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MODAL IDENTIFICATION AND STIFFNESS DEGRADATION OF CONCRETE BLOCK MASONRY BUILDINGS

Leonardo Avila[1], Paulo B. Lourenço[2], Graça Vasconcelos[3] and Nuno Mendes[4]

ABSTRACT

A new construction system for masonry buildings is analysed. Identification of modal parameters has been carried out in two structural solutions by using this system namely reinforced and unreinforced symmetric two-story concrete block masonry buildings. Experimental shaking table tests were developed on these buildings by incremental inputs. Traditional modal analysis by using low amplitude input motion was made before and after dynamic tests and stiffness degradation has been correlated. Mode shapes, mode frequencies, damping ratios and damage identification through the test are discussed

Keywords: Concrete block masonry, Shaking table test, Stiffness degradation

1. INTRODUCTION

For the design and strengthening of a structure, it is of great value the knowledge of its behaviour under seismic excitation. In particular the limit states and safety verification implemented for the design of buildings in resent codes aiming for the prediction of the structure´s response when subjected to seismic motion. Is then understandable why the knowledge of the dynamic properties of a structure not only on its early ages but also when it has been affected by important damage gives an important tool for the correct prediction of the structure´s behaviour when it has to deal with seismic loads and valuable data for the design or strengthening of future structures.

A new masonry construction system for low to medium residential buildings based on concrete block units is studied. With the aim of identifying its seismic behaviour, two models of a prototype building structure have been tested in a shaking table by performing incremental input motions. The prototype has symmetric geometry in plan and elevation. The models have been constructed with a scale 1:2 and correspond with a reinforced (RM) and unreinforced (UM) solution of the system. In order to identify the dynamic properties and possibly correlate structural damage the incremental seismic tests were

[1] PhD Candidate, ISISE-Department of Civil Engineering University of Minho, leoavila@civil.uminho.pt
[2] Professor, ISISE-Department of Civil Engineering University of Minho, pbl@civil.uminho.pt
[3] Professor, ISISE-Department of Civil Engineering University of Minho, graca@civil.uminho.pt
[4] Postdoc Researcher, ISISE-Department of Civil Engineering University of Minho, nunomendes@civil.uminho.pt

preceded by an equal series of modal characterisations tests, from which modal parameters were identified.

Through the centuries in which masonry buildings have been subjected to earthquakes it has been evident the low resistance capacity to lateral movements that this construction typology possesses. The low shear strength appears has the main reason for that [1]. In addition, as appointed by other authors, the dynamic behaviour of masonry buildings differs from that of concrete or steel ones due to the not rigidity connection between structural elements, like walls and floors, typical in this kind of construction and one of the reason of the poor performance during earthquakes. Aiming to increase the response against earthquakes, new codes request for a minimum of steel reinforcement for masonry buildings constructed in seismic prone areas. Is then one of the objectives of the paper first to find what are the dynamic properties of the models and then to analyse what is the influence in the modal parameters of the implementation of steel reinforcement for the construction system here studied. Finally the construction of vulnerability curves, in which the relation of the stiffness degradation, in this case directly related with the increase of damage is proposed.

2. CONSTRUCTION OF THE MODELS AND EXPERIMENTAL PROGRAM

The prototype building consisting of a two-story rectangular structure about 6m high with reinforced concrete slabs and with openings in only two opposite facades. Due to geometrical and capacity limitation of the shaking table placed at national laboratory of civil engineering (LNEC) in Lisbon, Portugal a reduce scale of 1:2 had been implemented. For it Cauchy similitude law was chosen. Table 1 describes the principal properties involved in the scale process, here *P* refer to prototype and *M* to model.

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]	$f_p/f_M = \lambda^{-1}$

The final scale geometry of the models is presented in Figure 1. The reinforced masonry building (RM) was constructed with a steel ratio of 0.05, in agreement with the Eurocodes [2-3]. For their construction several important differences arose, here beyond the implementation of reinforcement, was the consequences of it. The steel reinforcement was placed in both horizontal and vertical joints, then in order to make easily and faster the construction process, vertical continuous joints in which vertical steel reinforcement was presented were desirables, these joints were filled out with the same mortar used for the placement of the horizontal joints. For the UM model it was not necessary the construction of vertical continuous joints, then the bond pattern of the models is different.

