIMENTAL PROGRAM AND DESCRIPTION OF TESTS

Two masonry building models with scale 1:2 were built and tested in a 3D shaking table. The models are intended to represent a prototype masonry building structure. The residential prototype building consists of an individual two story house with regular geometry, commonly found in residential aggregates, with an interstory height of 3.0m, 2 opposite façades with a percentage of openings of approximately 14% and 2 opposite façades without openings corresponding to the walls bounding the contiguous houses. The slab floors are in reinforced concrete and work as a rigid diaphragm.

Cauchy scale law was adopted for the construction of the experimental models and for the input design. Figure 1a 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 in the case when the stresses induced by the gravity loads are small and negligible with respect to the stresses induced by the seismic forces. This is the case when the lateral load resisting system consists primarily of shear walls.
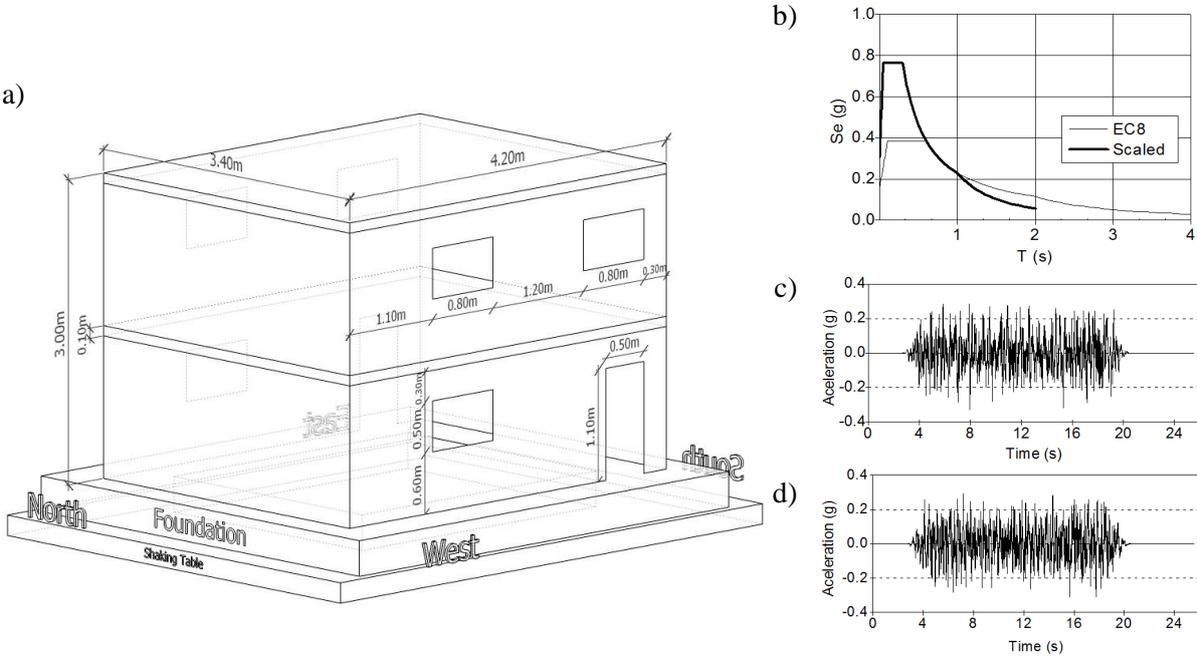
Figure 1. (a) Geometry for RM and UM experimental models; (b) standard and scaled elastic response spectrum; (c) artificial accelerogram North-South direction and (d) artificial accelerogram East-West direction

The two models are intended to represent a reinforced (RM) and unreinforced (UM) masonry building

solution respectively. Aiming to assess the performance of the new system under the present European code regulation, for the RM model, minimum requirements about reinforcement were implemented. Both models used a concrete beam foundation that was connected to the shaking table by mean of steel rods and reinforce concrete slabs for the floors. In the RM building the vertical steel reinforcement was anchorage to the beam foundation and embedded through the slabs.

The tests were developed by imposing simultaneous base excitations, acting along two orthogonal horizontal directions. Two uncorrelated accelerograms based on the type 1 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 show in Figure 1b, and corresponding accelerograms in Figure 1c and 1d. Each building was subjected to a series of dynamic input tests with increasing ground motion, obtained by scaling the accelerograms. The resulting input accelerograms have duration about 17s and spectral acceleration in the plateau region of about 0.75g.