The data acquisition system was similar for both models and consisted of 26 accelerometers and one acquisition unit. The location of the sensors is shown in Figure 1, they were placed in correspondence with the slabs and opening's corners. In addition an accelerometer was placed at the middle of each level in the north façade. The selected position of sensor allows for the identification of in-plane and out of plane deformation of the structure.

On the shaking table the two masonry buildings were first subjected to low level forced vibration tests, by means of a uniform white noise signal in the two orthogonal directions. The signal was processed with a low pass filter of 125Hz and imposed to the models with a sample frequency of 250Hz for about 3 minutes. For the main seismic inputs, first an accelerogram from the elastic response spectrum given by the Eurocode 8 [2] for the area of Lisbon was obtained then it was taken as the reference. In congruence with the adopted scale law, the reference input was shortened in time and increased in acceleration, this final input was taken as 100%. In order to obtain a successive damage assessment the final signal was introduced to the table with increasing percentage intensity. Table 2 presents the tests specification and the PGA obtained during each test for the two models, on the table NS refer to longitudinal and EW to transversal direction respectively. Dynamic characterization was performed before and after each seismic testing.

Table 2. Input series and corresponding PGA

	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)	-	-

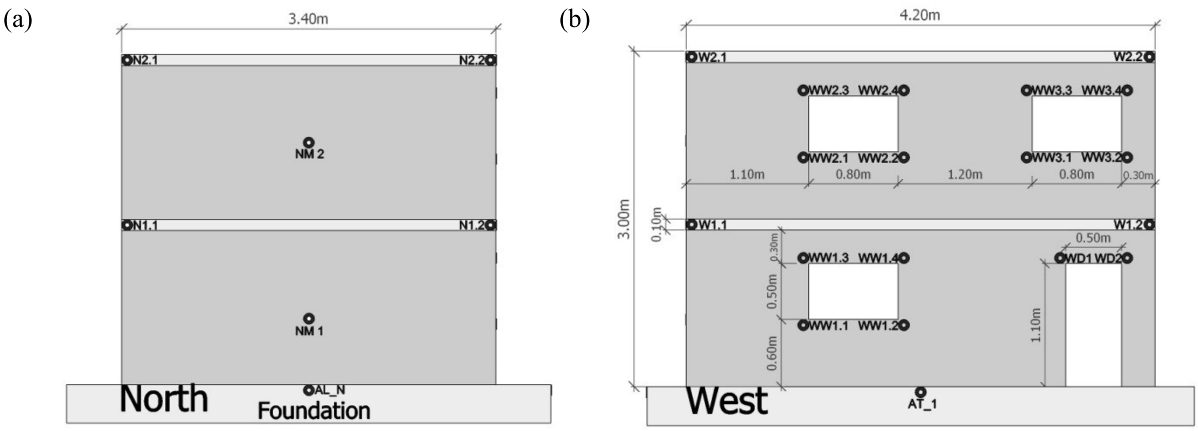

Figure 1. Scaled model geometry and location of the sensors: (a) front and (b) lateral view

RM building attains the maximum input that the shaking table can support without significant damage, with a registered PGA of 1.61g. On the other hand seismic input in UM was stopped due to imminent collapse of the building, here a maximum PGA of 1.33g was recorded.

3. TEST RESULTS

The solution of the eigenvalue problem, 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 identification was based on the frequency response functions (FRFs), phases and coherences, estimated through traditional methods of signal analysis [4].

Results from these identifications have been processed by using the software LNEC-SPA and ArteMis [5-6]. A first identification without any filter has been done aiming to identify the higher quantity of modes, however for both buildings no 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 the signals. The results were obtained and processed in thousands of gravity (mg). For the correct comparison and accuracy on the actual peaks of the time history records, if needed, DC offset was also implemented. Intended to avoid aliasing, no decimation was applied to any signal, so that a sampling frequency of 250Hz corresponding to a sampling rate of 0.004s was accepted and finally, boundary noise reduction by means of cropping of the extreme registered data was performed. As second step for the correct identification of the natural frequencies consist in the implementation of frames with a standard overlap of 2/3 and a reduction of leakage by the introduction of a Hanning window.

Figure 2 shows the transfer function curves for the 26 accelerometers placed on the buildings. Due to the symmetry of the two models only two essential natural frequencies were identified in the frequency range analyzed.

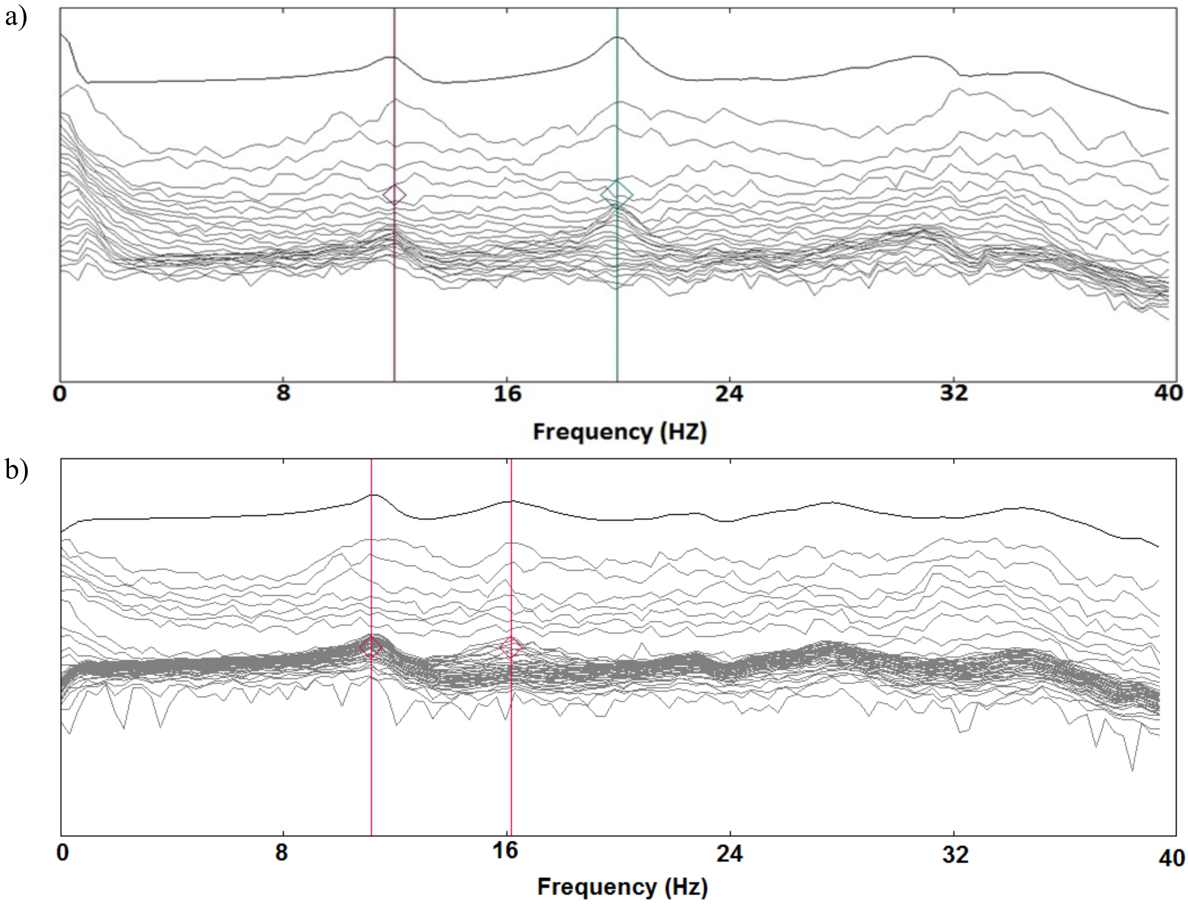

Figure 2. Transfer function curves for: a) RM and b) UM model

As seen, for the masonry buildings the first two natural frequencies are clearly identified by well define peaks, summarizing the frequency response of all the accelerometers. In a frequency around 30Hz was expected to find a third mode but after analyzing the mode shape in both cases it was concluded that a not logical behavior was related and it was not taken into account. For the models the first natural frequency occurs in the transversal direction. RM registered a value of 11.90Hz and UM a value of 11.27Hz. 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. In a first look to the graphs and the values the first natural frequency presents no significant differences between models, however opposite situation is presented for the second frequency, this will be discussed later with the analysis of the whole response through seismic tests.