For all the input tests the measurements of the models were made by a total of 28 accelerometers, which were placed at the base as well as in the slabs and openings corners in both directions. From the acceleration time history registered from them and by means of mathematical double integration displacements were also obtained. Table 1 summarizes the inputs and corresponding PGA registered for each building.

Table 1. Input series and corresponding PGA

Test	Reinforced model		Unreinforced model	
	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, here the test was stopped due to it was attained the maximum displacement capacity of the shaking table´s actuators. While maximum PGA for UM registered a value of 1.33g, here a second test of 250% was developed, but due to imminent model collapse some equipment was removed and no data at the base was registered. The test was finally stopped as result of no collapse.

4. TEST RESULTS AND ANALYSIS

After each test of given sequence, the accelerations from shaking table and buildings were processed and analysed. The signal processing made consisted mainly in the removal of quasi static components and noise by means of a Fourier filter between 0.4Hz and 35Hz. Signal offset correction and signal crop were also implemented. For each test series the cracks were accurately mapped as well, from where failure and collapse mechanism were identified.

4.1. Damage

The final damages after an input of 1.61g for RM and 1.33g for UM buildings are presented in Figure 2. It is noted the few damage observed in the RM building after the high strong motion imposed. For this model first cracks appeared after the input test of 100% (0.39g) however it is only until test of 300% (1.33g) that significant quantity of cracks were developed. The model does not present any sign of damage at the second level. The north façade shows only horizontal cracks developed in the mortar joints at the first two rows of units, whereas many smeared horizontal cracks distributed around the

first level wall are found in the south façade, however are also presented similar cracks to the ones observed in the north façade. It was noted that for these two walls 80% of the damaged occurred after the input of 300% (1.33g). In the walls with openings smeared cracks following a diagonal direction within openings are clearly identified. Most of the cracks are horizontal ones at the mortar-unit intersection. It should be mention that the damage in this building started in these walls. This damage is related to the in-plane shear resistant mechanism developed at the firs story where stresses are higher due to lateral movements. At the end of the tests maximum crack openings were lower than 1mm.

Major cracks and damages are presented in UM building in which collapse of masonry units were observed at the two levels. For this model significant damage was found at more early stage that the one registered for the reinforced solution. Long horizontal cracks together with stair step ones were found in all walls for the two levels after the input motion of 100% (0.51g). Not small cracks or damage were presented for the test of 50% (0.27g). Most of the stair step cracks started from the windows openings. Subsequence tests developed a mixed in-plane and out-of-plane resisting mechanism of the walls increasing rapidly the initial damages and causes severe horizontal cracks that joined around the whole structure perimeter. This fact develops the separation of the model in well-defined horizontal blocks that promotes sliding resisting mechanisms during shaking tests. Consequently during the final test of 250% (1.33g) several masonry units from two levels started falling out.
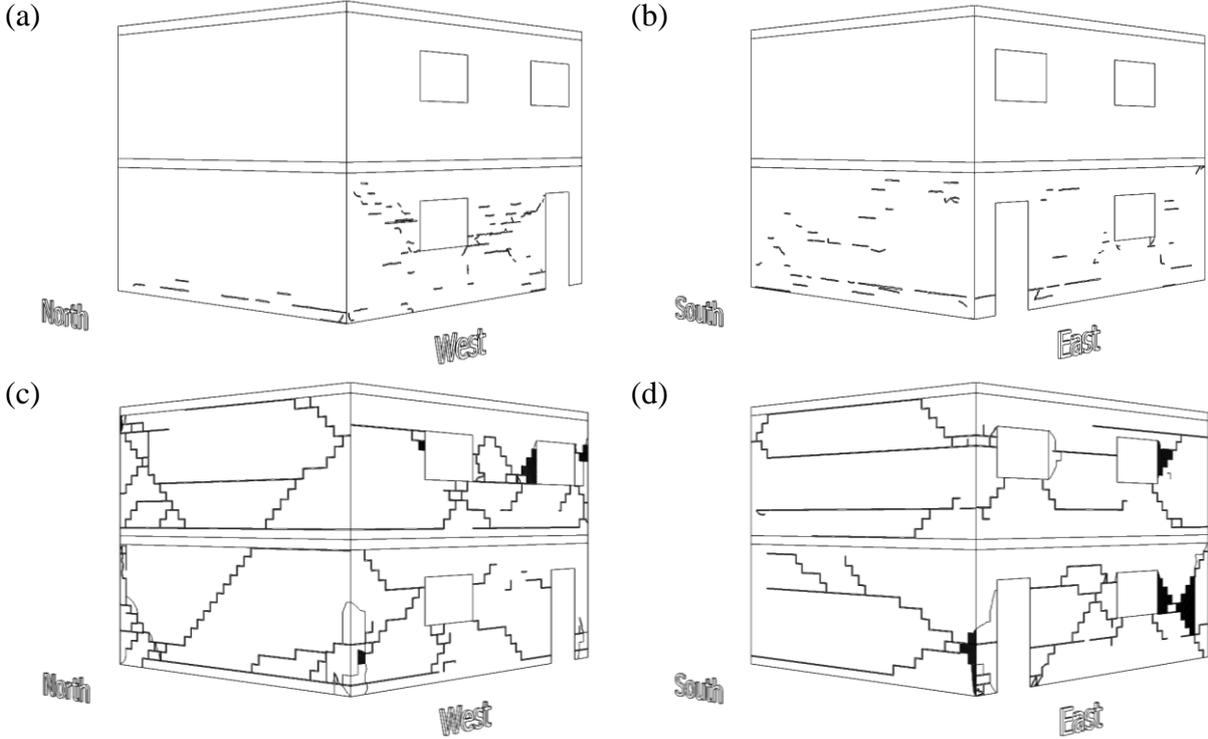
Figure 2. Final damage of: (a, b) RM after 9 test runs; (c, d) UM after 5 test runs