For the estimation of the coefficients of equivalent viscous damping Half-Power bandwidth method has been used. The identification of damping ratios was made for each orthogonal direction in agreement with the frequency identification, thus it was found a value of 2.62% for the first mode of RM and a value of 3.85% for UM, the second mode present respectively values of 1.34% and 3.52%. Table 3 summarizes the initial modal parameters found for the models.

Figure 3 presents the mode shapes obtained for both models. The behavior obtained is considered global for the buildings, even it was found a small local behavior in the second mode for the lower part of the west wall in the first level in UM. However for both models it was noted that even the frequency values are different the directions and shapes of the modes are similar. As expected this means that the buildings present similar global behavior, but RM possesses higher frequencies values as it is noted in Figure 2. 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 notoriously increase the stiffness of the RM structure in both directions.

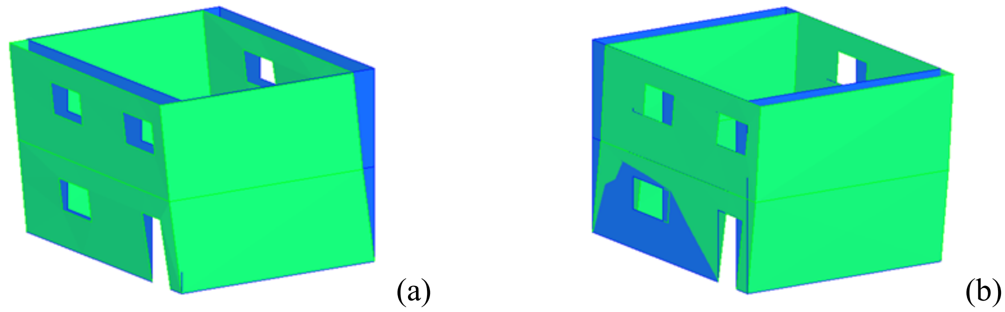

Figure 3. Global mode shapes for the masonry building models: (a) first mode-transversal direction and (b) second mode-longitudinal direction.

The two mode shapes are in relation with the orthogonal directions of the models, the rotational mode was not clear identified from the data obtained. According with the frequency values there is a difference of 5.3% in the first mode between the RM and UM building with a higher value for the first one, the difference increase to 19.42% for the second mode. Then the difference in stiffness between the two solutions was obvious.

It is well known that the evolution of the natural frequencies can be related to the progress of damage associated to the increasing of the seismic action imposed to the models by the shaking table. Indeed, a reduction of the natural frequencies can be explained by the reduction on the stiffness of the models, which is associated to the existence of damage. Therefore, it was decided to obtain the natural frequencies and damping ratios before and after each seismic test run corresponding to the imposition of increased level of seismic loading, the evaluated results are summarized in Table 3.

Table 3. Evolution of the fundamental frequency and coefficient of equivalent viscous damping

		Test run	Initial	50%	75%	100%	150%	200%	250%	300%	400%_1	400%_2
f (Hz)	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	-	-	-
ξ(%)	RM	long	1.34	1.52	2.65	3.20	3.30	5.32	6.45	7.63	5.84	6.24
		trans	2.62	2.75	3.06	3.18	3.40	4.70	4.73	4.81	3.73	3.90
	UM	long	3.52	4.76	-	5.34	5.52	6.06	7.68	-	-	-
		trans	3.85	4.06	-	4.62	5.04	4.90	5.47	-	-	-

From Table 3 it is observed that in RM there is a decreasing trend of the frequencies through the development of the tests. In the longitudinal direction is presented a decreasing rate about 1Hz per test. Different behavior is observed in the transversal direction in which the decay occurs in a rate about 0.3Hz. From the observed damage after each tests and at the end of the seismic inputs it was notorious a higher distribution of cracks in the longitudinal walls, mainly starting from the corners of the windows and following a diagonal path to an opposite corner. The transversal walls only presented damage at the bottom of them up to the two first courses of blocks, caused by the low shear resistance of the bed joint mortar. In this model no any damage affected any block unit.