As observed the differences in damages between RM and UM models are notorious. The behavior shown for RM building during all the input tests is explained by the contribution of both horizontal and vertical steel reinforcement which increase the connection between structural members. As observed the final condition of this model is very far away from collapse. In the UM building, even the damages were serious, the whole structure did not collapse after and input of 1.33g. The cracking pattern observed for this model differs considerably from the reinforced solution. On it is possible to find clear diagonal cracks that joined with long horizontal ones in both levels. The sliding mechanism generated, increased significantly the displacements of the structure, but at the end of the tests it remains standing. It should be stressed that from the recent past earthquakes, in which masonry

buildings have been seriously affected, it has been found peak ground acceleration (PGA) values of 0.49g for the 2007 Pisco earthquake, 0.25g for the 2010 Christchurch earthquake and 0.65g for the 2010 Chile earthquake.

4.2. Interstory Drifts

The in-plane behavior of the two structures is analysed in terms of interstory drifts, which is defined as the percentage of the relative horizontal displacement of adjacent floors in relation to its story height. Figure 3 shows the interstory drifts in both directions for the two models.
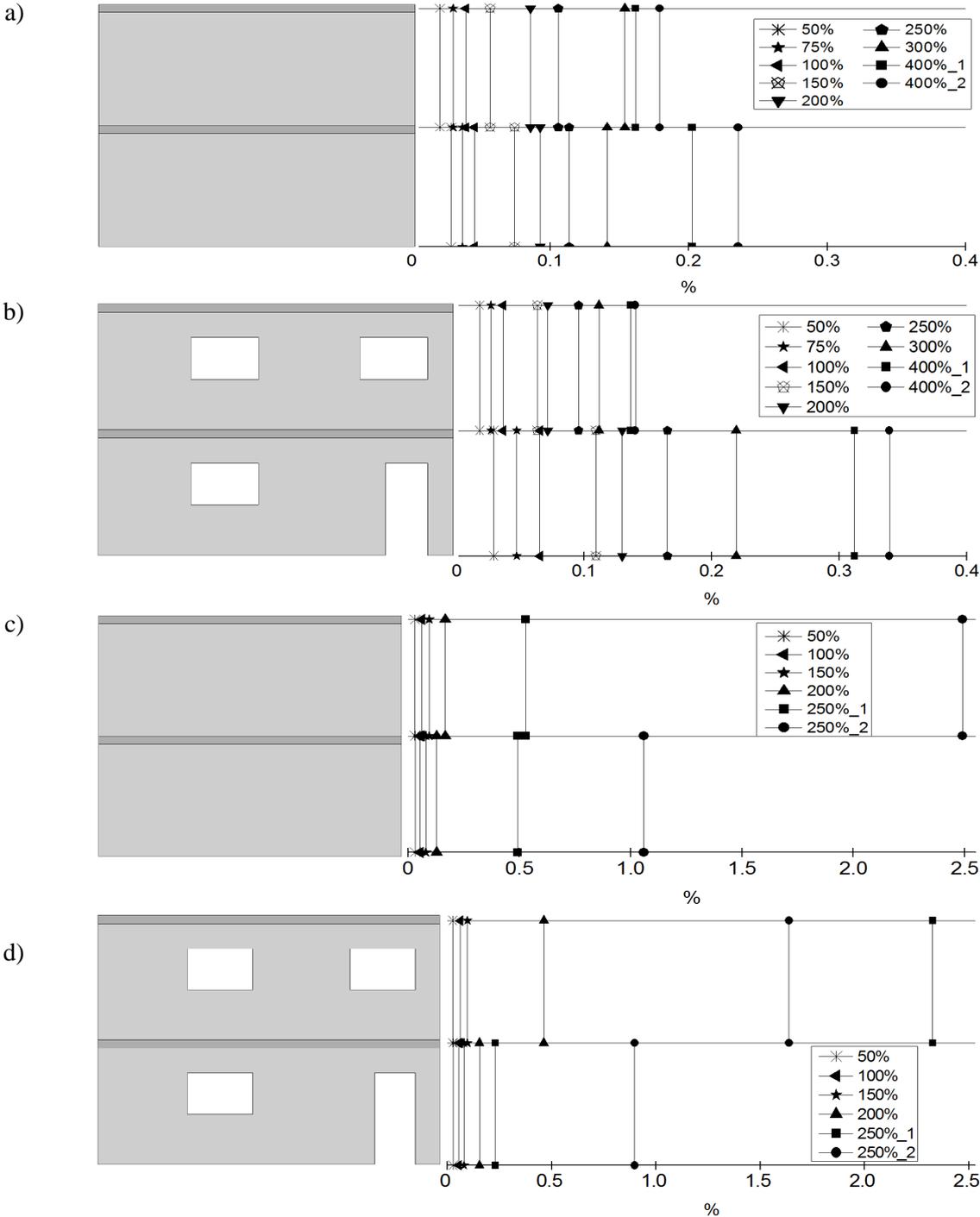
Figure 3. Interstory Drifts: (a,b) RM building and (c, d) UM building

Results from the RM building show a correlation between the incremental input series and the maximum interstory drift registered in both directions. It is observed that higher values are at the first level as well for both directions with a maximum value of about 0.35% in case of longitudinal walls. Maximum values for the second level are less than 0.18%. The results are in agreement with the damage in the sense that for this model cracks were only noticed at the first level. In general visible cracks are presented when story drift at the first level are over 0.15%.

Opposite situation is found in the UM building. In this model similar behavior was observed in both directions in which maximum interstory drifts were registered at the second level, with a value of about 2.5%, whereas the first level registered a maximum value of about 1%. From the plots it is observed that second´s level drifts are higher than the first level ones since the test of 150% in which the maximum base acceleration of the shaking was 0.8g. This behavior is explained due to the accumulated damaged occurred at this stage that caused the development of a sliding mechanism through the horizontal cracks that joined around the perimeter of the building. According with (Magenes and Calvi, 1997) for unreinforced masonry walls the ultimate deformation capacity in shear is very stable when expressed in terms of drift and corresponds approximately to 0.5%. As observed, the damaged presented for the unreinforced building when interstory drifts are about this value is severe, but due to global connection between structural elements given mainly by the mortar and the bond pattern it was possible to attain higher drift without building collapse.

4.3. Out of Plane Displacements

Another important effect studied from the tests was the out-of-plane displacements. As known the vulnerability of masonry to out-of-plane loads can result in the early collapse of structural masonry walls. Figure 4 shows the out-of-plane displacements for both models. Displacements are presented as related to the slabs, in this way the actual out-of-plane displacement carried out by the walls is studied.
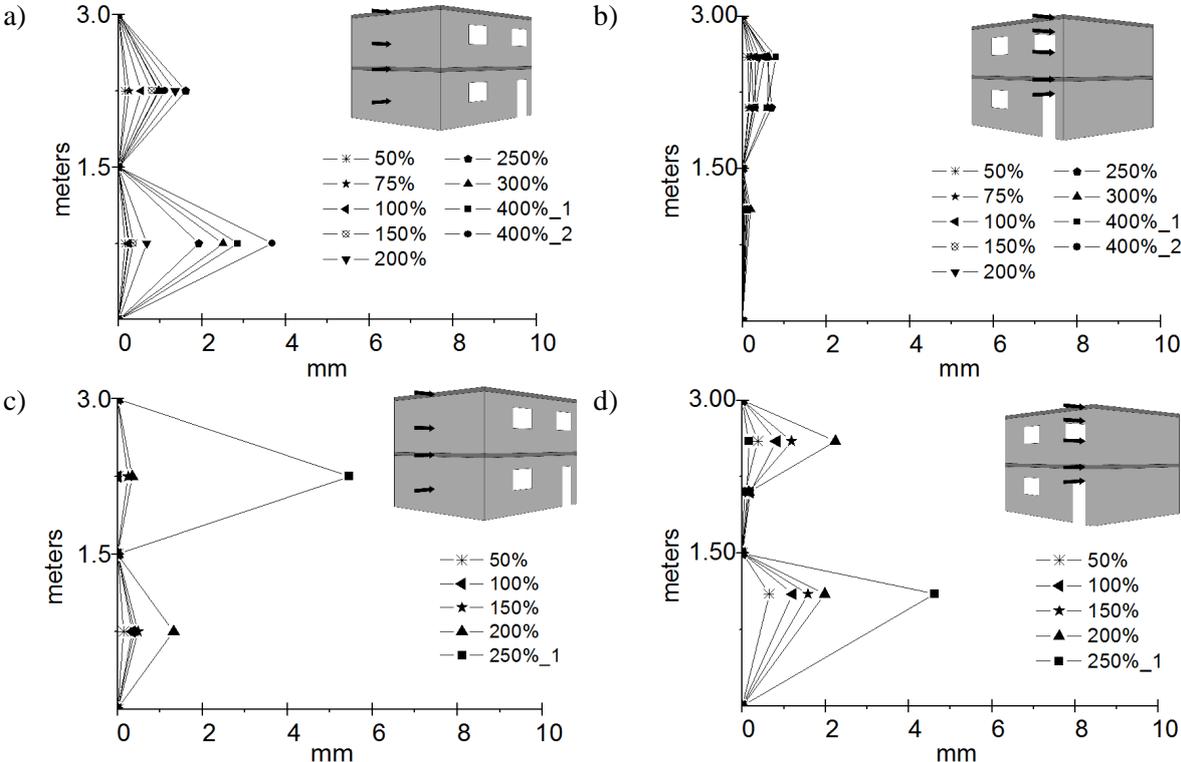
Figure 4. Out of plane displacements: (a,b) RM building and (c,d) UM building

The structural behavior of RM building to out-of-plane forces reveals a stiff structure. In the longitudinal direction a maximum of 4mm was registered, as expected according with the damage, it

happened in the first floor. Similar to the interstory drifts, for this level an increase on input motion represent an increase in the out-of-plane displacements. This situation is different at the second floor in which maximum values are less than 2mm. Here, although with small differences, the displacements do not show relation with the increased input sets.

In the transverse direction it was planned to measure the out-of-plane displacements at three different sections, aligned with the windows corners at the two story levels as well as window and door corners. The accelerometers were glued with epoxy resin to the masonry units and some of them felt during the tests, providing incomplete displacement profiles. The less affected cutaway section corresponded with the door opening at the west façade, this section is analysed, even is known that its locations could not be the most representative for the whole wall in which higher displacement would be expected at the middle of the wall length. For the transversal direction of RM building it is observed considerable lower values that the ones registered in the longitudinal direction, with a maximum below 1mm. In general not important differences are found in comparison between two floors. Here it should be stressed that no damage was observed at the right side of the door for this façade, which could explain the results obtained.

As expected higher maximum displacements are found in the UM building. In the longitudinal direction records from the last input test at the first floor could not be measured due to loss of equipment during the test. Maximum value at this floor is about 1.8mm for an input of 1.11g. The second floor attained a maximum out-of-plane value at the last test, input of 1.33g, near 6mm. However previous test presented, with an input of 1.11g, a maximum of 0.3mm, which means a difference of more than 5mm between consecutive tests.