The damage observed in UM was presented with a considerable high rate than the one in the RM and it was corroborated by the evolution of the frequency values. On this model the frequencies through the tests also decreased, as similar to the RM higher rate of decrease is presented in the longitudinal direction. This tendency reveals the influence of openings in the resistant capacity of the masonry walls in which the resistant mechanism is concentrated in the piers formed between openings. The damage on this building was considerable at earlier stages, so that severe cracks were observed after the second tests developed, corresponding to 100% of the reference signal and with a PGA of 0.43g. Subsequence seismic input increasing the damage at the point to be needed stop the test due to the imminent collapse and protection of the equipment. After the mentioned test a no logical trend was obtained in the frequency values. In both direction from test of 150% to 250% values increasing and ended with a slight difference to the ones obtained in 100%. This behavior is explained for the loss of connection in the walls in which continuous horizontal and diagonal cracks were presented through the bed and head joint forming a sliding mechanism that divide the building horizontally in almost independent structures. It started from test run of 150% with a PGA of 0.68g, in which large horizontal cracks developed in the UM model, extending to different walls. Namely, a horizontal crack located at the first course of the first story with a length of 80% of the perimeter of the building. These typologies of crack path modify the vibration properties. This type of crack occurred because with the increase amplitude of the inputs the same crack followed the perimeter around the model and finally joint together, then the characteristic and global stiffness of the building change with the loss of connection through the high. For test until 100% the frequencies of RM were higher than the ones from UM. Due to the previously discussed issue this was no longer observed.

A comparison between the highest and lower frequency obtained shows a reduction of 37% for RM and a 13% for UM. Both reductions were obtained in the longitudinal direction. However it should be noted that natural frequencies of UM at initial stage without any damage are similar to the ones in the RM when the last one overcome the fourth seismic input i.e. 150% of the reference input signal. But even with the prior results the damage were more significant in UM were structural elements were affected to the point of near collapse. The placement of steel reinforcement in the form of vertical continuous joints appears as the reason for this behavior. It is know that reinforcement increase the resistance behavior of the structures against earthquakes, in the present study a minimum of reinforcement ratio given by the Eurocode 8 [2] was implemented, but a relation between this parameter and global stiffness would be desirable for the discussed constructive system.

The dissipation of the kinetic and strain energy of the vibrating structure is representing by the damping ratio. In literature is possible to find different options for the estimation of the damping values. As a rule, difficulties come to its estimation as the energy is dissipated by various mechanisms that are not related to a unique physical phenomenon and make hardly possible to identify or describe mathematically each of these energy-dissipating mechanism in an actual building. The damping depends on the hysteresis rule appropriate for the structure being designed. Normally, for masonry structures, the damping ratio is taken as 3% related to critical damping, a value that has been supported by several investigations. A relation between the stiffness and the energy dissipation (damping) properties of the structure can be also found. Over the past decades the equivalent viscous damping has been used as a key parameter in the predictions of the maximum nonlinear response. However due to the uncertainty in its experimental estimation and the difficulty in the appropriate modeling method for include it in the design process it constitutes at the moment an open parameter to experimental and analytical research.

From Table 3 in congruence with the frequency behavior it is observed a change in the damping ratio, but contrary to what was described for the frequency and as expected, damping values increasing after each incremental seismic test. This is observed partly because damping is also a measure of the parameter controlling peak amplitude of the response to dynamic load, then damping would suggest in that way more energy dissipation mechanism, indicating the possibility of cracks in the structure. It is notice from the results that initial damping for RM are about half of the respectively values of UM. At the end of the seismic inputs damping ratio increase about 6 times for RM whereas only 3 times for UM in the longitudinal direction. In similar with the frequency the influence of the openings and steel reinforcement explain that situation.

The damping, as the other dynamic properties, change with the stiffness of the structure and could be related with the progress of the damage. However, the variation on the natural frequencies is more commonly correlated with the cumulative progress of damage. With the obtained data and for the RM masonry building it is possible to relate the progress of damage with the stiffness degradation, meaning that a relation between the damage and frequency reduction can be defined. Considering that there is no loss of mass during each of the i tests run, then a simplified damage indicator $d_{k,i}$ for the mode shape k of the masonry buildings can be estimated by means of Eq.1:

$$d_{k,i} = 1 - \left(\frac{f_{k,i}}{f_{k.0}}\right)^2 \tag{1}$$

where $f_{k,0}$ is the initial first natural frequency, determined before the series of shaking table tests, and $f_{k,i}$ is the frequency obtained after each input loading series. Figure 4 summarizes the relation between this damage indicator and the imposed PGA for RM. It is important to stress that the damage indicator refers to the cumulative damage.