In correspondence with the RM building plots, for UM building was analysed the same cutaway section in the transverse direction. Here a more consistent behavior is found in relation with the inputs in which progressive out-of-plane displacements are presented. The maximum value is about 4.5mm and was recorded at the firs level. At the second level the maximum attained a value over the 2mm. The behavior of this model for the out-of-plane displacements is clearly explained for the sliding mechanism occurred and previously mentioned. As observed in recorded videos the sliding behavior was more obvious to happen in the transversal direction. This mechanism could also explain why the small out-of-plane displacements in the longitudinal direction for high input motions. Finally it is important to highlight that for the values found in the proximity of the corner the traditional bond pattern adopted presented an adequate behavior even for large values of seismic loading.

4.4. Hysteresis Envelopes

The relation between the shear force at the base of the buildings and the global model drift allows a good understanding of the global behavior of the models. Figure 5 shows the monotonic envelopes of the dynamic behavior developed in the models during each individual test run.
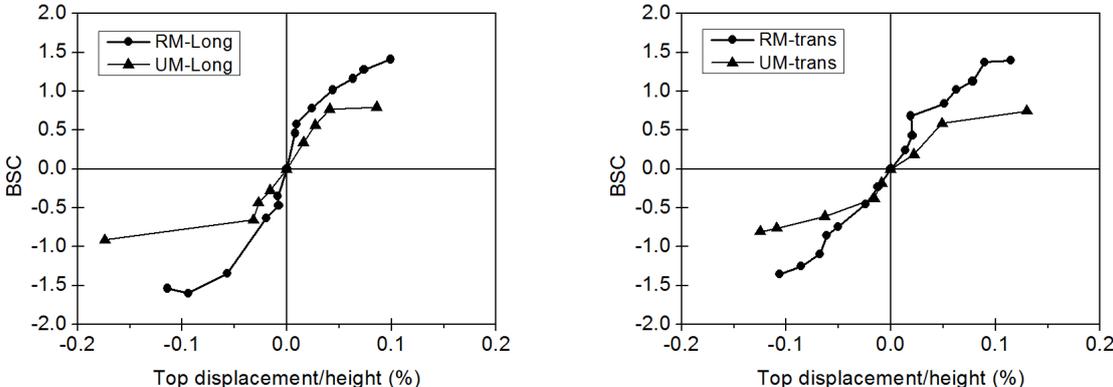
Figure 5. Hysteresis envelopes

The base shear has been calculated as the sum of the inertial forces developed at each story, and is given in terms of base shear coefficient (BSC), calculated as the ratio between the base shear developed in the model during each shaking test (BS) and the weight of the model above the base (W), i.e. BSC= BS/W.

From these plots it was evident that the models presented stiffness degradation. Here it is possible to observe how the stiffness of the RM model is higher than the stiffness of the UM building, both in the longitudinal and transversal directions. This behavior is observed since early stages and corresponds with both the higher level of damage and the larger displacements found in the UM building. However a similitude between models is found, on them stiffer behavior is presented in the longitudinal direction.

The comparison between hysteresis loops, from where the envelopes are obtained, for the two models reveals the difference in energy dissipation. The hysteresis loops found for the UM building involves more energy dissipation, particularly in case of the transversal direction, due to the predominance of the sliding mechanism. In comparison between the same input tests the displacements obtained for the UM model are considerably higher than the ones found in RM model. This behavior clearly shows the improvement in resistant capacity, stiffness performance and displacement response given by the steel reinforcement when it is used in masonry structures.

As observed the maximum values of BSC in UM building are less than 1 for both directions, whereas RM building attained values around 1.5, however the results also indicate clearly that the UM model reached the maximum capacity, while the RM model is possibly also rather close to the maximum capacity as some of the envelopes seem to present already a plateau.

5. CONCLUSIONS

The results of two shaking table tests experiment have shown the seismic behavior of a new construction system by implemented masonry materials. Two symmetrical models with scale 1:2 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. The models were subjected to increasing seismic input loading from which it was possible to record the crack patterns and deformation features.

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. For this model was clear the in-plane shear mechanism developed. Consecutive tests increase rapidly the horizontal cracks and the loss of connection between structural elements through the high of the model.

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. It is believe as well that the behavior through the tests was influenced by the mortar vertical joints in which the steel reinforcement was placed. However, the UM 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 results confirm the adequacy of modern unreinforced masonry by using traditional bond pattern, with robust units, to withstand seismic loading, as not damage was presented after and input of 0.25g and not collapse after an input of 1.33g.

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