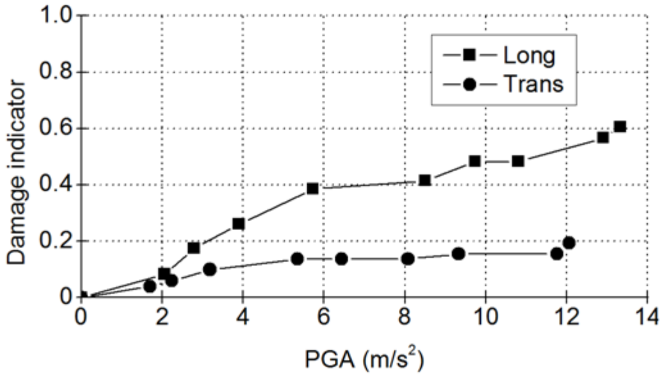

Figure 4. Simplified damage indicator for RM

As already discussed the stiffness degradation in the transversal direction is less significant than in the longitudinal for all the inputs, which results from rough diagonal cracking developed on the longitudinal walls with openings. It should be mention that the longitudinal walls, present higher concentration of reinforcements than the transversal walls due to the presence of openings. Then because to the only localized horizontal crack developed in the transversal walls, the change in the damage indicator looks almost horizontal.

4. CONCLUCIONS

In this paper particular emphasis was made on the dynamic identification of the modal parameters of two concrete block masonry buildings built with a new concrete block concept in order to identified damage correlation through a series of shaking table inputs. Following European codes, one building was designed and built with horizontal and vertical reinforcement and one as a simple and traditional construction. Low amplitude forced vibration tests were performed between the incremental seismic inputs by using 26 accelerometers to measure the structural response. In spite of the same seismic inputs were applied to both models the modal parameters of the buildings showed notorious differences, a relation between stiffness and frequency degradation was made, as well as stiffness degradation and damping increment.

The study revealed 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 allows 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. Given the arrangement of the vertical reinforcements, it can be considered that the RM model is composed of partial filled vertical joints, contrarily to the UM model, where only dry head joints are presented. The differences in the damage distribution as well as in the responses through the test runs in terms of displacements showed that reinforced masonry buildings are able to sustain relatively high dynamic excitation due to significant level of structural overstrength.

The failure mode presented in UM was of shear failure due to the bed joint sliding. This mechanism presents clear continuous connection among cracks that affecting the whole building in the first and second story. More severe damage was observed in this building than in RM. 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. then stiffness reduction was related, frequency and damping values shows a correlate relation that allows the identification in the case of RM of damage vs PGA giving an important tool for the study and analysis of new buildings design as well as data for the future identification of damage in structures made with this construction system.

Finally from the experimental results it seems that the occurrence of large horizontal cracks promotes the development of sliding deformation mechanism in unreinforced masonry, which should hide the reduction of stiffness due to the increase on damage and hence the reduction on the frequencies. However it shows 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.

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REFERENCES

[1] Bruneau M (1994) State of the Art Report on Seismic Performance of Unreinforced Masonry Buildings. J Struct Eng 120(1):22

[2] Eurocode (2004) EN 1998-1. In: Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings.

[3] Eurocode (2005) EN 1996-1-1. In: Eurocode 6 - Design of masonry structures - Part 1-1: General rules for reinforced and unreinforced masonry structures.

[4] J. S. Bendat and A. G. Piersol. (2000) *Random Data: Analysis and measurement procedures, Third ed.* New Jersey: John Wiley & Sons.

[5] Mendes L (2009) LNEC-SPA Signal Processing and Analysis tools. In: National Laboratory of Civil Engineering, Lisbon

[6] S. V. S. A/S, "ARTeMIS Extractor Pro," ed. Denmark, 2009.